Foundation Failure of an Oil Tank in Fredrikstad, Norway

Rupture des Fondations d'un Réservoir à Fredrikstad, en Norvège

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Summary

In 1952 an oil tank in south-eastern Norway tilted during the test filling, due to a foundation failure.

Based on a recent investigation of the soft clay below the foundation, comparison has been made between the calculated and observed failure loads. Skempton's bearing capacity theory results in the safety factor 1·70. If it is, however, taken into consideration that the clay is softer below the eastern part of the tank than below the western part, the danger of a local failure can be computed, resulting in the safety factor 1·05.

Sommaire

En 1952 une citerne à huile dans le sud-est de la Norvège se renversa pendant le remplissage d'essai par suite d'une rupture de la fondation.

Sur la base de recherches récentes sur l'argile silteuse au-dessous des fondations, l'auteur fait une comparaison des charges de rupture calculées et observées. La théorie de la capacité portante d'après Skempton fait ressortir un facteur de securité de 1·70. Toutefois, si l'on tient compte du fait que la résistance au cisaillement de l'argile est inférieure de la partie Est de la citerne à celle sous la partie Ouest, on peut montrer que le facteur de securité n'est que de 1·05 par rapport à une rupture partielle.

Introduction

The number of failures of buildings on soft clay which have been analysed for comparison of theoretical and observed bearing capacities is still limited to 6 or 7 cases. In connection with the investigation of a site in Fredrikstad, the Norwegian Geotechnical Institute was informed about a failure of an oil tank, which took place in 1952. In order to obtain the necessary information about the soil conditions, borings were made in 1956. The present paper gives a description of the available information, and a comparison is made between observed and theoretical failure loads.

Failure of the Tank

The refinery of De Nordiske Fabriker A/S in south-eastern Norway stores whale oil in a tank park located on a level area with soft marine, silty clay.

Since the first tank was constructed in this area, a special technique has been developed for constructing large tanks on soft clay. By this technique the plates are welded from a floating platform inside the tank, and as the height of the tank increases the platform is continuously raised by pumping water into the tank. In this way a certain increase in the strength of the clay is successfully obtained due to consolidation during the construction period, and in spite of large settlements, 50 to 100 cm, no failure has occurred among the great number of tanks already constructed.

In the spring of 1952 a sudden increase in the delivery of whale oil necessitated a rapid construction of a new tank, and the tank in question was erected without simultaneously filling with water.

The diameter of the tank was 25 m and its capacity was 6000 m^3. It was founded on a 35 cm thick gravel layer covered with a 15 cm thick concrete slab connected to a ring-formed outer wall. The weight of the empty tank was 550 tons. When the construction of the tank was completed, a test loading with water was made in March 1952; 5000 m^3 of water was pumped into the tank in the course of 35 hours. During and after the test filling the tank was kept under observation, and a couple of hours after the filling was finished a failure occurred. As the tank started to tilt, a heave at the area near the tank was observed, see Fig. 1. The differential settlement of the tank after unloading was approximately 50 cm and the maximum heave was about 40 cm.

The total weight of the tank at the moment of failure was 5500 tons, representing a net load of 11·2 tons per m^2.

After the failure, the tank was stepwise filled with water, and after each step a consolidation was awaited. Since 1954, i.e. two years after the construction, the tank has been used with full capacity. The settlement of the tank is today 50 to 100 cm.

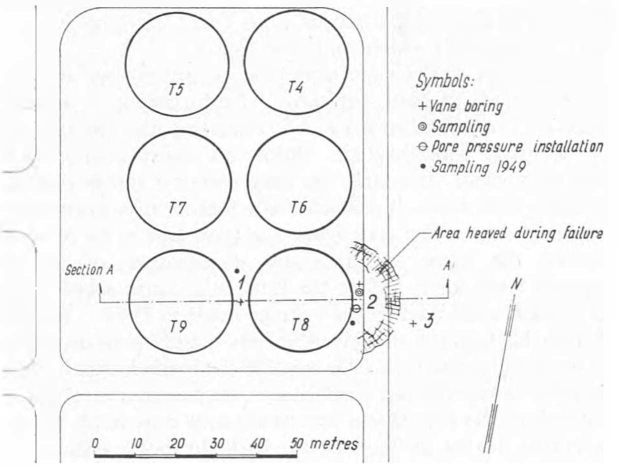

Fig. 1. Plan of the site
Plan de l'emplacement

Soil Conditions

In 1949, three years before the tank was constructed, some borings were made on the site by a private firm. This investigation also included a determination of the undrained shear strength by cone tests on undisturbed samples.

In 1955, about three and a half years after the construction of the tank, comprehensive field investigations were carried out by the Norwegian Geotechnical Institute. The investigations included three vane borings, a boring with undisturbed sampling and measurements of pore pressures in different depths.

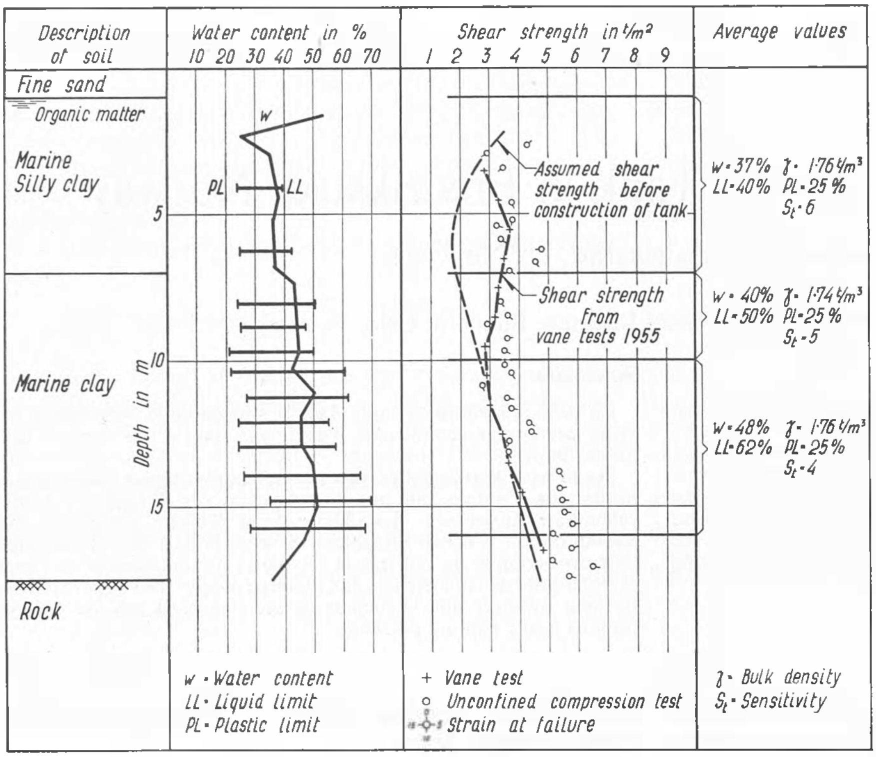

Fig. 2 Section A–A with shear strengths as assumed before construction of tank

Section A–A avec résistances au cisaillement présumées avant la construction de la citerne

A soil profile with the results of the laboratory investigations of the undisturbed samples is shown in Fig. 2. As seen from the boring, the soil consists of an upper layer of a silty clay (in the depths 1 to 7 m below the surface) and a lower layer of a soft marine clay (in the depths from 7 to 17·5 m below the surface), underneath which rock was found.

The results of the vane tests (Fig. 3) are plotted in a cross-section (A–A) through the tank. From boring 3, which was made on the unloaded area, it is observed that the clay layers are normally consolidated. Below an upper weathered crust with high shear strengths, the clay shows a linear increase in strength with depth, a characteristic feature of a normally consolidated clay. The clay below the tank had in all probability showed the same characteristic distribution of the shear strength with depth before the tank was constructed. This is very much confirmed by the borings made in 1949. The results of these borings are also given in Fig. 3, and based on these and on the borings made in 1956 outside the loaded area it has been possible to reconstruct a relatively reliable picture of the shear strength of the clay layers before the tank was built, illustrated by curves drawn through points with the same assumed shear strength.

The curves in Fig. 3 indicate that there was a certain regular variation in the original shear strengths below the tank. The clay below the western part of the tank thus showed considerably higher values of the shear strength than the clay below the eastern part of the tank. The minimum values of the shear strengths at two opposite points are thus assumed to have been respectively 2·7 and 1·7 tons per m². Similar variations in the shear strength have been observed in neighbouring profiles, and it is frequently observed in Norway that at sites where the rock surface is inclined the softest clay is found where the depth to rock is smallest.

The borings 1 and 2, which were made in 1955, show that since the construction of the tank an essential increase in shear strength has taken place in the clay layers under the tank. Owing to the consolidation under the weight of the tank, the upper clay layers have increased their shear strength by up to 2 tons per m². The increase in strength is, however, limited to the clay layers above a depth of 12 to 14 m.

Comparison of Observed and Calculated Failure Loads

A general, semi-empirical solution for calculating the bearing capacity of footings on saturated clay has been proposed by A. W. Skempton (1951) (The bearing capacity of clays *Building Research Congress Papers*, Div. 1, pp. 180–189):

$$q_f = N_c c_u + \gamma D$$

$$N_c = 5{\cdot}0\left(1 + 0{\cdot}2\frac{B}{L}\right)\left(1 + 0{\cdot}2\frac{D}{B}\right)$$

in which q_f = failure load; c_u = undrained shear strength of clay; γ = bulk density of clay; N_c = bearing capacity factor; D = depth of footing; B = width of footing; and L = length of footing. According to Skempton, an average value of the shear strength in a depth of $2/3B$ below the foundation level should be used in the equation provided that the variation is smaller than $\pm$ 50 per cent.

In the present case, the average value of the undrained shear strength along an assumed sliding surface reaching depth $2/3B$ below the tank is 3·1 tons per m² and the variation is approximately $\pm$50 per cent of this value. For a circular tank N_c is 6·2. Using Skempton's formula for calculating the theoretical safety factor at the moment of failure with a net load of 11·2 tons per m², it is found that

$$F = \frac{6{\cdot}2.3{\cdot}1}{11{\cdot}2} = 1{\cdot}72$$

There is, thus, a serious disagreement between the theoretical and the observed failure loads. This disagreement may be explained from a study of the above mentioned variations in shear strength below the tank and the way in which the failure took place.

As mentioned above, the clay below the eastern part of the tank is essentially softer than below the western part of the

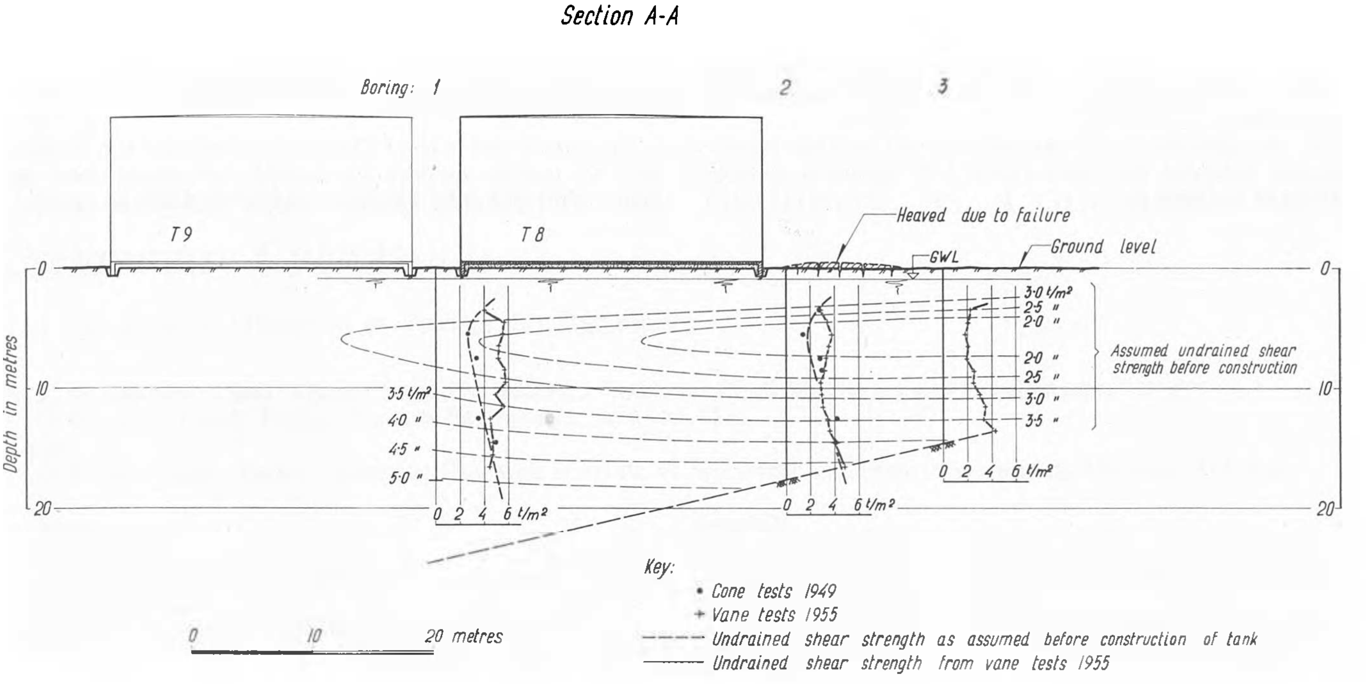

Fig. 3 Results of a boring and vane test, boring No. 2
Résultats d'un essai de forage et au moulinet, sondage No. 2

tank, see Fig. 3. There is, thus, reason to believe that the clay below the western part of the tank shows a sufficiently high shear strength to carry the load without a danger of failure. In contrast, the soft clay layer immediately below the eastern part of the tank shows such low strength values that it can hardly be loaded with 11·2 tons per m² without risking a local failure. As a working hypothesis, it may thus be assumed that the tilting of the tank under the test filling is not due to a complete failure of the whole tank but is a result of a local edge failure in the soft clay below the eastern part of the tank.

This hypothesis is strongly supported by the observations made during the test filling. The tank tilted in the eastern direction where the shear strength of the clay is low, whereas the settlements of the western part of the tank were relatively

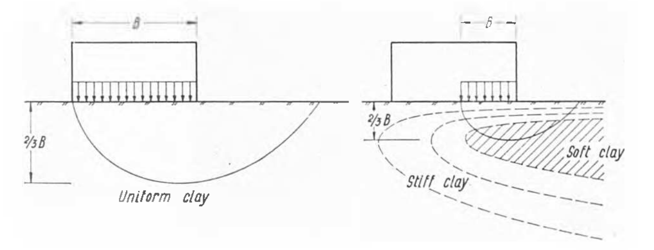

Fig. 4 Schematic illustration of total failure and local edge failure
Illustration schématique de la rupture totale et de la rupture locale à l'arète

smaller. Moreover, the heave, which was observed at the surface, was limited to a rather narrow zone with a width approximately equal to half the diameter of the tank. This observation points to the fact that the failure occurred in a limited depth below the eastern part of the tank.

Skempton's bearing capacity formula may easily be used also for calculating the danger of a local failure of the tank. In Fig. 4 is illustrated what is meant by a local edge failure of a footing, and it is indicated how the bearing capacity formula can be applied to a limited part of the tank. The N_c value, which has to be used for such a calculation, will—depending on B—be a value between 6·2, which is valid for a circular footing, and 5·2, which is valid for a narrow ring-strip.

An analysis of a local failure of the considered tank has been made using different values of B, the width of the part of the tank which is supposed to have failed. For each value of B is calculated the average shear strength along a sliding surface in a depth of $2/3B$, using the shear strength distribution in Fig. 3. The safety factors found for the different values of B are given in the table.

Table

Safety factors for local failure
Facteurs de securité pour ruptures locales

B m	N_c	c_u tons per m²	F	
$5·0 = 1/5 B_o$	5·4	2·9	1·40	
$6·3 = 1/4\ B_o$	5·45	2·3	1·12	
$8·3 = 1/3\ B_o$	5·55	2·2	1·09	
$10·0 = 1/2·5\ B_o$	5·6	2·1	1·05	Minimum value for local failure
$12·5 = 1/2\ B_o$	5·7	2·2	1·12	
$25·0 = B_o$	6·2	3·1	1·72	Total failure

The results in the table show that a safety factor close to unity is found for a local failure of a part of the tank with a width of 10 m. The position of the sliding surface for a local failure of an area with this width agrees approximately with the width of the area on which a heave was observed when the tank tilted over.

It is, thus, very probable that the tilting of the tank is due to a local failure in the soft clay below the eastern part of the tank. As illustrated above, the failure could have been predicted by the conventional bearing capacity formula only considering the stability of a part of the foundation.

Conclusion

An analysis of the tilting of an oil tank has demonstrated the necessity of considering the danger of a local failure in the design of footings on clay with a non-uniform shear strength distribution.

In the described case, the conventional bearing capacity theories indicated the safety factor 1·72 against a complete failure of the tank.

Agreement between theory and observation is, however, found by considering the danger of a partial failure in a local soft clay layer. A calculation of the safety factors against a local failure gives, thus, the minimum value 1·05 if only a width of $2/5B$ is considered.

The authors wish to thank De Nordiske Fabriker A/S, Fredrikstad, for making available the valuable information about the failure of the tank and for assistance during the field investigations.