The Improvement of a Tank Foundation by the Weight of Its Own Test Load

Amélioration de la fondation d'un réservoir par le poids de sa propre charge d'essai

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SUMMARY

This paper describes the design and testing of a tank foundation on soft hydraulic fill which was initially too weak to support the full tank. Use was made of the increase in strength of the fill caused by the weight of the tank itself, and calculations were made of the maximum permissible pore pressures which could be allowed consistent with stability of the tank. Pore-pressure and settlement observations were made to control the test loading, and the tank was successfully filled, without any damaging differential settlements, in a much shorter time than had been predicted.

SOMMAIRE

Cet article donne une description de la réalisation et de la mise à l'essai des fondations d'un réservoir sur remblai mou pompé qui, à l'origine, étaient trop faibles pour supporter le poids du réservoir à l'état plein. On a utilisé l'augmentation de la résistance du remblai causée par le poids du réservoir même, et on a calculé les pressions interstitielles permises maximum compatibles avec la stabilité du réservoir. On a mesuré les pressions interstitielles et le tassement pour contrôler l'essai de chargement et on a réussi à remplir le réservoir sans causer de tassements dangereux, et cela dans un délai beaucoup plus court qu'on ne l'avait prédit.

AN AREA at the mouth of the River Tees consisting of about 20 ft of sand with its surface at about mean tide level, overlying Keuper marl, had been reclaimed from the sea in recent years and is being developed as a tank farm for the storage of liquids. Reclamation began in 1932 and hydraulic filling of the area with dredgings to a depth of about 15 ft continued until 1953.

The first storage tanks, 56 ft high, were built during 1961 on reinforced concrete piles driven into the Keuper marl. The second group of tanks, 54 ft high, were supported by piers of compacted rock taken through the hydraulic fill on to the sand. At this time consideration was given to the possibility of pre-loading the ground to stabilize the foundations for the next group of tanks, as described, for example, by Di Corcia (1960) and Lambe (1962), but it was thought that an even greater economy could be effected by consolidating the hydraulic fill with the weight of the tanks themselves during the initial test loading with water. A water storage tank, 45 ft diameter and 48 ft high, without a roof, was made available to try out this technique, and the purpose of this paper is to describe the test.

SITE CONDITIONS

An initial exploration of the ground was made in 1960 using shell and auger borings. The hydraulic fill was described as grey clayey silt and samples were taken with a 4-in. diameter sampling tube of the type commonly used in Britain (Site Investigations Code, 1957, Fig. 5). Laboratory vane tests made on these samples gave values of $c_u = 250$ psf, and oedometer test results had the following range: $c_v =$ 2.55 to 22 sq.ft./yr and $m_v = 0.047$ to 0.165 sq.ft./ton. The findings from the borehole nearest to the site for the test tank are given in Fig. 1. Hand auger borings, made during the installation of some Casagrande type stand-pipe piezometers, revealed layers of sand in the clayey silt: if these were drained they would reduce the time for consolidation of the clayey silt considerably.

Dutch-cone probe tests were made at the centre of the tank site and at four points equally spaced on its periphery. Unfortunately the cone was too insensitive to measure the strength of the hydraulic fill, but it indicated the surface of the sand and showed that the depth of the fill was constant at 13 ft under the area to be occupied by the tank. It also showed that the density of the underlying sand was similar at each position and would not be the cause of large differential settlements.

Four vane borings (V.B. 2, 5, 6, and 8) were made with a vane 2 in. in diameter and 3 in. long through the hydraulic fill at the positions shown in Fig. 1. The test values are given in Fig. 2. Ten samples of the fill were taken when the piezometers were placed, using sampling tubes with a 2.9-in. internal diameter, an area ratio of 6.4 per cent, and an air-pipe connected to the cutting edge to avoid suction during sample withdrawal. Drained, undrained, and K_0 triaxial tests were made on these samples, using the procedures described by Bishop and Henkel (1962).

DESIGN OF THE FOUNDATION

From the results of the oedometer tests, maximum settlement for the centre of the tank was estimated as 2 ft. The bottom of the tank had to be 2 ft above ground level, so provision was made to place a base of compacted rock 4 ft thick. The four vane borings gave similar strength-depth profiles in the hydraulic fill and the minimum values were found in V.B. 8 (see Fig. 2). It was considered that shear failure might occur at the edge of the tank adjacent to this position, and a stability analysis (Fig. 3), by the method of Bjerrum and Overland (1957), showed that not more than 38 ft of water could be put in the tank if the strength of the hydraulic fill remained constant during loading.

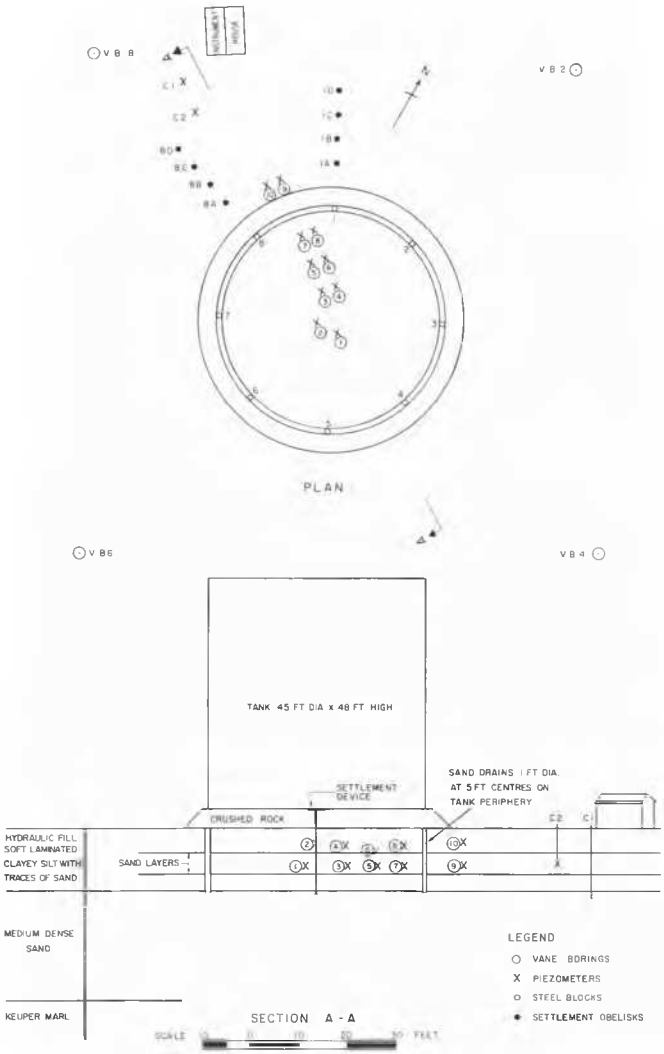

FIG. 1. Plan and section through the tank showing the positions of the piezometers, vane borings, and settlement observation points.

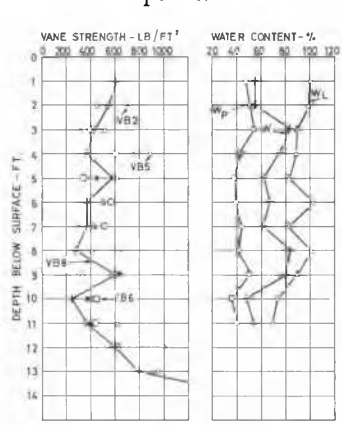

FIG. 2. Profile of vane strength *v.* depth, showing natural water content, w_P and w_L.

In order to allow for the increase in strength of the hydraulic fill when the tank was loaded, a stability analysis based on effective stresses was made, using the effective stress parameters obtained from the triaxial tests. In this analysis

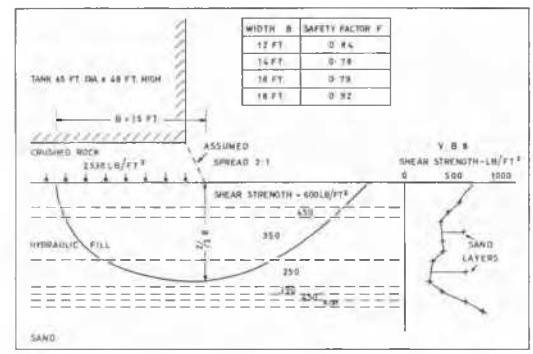

FIG. 3. Stability analysis based on total stresses.

it was assumed that the slip surfaces were two-dimensional, circular, and tangential to the sand surface, as illustrated by Fig. 4. Each circle was divided into slices, and the total pressure at the base of each slice was calculated from the weight of the tank plus the weight of soil in the slice, using submerged density for the weight of the soil below the groundwater level. The factor of safety for each circle was calculated by the method given by Bishop (1955), for values of $\bar{B}$ varying from 0 to 0.8. A large number of circles were analysed with the aid of a computer (Little and Price, 1958), and the lowest factor of safety for each value of $\bar{B}$ is shown in Fig. 4. For a safety factor of 1.14, the maximum pore pressure under the tank should be less than 60 per cent of the pressure imposed by the full tank.

Estimates of the time required for a 40 per cent dissipation of pore pressure depend on the assumptions made about the nature of the sand layers in the hydraulic fill and about which there was insufficient information. It was decided to assist the drainage of these layers by means of a curtain of sand drains 1 ft in diameter by 16 ft long at 5 ft centres around the periphery of the tank. In particular, it was considered that these drains would reduce the spread of pore water from the loaded soil under the tank to the unloaded soil outside its periphery. While these drains were being placed, two distinct and continuous layers of sand were revealed which separated the hydraulic fill into three layers, each approximately 4 ft thick.

Values for the decay of pore pressure at the centre of consolidating layers with drainage from both faces have been given by Taylor (1948): 40 per cent decay corresponding to a time factor $T_v = 0.3$. With a mean value of $c_v = 12$ sq.ft./yr and a drainage path $H = 2$ ft, the value of $t = T_v H^2/c_v$ becomes 36 days.

INSTRUMENTATION

Immediately after the sand drains were completed, 10 twin-tube hydraulic piezometers, 2 Casagrande type standpipe piezometers, and a settlement device were placed as shown in Fig. 1. The piezometer tips were at two levels, one above and one below the groundwater level.

The settlement device consisted of a steel tube, protected by an outer tube, taken through the hydraulic fill to a plate placed on the surface of the underlying sand, beneath the centre of the tank. When the base of compacted rock was complete, an overflow orifice of a water level gauge was attached to the top of the steel tube and covered with a steel cylinder which housed a second orifice. The cylinder was welded to a large end plate which was placed at the surface of the rock base and in contact with the bottom of the tank. In this way settlement of the centre of the tank, and of the

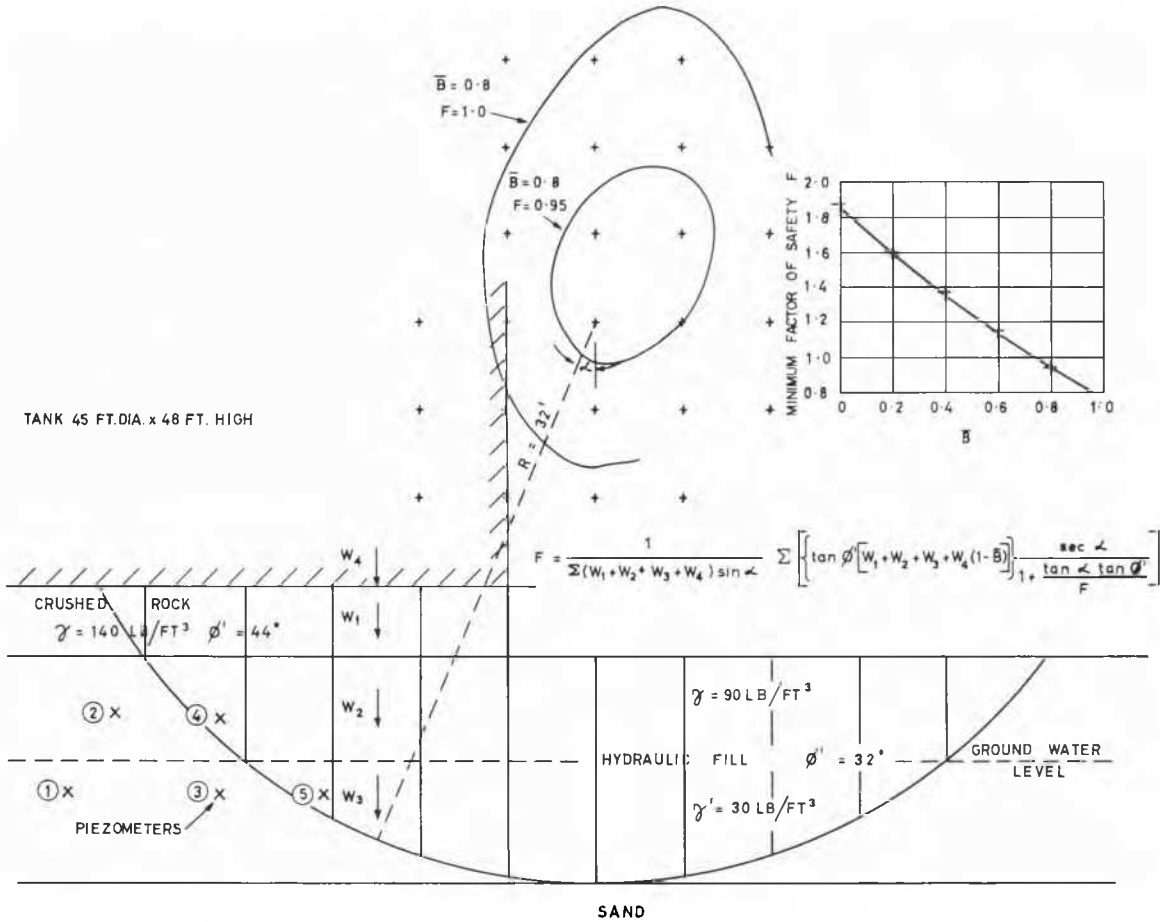

FIG. 4. Stability analysis based on effective stresses.

surface of the sand vertically below, could be measured at stand pipes in the adjacent instrument house. Details of the settlement device, the piezometers, and the method of installation are similar to those described by Penman and Watson (1963).

Eight steel blocks, approximately ¾ in. thick, were welded to the base rim of the tank to form settlement observation points (Fig. 1) and two groups of four obelisks were placed on the ground surface so that settlements or heave of the adjacent ground could be measured. These obelisks were placed near V.B. 8, which had shown the lowest *in-situ* strength for the hydraulic fill. A reference point for levelling was founded in the sand at a position considered to be outside the zone of influence of any loaded tank.

Flow tests at constant head were made from each piezometer in order to measure the *in-situ* permeability of the fill before the tank was loaded.

LOADING TEST

The maximum supply of water which could be obtained for the test would fill the tank in about 48 hours and it was decided to use this rate for as long as possible, and to keep a close watch on the pore water pressures and settlements so as to maintain stability of the tank foundation. Twenty hours after filling began the water supply was inadvertently cut off for 35 minutes and, nine hours later, there was a further interruption of 50 minutes caused by a burst in the connecting hose, which slightly lowered the water level in the tank. During these two interruptions there was a very marked drop in the pore pressures measured in the hydraulic fill under the tank, and it was evident that the increase in pore pressure could be stopped at any stage by stopping the flow of water into the tank. In fact this action was not necessary, and the tank was filled in 44 hours. Observations were maintained continuously for a further 30 hours and then repeated at intervals for 20 days. At the end of this period a second set of flow tests at constant head was made at each piezometer. The pore pressures measured by each of the 10 twin-tube piezometers, the settlements of the centre of the tank, the maximum and minimum values at its edge, and the settlement of the sand surface under the centre of the tank are shown in Fig. 5.

DISCUSSION OF RESULTS

The piezometers at the upper level, Nos. 2, 4, 6, 8, and 10, showed much lower pressures than those at the lower level, Nos. 1, 3, 5, 7, and 9, probably because the fill above the groundwater level was not fully saturated. Maximum pore pressures occurred as the tank became full: the distribution of maximum pore pressure is shown in Fig. 6, which shows that the maximum pore pressure under the centre of the tank was higher than that under the edge. Although the pore pressure at piezometer No. 1 reached 69 per cent of the applied pressure, the average pore pressure on the part of the critical slip circle under the tank was much less than this. The critical circle passed between the tips of piezometers Nos. 2, 4, 3, and 5 (see Fig. 4) and the average pore pressure measured at these tips was 41 per cent of the applied

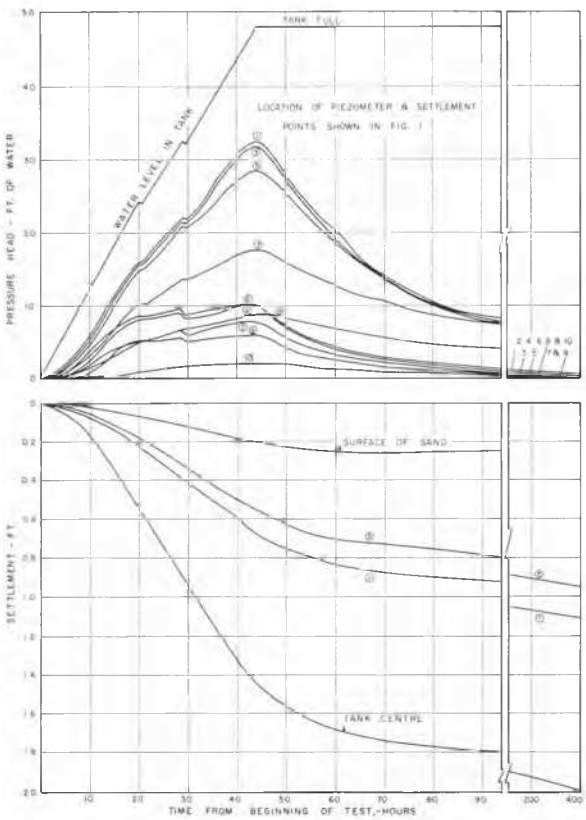

FIG. 5. Measured pore pressures and settlements *v.* time.

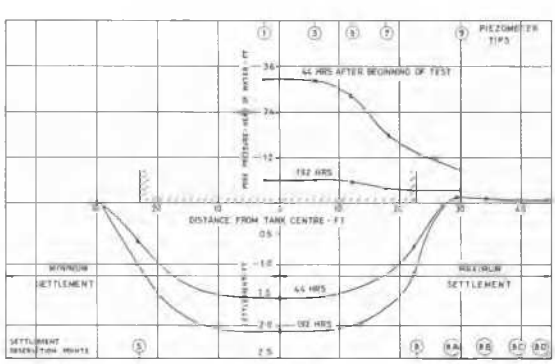

FIG. 6. Distribution of maximum pore pressure and settlement across the tank.

pressure, i.e. $\bar{B} = 0.41$, corresponding to a factor of safety of 1.36, if conditions assumed in the stability analysis are correct. However, piezometer No. 9, outside the curtain of sand drains, showed an increase of 18 per cent of the applied pressure. This may have been caused by lateral flow of the soil under the tank, which would increase the total pressure on the adjacent soil, and does not necessarily indicate that pore water from under the tank passed the sand-drain curtain. Evidence of lateral flow was given by heave of the ground surface adjacent to the tank (Fig. 6) and by excessive settlement at the tank centre. K_0 triaxial tests made on samples covering the whole depth of the hydraulic fill gave an average value of $m_v = 0.060$ sq.ft./ton, which corresponds to a compression of the fill of 1.04 ft under the centre of the tank. The measured compression was 1.45 ft, which suggests that some lateral yield occurred.

The rate of dissipation of pore pressure was about 30 times faster than that estimated from oedometer results. If the c_v values obtained from the oedometer tests were representative of the hydraulic fill, the faster rate can only be explained by much shorter drainage paths than those assumed. Free-draining seams, 7 in. apart, would satisfy this condition: examination of dried samples showed that there were large numbers of very fine sand partings, only a few sand grains thick, less than 1 in. apart. While these may not be completely free-draining they may have had sufficient over-all effect to cause the high rate of dissipation.

The values of m_v obtained from the K_0 tests made in the triaxial tests apparatus can be expected to be more accurate than those obtained from oedometer tests because much longer samples (7 in. instead of ¾ in.) were used and they were free from side friction. Unfortunately, values for c_v can only be obtained indirectly from these tests if values of k (permeability) are known, using the relationship $c_v = k/m_v\gamma_w$. Measurement of k was made in the triaxial apparatus by passing water through the samples from end to end under constant head. Values were found to decrease as the consolidation pressure on the sample was increased, 1.96 ft/yr being an average value. With $m_v = 0.060$ sq.ft./ton, this corresponds to $c_v = 1{,}175$ sq.ft./yr. If the drainage path is 2 ft as assumed in the design, this value of c_v would give a time for 40 per cent dissipation of pore pressure of $t_{40} = 0.4$ days.

In-situ values of k were obtained for the soil surrounding each piezometer tip by measuring the flow through the piezometer under constant head both before and after the soil had been consolidated by the weight of the tank, using the calculation method given by Gibson (1963). The values are given in Table I. Comparative values of settlement and cost for the three types of tank foundation used at this site are given in Table II.

CONCLUSIONS

The bearing capacity of the hydraulic fill was not sufficient to support the full tank. This was shown both by the $\phi = 0$ analysis given by Bjerrum and Overland (1957) and by an effective stress analysis based on methods given by Bishop (1955), which substantiated the founding of the earlier tanks on piles or piers. The effective stress analysis showed that the bearing capacity could be improved if the fill was allowed to consolidate under the weight of the tank as it was filled and that a factor of safety of 1.14 against edge failure could be achieved if the average pore pressure on the part of a potential slip surface under the tank was not allowed to

TABLE I. PERMEABILITY MEASURED BY FLOW FROM PIEZOMETERS BEFORE AND AFTER LOADING THE TANK

	Piezometer no.									
	1	3	5	7	9	2	4	6	8	10
Before loading k (ft/yr)	51	35	3.7	17.3	15.4	5.6	85	9.2	15.8	9.9
After loading k (ft/yr)	0.90	0.56	0.68	16.7	0.85	11.4	27.6	3.8	13.4	0.72

TABLE II. COMPARISON OF THE SETTLEMENTS AND COST OF THE THREE TYPES OF TANK FOUNDATIONS

Type of foundation	Height of tank (ft)	Cost (£ per sq. ft. floor)	Settlement (in.) Edge max.	Edge min.	Centre
R.C. piles	56	2.2	negligible		—
Rock piers	54	1.5	7.7	7.0	—
Peripheral sand drains	48	0.5	14.1	12.3	25.2

rise above 60 per cent of the head of water in the tank when it was full. When the pore pressures had completely dissipated, the factor of safety would be 1.86. The original estimate for the time required to allow 40 per cent dissipation of pore pressure was 36 days, but it was found possible to fill the tank in 44 hours because the average pore pressure on a potential slip surface under the tank only increased to about 40 per cent of the bearing pressure imposed by the tank. Some pore pressure increase was measured in the unloaded soil outside the tank. This would tend to reduce the tank stability and was not taken into account in the analysis. The analysis proved that slightly higher tanks could be constructed on this site by this method, using controlled filling, and that the stability could be improved further by surcharging the ground adjacent to the tank.

The cost of this foundation was much less than those using piles or piers even though a considerable amount of supervision was required for the test loading. The settlements were much greater, but were predicted with reasonable accuracy and allowed for in the thickness of the tank base. The maximum differential settlement round the edge of the tank was only 1.8 in. and reflects the uniformity of the soil under the tank. The maximum differential settlement between the edge and centre of the tank was 12.9 in. and even though the tank was only 45 ft in diameter, this caused no distress to the tank structure.

It is of interest that the greatest edge settlement of the tank occurred adjacent to the vane boring which showed the lowest *in-situ* shear strength.

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