THE INSTRUMENTED TOWER SILO AT HAMMOND, ONTARIO

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Received 9 March 1979

Bozozuk, M. 1979. The instrumented tower silo at Hammond, Ontario. Can. J. Agric. Eng. 21: 79-84.

A concrete stave silo 7.32 m in diameter, 24.28 m high, with foundations designed for the allowable bearing capacity of the marine clays at the site, was instrumented with settlement pins, earth pressure cells and piezometers. The performance of the structure was monitored from 1973 to 1978. The predicted and observed settlements were compared and the distribution of the pressures under the silo was measured. Up to 50% of the silage load was transmitted by friction through the walls to the foundations.

INTRODUCTION

On 28 Feb. 1973, the Ontario Ministry of Agriculture and Food held a special meeting in Ottawa to discuss problems related to the performance of tower silos. One of the major concerns was that an increasing number of foundations were failing due to inadequate design. Before improvements could be made, however, a number of investigations had to be carried out to:

(a) determine the distribution of vertical pressure applied to the soil below the foundation of a filled silo;
(b) determine the proportion of the total silage load transmitted by friction through the silo walls to the footings;
(c) determine the nature and magnitude of vertical settlements for a filled silo supported on compressible clay.

In the summer of 1973, the Division of Building Research of the National Research Council of Canada, in cooperation with Agriculture Canada and the Ontario Ministry of Agriculture and Food, instrumented a concrete stave silo at Hammond, Ontario, to carry out these investigations. This paper describes the instrumentation used and presents the observations made from 1973 to 1978.

PROPERTIES OF SOILS AT THE SITE

The silo was located on poorly drained flat agricultural land about 30 km east of Ottawa. The natural groundwater table was at a depth of 1 m, but a 30-m deep, free-flowing artesian well situated at the southwest corner of the farmyard indicated high excess pore water pressures below the clay formations. The silo was constructed about 10 m from two 15-m high tower silos.

Soil Profile

A soil boring showed 2.1 m of light brown sand overlying marine clay (Fig. 1). The sand formation consisted of fine to medium grain sand with 6% silt.

From 2.1- to 8.2-m depth, the marine clay for the most part consisted of horizontal bands of red and gray layers of silty clays, and some contorted layers. Black mottling occurred between 3.0 and 5.8 m, and distinct layers of sand were encountered at depths of

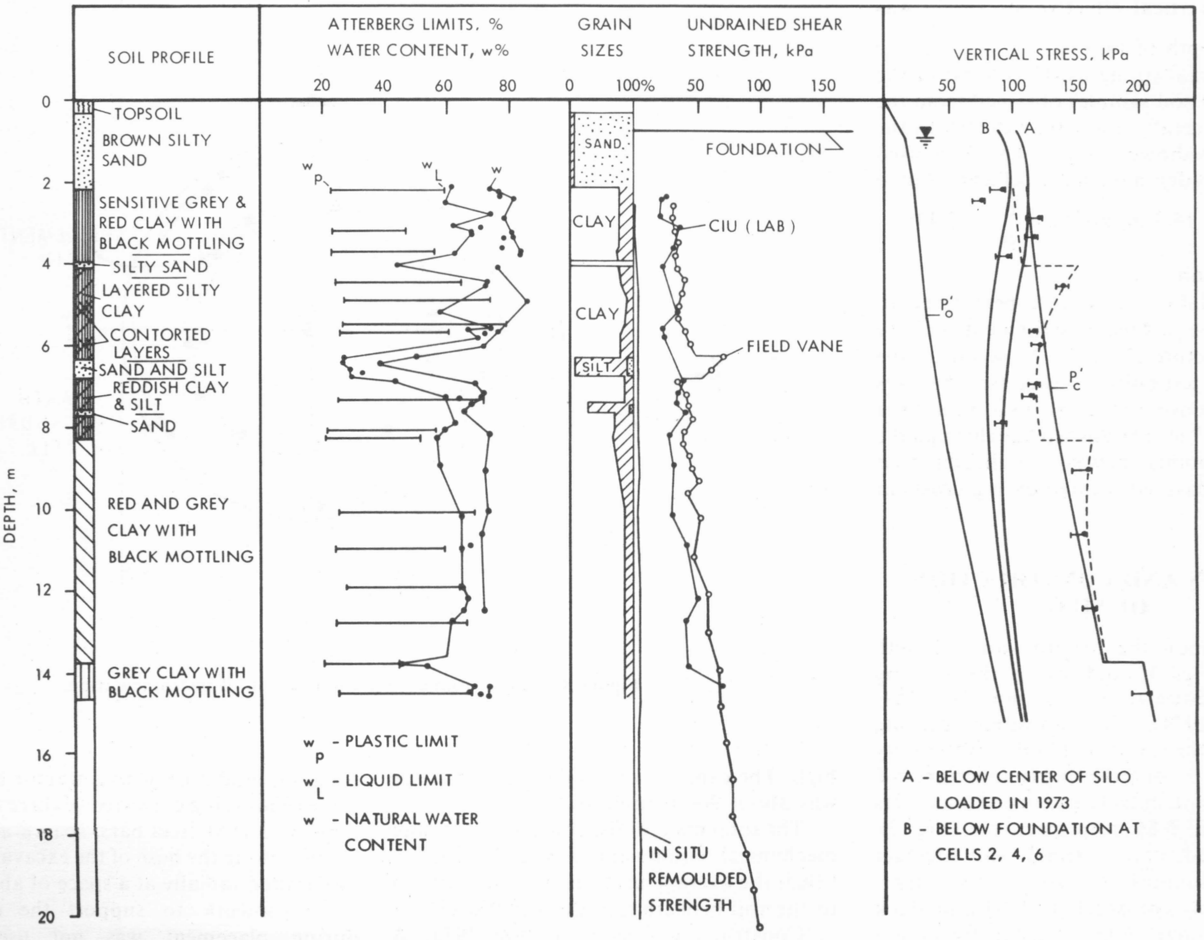

Figure 1. Composite of soil test results at Hammond Silo.

4.0, 6.4 and 7.6 m. The water content of the clays exceeded the liquid limits which varied from 47 to 80%, indicating that the clays were highly sensitive to remolding. The plasticity index varied from 31 to 46%, the clay size fraction exceeded 70%, and the activity varied from 0.31 to 0.61 indicating that this clay was inactive.

From 8.2 to 13.7 m, the soils were predominantly red silty clays with black mottling and some layers of gray clay. The water content exceeded the liquid limits which varied from 51 to 69%. The plasticity index was about 37%, the clay size fraction varied from 69 to 89% and the activity, which varied from 0.43 to 0.49, indicated inactive clays.

From 13.7 m to the bottom of the borehole at 14.6 m, the soil changed to a gray silty clay with black mottling and contained small shells. The water content was greater than the liquid limits. The plasticity index of 42%, clay size fraction of 84% and an activity of 0.50 showed this clay formation to be inactive.

Strength

The in situ shear strength of the soil was measured with a 55-mm diameter, 110-mm high Geonor vane (Andresen and Bjerrum 1956). It increased approximately linearly with depth from 31 kPa at 2.6 m to 98 kPa at 20.3 m except at the sand layer encountered at 6.4 m (Fig. 1). The ratio of shear strength to in situ vertical effective stress, $\frac{c'}{p'}$ was 0.51 to a depth of 14.6 m.

The triaxial strengths (CIU tests) of the undisturbed soil samples obtained from the site were generally lower than the field vane strengths, as shown in Fig. 1. They increased linearly with depth from 22.6 kPa at 2.4 m to 69.6 kPa at 14.3 m, giving a $\frac{c'}{p'}$ of 0.44.

Consolidation

Consolidation tests were performed to estimate the ultimate settlement of the loaded structure (Fig. 1). An average line through the test points showed that the clays were overconsolidated by 88.3 kPa to a depth of 13.7 m. The dashed line through the same test points, however, indicated that there may have been three drying crusts at the site.

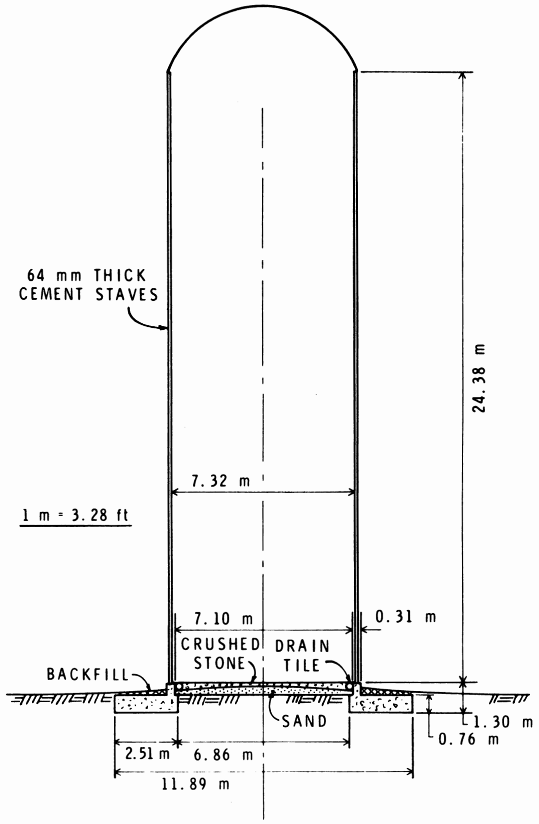

Figure 2. Design details of tower silo and foundations.

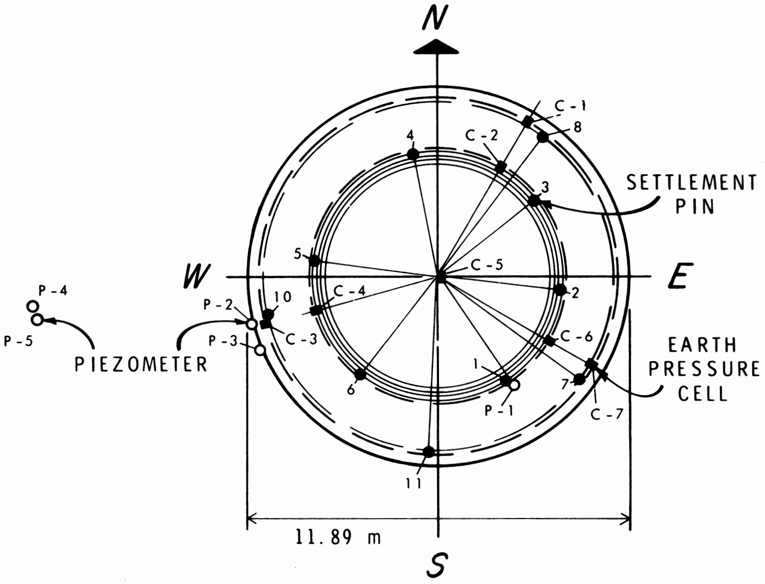

Figure 3. Plan of foundation and location of instrumentation.

DESIGN AND CONSTRUCTION OF SILO

Based upon the in situ vane strength given in Fig. 1, and using the bearing capacity equations (Skempton 1951; Bozozuk 1974), the allowable bearing capacity of the soil was 117 kPa with a safety factor of 3. A concrete ring foundation, 760 mm thick, with outside and inside diameters of 11.89 and 6.86 m, respectively (Fig. 2), was designed after Turnbull and Metzger (1974) to support the tower. The superstructure was constructed of 64-mm thick precast concrete staves (Fig. 2), forming a circular tower, 7.32 m in diameter, 24.38 m high. The capacity of the silo for corn silage was about 900 t (tonnes).

The total mass of the entire structure and mechanical equipment was 224 t. When filled, the average uniform pressure applied to the soil beneath the silo was 100 kPa.

Construction began in July 1973. A circular trench 2.51 m wide was excavated for the foundation with a tractor backhoe. Steel reinforcing consisted of three hoops of deformed 15M steel bars, spaced at 0.61-m centers along the base of the excavation and distributed radially at a space of about 1.22 m. Formwork to support the concrete during placement was not used. The excavation was filled with 56.43 m³ of

TABLE I. LOCATIONS AND DEPTHS OF VIBRATING WIRE PIEZOMETERS

No.	Distance from center of silo (m)	Depth (m)
P-1	4.15	3.48
P-2	5.93	8.90
P-3	5.94	3.41
P-4	12.80	8.60
P-5	12.89	3.20

"ready-mixed" concrete to form the foundation.

A concrete collar 305 mm wide, 530 mm deep, was cast on the inner edge of the footing to support the tower (Fig. 2). Sand and crushed stone were used to form a floor. Field drain tiles were installed along the inside circumference of the foundation to drain away excess silage juices which develop when the corn silage is too wet. The construction of the foundations and the superstructure was completed in September 1973.

INSTRUMENTATION

Field instrumentation was required to monitor the engineering performance of the structure during loading and unloading. The instrumentation consisted of 11 settlement pins to measure the settlement of the structure with time, seven rectangular 20 X 30-mm Gloetzl earth pressure cells to measure the contact pressure beneath the structure, and five vibrating wire Geonor piezometers to monitor the pore water pressures in the clay soils. The locations of the instruments are shown in Fig. 3. They were selected according to the geometry of the structure and changes in the soil profile, with consideration of the direction of the prevailing strong winds in the region.

The earth pressure cells were placed directly on a carefully prepared surface of undisturbed soil and immediately covered with concrete. The settlement pins were driven into the hardened concrete with a Ramset gun after the foundations were completed. The vibrating wire Geonor piezometers were pushed vertically into the ground with "E" size steel drill rods to the depths given in Table I.

DISTRIBUTION OF VERTICAL PRESSURE

During filling in 1973 and 1974, the number of loads of corn silage blown into the silo and the resultant height of the silage were recorded each day. The moisture content measured by the owner was 69 — 70%. From the height:density relationship given by Bozozuk (1972), it was possible to estimate the weight of silage. The weight of silage was also determined from the observed changes in contact pressures measured with the earth pressure cells. The

TABLE II. PROPORTION OF TOTAL LOAD IN SILO TRANSMITTED BY FRICTION THROUGH WALLS TO FOOTINGS OR CARRIED BY INTERIOR FLOOR

Year	Maximum load (tonnes)	Load transmitted through silo walls (%)	Load carried by interior floor (%)
1973	795	47.5	52.5
1974	795	51.7	48.3
1975	708	45.9	54.1
1976	373	45.5	54.5
1977	379	42.5	57.5
1978	518	46.8	53.2

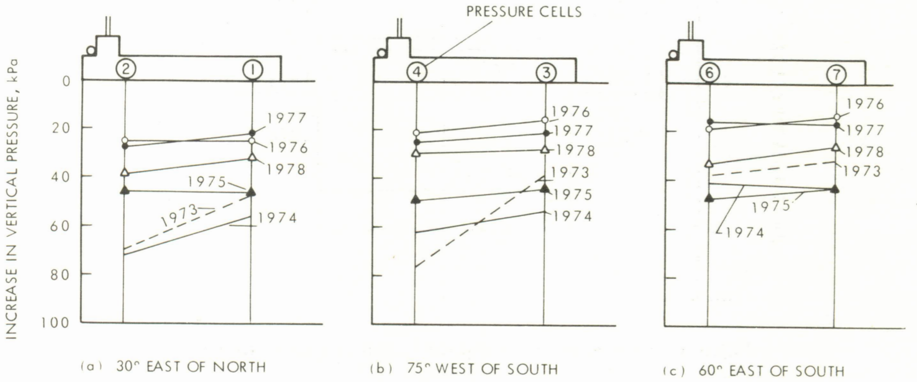

Figure 4. Distribution of measured changes in vertical contact pressure after loading from 1973 to 1978.

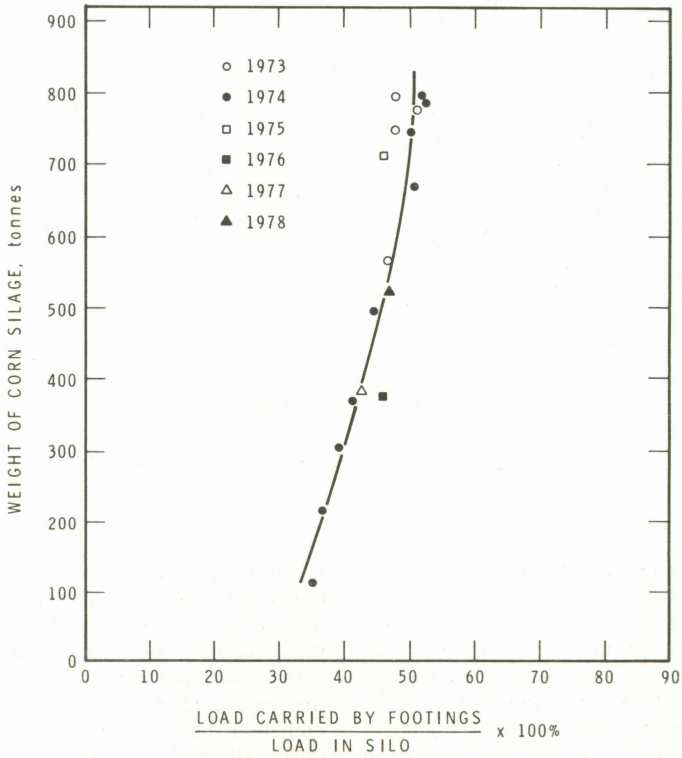

Figure 5. Proportion of silage load transmitted through walls to footings, measured during filling.

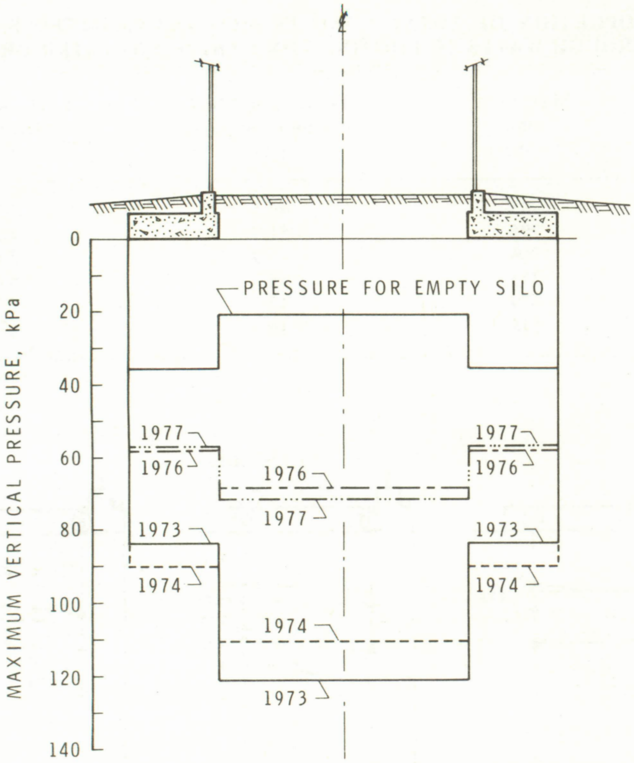

Figure 6. Vertical pressure applied to soil at base of foundation.

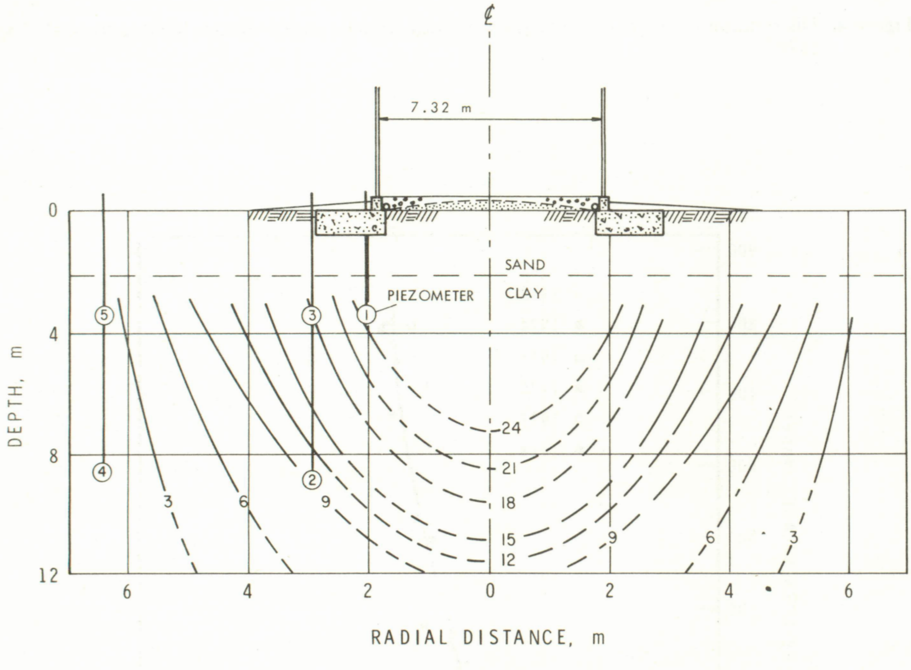

Figure 7. Excess pore water pressures after first filling, kPa.

estimated and measured maximum weights agreed to within 1%.

The distribution of the contact pressures around the ring foundation was measured each time the silo was loaded during the period 1973 - 1978. As shown in Fig. 4, the contact pressures were normally higher at the inner edge or heel of the ring foundation than at the outer edge or toe, at each of the instrumented sections. The difference was greatest (up to 50%) when the silo was new, but it diminished and the pressures became more uniform (to within 5 - 10%) after the third loading. Because of the orientation with respect to the prevailing strong winds, the measured pressures were consistently higher in the sector 30° east of north, and lower, 60° east of south.

From the contact pressures measured with the earth pressure cells, it was possible to determine the proportion of the total silage load transmitted by friction through the walls to the footings. The maximum amount of corn silage placed in the silo each year from 1973 to 1978 is given in Table II. At no time was the silo filled to its total capacity of 900 t. The mass varied from 373 t in 1976 to a maximum of 795 t in 1973 and 1974. The proportion of the maximum load transmitted by friction through the walls to the footing each year varied from 42.5% to 51.7% with an average of 46.7%. This was significantly below the 67% that would be determined from the equation proposed by Bellman (1972).

The relation between the amount of silage in the silo and the load transferred by friction through the walls to the footings during filling is shown in Fig. 5. The load transfer increased from 35% when the mass was about 100 t to a maximum of 50 ± 3% at 800 t. This load transfer relation is most likely affected by silo diameter as well, but this could not be verified with observations from only one structure. Published data to corroborate this statement are not available.

The distribution of the contact pressures beneath a filled silo has an important bearing on settlement estimates and in determining the factor of safety against a bearing capacity failure. The observed average vertical pressures beneath the ring foundation and under the granular floor inside the silo are shown in Fig. 6. For the first load in 1973, the vertical pressure inside the silo was about 37 kPa more than under the ring foundation. This difference decreased with each successive loading cycle, and was about 10 - 15 kPa in 1976 and 1977. It appears that vertical pressures tend to become relatively uniform beneath the whole structure over the long term, thus supporting the general assumption of uniform pressures for stability and settlement calculations.

PORE WATER PRESSURES DUE TO LOADING

Vibrating wire piezometers were selected because they responded well to rapid changes in pore water pressures. They reached equilibrium with the surrounding groundwater conditions soon after they were installed and performed very well during filling of the silo in 1973. The peak pore pressures were registered at the maximum load and subsequently decreased with time, partly because of consolidation of the clay subsoils, but mostly because the load decreased as the silage was used for feeding purposes. After the silo was first filled, the excess pore water pressures dissipated within 2 mo. In 1974, two of the piezometers ceased to function, and the zero reading had drifted in the other three. Consequently it was only possible to measure confidently the changes in the pore water pressures during rapid loading, and not the slow dissipation with time. The pattern was repeated in 1975, and in 1976 all piezometers had failed.

The peak excess pore pressures measured in 1973 are plotted in Fig. 7. Assuming symmetry about the center of the silo, the pore pressure contours formed a pore pressure bulb below the structure. The maximum excess pore pressure of 24 kPa was measured in the clay just below the inside edge of the ring foundation. This represented about 40% of the applied vertical stress at this location.

A summary of the peak excess pore pressures measured from 1973 to 1975 is given in Table III. The pore pressures were greatest in 1973 when the silo was first loaded and then became progressively less with each loading cycle. Since the shear strength of soil is directly affected by the excess pore pressure, the factor of safety against a bearing capacity failure is generally at a minimum when the silo is first filled to capacity.

SETTLEMENT

The soil had never been fully charged since it was erected in 1973. The heaviest mass was 795 t in 1973 and 1974 (Table II). The distribution of the increased vertical stresses from this load below the center of the silo floor, and below the ring foundation at pressure cells C-2, C-4 and C-6 (Fig. 3) determined after Egorov (1965) or Jumikis (1973), is plotted in Fig. 1. If this load were maintained continuously, the estimated total settlements based on the consolidation test results would be 230 mm below the center of the structure and 135 mm at the base of the tower. Since the silo is loaded for a short time each year, the settlement proceeds in steps in direct relation to the magnitude and duration of the loading periods.

Settlement observations for the period 1973 - 1978, plotted in Fig. 8, reflect the intensity and duration of loading, as expected. The maximum settlement at the base of the tower occurred at point 2, and reached 25 mm under the 795 t mass in 1973, then decreased as the silo was unloaded. It increased to 51 mm under the 795 t mass in 1974, rebounded to 45 mm as the silage was removed, then increased to 55 mm under the 708-t mass in 1975. In 1976 and 1977 the masses were considerably smaller (Table II) and no settlement occurred. In 1978 it increased to 60 mm under the 518-t mass, which was 45% of the predicted settlement under a mass of 795 t.

TABLE III. MAXIMUM EXCESS PORE WATER PRESSURES MEASURED DURING LOADING FROM 1973 TO 1976

Piezometer no.	Excess pore water pressure (kPa) 1973	1974	1975	1976
P-1	24	17	11	—
P-2	9	—	—	—
P-3	18	12	5	—
P-4	0	—	—	—
P-5	1	0	1	—

The minimum settlement at the base of the tower was observed at point 5. The settlement history was similar to that of point 2, but its magnitude was about 14 mm less, causing the tower to lean about 50 mm (O° 7′) from the vertical. The settlements at points 3 and 6 were between those of 2 and 5. The cause of the differential settlement may have been variations in soil properties under the foundation, non-uniform loading of the structure or inferior performance of the footing due to inadequate reinforcement.

The maximum and minimum settlements at the toe of the ring foundation occurred at points 7 and 10, respectively. The settlement record reflected the intensity and duration of loading, but the measured settlements were generally 15 - 20 mm less than those observed at the base of the tower. Extrapolating this differential settlement over the 2.51-m width of the footing showed that the "toe" was 32 mm higher than the "heel" in 1978. A concrete ring foundation with this large deformation or warp could not provide the expected uniform support for the loaded tower. Fortunately the small amount of steel reinforcing kept the concrete footing together so that it was able to support the structure in spite of the large deformations.

DISCUSSION

A new silo loaded for the first time "beds" into the supporting soil. If properly designed

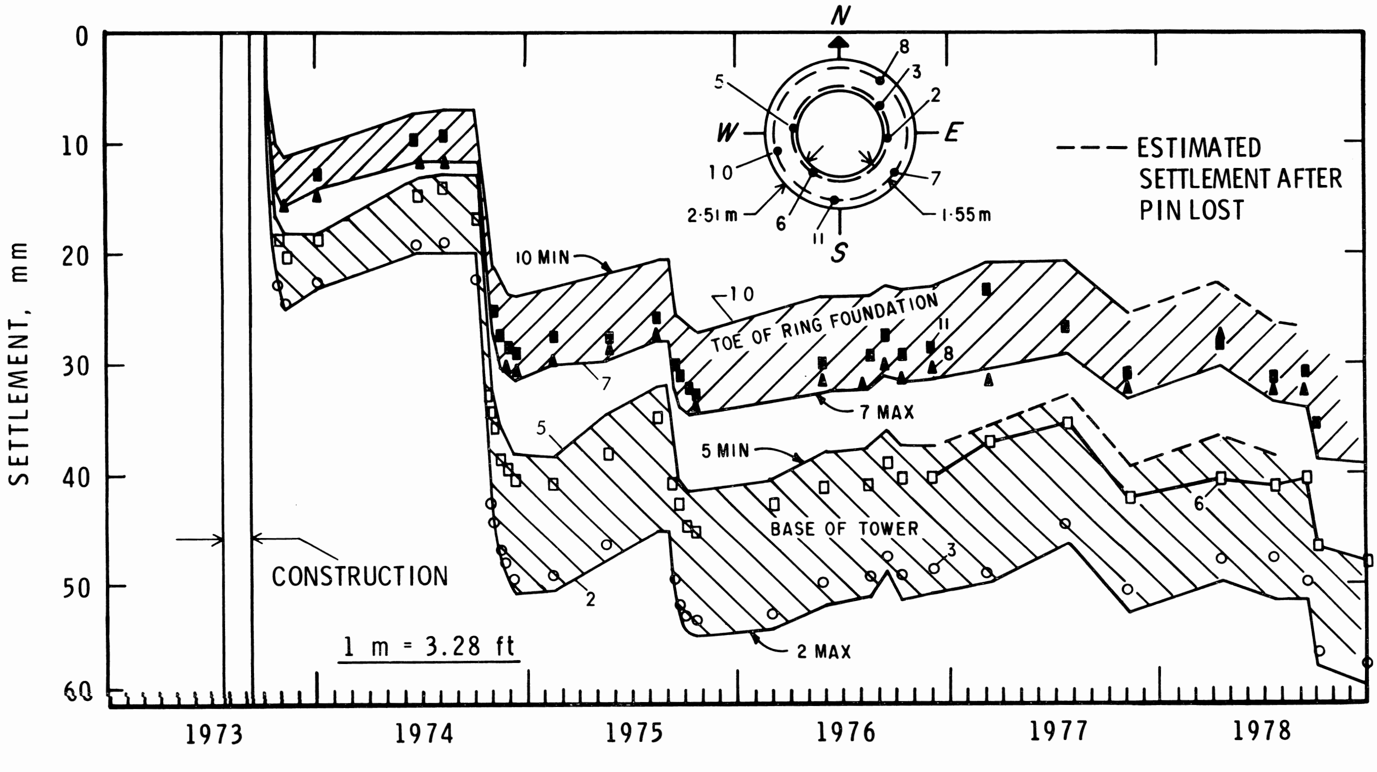

Figure 8. Settlements measured on foundations of Hammond silo.

and constructed some settlement will take place but the structure will perform satisfactorily. The factor of safety against a bearing capacity failure will be lowest at this time because the excess pore water pressures in the foundation soil will be highest. The stability will improve, however, with future loadings as the excess pore pressures become less and the foundation soils gain strength as they consolidate under the weight of the structure. Proper drainage of the silage juices is essential to prevent seepage into the foundation soil and reduction of the bearing capacity.

The "bedding in" also tends to equalize the pressures applied to the soils under the tower silo. It is possible, therefore, to base bearing capacity determinations and to predict settlements on the assumption that the total loads are applied uniformly over the soil enclosed by the outside diameter of the foundation.

Designing the foundation for the allowable bearing capacity of the soil with a suitable factor of safety is like buying insurance for the structure. It was evident from its deformation that the concrete foundation was insufficiently reinforced but the structure was adequately supported even though the total and differential settlements were large. Because of this lack of sufficient steel reinforcing, the owner should be very careful when attempting to fill the silo to its potential capacity of 900 t.

During filling with corn silage, the proportion of load transferred by friction through the walls of the silo to the footings increased with height of silage to 50 ± 3% of the maximum load. This proportion may vary if the interior of the silo is coated for protection against acid attack from the silage juices, which may change the coefficient of friction. It may also be different for other sizes and makes of silo.

Attempts were made to measure the tensile forces in the steel hoops confining the staves at the base of the silo, but the measuring points were damaged and no measurements were obtained. In future studies, the radial forces acting on the sides of the foundations and silo walls should be measured together with the stresses generated in the steel hoops. This information is vitally needed to improve the design of the structure.

Future work is also required to understand and predict the ultimate settlements due to the intermittent cyclic loading of the clay soils from the tower silos.

CONCLUSIONS

The 6-yr performance study of the concrete stave silo led to the following conclusions:

1. The proportion of load transferred by friction through the walls to the footings was 50 ± 3% when the silo was filled to 776 t which was 85% of the design capacity.
2. The size of the foundation, designed for the allowable bearing capacity of the soil using the average in situ vane strengths, was shown to be adequate for the structure.
3. The concrete ring foundation was not adequately reinforced with steel.
4. The large total and differential settlements, which caused the tower to lean 50 mm (0°7′) from the vertical, did not affect the performance of the structure. The inclination was well within the tolerance of 5.5 mm/m (0°19′) allowed by the Ontario Silo Association (1975).
5. To estimate the bearing capacity of the soil and the ultimate settlement of the structure, it can be assumed that the vertical pressure is uniformly applied to the clay soil under a filled silo supported on a ring foundation.
6. The Gloetzl earth pressure cells functioned very well during the life of the project.
7. The vibrating wire Geonor piezometers were satisfactory for monitoring short-term rapid changes in pore water pressures, but entirely unsuitable for long-term observations.

ACKNOWLEDGMENTS

The silo was constructed by Coopérative fédérée de Québec, under the direction of Mr. Lavoie of Bourget, Ontario. The cooperation of the owner of the silo, R. Leonard, and the assistance of the technical staff of the Geotechnical Section, Division of Building Research, who installed the instrumentation, carried out the surveys and performed the soil tests, are gratefully acknowledged. This paper is a contribution from the Division of Building Research, National Research Council of Canada, and is published with the approval of the Director of the Division.

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