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CORRESPONDENCE

The Editors, *Géotechnique.*
Dear Sirs,

ϕ=0 ANALYSIS

TEST ON A LARGE SURFACE FOUNDATION.

In response to Mr. Nixon's valuable contribution to the Correspondence published in your last number, the following account of a test on another oil tank foundation is presented by permission of the Anglo-Iranian Oil Co., Ltd.

The site is that described in Professor A. W. Skempton's paper on " Vane Tests in the Alluvial Plain of the River Forth near Grangemouth " (*Géotechnique*, December, 1948). The soil characteristics at the site of the test here described were as shown on Fig. 1. The stony layer at depth 30 ft. mentioned by Prof. Skempton was hardly noticeable at this part of the site.

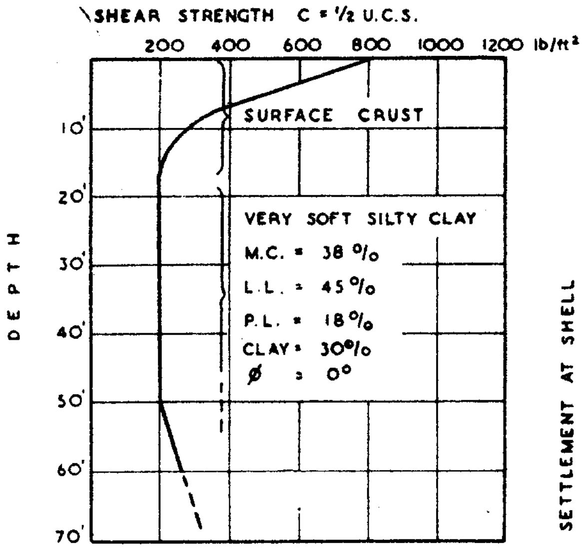
The tank, erected in 1948, was made of welded steel plate, 116 ft. diameter and 30 ft. high. It was built on a slightly cambered mound of spent oil shale about 2 ft. high.

The tank was tested by filling it with fresh water at the rate of 1 ft. per day for 30 days. Each day, before the day's increment of water was added, levels were taken on the shell and on the ground nearby. The tank then stood full for 42 days, measurements of settlement being continued. After emptying, the contour of the bottom was determined for comparison with its original shape. The results of the test are shown in Figures 2 and 3. Slight ground heave occurred amounting to $2\frac{1}{4}$ ins. at a distance of 20 ft. from the shell opposite the point of greatest settlement. An adjacent tank was filled to the top in a few days without ill effect. Other tanks nearby have been used for many years for the storage of oil products with a specific gravity of up to unity to a height of 30 ft.

In the case of this test the quotient :—

$$\frac{\text{Ultimate Bearing Capacity}}{\text{Shear Strength of Soil}} \quad \frac{2072}{200} = 10{\cdot}4$$

The discrepancy between this figure and that of 7·4 given by Mr. Nixon may be explained thus :—Mr. Nixon in giving a figure of 1640 lb/sq. ft. for the ultimate bearing capacity has



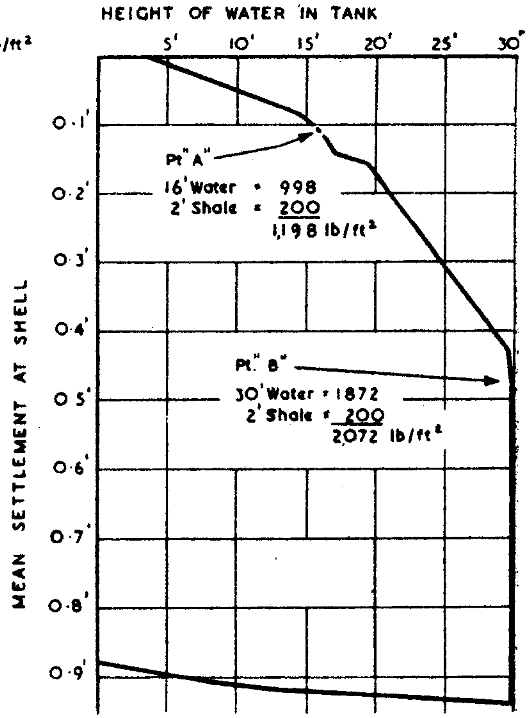
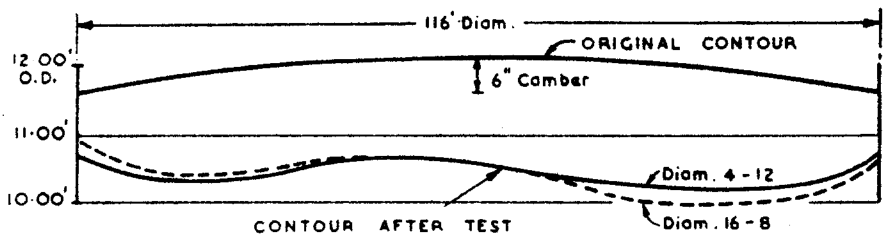


FIG. 1 (*above*). GRANGEMOUTH SOIL CHARACTERISTICS IN VICINITY OF TANK

FIG. 2 (*right*). LOAD SETTLEMENT CURVE

FIG. 3 (*below*). DEFORMATION OF TANK BASE



allowed for a spreading effect from the chalk mat ; this is perhaps hardly justified as the chalk mat is not rigid. Taking the full pressure of 30 ft. of water and 3 ft. 6 in. chalk, we have :—

$$\frac{\text{Ultimate Bearing Capacity}}{\text{Shear Strength of Soil}} = \frac{1872+350}{210} = 10{\cdot}5$$

An analysis of the Grangemouth results is suggested below, the following factors being taken into account :—

(a) Unavoidable disturbance of samples will reduce the apparent shear strength of the soil by (say) 10 per cent.

(b) The effective shear strength in the ground for a shallow slip in normally consolidated clay is greater than the unconfined test strength by, say, 10 per cent (see Hansen & Gibson, *Géotechnique,* June, 1949).

(c) The softness of the ground, the shape of the Load-Settlement curve (Fig. 2) and the very slight ground heave all suggest " local " rather than " general " shear failure.

If we take the effective shear strength in the ground as 200+10 per cent + 10 per cent = 242 lb./sq. ft. then, according to Terzaghi and Peck (p. 171), shear failure in these very soft soils can be expected to become apparent when the nett loading equals :—

$$7{\cdot}4\times\frac{2}{3}C=7{\cdot}4\times\frac{2}{3}\times 242=1{,}192 \text{ lb./sq. ft.}$$

This corresponds remarkably closely with the loading at which the Grangemouth tank commenced to settle rapidly (see Fig. 2, point "A"). It would be interesting to know whether a similar correspondence was observed in the case of Mr. Nixon's tank.

The distortion of the tank bottom shown in Fig. 3 suggests that final failure was about to occur through a progressive slip under the edge of the tank. This distortion, incidentally, commonly occurs in heavily loaded oil tanks. Near the edge of the tank the slip failure will no doubt be restrained by the strength of the soil crust, but this is unlikely to have much effect on general failure. It is of interest to note that Mr. Nixon's tank on a true clay soil did overturn when loaded with 30 ft. of water, whereas the Grangemouth tanks on a silty soil remained stable. Perhaps the explanation is that the latter soil, *in situ*, has some frictional property which is not apparent in triaxial tests.

I join with Mr. Nixon in inviting further contributions from your readers which may throw light on the behaviour of surface foundations in relation to soil properties.

B. F. SAURIN.

26th September, 1949.

The Editors, *Géotechnique.*
Dear Sirs,

ϕ=0 Analysis

Tests of Full-Scale Storage Tanks on Soft Clay Soil

Mr. Saurin's letter of the 26th September, 1949, with its description of a large-scale tank test is particularly welcome as it forms a second example of the type of test described in my previous letter dated 27th November, 1948. Technically it is all the more interesting and thought provoking in that Mr. Saurin suggests an alternative explanation of the two failures.

Beginning with the consideration of the *true* strength of the soil, Mr. Saurin contests the results of the unconfined compression test. Nevertheless, Professor Skempton* has shown that the *in situ* value of the shear strength using the vane test confirms the unconfined compression tests on soil samples down to a depth of 50 ft. at Grangemouth. Since writing my original letter I have carried out similar tests close to the Shellhaven tank; the comparison is given in Fig. 1.

In view of this similarity between the results of the two methods of testing the cohesion of the soil, I feel that at this stage there is little justification for "adjusting" the shear strength.

Secondly, there is the question of the spreading of the load with increasing depth. I would suggest that this is possibly the assumption about which least is known and the one therefore that should receive prior consideration.

By assuming there is no spread of the load through the mat of either tank, Mr. Saurin arrives at almost identical figures for the ratio.

*"Vane Tests in the Alluvial Plain of the River Forth near Grangemouth," by A. W. Skempton. *Géotechnique*, Vol. 1, No. 2, page 111.

ULTIMATE BEARING CAPACITY
SHEAR STRENGTH OF SOIL

But there is one fundamental difference in the two sites not included in this ratio. Whereas at Shellhaven the soft clay commences 5 ft. below the surface, at Grangemouth it is 20 ft. In both cases, above these depths there is a relatively strong surface crust of soil strong enough to bear the load of the tanks easily. This fact I suggest rules out any usefulness of this ratio that is implied by the favourable comparison between the two sites.

Figs. 2 and 3 illustrate the effect of "no spread" and a "22½° spread" through the relatively strong material of the surface crust for the failing load of both tanks plotted against the Ultimate Bearing Capacity (based on 7·4c.). It is seen that in both cases if "no spread" applies, failure is predicted for a much lower loading. Whereas the curve of a "22½° spread" just exceeds the Ultimate Bearing Capacity in the zone of the soft clay in both cases. The "22½° spread" is based on the approximate spread as given by the Boussinesq theory and was first suggested as a simplifying assumption by Glossop and Golder in Road Paper No. 15, Inst.C.E.

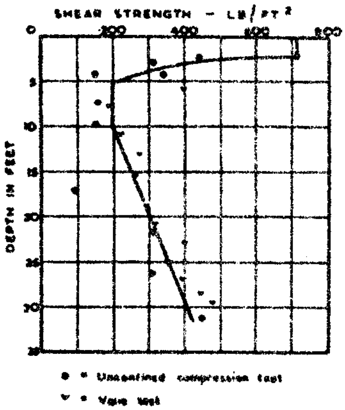


FIG. 1. SHELLHAVEN SOIL CHARACTERISTICS IN VICINITY OF B31 TANK

There is an alternative explanation for the difference between the results of the two tank failures. The relationship between the results of the Unconfined Compression test and the Vane test is different for each site. This suggests that we may not transfer the experience of one site to another, for example, in the form of a correction factor applied to the result of the unconfined compression test. It is noteworthy that the materials at these two sites are very different one from another (Grangemouth liquid limit 40 per cent, Shellhaven liquid limit 87 per cent). From the geology of the areas this would be expected. The Grangemouth soil is a silt formed by glacial action on crystalline rocks, whereas the soil at Shellhaven although also deposited in post-glacial times is derived from the sedimentary rocks of the Thames Valley. The Grangemouth silt would be expected to be much more sensitive to disturbance due to sampling and consequently the ratio

$$\frac{\text{Vane Strength}}{\text{Unconfined Compression Strength}}$$

would be expected to be higher than would be the case at Shellhaven.

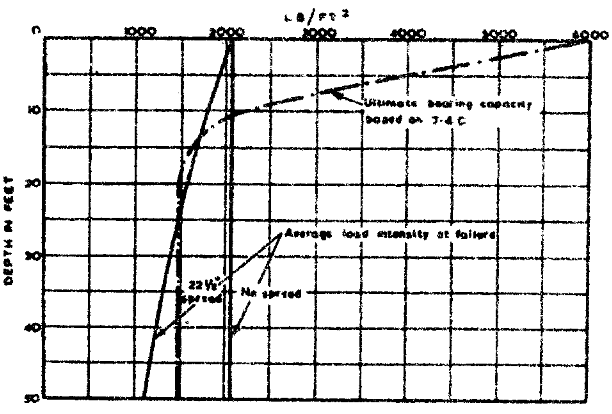


FIG. 2. GRANGEMOUTH TANK

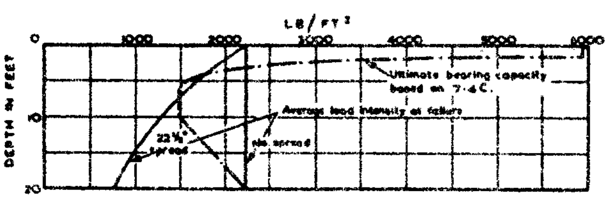


FIG. 3. SHELLHAVEN TANK

Finally, before closing I should like to make one or two points concerning the mechanics of each failure. I suggest that the Shellhaven tank overturned because the unusual height to base ratio of 3 : 2 when combined with the differential settlement, due to variation in the strength of the soil, produced a movement of the centre of gravity of the tank, which would increase the load on that area in which the soil was weakest.

Mr. Saurin suggests that the Grangemouth tank possibly failed only locally. It could be more easily judged whether or not this was the case if time *v.* settlement curves for each load increment could be studied; this would be especially interesting for the 42 days when the tank was full. Again, these settlement curves might help to explain the phenomenon which occurred when the loading equalled 16 ft. of water. I regret that no additional settlement records exist concerning the Shellhaven tank other than those already stated in my previous letter.

Mr. Saurin's diagram showing the contours of the base of the tank after the test is most interesting. It certainly suggests local rather than general failure, and this would agree with the calculations shown graphically in Fig. 2, giving 15 to 25 ft. as the depth at which the pressure exceeds the Ultimate Bearing Capacity. General failure would be indicated by a much deeper zone.

Yours faithfully,
I. K. NIXON.

1*st November*, 1949.
39, Walton Drive,
Harrow, Middlesex.

OBITUARY

The death of Professor Dr. Thord Johannes Brenner, on the 13th of March, 1949, was a severe loss to Finland and to all those who are interested in Geotechnology. Born at Helsingfors in 1892, Brenner was educated as a geologist and later worked under Professor W. Ramsey and Professor J. J. Sederholm, in Kola (1911), Hogland (1912), Oesterbotten (1913, 1914), Kola (1915), Altei and Northern Mongolia (1916, 1917), Swedish and Finnish Lapland (1919, 1920).

In 1921 the Finnish railways called him in to study methods for the prevention of landslips and settlements which had destroyed parts of their lines.

Brenner tackled these problems with energy and originality. He organized and guided the geotechnical bureau of the railways (which was destroyed during the second world war), introduced modern ideas in Soil Mechanics and applied them to landslides, power stations, water supply, harbours, runways for airports, bridge and building foundations.

In 1946 Brenner became lecturer in Geology at the University of Helsingfors and in 1948 he was elected Professor for the newly-created chair of Foundations and Soil Mechanics at the Technical College at Helsingfors, which post he held for the last few months before his death.

Among his papers on Geology and Geotechnology the most important ones are :—"Die physikalischen Eigenschaften der mineralischen Böden. Fennia 54/5 1931" and "On the Strength Properties of Mineral Soils. Bull. Comm. Geol. de Finlande Nr. 139 1946" in which he discussed among others the problem of the bound water in the soil.

Besides his open and winning personality which had gained him many friends, especially at the Scandinavian Congresses and at the Congresses in Zürich (1938) and Rotterdam (1948), Brenner's importance lay in the fact that he had started as a Geologist and Scientist and in his later years devoted himself to Geotechnology. With this background he achieved much, and extended the interest in and understanding of Geotechnology to many new fields.

A. VON MOOS, ***Zürich.***