The Post-Glacial Clays of the Thames Estuary at Tilbury and Shellhaven

Les Argiles post-glaciaires de l'Estuaire de la Tamise à Tilbury et à Shellhaven

A. W. Skempton, D.Sc., A.M.I.C.E., F.G.S., and D. J. Henkel, B.Sc., Imperial College, University of London, England

Summary

Investigations of the post-glacial clays of the Thames estuary have been made at two sites, near Tilbury and Shellhaven. The clays are about 45 ft. thick and are normally consolidated, except for the upper layers which have been subjected to drying. Shear strengths were determined both by in-situ vane tests and by undrained triaxial compression tests on undisturbed samples, and the results of the two methods were in complete agreement over the full range of depth. Shear strengths determined in the consolidated undrained triaxial tests were in general too high. The pre-consolidation loads as found in oedometer tests were in reasonably close agreement with the effective overburden pressure (except of course, in the upper dried zone). The ratio of shear strength to effective overburden pressure correlated closely with plasticity index, and the correlation is almost identical with that obtained from other sites in normally consolidated clays. Information is given on the mineralogy, the activity and sensitivity of the clays.

Sommaire

Des investigations sur les argiles post-glaciaires de l'estuaire de la Tamise ont été faites à deux endroits, près de Tilbury et de Shellhaven. Ces couches d'argile ont à peu près 15 mètres de profondeur et sont normalement consolidées, sauf les couches superficielles qui ont été asséchées. La résistance au cisaillement a été déterminée in-situ par des essais avec l'appareil à palettes de même que par des essais de compression triaxiale à teneur en eau constante sur des échantillons intacts; les résultats donnés par les deux méthodes correspondaient parfaitement à toutes les profondeurs. Les résistances au cisaillement obtenues dans les essais triaxiaux à teneur en eau constante, après consolidation, étaient en général trop élevées. Les pressions de préconsolidation données par les essais œdométriques correspondaient assez bien avec la pression des terres de couverture (sauf, évidemment, dans la zone superficielle asséchée). L'indice de plasticité correspond exactement au rapport de la résistance au cisaillement à la pression des terres de couverture et cette relation est presque identique à celle obtenue à d'autres endroits dans des argiles normalement consolidées. Des renseignements sont donnés sur la minéralogie, l'activité et la sensiblité au remaniement des argiles.

Introduction

In 1948 *Carlson* published a paper showing that, in two beds of clay in Sweden, the shear strength as determined by unconfined compression tests on undisturbed samples was considerably less than the strength as determined in-situ by the vane test; the difference increasing with increasing depth. But he also showed that the vane test strengths were in reasonable agreement with the strengths calculated from landslips which had taken place in these clays. In 1950 *Cadling* and *Odenstad* gave details of investigations, at several other sites in Sweden, which led to the same conclusions. Moreover *Skempton* (1948 b) and *Hansen* (1950) have also published records where, at depths of more than about 30 ft. to 40 ft., the strengths as measured by the vane test were greater than those obtained from unconfined compression tests (or undrained triaxial compression tests) on undisturbed samples.

These results have led some engineers to doubt the validity of the standard procedure of sampling and testing, at least at depth in beds of soft clay, and there has been a tendency to insist on the use of vane tests, even in cases where it is ill suited to the site conditions. A more correct view of the matter seems to be that, in general, perfectly satisfactory results are obtained from compression tests on undisturbed samples, and that trouble with this standard procedure is restricted to clays which have

(i) a high liquidity index, and
(ii) an activity of less than 0.75 (*Skempton*, 1953)

$$\left(\text{activity} = \frac{\text{plasticity index}}{\text{percentage clay fraction}}\right).$$

It is true that such clays appear to be of frequent occurrence

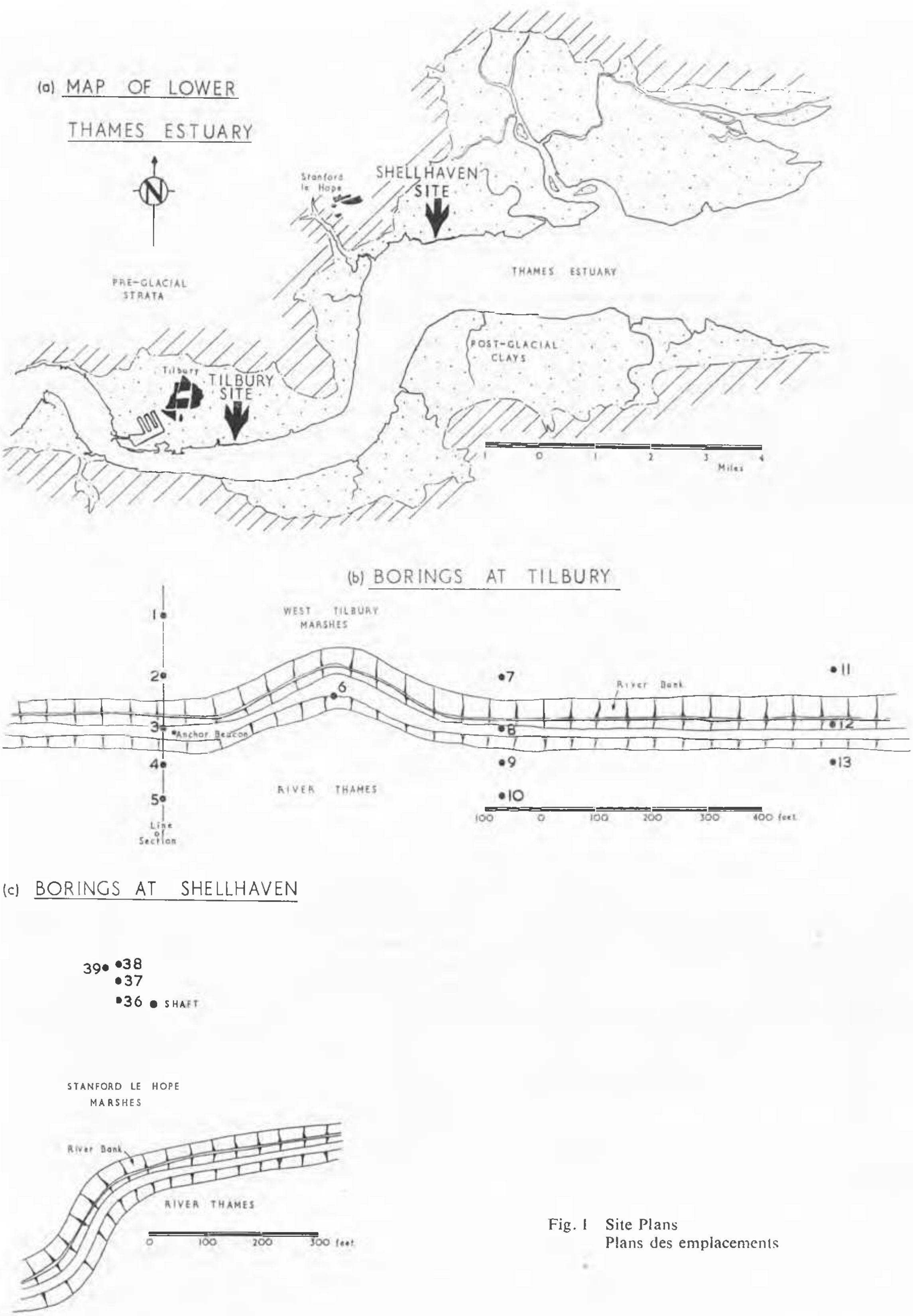

Fig. 1 Site Plans
Plans des emplacements

in Scandinavia, yet, on a broader view, they must be considered exceptional even among normally-consolidated clays (as defined by *Terzaghi*, 1941). And in over-consolidated clays in which the liquidity index is almost always low, sampling difficulties are practically unknown.

In the present paper investigations are described at two sites in the normally-consolidated, post-glacial clays of the Thames estuary at Tilbury and Shellhaven. At all depths from the surface to the base of the strata, some 45 ft. below ground level, the vane tests and compression tests on undisturbed samples were in complete agreement, in accordance with the high activity of the clays, namely about 1.3 or 1.4.

Another point, concerning which there has been controversy, relates to the question as to whether normally consolidated clays do or do not show an increase in strength with depth. Prior to 1948 the evidence was conflicting, but since then a number of case records have been published which show that, in general, the strength of such clays does increase with depth. Evidence to the contrary is due either

(i) to the clay not being truly normally-consolidated (usually on account of intermittent drying during formation), or
(ii) to the clay belonging to the category in which satisfactory sampling is not possible at depth.

Yet in the latter cases the strength is found to increase with depth when the vane test is used. But this question has been taken a stage further. *Skempton* (1948 b) suggested that in normally-consolidated clays the rate of increase in shear strength c with effective overburden pressure p, as expressed by the ratio $(c/p)_n$, is a function of the type of clay and, in particular, of the liquid limit. Subsequent data has confirmed this suggestion, although it now appears that the correlation between $(c/p)_n$ and plasticity index is slightly better than the correlation with liquid limit, and the test results from the two Thames estuary sites provide important additional information in this matter. (A paper giving all the available evidence on values of $(c/p)_n$ is in preparation by the senior author and will be published in Proceedings Inst. Civil Engineering.)

The Sites and the Borings

The geographical positions of the sites are shown in Fig. 1 (*a*). Both are situated on the post-glacial marshes along the north bank of the Thames estuary. More detailed site plans, showing the positions of the borings, are given in Fig. 1 (*b*) and (*c*). A cross section through the marsh and the river embankment at Tilbury is shown in Fig. 2, and this is typical of both sites. The Tilbury borings were made in connection with problems of making a jetty for a new power station, and at Shellhaven the investigations concerned the construction of a deep cofferdam for a pump house for an oil refinery. (For records of the loads in the bracing of this cofferdam see *Skempton* and *Ward*, 1952.)

At Tilbury undisturbed samples 4 inches in diameter and 18 inches long were taken at intervals of about 5 ft. from boreholes 7, 8 and 10, while vane tests were carried out in the other

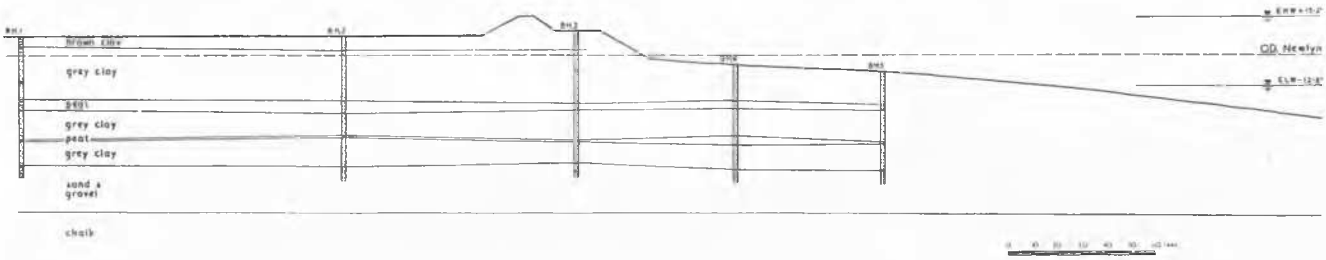

Fig. 2 Tilbury Section on Line of Anchor Beacons. Depth of Chalk Given by Previous Borings
Coupe à travers la levée de la rivière à Tilbury

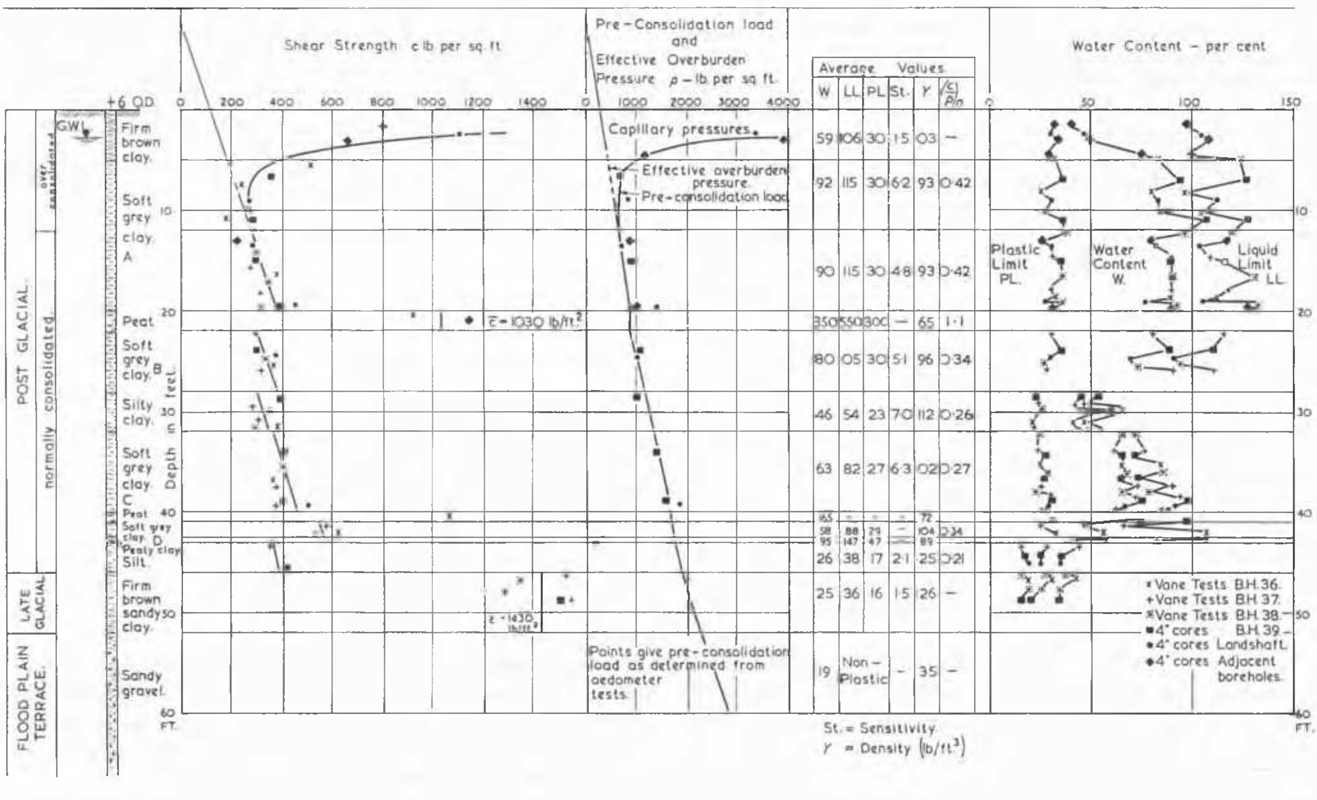

Fig. 3 Shellhaven Test Results
Résultats d'essais de Shellhaven

10 boreholes. At Shellhaven, vane tests were carried out in boreholes 36, 37 and 38, and undisturbed samples were taken in borehole 39. In addition, undisturbed samples were taken, by hand, as excavation proceeded in an 18 ft. diameter shaft immediately adjacent to the borings (see Fig. 1 (*c*)). For purposes other than the design of the cofferdam at Shellhaven,

Mineralogy and Activity

Professor *R. E. Grim* kindly carried out a mineralogical analysis of the clay fraction of a representative sample of the Shellhaven clay, and found that the predominant mineral was Illite, with some Kaolinite and no Montmorillonite.

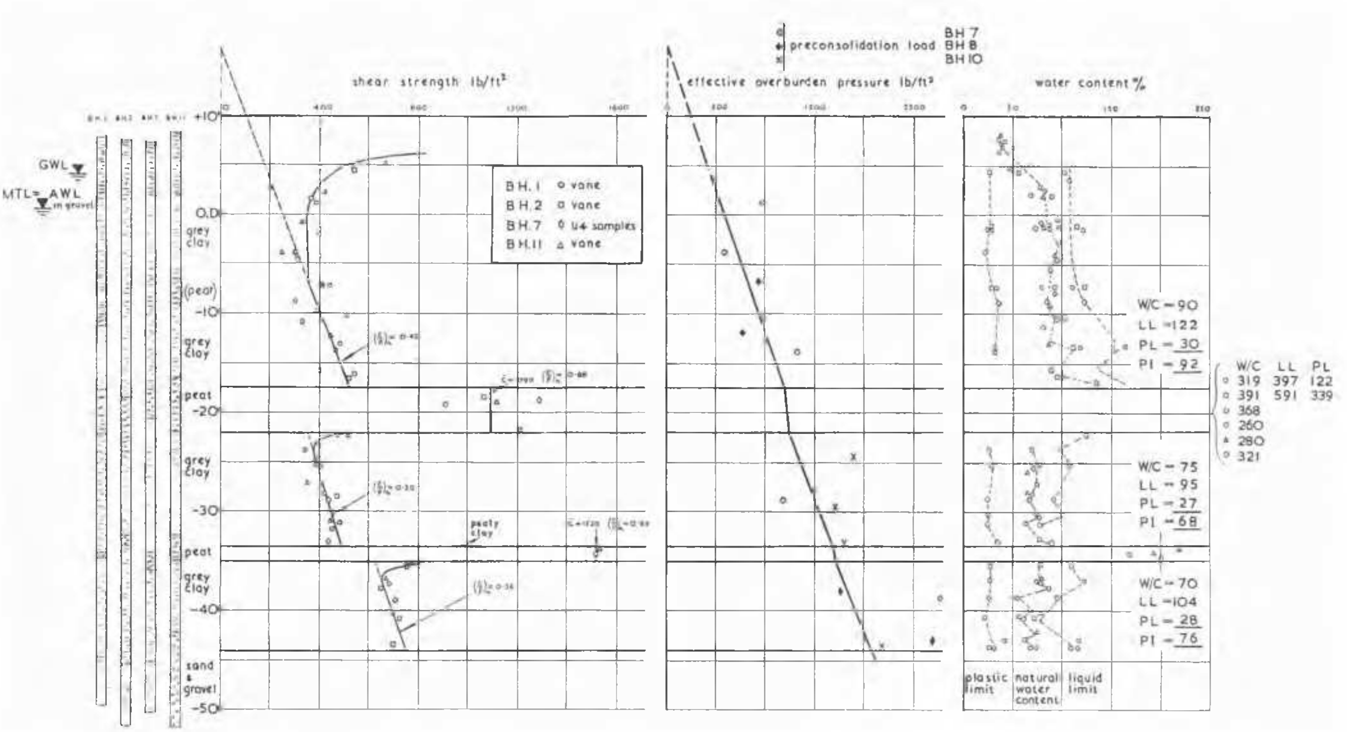

Fig. 4 Tilbury Marsh Borings
Sondages de marais à Tilbury

samples were taken from neighbouring boreholes and, where necessary, a few results from these boreholes have been used to supplement the data from boreholes 36 to 39.

The principal strata at Tilbury are shown in the section Fig. 2 and are summarised for Shellhaven and Tilbury in Fig. 3 and 4. The only important differences in stratigraphy are that at Shellhaven there is only one layer of upper peat and there are layers of silty clay and silt. At Tilbury all the post-glacial material other than peat consists of a moderately homogeneous clay.

At both sites the water level in the underlying gravel stood about 2 ft. below ground water level in the clay. There was as a consequence a slight hydraulic gradient acting downward through the clay, and this has been allowed for in calculating the effective overburden pressures.

Water Content, Atterberg Limits, Density

The variations of water content (w) liquid limit (LL) and plastic limit (PL) with depth are plotted in Fig. 3 and 4. For Tilbury the results are given only for the four borings made in the marsh. The results from the other borings are closely similar. The average values of these properties for each stratum, at both sites, are given in Tables I and II, where the average density is also quoted, together with the liquidity index (LI) as defined by the expression

$$LI = \frac{LL - PL}{w - PL}.$$

In Fig. 5 the PI and percentage clay fraction (less than 2 microns) have been plotted for a number of samples from both sites. The activity (= PI/clay fraction) at Shellhaven is independent of the particle size distribution and has the average value 1.33. At Tilbury the range of PI is less, and the average activity is 1.40. In view of the absence of Montmorillonite it is probable that these moderately high activities are due to the presence of organic colloids. There is no essential difference between the particle size distribution curves for the clays from the two sites.

Vane Test

Vane tests were carried out with an apparatus similar to that previously used at Grangemouth (*Skempton*, 1948 b) but with some improvements in mechanical design due to the authors' colleague Dr. *A. W. Bishop*. The time of loading to failure was normally about 2 minutes, and corrections[1]) were made to reduce the measured strength to a standard time of loading of 5 minutes. The results are plotted for Shellhaven in Fig. 3 and for three boreholes at Tilbury in Fig. 4. The vane tests from the other boreholes at Tilbury show no essential difference and lack of space prevents their inclusion in this paper.

[1]) Using data obtained by special tests at Shellhaven to ascertain the effect of rate of loading on the measured strength. This data was quantitatively similar to the results published by *Skempton* (1948), *Cadling* and *Odenstad* (1950) and *Casagrande* and *Wilson* (1951).

Table 1 Properties of Shellhaven Post-Glacial Strata (Average Values)

Stratum	*Depth above or below O.D. ft.*	*Consistency*					*Activity*	*Density*	*Shear Strength*		*Consolidation*	
		w	*LL*	*PL*	*PI*	*LI*	$\frac{PI}{clay}$	*lb/ft*^{3}	$\left(\frac{c}{p}\right)_n$	S_t	$\frac{K_v}{c}$	C_v *cm*^{2}*/sec*
Firm brown clay	6 to + 1	59	106	30	76	0.38	1.1	103	—	1.5	—	—
Soft grey clay A	+ 1 to —14	91	115	30	85	0.72	1.3	93	0.42	5.5	28	7.10^{-5}
Upper peat	—14 to —16	350	550	300	250	0.20	—	65	1.1	—	—	—
Soft grey clay B	—16 to —22	80	105	30	75	0.67	1.3	96	0.34	5.1	31	4.10^{-5}
Silty clay	—22 to —26	46	54	23	31	0.74	1.3	112	0.26	7.0	38	12.10^{-5}
Soft grey clay C	—26 to —34	63	82	27	55	0.65	1.4	102	0.27	6.3	33	6.10^{-5}
Lower peat	—34 to —35	165	—	—	—	—	—	72	—	—	—	—
Soft grey clay D	—35 to —36.5	58	88	29	59	0.49	—	104	0.34	—	—	—
Peaty clay	—36.5 to —37	95	147	47	100	0.48	—	89	—	—	—	—
Silt	—37 to —40	26	38	17	21	0.43	1.4	125	0.21	2.1	—	—

Undrained Compression Tests

The simplest method of determining the undrained shear strength of satured clays is, of course, to measure the unconfined compression strength and divide the result by 2. But if, as at Shellhaven, the clay is slightly fissured, the unconfined compression test is not satisfactory and at Imperial College it is now a matter of routine to use the undrained triaxial test. Suitable corrections are made for the strength of the rubber envelope (for soft clays this is an important correction, see *Henkel* and *Gilbert*, 1952) and all results are reduced to a standard time of loading of 5 minutes.

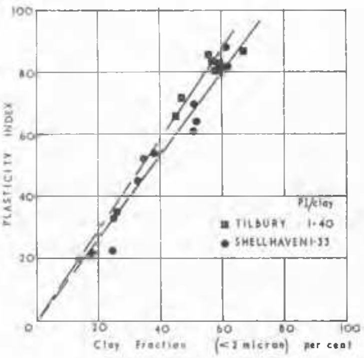

Fig. 5 Relation between Plasticity Index and Clay Fraction
Relation entre l'indice de plasticité et le pourcentage d'argile

It is, perhaps, rather surprising that a post-glacial normally-consolidated clay should be fissured (possibly a consequence of syneresis, see *Skempton* and *Northey*, 1952). But it was evident that the Shellhaven clay did contain numerous very small fissures and, on the average, the unconfined compression strength was some 10 to 15 per cent lower than the corrected strength as measured in the undrained triaxial test. Apart from this effect the strength of the clay in the undrained triaxial tests was absolutely independent of the chamber pressure, thus proving that the undrained angle of shearing resistance $\varphi_u = 0$. The Tilbury clays gave results very similar to these from Shellhaven, but the fissures were less evident.

The great majority of the triaxial tests were carried out on specimens 1½ inches diameter and 3 inches long. But in order to see whether there was any scale effect, undrained triaxial tests were made on six specimens of Shellhaven clay 4 inches diameter and 8½ inches long. Each of these specimens was taken from the centre portion of a core, and 1½ inch diameter specimens were also prepared from the core, two from above and two from below the central portion. It was found that the average strength of the six 4 inch and of the twenty four 1½ inch diameter specimens were identical. The stress-strain curves of the larger specimen were, however, rather steeper than those of the smaller specimen. A typical stress strain curve for an undisturbed 4 inch diameter specimen is shown in Fig. 6 together with the stress-strain curve for the same clay after remoulding.

The results of the undrained triaxial tests are plotted in

Table 2 Properties of Tilbury Post-Glacial Strata (Average Values)

Stratum	*Depth above or below O.D. ft.*	*Consistency*					*Activity*	*Density*	*Shear Strength*		*Consolidation*	
		w	*LL*	*PL*	*PI*	*LI*	$\frac{PI}{clay}$	*lb/ft*^{3}	$\left(\frac{c}{p}\right)_n$	S_t	$\frac{K_v}{c}$	C_v *cm*^{2}*/sec*
Firm brown clay	G.L. to + 2	40	115	30	85	0.12	—	115	—	—	—	—
Soft grey clay	+ 2 to —18	89	115	30	85	0.70	1.4	95	0.43	3.3	50	10.10^{-5}
Upper peat	—18 to —22	310	420	180	240	0.54	—	68	0.85	c.3	—	—
Soft grey clay	—22 to —33.5	70	90	28	62	0.68	1.5	100	0.31	3.6	40	11.10^{-5}
Lower peat	—33.5 to —35	—	—	—	—	—	—	70	0.89	—	—	—
Soft grey clay	—35 to —45	71	109	29	80	0.53	1.4	100	0.36	3.2	50	9.10^{-5}

Fig. 3 and 4 and it will at once be seen that no significant difference exists between these results and those of the vane tests, throughout the whole depth of the post-glacial strata.

Sensitivity

The remoulded shear strength c_r was determined on practically all samples, and the sensitivity (defined by the ratio $S_t = c/c_r$, see Fig. 6) obtained. The average values of S_t for each stratum are given in Tables 1 and 2. The Shellhaven clays are appeciably more sensitive than those at Tilbury. The salt content of the pore water of both clays is about 25 gm/litre and the probable explanation of the difference in sensitivity is that, at the time of deposition, the estuary was more salty at Shellhaven than at Tilbury. Some leaching has taken place in the Shellhaven clays (*Skempton* and *Northey*, 1952), but it is possible that little if any leaching has occured at Tilbury. Whether this is correct or not, it may be noted that the sensitivity of about 3.5 at Tilbury could be accounted for entirely, or almost entirely, by thixotropy whereas it has been shown (loc. cit.) that the sensitivity of the Shellhaven clay cannot be attributed solely to this cause. Also, at Shellhaven, even at the present time, the salt content of the estuarine water is 31 gm/litre and some leaching in the clay must have occured, while at Tilbury the salinity of the river water is now rather less than that of the pore water in the clay.

Effective Overburden Pressure, Pre-Consolidation Load

The effective overburden pressure at any depth is equal to the total pressure due to the full weight of all the soil above the point under consideration, minus the ground water pressure at that point. The variations of effective overburden pressure with depth for the two sites are plotted in Fig. 3 and 4.

Typical oedometer test results for two undisturbed samples of Shellhaven clay are given in Fig. 7, together with the curve for one of the samples in the remoulded state. The pre-consolidation load can be readily determined for the undisturbed samples (*Casagrande*, 1936) and the results are plotted for all the oedometer tests, in Fig. 3 and 4. At Shellhaven there is excellent agreement between the pre-consolidation load and the effective overburden pressure at depths below 12 ft. At smaller depths the pre-consolidation loads give clear evidence that the clay has been subjected to drying.

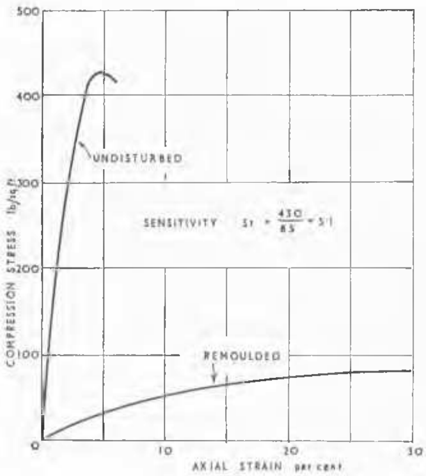

Fig. 6 Typical Stress Strain Curves for Shellhaven Clay
Courbes typiques effort/déformation de l'argile de Shellhaven

At Tilbury the agreement is not so close as at Shellhaven, but no reason can be advanced for the larger discrepancies between the pre-consolidation loads and effective overburden pressure.

The Ratio $(c/p)_n$ from Field Data

Once the variations with depth of shear strength c and effective overburden pressure p are known, the ratio (c/p) for any stratum can immediately be found. The values of this ratio for each layer of soil are given in Fig. 3, for Shellhaven, and

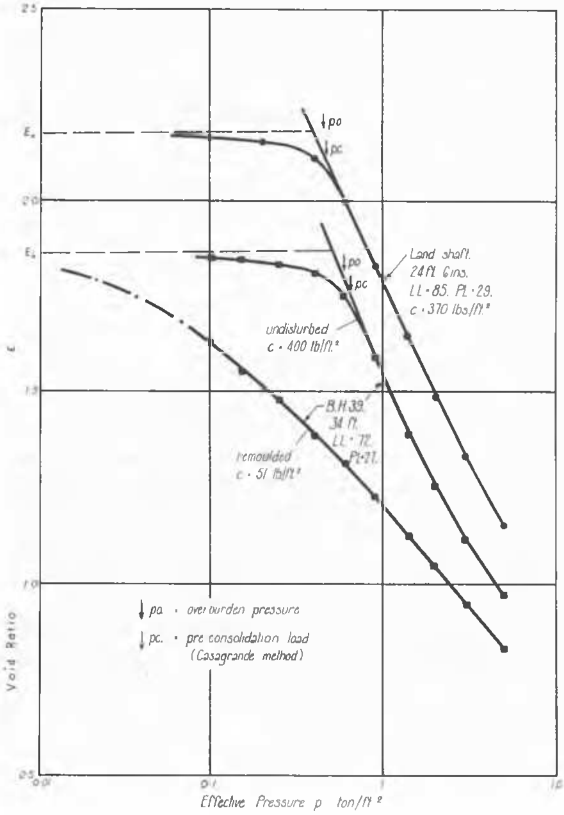

Fig. 7 Shellhaven Clay
Argile de Shellhaven

in Fig. 4 for the marsh borings at Tilbury. The average values for each stratum at Tilbury, as determined from all thirteen borings, are quoted in Table 2 and, for ease of reference, the Shellhaven values are given in Table 1.

It is clear from Fig. 3 that the ratio $(c/p)_n$ is different in the various strata and that, in general, the ratio is higher the greater the liquid limit or plasticity index. The values of $(c/p)_n$ are plotted against PI in Fig. 8 and 9 and a reasonably close correlation is seen to exist. In Fig. 9 published values of $(c/p)_n$ from five other sites in normally consolidated clays[1]) are plotted and these agree with the data derived from the Thames estuary clays. The points representing the peats at Tilbury and Shellhaven have been given in this Figure, and they appear to lie on the same correlation as the points for the clays and silt. But this may be fortuitous.

[1]) Data for these sites will be found in the following references:— Gosport—*Skempton* (1948a); Grangemouth, Fens and Koping—*Skempton* (1948b); Horten—*Hansen* (1950).

The Ratio $(c/p)_n$ from Consolidated-Undrained Tests

It has been suggested by *Taylor* (1948, p. 397) that the laboratory test giving the closest approximation to the true in-situ strength of clays is that in which the undisturbed sample is consolidated under the in-situ pressure and then sheared without further change in water content. This procedure has been questioned by *Terzaghi* (1947) and by *Hansen* and *Gibson* (1949). In Fig. 8 the results of eight standard consolidated-undrained tests are plotted, and it will be seen that, except for samples with a *PI* of about 80, the values of $(c/p)_n$ obtained

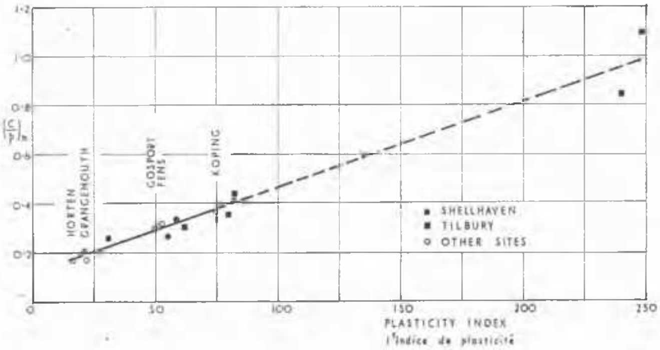

Fig. 8 Relation Between $\left(\frac{c}{p}\right)_u$ and Plastic Index

Relation entre $\left(\frac{c}{p}\right)_n$ et l'indice de plasticité

from these tests are appreciably greater than the "field" values obtained from vane tests and undrained triaxial tests on undisturbed samples. For samples with $PI = 55$ the error is of the order 30 per cent, and fragmentary data from other sites suggest that the over-estimate of strength from consolidated undrained tests is even greater for clays of lower *PI*.

In nature, clays are anisotropically consolidated under vertical and horizontal effective pressures p and K_0, where K_0 is the coefficient of earth pressure at rest; and in normally con-

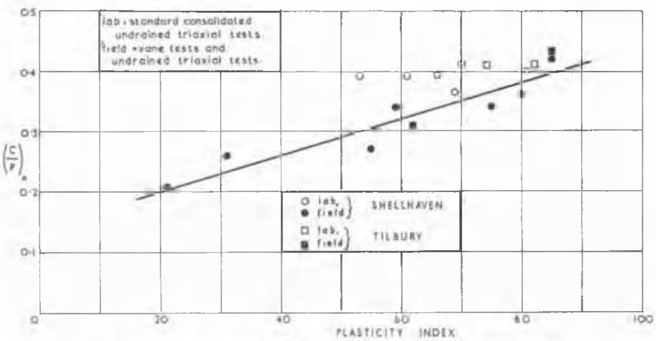

Fig. 9 Difference Between $\left(\frac{c}{p}\right)_n$ Measurid by Consolidated-Undrained Tests and the insitu Values

Différence entre les values $\left(\frac{c}{p}\right)_n$ obtenues dans les essais laboratoire et celles obtenues insitu

solidated clays K_0 is usually less than unity. Experiments have shown that if a clay is anisotropically consolidated, with $K_0 < 1$, the value of $(c/p)_n$ is less than in the standard consolidated-undrained test where K_0 is made equal to unity. Therefore the differences between the results of these latter tests and the "field" values of $(c/p)_n$ may be due, in part, to the possibility that K_0 is least in silts and silty clays and increases with increasing *PI*. Moreover, it is also possible that in those strata at Shellhaven and Tilbury with a *PI* of about 80 the value of K_0 may be approximately 1.0. Another cause of the differences may be the presence, in undisturbed materials of low *PI*, of a meta-stable silt structure (*Terzaghi*, 1947 and 1952).

Consolidation Characteristics

The coefficient of consolidation c_v was obtained by the $\sqrt{t}$ fitting method described by *Taylor* (1948, p. 239) and the average results for the various strata at both sites are given in Tables 1 and 2. The compressibility was calculated in accordance with the expression

$$m_v = \frac{\Delta e}{(1+e)\,\Delta\sigma}$$

where Δe is the decrease in void ratio corresponding to an increase in pressure $\Delta\sigma$ from the effective overburden pressure p to a pressure $(p + 2c)$, c being the shear strength of the sample under test. In Table 1 and 2 the modulus of compressibility $K_v = 1/m_v$ is expressed as a ratio of the shear strength c, and the average values of this parameter are given for each stratum. Typical $p-e$ curves are given in Fig. 7.

Acknowledgments

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