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FOUNDATION FAILURE OF A SILO ON VARVED CLAY

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By
W. J. EDEN and M. BOZOZUK

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ANALYZED

FOUNDATION FAILURE OF A SILO ON VARVED CLAY

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THIS PAPER is a case record describing the bearing capacity failure of a farm silo near New Liskeard, Ont. The silo failed suddenly in July, 1961, the day following its first filling to capacity. The failure of this silo, which was founded on a considerable depth of normally consolidated varved clay, provided an opportunity to assess the applicability of bearing capacity theories to a highly stratified clay.

In September, 1961, a field investigation was carried out in which the undisturbed strength of the clay was measured with a field vane apparatus and samples taken with a thin-walled tube piston sampler for laboratory strength determinations. The results of the site investigation are presented and discussed with regard to the required bearing capacity.

The Structure

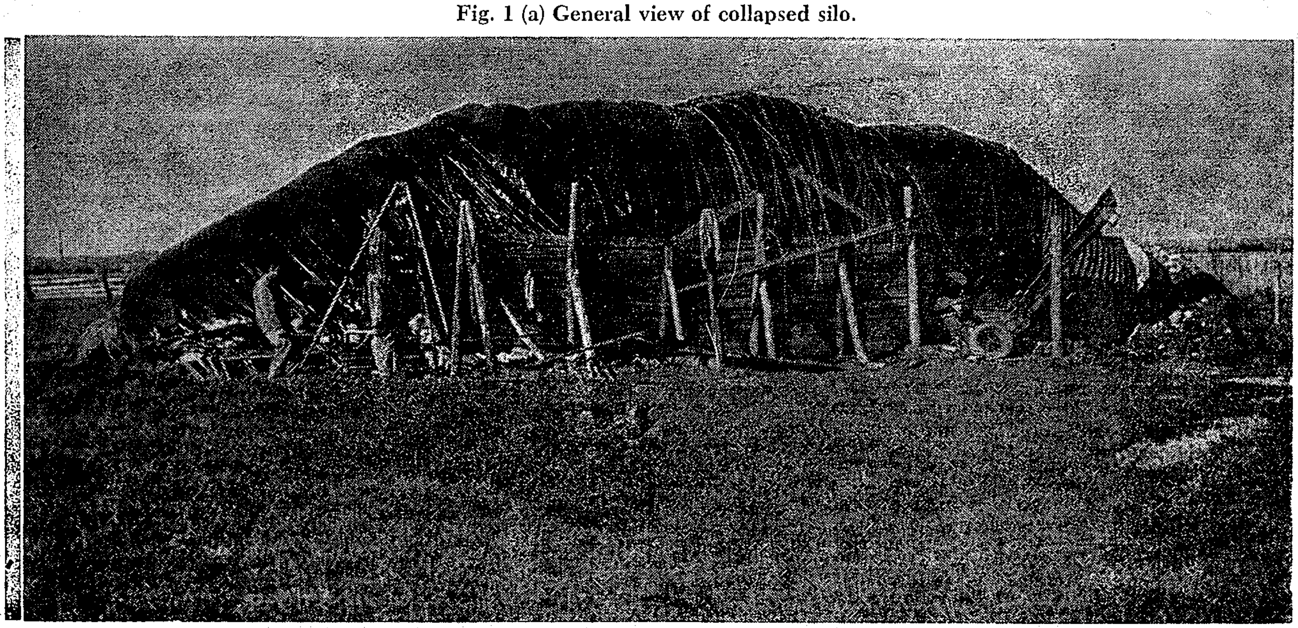
The cylindrical silo was constructed of precast concrete staves with a sheet aluminum dome roof. The staves were retained by steel tension hoops placed around the outside. The silo measured 20 ft. inside diameter by 50 ft. high and was founded on a concrete ring 22 ft. outside diameter by 18 ft. inside diameter. The ring extended from 1 ft. above grade to 4 ft. below grade. At grade level, four clay tiles passed through the concrete ring to drain off the excess silage juices. The foundation was a rough casting that provided good adhesion between the soil and the concrete. The soil retained by the concrete ring was left undisturbed.

The silo was located in the corner of a barnyard about 30 ft. from the corner of the barn. In preparation for paving the yard, about 1 ft. of soil had been removed from one side of the silo. This slight excavation extended from the barn to within 3 ft. of the silo.

The Failure

The silo was filled with grass silage from July 15 until July 23 at a variable rate due to poor harvest weather. It is believed that a significant portion of the full capacity load was applied on July 23. On the morning of July 24, one of the farm hands thought he noticed a slight tilt to the silo but not enough to be positive. By early afternoon the tilt was quite pronounced and it was decided to move the silage blower to safety from its

Fig. 1 (a) General view of collapsed silo.



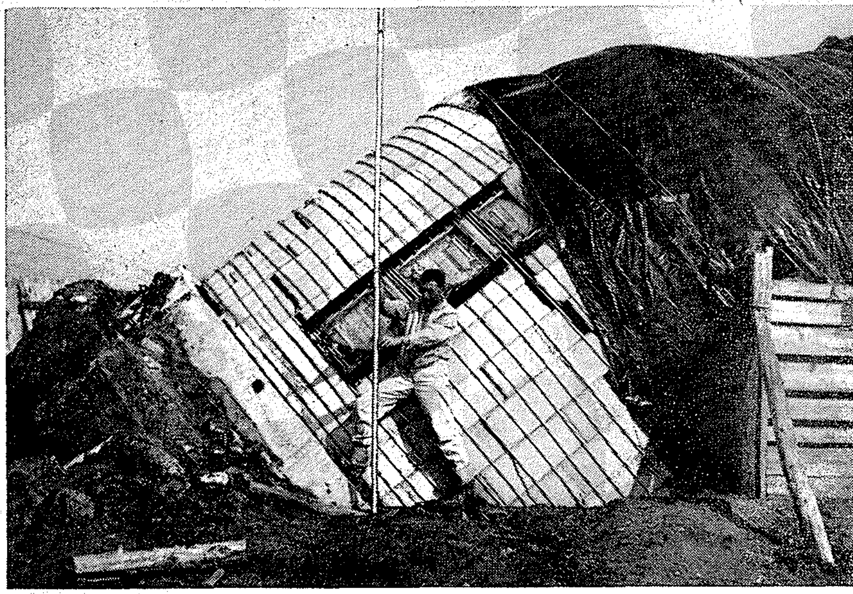


Fig. 1 (b) Attitude of the silo base after failure.

position beside the silo. Before the blower could be pulled clear however, failure of the silo occurred. As it failed, the silo structure broke. Fig. 1(a) shows the failed structure. To salvage as much of the silage as possible, tarpaulins were placed over the silo to protect the fodder. Fig. 1(b) shows the attitude of the foundation ring after failure.

Prior to the failure, it had been noted that the drain tiles did not function; the silage juices seeped under the foundation ring and bubbled up to the surface near the foundation.

Fig. 2 shows the final position of the foundation ring. The failure occurred in the direction of the shallow excavation in the barnyard.

Soil Conditions

The silo was located approximately four miles north of the town of New Liskeard which is situated at the north end of Lake Timiskaming in northeastern Ontario. This area is located in the "little clay belt",[1] a clay plain formed after the retreat of glacial lake Barlow. The chief deposits of the little clay belt are normally consolidated and slightly overconsolidated varved clays. The surface elevation at the silo is 724 ft., about 140 ft. above the level of Lake Timiskaming. It is believed that the silo was situated near the top of the little clay belt deposits.

Two borings were made about 12 ft. from the foundation outside the failure zone. In one, vane tests were conducted from the 4 ft. level to a depth of 46 ft. The apparatus used was the "Geonor" vane, 110 mm. x 55 mm. as described by Andersen and Bjerrum.[2] In an adjacent hole, thin-walled tube piston samples were taken from depths of 3 to 30 ft. for laboratory tests. An additional vane boring was made in an undisturbed area approximately 100 ft. from the silo.

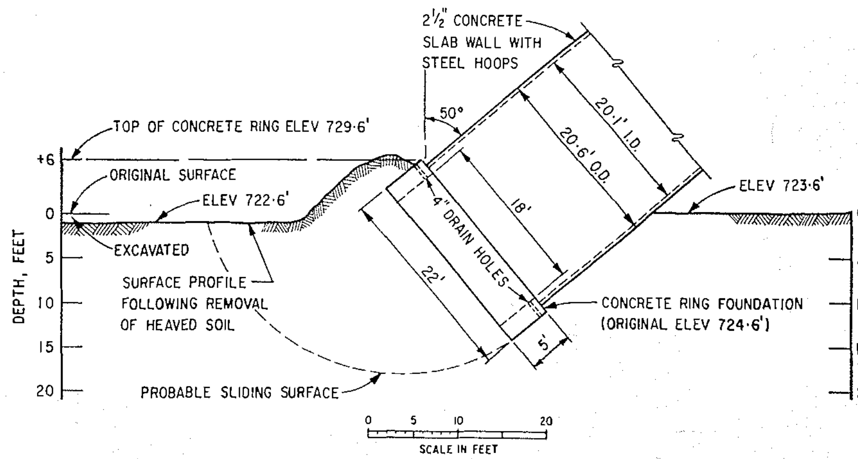
The results of the field and laboratory tests are presented in summary form in Fig. 3. The soil conditions consisted of normally consolidated varved clays to a depth of more than 46 ft., with a weathered crust from the surface to a depth of about 5 ft. Below 5 ft., to about 22 ft., the clay was very soft and sensitive. The soil strata consisted of very thin light layers between thicker dark layers. The dark layers averaged about ½ in. thick. Below 22 ft., the varves were much more distinct with layers of approximately equal thickness. Classification tests made on separated layers of the lower clay are presented in Fig. 3. The consolidation tests show the clay to be almost normally consolidated. The results of field vane tests are shown as solid lines in Fig. 3; the results of undrained laboratory tests are shown as points, each point representing a single test. The undrained triaxial tests were conducted at lateral pressures equal to the effective overburden stress. It can be seen that the laboratory strength tests are on the average lower than the field vane tests. The average of all the undrained laboratory strength results between 6 and 20 ft. gives a strength of 235 p.s.f. If the maximum value from each tube is averaged, a value of 295 p.s.f. is obtained. The average strength yielded by the field vane over the same ranges in depth in the two borings was 325 p.s.f.

Analysis of the Failure

To conduct the analysis of the failure, three factors must be determined: the physical dimensions of the structure, the load, and the shear strength of the soil. Only the physical dimensions can be determined accurately. It is necessary to estimate the load, and there is also some question whether vane and laboratory strength determinations on varved clay are reliable.

The load is made up of the weight of the silo structure, the weight of the contents, the weight of the foundation ring and the weight of soil retained in the ring. Using the dimensions of the silo, the weight of the structure was calculated to be 50 tons—a figure subsequently confirmed by the silo manufacturer. The foundation ring weighed approximately 47 tons and the soil retained in the ring at 100 pcf was about 50 tons. The weight of the contents is less certain. The farmer estimated it to be 400 tons based on the yield per acre of forage crop. The manufacturer lists a capac-

Fig. 2 Details of the silo base after failure.



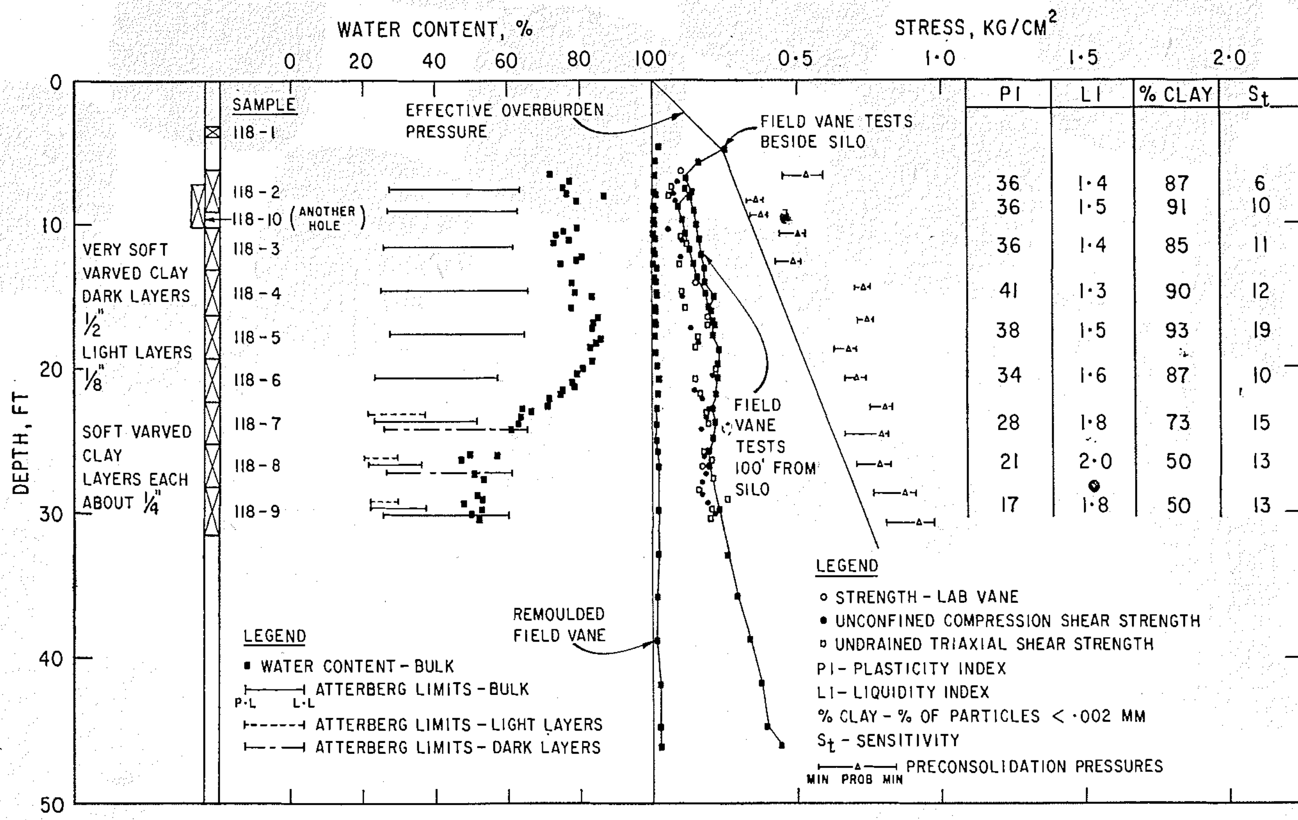


Fig. 3 Boring log and summary of test results.

ity of 450 tons of grass silage at 65% moisture content and 389 tons of corn silage at 69% moisture content. Gray[3] presents graphically the measured densities of corn silage and these measurements have been substantially confirmed in subsequent studies by Otis and Pomroy.[4] Using the density-depth relationships given by Gray[3] the weight of the silage was estimated to be 393 tons. Hence the total weight of the contents, structure, and soil retained in the foundation ring was estimated to be 550 ± 50 tons.

It has been the experience of the authors and has been confirmed by the test results, that field vane strengths are slightly higher than laboratory determinations. For bearing capacity computations the field vane strengths were used. Using Skempton's rule,[5] the shear strengths were averaged between the bottom of the foundation and a depth below the foundation equal to two thirds of its diameter, that is, 4 to 20 ft. Including the boring 100 ft. away, there were 26 strength measurements in this zone giving an arithmetic average strength of 325 p.s.f.

To apply the various bearing capacity formulae, it was decided to consider the foundation to be equivalent to a circular bearing area 4 ft. below the surface. The soil inside the foundation ring above 4 ft. was considered inert and contributed to the total load of 550 ± 50 tons.

Four methods of analysis were tried and are summarized in Table I. The factor of safety, F, listed is the ratio of the bearing capacity calculated from the shear strength to the estimated average bearing pressure. Table II lists the safety factors calculated using the average laboratory shear strength, and the average maximum laboratory shear strengths in one case.

The four methods of analysis were:

1) Skempton[5]
2) a modification to the above formula following the procedure of Skempton[6]
3) the formula suggested by Meyerhof[7]
4) the Fellenius circular arc method as described by Wilson.[8]

(1) Skempton's Method

For rapid loading on clay, Skempton[5] has proposed the following formula: $q_u = c.N_c + p$ where

- q_u is the ultimate bearing capacity,
- c is the average undrained strength of the clay,
- N_c is a factor depending on the shape of the foundation and its depth of embedment, and
- p is the overburden pressure at foundation level.

For this case, c was taken at 325 p.s.f. N_c for a 22 ft. diameter circular footing 4 ft. below the surface is 6.6. The overburden pressure, p, is equal to 400 p.s.fi q_u was calculated to be 2540 p.s.f. For the three load conditions assumed, 500, 550, and 600 tons, F, the safety factor, varied from 0.97 to 0.80.

(2) A Modification to Skempton's Method

Since the above formula does not take into account the adhesion between the soil above 4 ft. and the rough surface of the foundation ring, Skempton's formula was modified. In an earlier work, Skempton[6] recognized the adhesion and proposed that the bearing capacity be increased by a factor $(L/A).c'$, where L is the perimeter area of the footing in contact with the soil. A is the area of the base of the footing and c' is the adhesion between the footing and the foundation. Potyondy[9] has shown that the adhesion between soil and rough concrete is equal to the strength of the soil, in this case 500 p.s.f.

The modified formula becomes

$$q'_u = c.N_c + p + (L/A).c'$$

and gave a bearing capacity of 2900 p.s.f. and a range in F from 1.10 to 0.92.

(3) Meyerhof's Method

In the case of a buried circular foundation with a rough shaft bearing

TABLE I
Factor of Safety, for Four Methods of Analysis

Case	*Formula Used*	*F for Total Load Assumptions*		
		500 ton	*550 ton*	*600 ton*
1.	Skempton — $q_u = c \,.\, N_c + p$ (ref. 5) c = Average vane shear strength between 4 ft. and 20 ft. N_c = 6.6 p = 400 p.s.f. (Adhesion not considered)	0.97	0.88	0.80
2.	Modified Skempton (ref. 6) $q_u' = c \,.\, N_c + p + (L/A) \,.\, c'$ c and N_c as above L = perimeter area of footing A = area of footing c' = p.s.f. (Adhesion considered)	1.10	1.00	0.92
3.	Meyerhof (ref. 7) $q_r = c \,.\, N_{cqr} + K_s \gamma D$ N_{cqr} = 7.6 (rough concrete—full adhesion on sides) c = average vane shear strength between 4 ft. and 20 ft. K_s = 1 γ = 100 pcf D = 4 ft.	1.09	0.99	0.91
4.	Fellenius, as modified by Wilson (ref. 8) Circular arc through centre Adhesion included	0.94	0.88	0.78

on a purely cohesive soil, Meyerhof[7] proposes the formula:

$$q_r = c.N_{cqr} + K_s \gamma D$$

where

- q_r = ultimate bearing capacity.
- c = average shear strength = 325 p.s.f.
- N_{cqr} = a factor depending on shape of the foundation, the depth of the bearing surface and full adhesion between the soil and the shaft of the foundation = 7.6.
- K_s = coefficient of earth pressure between the soil and the shaft taken as 1.
- γ = density of the soil above the bearing surface = 100 pcf.
- D = depth of the foundation = 4 ft.

Hence,

q_r = 2870 p.s.f. and F varies between 1.09 and 0.91.

(4) The Fellenius Method

The Fellenius method is based on the premise that failure will take place on a circular arc. Thus, in the analysis, it is possible to take into account variations in the level at the surface of the clay. Wilson[8] has derived a method of locating the centre of the most critical surface for analysis by statics. Assumptions made were that only 3 ft. of soil were above the foundation level on one side, and that the soil above the foundation level had a shear strength of 500 p.s.f. The average shear strength below 4 ft. was taken as 325 p.s.f. To do the analysis, a slice one foot wide was taken through the centre of the silo. Because the failure surface has three dimensions, with a larger portion in the upper crust than in a two-dimensional slice through the centre, the Fellenius method can be expected to yield a safety factor somewhat low. Using this method, safety factors of 0.94, 0.88 and 0.78 were obtained.

Conclusions

The analysis of this failure has shown that there is reasonable agreement between the bearing capacity calculated from field vane strengths and the average bearing pressure applied by the structure. If the theoretical analysis is satisfactory the following implications may be stated as applying to soft normally consolidated varved clay with a range in properties similar to this site.

1. The field vane test will yield reliable undrained strength values for design purposes in medium to highly plastic varved clays. Since the maximum laboratory strength determinations are slightly lower than the vane strengths, the maximum, rather than average laboratory strengths, should be used in design.

2. Since Skempton's rule of using the average shear strength to a depth below the footing equal to two-thirds the width yielded reasonable agreement, the average vane strength, not the minimum, should be used for design.

3. It appears that the strength of the fissured crust and adhesion between the concrete and the soil deserve consideration in design. Hence, a bearing capacity formula such as Meyerhof's, which takes this into account, should be used for design.

This paper is a contribution from the Division of Building Research, National Research Council and is published with the approval of the Director of the Division.

TABLE II
Comparison of Test Methods as Applied to Meyerhof's Formula

Shear Strength p.s.f.	*Factor of Safety*		
	500-ton load	*550-ton load*	*600-ton load*
Average field vane strength = 325	1.09	0.99	0.91
Average laboratory strength = 235	0.83	0.75	0.69
Average of maximum strengths per sample = 295	1.00	0.91	0.84

Acknowledgements

"The authors wish to acknowledge the assistance of Mr. Arthur B. Campbell, Extension Specialist, Ontario Department of Agriculture, New Liskeard and Professor F. H. Theakston, Ontario Agricultural College, Guelph in this investigation."

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Discussion

FOUNDATION FAILURE OF A SILO ON VARVED CLAY

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The Engineering Journal, September, 1962, page 54

Discussion by Dr. G. G. Myerhof,
Head,
Department of Civil Engineering, Nova Scotia Technical College

The paper represents an important case record and is probably the first publication of a large-scale foundation failure on varved clay. In spite of the sensitive nature and relatively low strain at failure of this material, excellent agreement between the theoretical and observed failure loads has been obtained when an allowance for the adhesion between the foundation concrete and soil is made. Although the shearing strength of varved clays varies with the inclination of the failure plane in relation to the direction of the varves, the field vane test results give better agreement than the undrained compression tests which may be affected by some sample disturbance. Nevertheless, the maximum laboratory strength would also appear to be in reasonable agreement with the estimates, if an allowance for the variation of shearing strength with inclination between the varves and the probable sliding surface had been made.

Discussion by
D. L. Townsend, M.E.I.C.
Queen's University

As an instructor in soil mechanics, it is particularly gratifying to read of cases such as this where the theoretical calculations so closely agree with the actual results experienced in the field. In many cases the design factor of safety is so chosen that there is often little chance to verify theories with any precision and field measurement. The natural soil conditions are so varied that one wonders if the precise assumptions of the classroom are ever approximated in the field.

Such a record is of added personal interest in view of the varved clay which was encountered at the site, since there are few case records with this type of soil.

Investigations conducted at Queen's University by Gay[1] under this discussor's supervision have indicated that during shearing there is a definite movement of water from one layer in a varved soil to the other. For these tests, artificial varved samples were prepared from a slurry using two different soils which had the properties given in Table I. The strength tests were conducted at 0.4% strain/minute and the moisture contents measured after failure with the appropriate layer thickness are given in Table 2. From these results it can be seen that a definte moisture movement does take place between the layers.

Hughes[2] used small needles inserted into the individual layers of similar artificial varved soils to experimentally verify that the moisture movement was caused by a substantial difference in pore pressure between the layers. In addition, the mobilized angle of shearing resistance in terms of effective stresses, ϕ', varied with the amount of strain and the relative position within the layer. Typical results for one of the soils when tested at 0.4% strain/minute are given in Table 3.

Additional tests were run where 90% of the ultimate total deviator stress was applied within five minutes, and the needles were used to observe the dissipation or increase in the pore pressure in each layer and the time taken to reach equilibrium conditions. For an artificial sample with a drainage path of 1.5 inches, the time to reach equilibrium was 90 minutes, and a single test on a natural varved soil which had a 0.3 inch drainage path took up to 200 minutes to reach an equilibrium pore pressure condition.

In addition to these experimental results, Kenney[3] has suggested that the ratio between the coefficients of volume compressibility and expansibility of the layers in the varved soil may be critical for cases shortly after construction.

These preliminary results have been mentioned in some detail since they have not been presented before. Their purpose is not to divert attention from the value of the paper but to indicate that the rate of strain may be of more than considerable importance for the shear strength testing of varved soils.

A different rate of testing between two tests might allow more moisture to migrate from one layer to the other during the same interval. Hence the mobilized effective stresses could change, and this would be reflected in different total stress results between the field vane and the undrained triaxial result. Certainly some of the discrepancy will be due to sampling, (which may be manifested by distortion of the layers) but it is felt that the sampling should not produce a 40% discrepancy between the averages of the two test methods.

It would be appreciated if the authors of the paper could supply additional information on such items as the rates of strain used in both the laboratory and field test programmes. As an indirect measure of possible moisture movement, was the buldging of the triaxial samples confined to only one soil layer, or was there a uniform barreling throughout the sample?

Due to the thinness of the individual layers within the varves in the upper strata, it is unlikely that the properties of the individual soils in each layer have been determined, but it would also be interesting to know the bulk soil results for permeability, coefficients of consolidation and rebound, and compression indices for the soil.

TABLE 1
Soil Properties of Artificial Samples

	w_L	w_P	C_C	C_R	c_v cm²/sec	c_r cm²/sec	k cm/sec
Soil "L".........	36	18	0.22	0.02	2.0×10^{-3}	8.4×10^{-3}	4×10^{-8}
Soil "D".........	40	18	0.21	—	4.6×10^{-3}	—	$.7 \times 10^{-8}$

TABLE 2
Final Moisture Contents, Artificial Samples
at Cell Pressure 1.4 kg/cm²

	Assumed Initial m.c.	Layer Thickness (cms)			
		2.67	2.00	1.00	0.50
Soil "L"..................	21.0	21.4	21.9	22.7	23.9
Soil "D"..................	24.8	24.2	23.8	23.5	22.1

TABLE 3
Variation in ϕ'_{mo} with Position in Layer, Soil "L"

Distance from Interface	Percent Strain 1.5	2.0	4.2	5.8	6.7	15	Maximum Test Value
0.2 inch.........	19°	21°		26°		28°	33.5°
1.5 inch.........	15°		18°		22°	26°	33.5°

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Discussion by V. Milligan, M.E.I.C.,
Partner, H. Q. Golder & Associates Ltd.

As good things are invariably simple, so this paper, which describes the failure of a silo on soft varved clay, is simple and well presented. The data has been carefully assembled by the authors and various mechanisms of foundation shear failure examined.

If one assumes that the annular concrete foundation ring and the weight of silage resting on natural ground within the structure act in concert, and the fact that the silage juices which were observed to seep under the foundation ring and rise to the ground surface did not reduce clay adhesion, then the bearing capacity formulae proposed by Skempton[1] and Meyerhof[2] would adequately explain the failure. These are uncertainties neglected by the authors in their analyses which might be questioned.

The authors have further illustrated the value of in situ vane testing in this stratified deposit. Similar correlation of the vane shear strength with the undrained shear strength as measured in the laboratory has been noted in the study of an embankment failure on soft varved clay. This failure and some data relating to it are illustrated in Figures 1 to 3. It may be noted that normal methods of analysis for the failure using the undrained shear strength or $\phi = 0$ hypothesis, as one would apply for an earth structure on a homogeneous clay deposit, proved to be

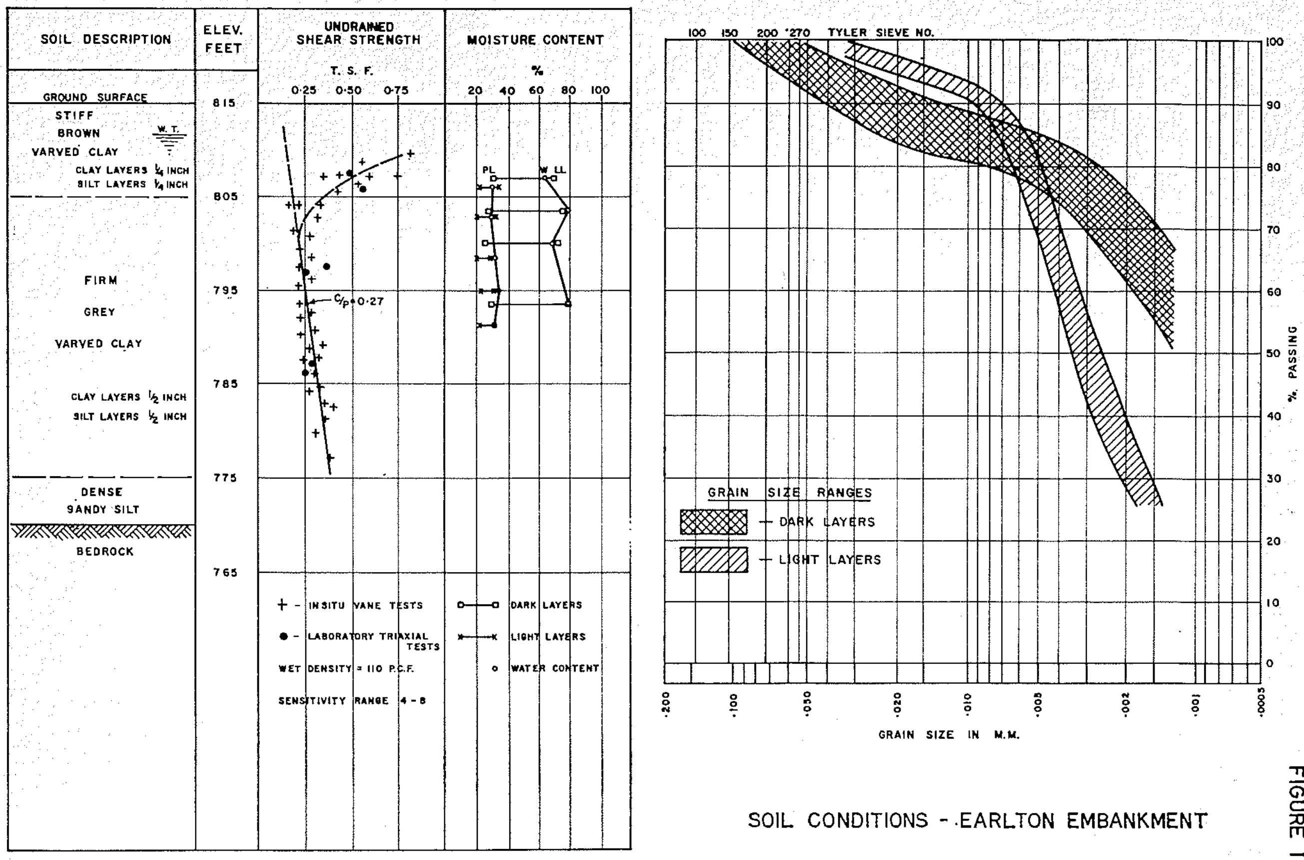

Figure 1.

sound for this stratified deposit. However, there is a danger in the extrapolation of this experience to suggest that the sole use of vane shear tests would be valid for all varved clays. Where the individual layers are thin, of comparable thickness and of similar degree of structure (generally liquidity indices for both layers close to 1) then experience has indicated that in situ vane shear tests give good agreement with undrained compression tests on thin wall tube samples. However, where the layers are thick and of dissimilar degree of structure, no agreement may be found. This point has yet to be studied.

It should also be pointed out that the liquidity indices given by the authors for the varved deposit, namely between 1.3 and 2.0, seem very high in comparison with available data[3] for similar stratified deposits in Ontario. It is possible that some of the tests as detailed in the paper, were carried out on bulk samples rather than on individual layers of the samples and thus do not give a true indication of the properties of the deposit.

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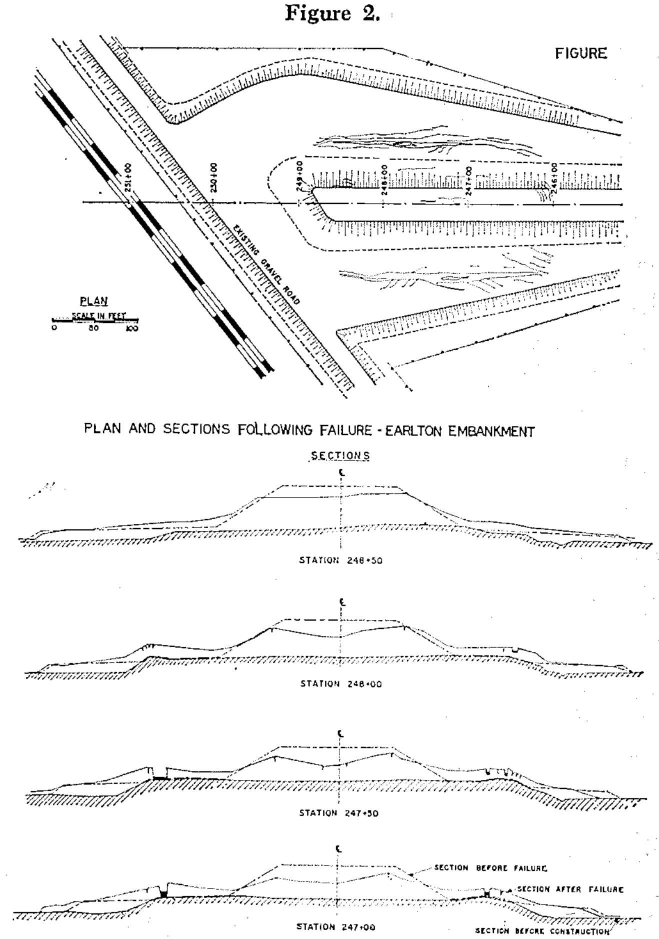
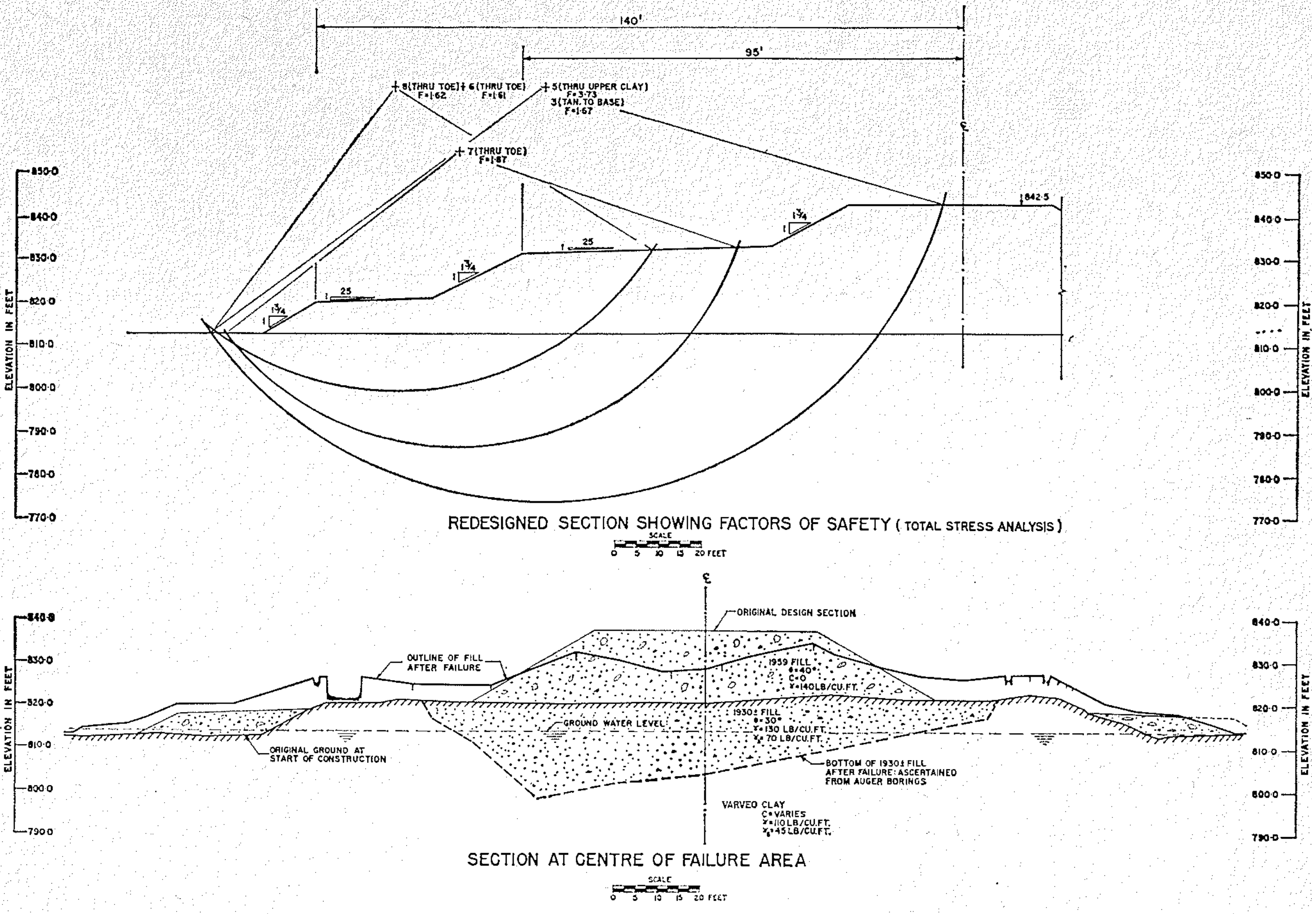
Author's Reply

The authors appreciate the comments of Dr. Meyerhof, Professor Townsend and Mr. Milligan concerning the behaviour of varved clay under shear stresses. Although the excellent agreement achieved in this case may have been fortuitous, the authors be-

Figure 2.





RE-DESIGNED SECTION - EARLTON EMBANKMENT

Figure 3.

lieve it demonstrates that the $\phi = 0$ analysis can be applied to the rapid loading of highly plastic varved clays where the bulk properties resemble those of more uniform clays.

In reply to the specific questions raised by Prof. Townsend, the rate of strain used in the laboratory undrained strength determinations was 1% per minute. The rate of strain used with the field vane tests cannot be definitely stated. Although the instrument head is rotated at 6° per minute during the application of stress, the long slender torque rods store energy which is released suddenly when the soil fails. Hence the field vane test is more nearly a constant rate of stress application than a constant rate of strain test.

In the undrained laboratory strength tests, the strain at failure varied considerably, presumably being related to the amount of disturbance which the sample had suffered. A number of samples failed at less than 1% while two specimens had failure strains of greater than 10%. The average failure strain for all the tests was 4.3%. Generally, the samples from the upper clay horizon failed on one or more well-defined shear planes without noticeable bulging. In the lower clay with the better defined varves, failure took place as Prof. Townsend suggests by bulging in the light layers which caused vertical cracks to develop in the more brittle dark layers due to the spreading action.

From the consolidation tests on bulk samples, the following results were obtained.

- Coefficient of permeability — approximately 1 x 10^{-5} cm/sec
- Coefficient of Consolidation — 0.003 to 0.006 cm^2/min
- Coefficient of Rebound — 0.02 to 0.06 cm^2/min
- Compression Index — 1.5 to 2.0, possibly greater than 2.0 for some tests.

It is believed that sample disturbance was a serious factor in the undrained laboratory strength test because of the high sensitivity of the clay. The sampling was conducted with a thin-walled fixed piston sampler with an area ratio of about 10%. The samplers were pushed in with a single thrust with care being taken to prevent over-driving. They were sealed on the site and transported by a light truck from New Liskeard to Ottawa. In the light of recent investigations in Sweden,[1] the transportation and subsequent storage probably contributed considerably to the sample disturbance and to the decreased strength of the laboratory test results.

Mr. Milligan raises the point of uncertainties in the application of bearing capacity theory to a case such as this silo. There are indeed many uncertainties in this case. For example, the foundation ring probably supported a very large portion of the load due to the tendency of the silage to adhere to the walls of the silo; there was quite likely some eccentricity in the loading and here there was considerabl variation in the clay since the vane tests varied from 500 p.s.f. to 170 p.s.f. to a depth of 20 ft. In spite of the many uncertainties, the bearing capacity theory has been reasonably successful as indicated by the studies of several case records by Skempton.[2] Mr. Milligan presents a case record of loading on varved clay where the $\phi = 0$ analysis was successful for an embankment load.

The point raised by Mr. Milligan concerning varved clays in which the layers had very dissimilar properties is well taken. It should be emphasized in this case that the failure was probably confined to the upper, more uniform clay and may not be indicative of the behaviour of the lower, more distinctly varved clay.

The liquidity indices given in Fig. 3 are for bulk properties. For the lower three samples it was possible to conduct tests on separate layers and the results of these are tabled below:

	Aver W.C.	*L.L.*	*P.L.*	*P.I.*	*L.I.*	*% Clay*
118 - 7 Bulk	65.2	51.1	22.8	28.3	1.8	73
118 - 7 Light layers	43.6	36.7	20.9	15.8	1.4	54
118 - 7 Dark layers	73.4	65.2	25.6	39.6	1.2	93
118 - 8 Bulk	51.6	35.8	20.6	15.2	2.0	50
118 - 8 Light layers	32.4	28.9	20.4	8.5	1.4	33
118 - 8 Dark layers	72.5	60.4	25.8	34.6	1.35	87
118 - 9 Bulk	50.3	37.3	20.5	16.8	1.8	50
118 - 9 Light layers	31.7	28.9	22.0	6.9	1.4	32
118 - 9 Dark layers	75.6	60.0	25.2	34.8	1.4	86

It can be seen that when tests are conducted on separated layers the liquidity indices of the separate layers are of the same order. The table further indicates that tests on bulk samples of varved clay can be somewhat misleading.

References

1. **Standard Piston Sampling.** A Report by the Swedish Committee on Piston Sampling. Proceedings, No. 19, Swedish Geotechnical Institute, Stockholm, 1961.
2. **The Bearing Capacity of Clays** by A. W. Skempton. Proceedings, Building Research Congress, London, 1951. pp. 180-189.