Foundations for storage tanks on reclaimed land at Teesmouth

A. D. M. PENMAN & G. H. WATSON

Mr A. L. Little and Mr G. C. G. Beavan, Binnie and Partners

The Paper presents a clear and detailed account of an interesting civil engineering operation of some ingenuity.

64. The contrast between the costs of a piled foundation, vibrated rock pier foundation and sand drain foundation is startling; the ratios are 44:30:10. We are not told the time occupied by the installation of the sand drains but it would almost certainly be less than that for the vibrated rock piers and perhaps considerably less than for piled foundations. The two days for filling the tank to provide the load is a small addition. Although the tank was left for a further 20 days, this was presumably for experimental purposes.

65. It is interesting to learn that the original estimate of the minimum safe time in which a tank could be filled was 3 months which was reduced to 10 days by the exploitation of two subsequently discovered intermediate sand layers in the hydraulic fill. Even this estimate was too high and the test loading was completed in less than 2 days. As the Authors modestly say, this illustrates the difficulty of estimating drainage times from tests on small samples of layered deposits. They pointedly refrain from commenting on the failure of the original site investigation to find the all-important intermediate sand layers but perhaps it is too much to expect the average commercial boring to detect layers of sand only 3 in. thick in hydraulic fill although the Authors' later investigations found them.

66. The Authors were fortunate to have a small tank with whose foundation they could experiment but would they not agree that without such experimental data it is undesirable to base the all-important estimates of consolidation progress solely on inspection and testing of hit and miss samples. Continuous sampling tools such as the Swedish foil sampler and the more recently developed Dutch 'stocking' sampler[17] appear to be more appropriate than short open drive tubes for finding all the thin layers and partings of sand upon which the consolidation rate largely depends. Would the Authors comment on this and could they make some useful suggestions on the best way to go about such an investigation.

67. Although construction required a slow filling of the largest tank, no appreciable excess pore pressures developed and the filling could, from this point of view, have been much more rapid.

68. Probably no commercial undertaking would be prepared to wait 3 months whilst the foundations of an apparently available tank became strong enough to bear its load but a few days is short enough for any reasonable person. It appears therefore that the conditions at Teesmouth were ideal for a consolidated foundation and the Authors are to be congratulated for finding this out.

69. It would be interesting to have an indication of the cost of the investigations, instrumentation, observations and specialist advice that were necessary to prove the feasibility of, and to carry out, the foundation consolidation technique. This would indicate to others in future whether the possible savings made by using the techniques would justify the costs. For instance, it appears that the foundations of the 45 ft dia. tank cost about £800 which meant a saving of the order of £2700 over the cost of

Paper published: *Proc. Instn civ. Engrs*, 1967, 37 (May) 19–42.

piling, less than this in relation to the cost of rock pier foundations. It seems unlikely that the research done was economically justified by this one tank alone although it was undoubtedly justified many times over for all six of the consolidated foundations.

Dr D. A. Greenwood, Cementation Company Limited

I was especially interested in the Authors' well documented case histories of tank foundations constructed on the North Teesside site because of my association with similar works on the south bank of the Tees, and elsewhere in UK, and overseas.

71. At the new Teesport refinery, Shell UK Ltd, have erected several crude oil and gas oil tanks on hydraulic fill which had been strengthened by methods similar in principle, but differing in execution, to the rock pier type of foundation described by the Authors. Vertical columns of compacted coarse grained slag, approximately 3 ft in dia. (according to the strength of the containing fill) were formed 6 ft 3 in.–7 ft apart through the hydraulic fill beneath each tank foundation pad. A large poker vibrator or 'vibroflot' washed a hole through the fill into which, with the machine still in the bore, slag was compacted and expanded into the surrounding soft fill. Water circulation through the vibrator prevented collapse of the sensitive soil and ensured that columns had high strength and free draining properties. The alternatives of use of compressed air or no jetting fluid, both of which were tried, led to unsatisfactory columns in the soft hydraulic fill.

72. The sequence of strata beneath the crude oil tanks was the same as that described by the Authors for the north bank site. The desiccated crust was 4–6 ft thick with c_u equal to 300–400 lb/sq. ft. The strength range of the soft part of the hydraulic fill was 150–350 lb/sq. ft with consolidation properties determined in oedometer tests $m_v = 0{\cdot}04$–$0{\cdot}11$ sq. ft/ton, and $c_v = 20$–70 sq. ft/year. Beneath the gas oil tanks, the former beach sand was only 3 to 4 ft thick and was underlain by a glacial varved clay ($c_u = 550$ lb/sq. ft, $m_v = 0{\cdot}02$ sq. ft/ton and $c_v = 19$ sq. ft/year) resting on stiff boulder clay. The compacted columns did not penetrate more than 3 ft into the sand in this area.

73. Table 2 shows the thickness of the hydraulic fill and the varved clay together with data on settlement for each tank. In column 3, total measured settlements are shown; the figures in brackets represent the estimated settlements of the gas oil tanks due only to the treated fill. The difference between the two sets of figures quoted for the gas oil tanks is settlement attributed to lower layers—largely to the varved clay.

74. Table 2 also includes results for a foundation constructed on natural alluvium of the Humber overlying stiff boulder clay. This alluvium had the following properties: 4 ft thick crust $c_u = 1150$ lb/sq. ft; soft alluvium $c_u = 150$–450 lb/sq. ft, $m_v = 0{\cdot}05$ sq. ft/ton, $c_v = 25$–150 sq. ft/year.

75. Each tank was founded on an asphalt surfaced rolled granular pad 2–3 ft thick.

76. In all cases, the time taken to fill the tanks was governed primarily by ability to pump the quantity of test water required and was generally three to four weeks. However, there is no doubt that, in the light of the Authors' results and with the short drainage paths provided by the columns, much more rapid loading could have been applied, safely, if necessary.

77. It is clear in retrospect that, in the hydraulic fill described in the Paper, rapid rates of settlement would always be achieved. In natural soils without sand partings it is conceivable that under the 'controlled loading' technique, rate of dissipation of pore pressure might sometimes be critical in limiting a construction schedule. With stone piers or columns this would never be the case.

78. The stone columns formed using the vibroflot appear to have resulted in considerably smaller total settlements than the stone pier and controlled loading techniques described by the Authors, despite greater depths of hydraulic fill. This

Table 2. Settlement of ground strengthened by stone columns

1 Tank designation	2 Size		3 Average edge settlement *ft*	4 Greatest tilt *ft*	5 Centre settlement *ft*	6 Ratio of tank dia. to depth of fill D/H	7 Depth of soft layers *ft*		
	Dia. *ft*	Ht *ft*							
							Fill		
Teesport						9·2 (min.)	24 (max.)		
104 Crude oil	220	54	0·25 (0·25)*	0·06	0·45	13·8 (max.)	16 (min.)		
106 Crude oil	220	54	0·20 (0·20)	0·17	0·33	11	20		
Teesport							Fill	Sand	Varved clay
123 Gas oil	96	54	0·43 (0·23)	0·20	Not measured	5·6	17	3	10
124 Gas oil	96	54	0·47 (0·27)	0·10	—	5·6	17	3	10
125 Gas oil	96	48	0·36 (0·19)	0·10	—	6·0	16	3	13
126 Gas oil	56	42	0·29 (0·14)	0·05	—	3·3	17	4	12
165 Gas oil	80	54	0·45 (0·25)	0·09	—	4·4	18	2	13
167 Gas oil	80	48	0·41 (0·24)	0·04	—	4·7	17	3	13
168 Gas oil	64	60	0·50 (0·28)	0·04	—	3·5	18	4	13
169 Gas oil	48	42	0·38 (0·23)	0·13	—	2·7	18	4	11
Humber site							Alluvium		
L.D.F.	144	54	0·42 (0·42)	0·05	—	6·3	23		

* The figures in brackets in column 3 represent estimated settlements attributable to the layer of treated hydraulic fill.

would be anticipated with controlled loading but in the case of the stone piers, it can probably be attributed to the more stable cylindrical shape of the columns since the quantity of stone per unit volume of treated soil used in each system appears to be almost identical.

79. Settlement of clayey strata strengthened by cylindrical stone columns is, in my experience, approximately one third of that calculated for untreated soil on the basis of consolidation properties determined in oedometer tests.

80. Since total settlements are reduced by stone columns, the differential settlements follow likewise. The large number and close spacing of the columns contribute to relatively small differential movements. By adjusting the distribution of column spacing, the opportunity is offered of compensating to some degree for varying thickness of the compressible stratum. This does not seem practicable with the controlled loading technique.

81. The cost in 1965 of the tank foundations on stone columns was comparable with that of the stone piers described by the Authors. The depth of the treatment averaged 20 ft, however, so that on the basis of equal depth, the cost of foundations on stone columns was 75% of that of piers, but rather more than that of foundations prepared by controlled loading.

Professor P. W. Rowe, Simon Engineering Laboratories, University of Manchester

The Authors have reported observations of the high consolidation rates of a layered deposit in contrast to those predicted on the basis of measurements on small samples in a Casagrande oedometer and have shown how recognition of this fact can effect marked economy in the cost of foundations for storage tanks. The same phenomenon can apply to foundations for other structures and to many cases of embankments and cuttings.

83. I have long advised that storage tanks on suitably layered deposits should be raised simply on a pad foundation without sand drains and filled initially at a controlled rate of loading. Careful sampling and testing, of even small samples for compressibility, will furnish estimates of total settlement sufficiently accurate to allow the use of settlement as a means of control. Filling may be rapid up to about two-thirds of the height where the water level can be maintained constant for a few days until the rate of settlement is seen to decrease. Thereafter filling may be resumed until the tank is full provided a running plot of settlement is maintained. With reference to Fig. 7 I do not advise filling of a tank overnight after the two-thirds level has been reached unless settlement observations are maintained.

84. It is interesting to note that the initial adoption of an expensive piled foundation followed from the use of shell and auger boring. Presumably this would have been conducted in the normal way without water in the casing until the 'seal sands' were met. But water pressure in sand layers within the clay, excess to that in the borehole and in the layered clay just below the base of the hole, will have resulted in flow towards the hole, partially softening the clay and disturbing the layers to a degree which could have precluded their detection. The degree of clay softening would be less for shallow holes driven continuously during part of a day but nevertheless the accident to the crane indicated a decrease from some 300 to 200 lb/sq. ft.

85. In their conclusions the Authors claim that the drainage of the permeable layers within the clay 'was assisted by putting vertical sand drains under the tank periphery'. Is there evidence to support this belief? The method of 'placing' the sand drains is not reported but if it had been by a driven mandrel the installation would have harmed the ground owing to the remoulding of the clay and destruction of the layers in the vicinity of the drains. Installation by shell and auger would have done much less harm but overdriving of the casing, and smear, would cause sufficient disturbance to render the drains relatively ineffective. Sand layers 3 in. thick (§ 12) would most certainly not require drainage assistance. By omitting the drains substantial further economy could have been achieved.

86. In order to be able to relate this experience to predictions on other sites it would be useful if the Authors could publish some of the following additional information:

(*a*) photographs of undisturbed samples partially cut, split and left partially to dry;
(*b*) the results of in situ constant head permeability tests on the piezometers;
(*c*) the results of consolidation tests on large samples subjected to vertical, and radial, flow.

Mr R. Dixon, Terresearch Limited

Would the Authors give their reasons for the selection of a factor of safety of 1·15 for the tanks subject to controlled test loading. The adoption of such a small factor of safety suggests that the stability analysis was regarded as somewhat conservative. If so, in what respects?

88. Actually the Authors have introduced an additional factor of safety in § 11 by identifying the value of $\bar{B}=0{\cdot}5$ with the pore-pressure equal to 0·5 of the full tank load. By definition, in Appendix 1, $\bar{B}$ is the ratio of the pore-pressure to the total load on a slice (including the weight of the soil) so $\bar{B}=0{\cdot}5$ corresponds, in the case of a deep circle, to a pore-pressure equal to about 0·6 of the pressure imposed by the full tank.

89. It is presumed from Fig. 3 that the piezometers were located with respect to levels of the known sand layers. If intermediate sand layers did not exist would the Authors recommend placing piezometers at various levels throughout the depth of the fill in order to determine the average pore-pressure?

90. Referring to Fig. 4 could the Authors explain the anomaly that between times 90 and 400 h the surface of the sand settled 0·3 ft whilst the measured total settlement of the tank at the centre was only 0·2 ft.? Incidentally how was the consolidation of the sand estimated?

91. With regard to settlement due to the lateral flow of the fill no indication is given as to how this may be estimated. Would it be possible to give a likely maximum as a percentage of the consolidation settlement?

Mr D. A. Fox, Senior Civil Engineer, British Petroleum Company

The Authors endorse the fact that oil tanks are often sited on weak ground with bearing pressures well in excess of those permissible for normal structural rafts and building footings. To the 'oil boiler', tanks represent a large investment in money and acreage which he would prefer to dispense with, if he could; they do not normally earn any useful revenue. Cheap foundations with large settlements are therefore acceptable within reason.

93. The Authors refer to a few published examples of tanks which failed during their initial water tests; there must be a very large number of unpublished failures, and near-failures which have had to be jacked up and re-levelled. In the majority of those cases failure probably occurred from too rapid loading during the initial water test.

94. This factor deserves to be emphasized in conjunction with the Author's conclusions in § 37, the operative words being 'controlled conditions' when filling the tanks for the first time at a new or unknown site, and especially sites when there is a dessicated crust overlying normally-consolidated silts and clays. It is quite possible at some tank farms where tanks of certain heights have been successfully established for several years to find that an adjacent area required for expansion of storage, and thought to have much the same subsoil profile, does in fact have a slightly thinner crust. New tanks water tested in a similar manner to those already established are then found to be breaking through, and water test procedure and perhaps tank heights have to be modified.

95. Every tank site must therefore be treated on its own merits, and those that are underlain by clays of higher plasticity are usually slower draining, and they require much slower rates of filling once the point is reached where plastic or radial yield becomes noticeable from external ground heave observations. It is preferable therefore to permit a high rate of loading up to this point, and this means a rate of say 4 ft of water per day. Beyond this point the rate may be decreased to 2 ft per day, and when this point is reached where failure may be imminent around the circular arc of the type shown in Fig. 9, loading is decreased to 1 ft per day or much less, even 1 ft per week in extreme cases.

96. A similar method of circular arc analysis was developed from Guthlac Wilson[18] and has been in use for nearly 20 years now, with however, an arbitrary assumption that for all practical purposes the centre of the arc coincides with the tank shell. Allowance also has been made for the wedge shape in plan area. The weakest arc of failure appears usually to coincide with the position of maximum ground heave outside the tank shell and the position of maximum local deformation of the bottom inside the tank shell. This is not so much a case of abrupt sliding of a failure surface, i.e. the weakest arc, relative to a non-sliding arc just beneath but rather it is an arc around which the overall strain is at its maximum. This is of course the arc with the lowest theoretical factor of safety, and adjacent arcs just inside and just outside with fractionally higher factors of safety will also yield but with slightly lower overall strains.

97. Most large tanks fail through local yielding at the weakest combination of the crust and the subsoil; overall failure of the whole plane of the tank tipping as reported by Nixon,[1] only occurs with smaller tanks. Design according to stability analysis requires an assessment of what is the lowest localized shear strength rather than a low average through all borehole results taken around a tank or across the tank farm. This is not necessarily the same as the results from one extra weak borehole alone.

98. At some sites where the subsoil consists of normally-consolidated clays of medium to high plasticity, and poor drainage characteristics, it is worth considering the excavation, down to about 12–15 ft depth, of a large ditch circular in plan and set below and just inside the tank shell. This ditch is filled with the cheapest available fine granular material, such as poor-quality sand, and these 'subsoil sandfill rings' function as shear rings (based on stability analysis) and as a means to accelerate the local vertical and radial drainage of the clays under the tank load, and additionally they help to bridge over local weak spots. The extra costs of such rings must of course be assessed against the expectations of more reliable water testing and of higher loads being attained, but to be economic it is essential that they be excavated without the use of sheet piling or other revetment.

Mr A. Lyndon, Simon-Carves Limited

Prior to the work described in the Paper being carried out, some useful background experience had been gained with the foundations to a nearby already established tank farm. The ground was naturally deposited and although not quite so favourable as the present reclaimed site, conditions were similar in that at a shallow depth a sufficient thickness of sand generally existed to confine the stability problem for surface loading to an upper layer of soft clay. The necessity to prevent lateral yield of the clay had traditionally been overcome by the installation of an interlocking ring of sheet steel piles about 15 ft deep under the perimeter of the tank. On one occasion, however, a tank (No. 83) tilted unduly and there was subsequently found to be a lense of peat beneath that sector of the sheet piling.

100. When further tanks were required a more economical method of founding was used, dispensing with piling and based on partial consolidation of the weak ground with controlled loading during test filling. The foundation design adopted aimed at maintaining stability in the soft clay and obtaining sufficient consolidation

settlement of the ground before the tank was commissioned, such that any further settlement and possible tilting would not affect the operation of the plant.

101. The method was first successfully applied to two fixed roof tanks, numbers 400 and 401, each 54 ft diameter by 22 ft high. The ground generally consisted of up to 65 ft of recent alluvium overlying heavily consolidated deposits. Boreholes taken at the tanks positions showed that the alluvium at tank 400 comprised: 10 ft of soft clay over 20 ft of sand, and 40 ft of silty clay, and at tank 401: 10 ft of soft clay, 30 ft of sand and 12 ft of silty clay. In this instance it was decided to use a spread foundation consisting of 6 ft thick base of rolled blast furnace slag, which also acted as a consolidation load whilst the tank was being erected. The rate at which the tanks were initially filled with water was controlled by allowing the settlement at each stage of loading to become steady, for this purpose a settlement post, consisting of a steel joist embedded in a pad footing was installed in a suitable perimeter position to each tank. Settlement observations were made throughout the 10 month loading period the results of which are shown in Fig. 12.

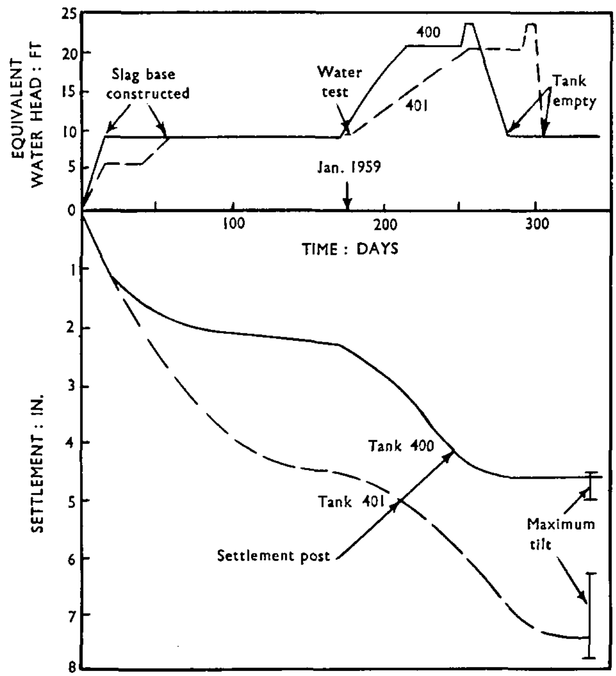

Fig. 12. Loading and settlement with time of tanks 400 and 401

102. It will be noted that although a lesser depth of alluvium existed, the greater settlement occurred to tank 401. This was attributed to the comparatively thick sand layer so much so, that by the end of the test loading it was calculated that the consolidation process was virtually completed.

103. The gain in strength of the upper soft clay was assessed assuming that the relation between shear strength and effective overburden pressure, as derived from the lower normally consolidated clay could be applied. Consequently referring to

Fig. 13 the initial minimum shear strength of 200 lb/sq. ft would have increased to over 600 lb/sq. ft by the end of tank filling, ensuring that the soil was not overstressed.

104. A more rigorous effective stress analysis was also applied to the stability of the tank foundations based on the slip-circle method and programmed for the computer. This enabled the minimum factor of safety to be readily determined for any given average pore pressure dissipation. In this case, as field measurement of pore pressure was not made, it merely provided an alternative approach to the stability problem.

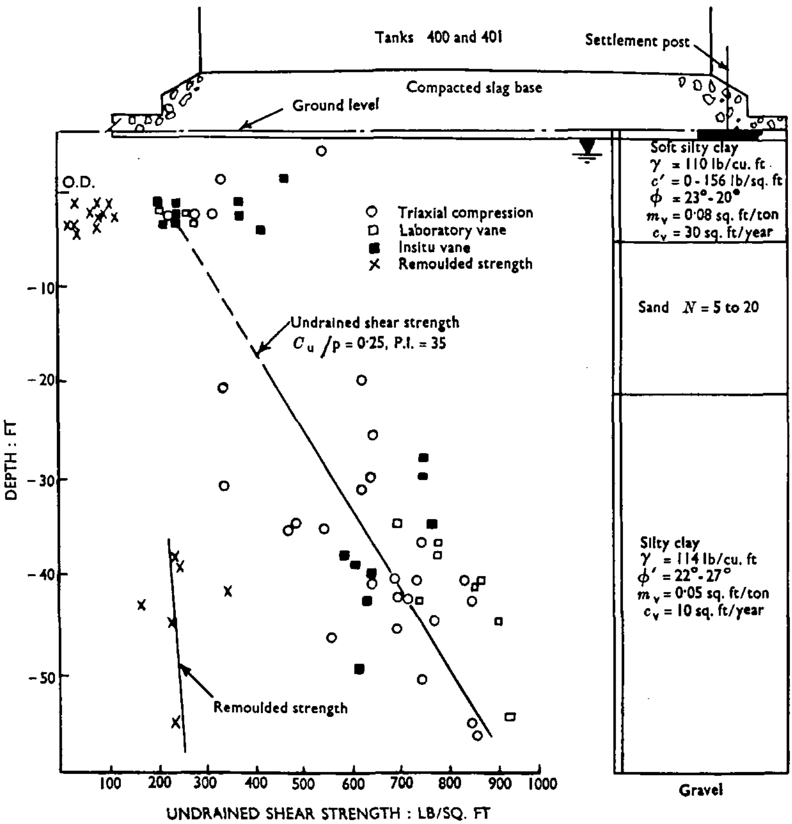

Fig. 13. Foundation to tanks 400 and 401 showing soil properties

105. As mentioned earlier tank 83 had tilted unduly during test filling, it had settled a total of 2 ft tilting 9 in. when the height of water in the tank had reached 35 ft over a period of 40 days. A stability investigation, based on local edge failure, showed that a factor of safety of 1·1, calculated from the vane strengths was operative. However, due to the peat layer extending beneath the sheet piling, under test filling, migration of pore-pressure to outside the edge of the tank could readily have occurred and probably constituted the true failure condition. Irrespective of the method of test and analysis used for the stability computation, the field evidence presented by

tank 83 confirms that local edge failure and the effect of pore-pressure migration should be considered in the foundation design.

106. It was established from the experience gained that the most successful way to control the filling of a tank and to ensure stability was by pore-pressure measurement. A further small tank, No. 89, was then constructed and used to demonstrate the application of a field pore-pressure apparatus. The ground was similar to that as found to tanks 400 and 401, and to expedite the consolidation process sand drains, about 12 ft deep, were installed by hand auger. The results obtained during test filling are given in Fig. 14 and show that this method is extremely effective.

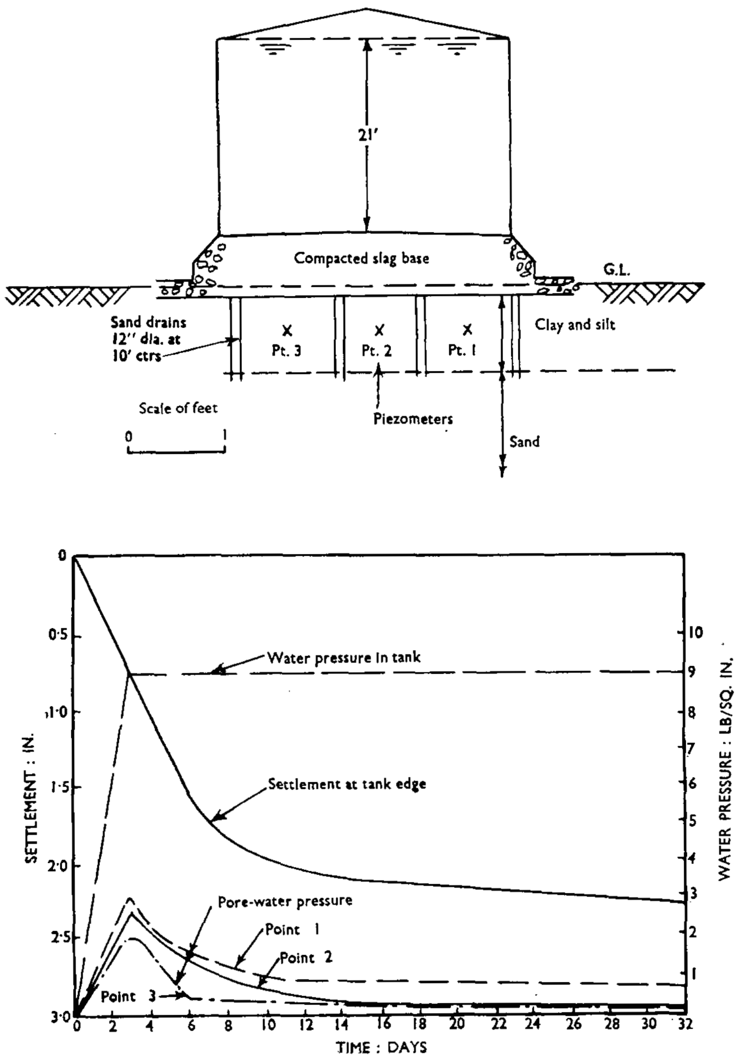

Fig. 14. Tank 89 observed settlement and pore pressure

107. The feasibility of this technique for the founding of large storage tanks, where one is only concerned with the stability of a comparatively thin upper layer of soft clay has been readily established in the Paper. It would have been interesting, however, if the Authors had the opportunity to determine, by direct measurement, the shear strength after consolidation to compare with the gain in strength as calculated.

Mr E. S. Webber, Howard Humphreys and Sons

Overflow-type remote-reading settlement gauges, similar to those described in Appendix 2, have been installed in the Guma Dam, Sierra Leone. Certain difficulties encountered in their operation may be of general interest.

109. The Guma Dam was constructed for the Guma Valley Water Company, and Howard Humphreys and Sons were the consulting engineers. It is an earth and rockfill embankment some 200 ft high, and 12 overflow settlement gauges were installed in the earthfill only, at four different elevations, each with its own gauge house. The two chief advantages for an embankment of this type of gauge, as compared with any form of extending standpipe, are the ease and permanence of reading, and the minimizing of obstruction to earthfill operations. There is also greater security against damage during construction.

110. The gauges at Guma Dam were similar to those described in the Paper, but with only a single overflow point similar to D in Fig. 10. They were constructed from 4 in. diameter steel tube, with screwed end caps. The air tube was connected through the top end cap.

111. The operating instructions required that 24 hours should elapse between filling the standpipe and taking the reading, to ensure equilibrium. Surprisingly large apparent settlements occurred soon after installation, and readings were taken at daily intervals. Water rapidly disappeared from the bottom of the standpipes, representing a settlement of some 4 ft in four weeks. This appeared to be very unlikely, and leakage was suspected. However, the installations had been pressure tested prior to backfilling, so that leakage at joints could almost certainly be ruled out and loss by diffusion through the walls of the tube was suspected.

112. The tube used to connect the overflow tips to the standpipes in the instrument house was 0·111 in. i.d. nylon tube as at Teesmouth. The tubing was manufactured from nylon 66.

113. Similar tubing was used for the connecting tubes of the pore pressure installation, but for that part of the work the nylon tube had been sheathed with polythene to prevent possible build up of pressure in the circuits due to osmotic effects. However the sheathing of nylon tubing with polythene was not at that time a standard process and owing to supply difficulties it was necessary to use unsheathed tubing on the settlement installations. It was not foreseen that losses through the walls of the tube would give any significant error over the period of 24 hours required for a settlement gauge to attain stability.

114. A laboratory test of diffusion through the tubing was then carried out under a pressure of 5 ft of water, similar to conditions in the embankment. The losses, expressed as percentages of the initial volume of water contained, were as follows, for a near constant air and water temperature of 80°F:

Unsheathed nylon tube, immersed in water	0·17% per day
Nylon sheathed in polythene, in air	0·11% per day
Nylon sheathed in polythene, immersed in water	0·08%–0·40% per day

115. These results demonstrate the effectiveness of the polythene sheathing, and suggest that the apparent settlement was most likely due to diffusion through the unsheathed tubing. Have the Authors had any similar trouble? It would appear from § 54 that they have not. It is noted that no water was added to their standpipes, so that it would not be possible to tell whether some of the recorded settlement was in fact due to diffusion through the tubes.

116. The difficulty was overcome at Guma by measuring the rate of fall in the standpipes at one minute intervals over 15 minutes after adding water. To determine the level of the overflow in the settlement cell, the mean level h for each time interval was plotted against $\Delta h/\Delta t$, and the straight line plot extrapolated to find the value of h for which $\Delta h/\Delta t = 0$.

117. A second difficulty encountered at Guma was due to flooding, through rupturing of the air pipes. Although each cell had an individual drain pipe, for the three lower elevations, the cells at each elevation were provided with a common ¾ in. diameter air pipe. This was perfectly satisfactory while it remained intact, but as soon as it was ruptured, all the cells at that elevation became very difficult to operate. In consequence, individual air pipes were provided to each cell at the upper elevation, and these cells have continued to operate satisfactorily. It is not clear from § 50 whether individual air pipes were fitted in the Authors' installation.

118. The fact that the instrument house was a short distance from the gauges under the Teesmouth tanks suggests that both of the difficulties encountered at Guma would be minimized in this case, but they should be borne in mind where longer connecting tubes are required. The connecting tubes at Guma were up to 1500 ft long. The effect of temperature on diffusion through the tube may also need consideration.

Mr E. H. Steger, C. J. Pell and Partners

Messrs Penman's and Watson's interesting Paper confirms many points which have been suspected for some time in the character and behaviour of what is essentially a deposit of recent alluvium.

120. The most important of these is the existence of thin layers of sand. Even if not detected during routine borings, such layers invariably occur within recent alluvium, generally in the form of lenses rather than continuous strata. One result is that often the rate of consolidation is more rapid than that derived from theoretical considerations. Another is an increase in differential settlements due to this lack of uniformity in the foundation soils.

121. Unfortunately, one is frequently faced with the construction of floating roof tanks on such layers, the thickness of which may well exceed 100 ft, giving a D/H value near unity. At such depths, the provision of any form of special foundation, be it piles, sand-drains, a buoyant foundation, etc., becomes expensive: the more so because tanker terminals are frequently in areas where the cost of transport of specialist plant and materials add enormously to the cost of the construction. Where the cost of the foundation approaches that of the tank itself, it becomes an uneconomical proposition and it may be preferable to allow years until the tank can be fully loaded, or to construct smaller tanks.

122. As regards settlements and differential settlements, could the Authors please explain one apparent contradiction: in § 14 the order of magnitude of the estimated differential settlements between centre and periphery appears to be 6 in., whereas in § 18 the measured difference was 12 in., and it is said that this had been expected. I should think 12 in. to be the more likely figure but in view of the expected, and measured, evidence of local overstressing, find the 1·8 in. differential movement around the edge very small indeed.

123. It is fully appreciated that the settlements reported in the Paper are largely immediate or elastic settlements. Have the Authors any figures for subsequent consolidation settlements?

Mr R. G. Brickell, Brickell, Moss, Rankine and Hill, Lower Hutt, N.Z.

An identical programme to that described in the Paper was carried out by my firm in 1959.[19] An unusual feature of the work on the pretreatment of the foundations for the Shell bulk bitumen tank at Lyttleton, N.Z., was the use of a model of the foundation comprising a 4 in. mould of undisturbed soil.

125. This model was given the preloading treatment to a trial time scale and the improvement of shear strength was measured with a laboratory shear vane inserted in one of 8 small holes drilled in the loose top of the mould at $\frac{2}{3}$ radius from the centre. With this model the minimum times for preconsolidation were determined.

126. In the prototype the time available for preconsolidation was limited by shipments of bitumen en route and the programme had to be carried out in the minimum time practicable. Another feature of the work was the use of remote water level gauges brought to a central reading point.

127. The programme on the prototypes was completely successful, as to improvement in shear strength. However, settlement predictions were not accurate as the hot bitumen in the tanks apparently steamed off moisture and caused greater settlements over the years than the consolidation tests on the preconsolidated samples predicted.

Mr Penman and Dr Watson

Since the Paper was published, the failure of two tanks on Thames-side has been brought to our attention. The tanks were 80 ft and 48 ft dia., both 42 ft high and were built with their base-plates laid to a fall of 1 in 50 towards the centre. Other similar tanks on the site were tested by filling with water over a period of about one week, they were then kept full for a week and then emptied into another tank in about a week. The resulting settlements were of the order of 1 ft at the edges with a centre-edge differential settlement of about 5 in. The tanks which failed were kept full of water: the 80 ft dia. tank failed after about one month and the 48 ft dia. tank some time later. Failure was due to rupture of the base plates. From the behaviour of the other tanks it has been estimated that the centre-edge differential settlement was about 9 in. at the time of failure. Without radial movements of the tank edge, this could be expected to produce a strain of only about 5×10^{-4} in the base plates—presumably this was concentrated at one lap-weld and was sufficient to initiate failure.

129. These examples show forcibly that where centre–edge differential settlements are to be allowed, the centre of the base-plates must be raised by an amount at least equal to half the maximum expected differential settlement to prevent the plates from carrying tension. Although §§ 18 and 41 of the Paper state that no structural damage has been observed in any of the tanks at Teesmouth, it must be emphasised that the base-plates of all those tanks were laid with a central rise.

130. Where very large centre–edge differential settlements are expected it may be wise to use full depth butt welds for the base plates (used in Holland), the extra cost could be expected to be small when compared with the cost of providing special foundation to reduce the differential settlement.

131. Mr Little and Mr Beavan draw attention to the importance of a knowledge of the spacing of sand layers in estimating drainage times, and equally, of course, a knowledge of the drainage facilities of the sand layers is necessary: it was to ensure that the horizontal layers could lose their water readily when it reached a position under the edge of the tank that the Authors placed sand drains at this position. Continuous sampling tools would be ideal in this soft foundation soil to show all the sand layers, although a lot can be done by 'continuous' sampling with a thin-walled sampling tube fitted with an air pipe (§ 12) provided that the water in the borehole is kept high enough to prevent ingress of water into the hole and consequential disturbance. When the main sand layers have been established in a number of holes their intermediate positions and continuance can be checked by a field vane or cone which will indicate the sand by increased resistance.

132. Even an exact knowledge of sand layers and partings might not lead to an exact time estimate due to the facts outlined in § 19 but it would be a useful piece of research to take continuous samples of the type discussed by Messrs Little and Beavan

and re-estimate the time required for loading Tank 2402-F, now that we know the answer.

133. The cost of the investigations, instrumentation, observations and specialist advice relating to the small experimental Tank 2402-F has been calculated on the basis of the time spent by all persons involved in this work, plus full cost of the field apparatus used, ignoring the fact that much of it was re-used several times subsequently, and the total amounts to £2275. This figure includes the site work outlined in §§ 12, 13, 15 and 16 together with the laboratory work indicated by § 20, but does not include the original site investigation, which covered the whole area of the tank farm, nor the cost of the cone penetrometer (§ 12).

134. Dr Greenwood has given us most useful data on the behaviour of tanks founded on compacted cylinders formed by the vibroflot, a descendant from the German Rütteldruckgerät which came to this country via North America. The piers (§ 9) were compacted by a German machine, and the main differences between the two methods of construction were that while the piers began with a grabbed hole from which the rock was compacted sideways into the soft fill without the use of water, the cylinders were formed from a hole jetted into the soft fill by the vibroflot itself, to which rock was added. The differences in the settlement produced in these two similar types of foundation by tanks of the same height (compare Tables 1 and 2) may reflect differences in the hydraulic fill at Teesmouth (North side) and Teesport (South side) rather than the differences in constructional technique. The quoted values of m_v are similar for the two sites (§§ 3 and 72) but at Teesport the values of c_v are higher than those at Teesmouth, which suggests that the Teesport hydraulic fill was the more free draining. This is substantiated by the fact that holes could be grabbed out at Teesmouth whereas water was needed in the Teesport holes to prevent collapse. The degree of compaction achieved may well have been higher in the more free draining material, resulting in the observed smaller settlements. Although the Authors were not primarily concerned with the construction of the piers, they understand that the vibrating poker used at Teesmouth could not be jetted down satisfactorily because the fill was not sufficiently free draining and hence the more expensive method of grabbing out a hole had to be used.

135. The idea (§ 80) of adjusting the compressibility of the fill by varying the spacing of the columns (to reduce tank tilt when the fill thickness varies over its diameter) is interesting and it would be useful to know from field tests how compressibility can be varied from the condition of no rock columns or piers, to the condition when they are placed at their closest spacings.

136. The Authors agree with Professor Rowe's advice on overnight filling (§ 83) and suggested in the Paper (§§ 22 and 24) that slow filling only during normal working hours should be used if the sole observations of tank behaviour were those of settlement. Fig. 7 shows the results of readings taken every 2 hours during the 99 hours required to fill the tank and the next 12 hours (regardless of day or night) and further readings taken once or twice a day while the tank remained full.

137. It is not clear to the Authors how 'a running plot of settlement' (§ 83) should be interpreted to show tank stability; presumably rapid filling up to about two-thirds of the height could cause failure if the tank were sufficiently high in relation to initial soil strength.

138. Holes for the sand-drains were made with lorry-mounted power augers, and the sand-drains for Tank 2402-F were completed in three days, at a cost of about 10s/ft. The purpose of the curtain of sand-drains was to prevent the flow of pore-water to the unloaded soil outside the tank where it was feared that it would increase pore-pressures and so reduce stability. In fact piezometers outside the curtain (Tip 9, Fig. 4 and Tips 13 and 15, Fig. 7) showed pore-pressure increases; this may be attributed to the lateral flow of soil (see Fig. 8) causing an increase in total pressure in the soil outside the sand-drain curtain. An indication of the effect of the sand-drains can be inferred from the low pore-pressures measured by Tips 12 and 13 (Fig. 7)

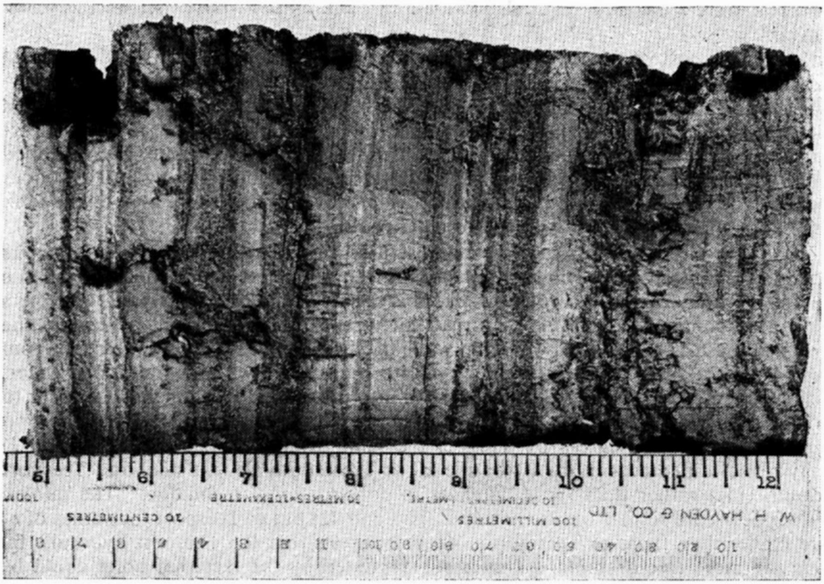

Fig. 15

400 hours after the tank became full. The true value of the sand-drain curtain could be assessed from tests on two tanks of the same size, one with and one without sand-drains, but unfortunately such an opportunity did not arise at Teesmouth. Mr Brickell has given an example of the effectiveness of a sand-drain curtain, which is quoted towards the end of this reply.

139. Referring to § 86 (*a*), an undisturbed sample has been obtained from a depth of 2 ft, and it is shown in Fig. 15; unfortunately no photographs were taken of the undisturbed 3 in. dia. samples which were tested prior to tank construction. Constant head permeability tests, § 86 (*b*), on the piezometers under Tank 2402-F were made before and after loading; values of permeability were obtained by the method given by Professor Gibson[20] and are in Table 3. No tests of the type described under § 86 (*c*) have been carried out on soil from Teesmouth.

Table 3. Values of permeability from constant head tests

Piezometer No. (Tank 2402-F)	1	2	3	4	5	6	7	8	9	10
Before loading k (ft/year)	51	5·6	35	85	3·7	9·2	17·3	15·8	15·4	9·9
After loading k (ft/year)	0·90	11·4	0·56	27·6	0·68	3·8	16·7	13·4	0·85	0·72

140. The Authors are grateful to Mr Dixon for pointing out their incorrect use of $\bar{B}$ in Professor Bishop's analysis to express pore-pressures in terms of tank load only, excluding soil weight. It should also be noted that shallower circles can show lower factors of safety; the depth of the worst circle may be controlled by the thickness of any dried crust or may be kept down by providing a zone of granular material

under the tank edge, as suggested by Mr Fox. A very high water table and pore-pressures produced by lateral flow of soil can also reduce the factor of safety. Both conditions can be improved by the provision of a surcharge around the tank.

141. Mr Dixon has also pointed out an error in Fig. 4, which originated in an error in Table 1. Only one observation of the settlement of the sand surface taken 400 hours after the start of the test was available (after 150 hours after the start of the test), and this was 0·38 ft, incorrectly quoted as 0·58 ft. Because of the limited information, this part of the sand surface curve was not given in an earlier publication.[21] Line 9, column 9 of Table 1 should be altered to read 0·38 instead of 0·58, and the 'surface of sand' curve in Fig. 4 removed over the period 200–400 hours and replaced by one observational point at 400 hours at a settlement of 0·38 ft. The settlement of the sand was estimated from the average standard penetration resistance by the methods suggested by Terzaghi and Peck.[22]

142. The factor of safety against edge failure caused by a rotational slip under part of the tank decreased as the tank was filled due to the build-up of pore-pressures. As soon as filling stopped, the factor of safety could be expected to begin increasing due to the decay of pore-pressures and it was decided to use a high rate of filling for the test on Tank 2402-F until the factor of safety had fallen to a fairly low value in order to see how quickly the tank could be filled. Factors of safety of 1·2 are not uncommon for temporary earth-works and because careful, continuous observations were to be made on the tank it was felt reasonable to make 1·15 a lower limit during the test. Had the analysis erred on the unsafe side it was hoped to be able to see signs of distress by the development of uneven edge settlement in time to prevent complete failure. In the event, the water supply was inadvertently cut off for two short periods during the test, and each time, the pore-pressures began to fall so rapidly (see Fig. 4) that it was clear that the factor of safety would rise quickly as soon as filling was stopped, consequently a very low minimum factor of safety was acceptable. Such a low factor of safety could not, of course, be used without detailed and continuous observations on the tank. In future special tests of this type the technique of causing a short halt during filling will be used to observe the rate of pore-pressure dissipation.

143. The amount of lateral flow which occurs depends on the relative weakness of the soil in relation to the tank load and the size of the tank in relation to the thickness of the weak layer. Rapid filling, with resultant high pore pressures (low soil strength —low factor of safety), produces maximum lateral flow. Tanks of large diameter (relative to the thickness of weak soil) have a restraining effect on lateral flow (see §§ 39 and 40). Filling which is slow in relation to the value of c_v for the soil will minimize lateral flow; the maximum value would be when failure occurred. Settlement due to lateral flow cannot be related to settlement due to consolidation: it may be related to the minimum factor of safety permitted during loading and is probably best obtained from observations on the first tanks constructed on any given site. Meyerhof[23] gave a method based on total stress analysis in which settlement due to lateral flow was related to the plastic strains of samples during triaxial compression tests with external pressure conditions and rates of strain similar to those expected to occur under the actual foundation.

144. In the absence of pronounced sand layers, the piezometers would have been placed at various levels and grouped near potential failure planes.

145. Mr Fox has given valuable practical experience to the discussion and has emphasized that every tank site must be treated on its own merits, as well as the dangers of too rapid loading. The rate of loading must be adjusted to suit the rate of pore-pressure dissipation at the particular site so as to maintain stability.

146. The Authors are grateful to Mr Lyndon for providing details of the earlier work on tank foundations at Billingham Reach, and agree that it would be useful to measure the existing in situ strength of the soil under a full tank using a vane or similar in situ testing equipment.

147. The overflow-type remote-reading settlement gauge was developed for use in earth dams and its use under tanks was regarded as very straightforward particularly in view of the comparatively short lengths of connecting tubing. The experiences described by Mr Webber are most useful in connexion with installations for earth dams. Polythene tube, the walls of which were completely impervious to water, was used by the Building Research Station for early piezometer installations in earth dams but its walls were found to be slightly permeable to air so a change was made to tubes made from nylon 66, which was found to absorb water. Absorption by freshly manufactured tube, which caused it to swell, produced an initial water intake of about 1·2% volume per day which fell to a steady evaporative loss of 0·17% per day (in agreement with Mr Webber's figure) when the tube was in air.[24] When the tube was immersed in water the initial intake was even more rapid because swelling occurred from both sides, but there was no steady loss from the tube. It was concluded that there would be no loss from tubes buried in wet clay as were the connecting tubes for the settlement gauges in a trench under the tank. To check this a dummy length of tube was placed in the same trench under the tank. All tubes were filled with water several weeks before the loading test to allow complete swelling, and the only loss from the dummy tube was consistent with evaporation from the short length exposed inside the instrument house. This was 12 ft and a loss of 0·17% per day would correspond to a fall of water level in the copper end pipe and Perspex stand pipe, of only 0·0043 ft/day, so that provided the rate of tank settlement remained greater than this, no error should have occurred. The obvious solution to the problem was, of course, to coat the nylon tube with polythene, so as to avoid both air and water percolation, and this type of tube is now generally used for earth dam installations. A common air pipe was used for the gauges placed under Tanks N3000-F and 2101-FD, without causing any difficulty, but, of course, the lengths were very much shorter than those used at Guma and they were not damaged.

148. Mr Steger's remarks on the cost of transporting plant and materials to many sites in the world where tanks are required emphasizes the usefulness of simple foundations even though detailed measurements have to be made to ensure stability. The 6 in. rise given to the centre of the foundation pad, mentioned in § 14, was to allow for a central settlement of 12 in. without putting the tank base-plates into tension; from the evidence given at the beginning of this reply, failure can be caused by excessive tension in the base-plates. In fact the central settlement was only 12 in. greater than the edge settlement, so that the initially upward pointing cone of 6 in. height became a downward pointing cone of 6 in. depth. Greater settlement could be accepted, up to the limit where base-plate tension would produce leakage, but it would clearly be preferable to provide an even greater initial central rise so as to avoid base-plate tension altogether.

149. The 1·8 in. differential settlement round the edge reflects the uniform soil conditions under the tank. The tanks were kept full of water until the excess pore-pressures had fallen almost to zero, indicating that a large amount of the consolidation was complete. Subsequent edge settlements of the tanks appear to be of the order of ½–¾ in., although due to rearrangements on the site when the tanks were put into service, the accuracy of these observations is not very high.

150. The Authors are grateful to Mr Brickell for drawing their attention to his paper,[19] which describes the use of preloads to consolidate foundation soils and gives examples of two tanks (largest 80 ft dia. × 42 ft high) built over sand drains 8 ft apart, in which the initial water load in the tank was used to consolidate the soil. Instead of using in situ piezometers to check that the gain in shear strength of the foundation soil was at all times adequate, Mr Brickell obtained the relationship between consolidation and shear strength of the soil from the model of the foundation which he has described in his contribution to this discussion and used the measured settlement of the tank to indicate improvement of shear strength.

151. The paper describes a method used to observe the settlement under the centre

of a gravel preload (at Hastings Memorial Hospital site) by observing a short central staff with a surveyor's precise level through a horizontal 6 in. dia. pipe laid under the gravel, although no mention was found in the paper of remote reading water level gauges of the type used at Teesmouth.

152. In the paper Mr Brickell warns against the dangers of increasing the load on sand-drain projects faster than the rate at which shear strength is increasing and says, 'There have been several failures overseas from this cause'. He also gives a most interesting example of a 54 ft high tank which began to show evidence of plastic yield of the foundation soil when it contained only 20 ft of test water. To quote, 'In this case, sand-drains could be constructed round the tank only. By this expedient, however, the sub-soils were consolidated sufficiently to allow full product load in a period of 8 weeks'.

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