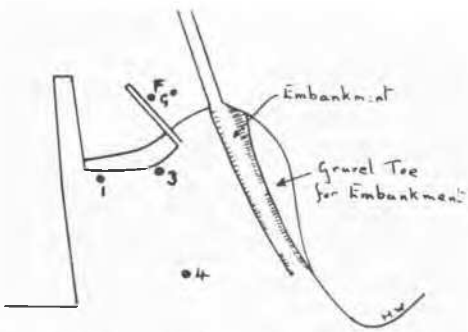
II a 2 THE GEOTECHNICAL PROPERTIES OF A DEEP STRATUM OF POST-GLACIAL CLAY AT GOSPORT

A.W. SKEMPTON M.Sc., A.M.I.C.E., F.G.S.
University Reader in Soil Mechanics and Assistant Proffessor at Imperial College, University of London.

1. INTRODUCTION.

In 1942 following the collapse of a steel sheet pile wall in the Royal Clarence Yard at Gosport, the Admiralty requested the Building Research Station to report on the causes of failure and to advise on the re-design. Three borings were made, in the positions shown in Fig. 1. and they revealed



Key Plane

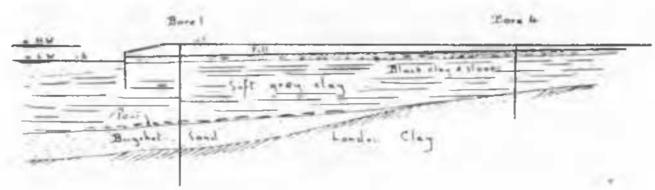
FIG. 1

about 60 ft. of soft, post-glacial clay,overlying the Bagshot Sands and London Clay. A number of undisturbed samples of the soft clay were obtained and tested in the laboratory. The results show that this stratum consists of a normally consolidated clay (as defined by Terzaghi 1941) in which the shear strength and preconsolidation load increase with depth in a regular manner. This increase in strength and pre-consolidation load would be expected from general soil mechanics principles, yet the author does not know any other published examples giving complete data on a normally consolidated clay stratum.

In contrast, however, there have been descriptions of several cases where a bed of clay which might be expected to be normally consolidated has shown no regular increase in strength with depth (Terzaghi 1941, Casagrande 1944 (a) and 1944 (b)).
Consequently the present results are published in order to show that the expected behaviour of a normally consolidated clay can be observed in nature. If this conclusion is accepted it follows that the other published results relate to strata which are not true normally consolidated clays, and that there must be a reason for their apparently anomalous behaviour. Casagrande (loc.cit) has suggested one possible reason, involving the drying of the clays to depths of 30 or 40 ft. below their surface. Whether this is correct or not, the Gosport clay at least shows the necessity for some such explanation of the anomalous strata, and discounts the assumption that an approximately constant shear strength with depth is the standard behaviour pattern of geologically recent clays.

2. GEOLOGY.

From the borings made in the Royal Clarence Yard and from the records of several wells in the neighbourhood the geological section shown in Fig. 2 has been prepared.The



Geological Section

FIG. 2

Bagshot Beds and the London Clay are of Tertiary Age. Overlying these deposits there is a thin bed of peat. A sample of this peat, from Borehole No.3 was submitted to Dr. H. Godwin F.R.S. for pollen analysis. He found (Godwin 1945) that the peat was of Pre-Boreal age; Zone IV in the British classification. At this period, about 9,000 or 10,000 years ago, the sea was substantially lower than at present. Various submerged peats around the coast show, however, that shortly after Zone IV peat was formed , a comparatively rapid rise in sea level took place and the soft clays found in many of the estuaries were deposited during this period. The marine transgression was probably almost complete by the end of Boreal times, roughly 7000 years ago; since when only minor oscillation have occurred, with a general slight rise in sea level. Evidence from a peat surface at Amberley Wild Brooks, not far from Gosport,(Godwin 1943) suggests that the transgression has not increased during the past 2000 years.

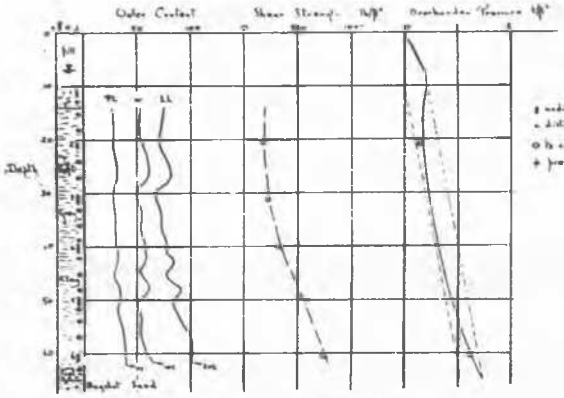
The Gosport clay is therefore post-glacial in age; deposited mainly if not entirely during the Boreal transgression.
Therefore there can be no doubt that it is fully consolidated under its own weight. In about 1890, however, a fill was placed on the clay, in the area of the borings, to + 6 O.D. and in 1936 the fill was raised by another two feet.

3. BORINGS.

The boring and sampling operations were carried out by Messrs.Soil Mechanics Ltd.,under the supervision of Mr. H.J.B.Harding and the author. The boreholes were 6 ins. diameter, lined with casing tubes. Undisturbed samples were taken at the depths shown in Figs. 3,4 and 5 with the tool described by Cooling (1946). This has a diameter of 4⅛ in. and an area ratio of 23%. Visual examination showed that distortion of the samples was limited to the outer 1/10 inch, although some slight disturbance of the clay is inevitable in sampling owing to the change in stress conditions.

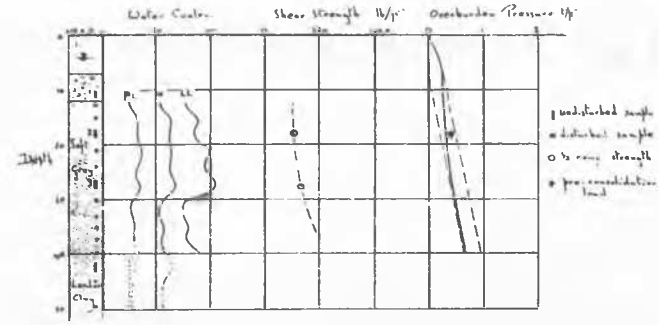
In addition, small samples were taken at about every 2 or 3 ft. in depth, for determination of water content and Atterberg Limits.

The author also took two samples of the clay (samples F and G) where it was exposed on the foreshore, at low water mark. Here the



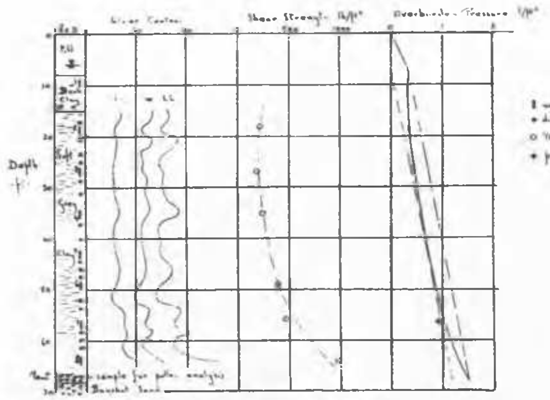
Bore no. 1

FIG. 3



Bore no. 4

FIG.5



Bore no. 3

FIG. 4

Table I.

	Average	Range
Water content w	56	46 - 73
Liquid Limit LL	80	62 -127
Plastic Limit Pl	30	25 - 50
Consistency index $\frac{LL - w}{LL - PL}$	C.48	0.31 - 0.73

In Boreholes 1 and 3 there is a general tendency for the Atterberg Limits to increase with depth, but the water content remains essentially constant. The consistency index therefore increases with depth. In Borehole 4 no general drift can be seen. The variations in water content are reflected by variations in the Atterberg Limits to a very marked extent. The relation between liquid limit and plasticity index, except for a few of the high liquid limit clays, is typical of that given by Casagrande (1947) for inorganic clays.

5. PARTICLE SIZE.

Mechanical analyses were carried out on four samples, all from Borehole No.1. The results are given in Table II.

TABLE II

Sample	Liquid Limit	Fine sand .2-.06 mm	Silt .06 - .002 mm	clay <.002 mm	H_2O_2 loss	H Cl loss
1 - 2	71	2.9	48.3	42.6	5.9	0.3
1 - 3	65	6.1	44.7	37.6	5.4	6.2
1 - 5	82	0	27.8	50.9	2.6	18.7
1 - 6	98	0.3	22.9	59.0	9.7	8.1

clay was very soft, and the samples were obtained by carefully pressing into the clay a thin walled tin tube 5 ins. diameter and 15 ins. long. This tube had an area ratio of only 4% and the samples were not visibly disturbed in any way.

4. ATTERBERG LIMITS AND WATER CONTENT.

Water contents were determined on every test specimen and on each of the small samples. Atterberg Limits were determined on at least two and in some cases on every test specimen cut from the undisturbed samples and on each of the small samples.

The results are plotted in Figs.3,4 and 5. Average values and total ranges are given in Table I (excluding the foreshore samples).

These results show a definite correlation between liquid limit and clay fraction, typical of marine clay (Cooling 1946). They also show that the liquid limit is influenced more by the clay fraction than by the organic content (H_2O_2 loss)

6. MINERALOGY.

The four samples on which mechanical analyses had been made were submitted to Dr. Nagelschmidt for X-ray analysis. For the whole clays he found chiefly quartz and calcite, the calcite content corresponding roughly with the HCl loss given in Table II. For the clay fraction all the samples were similar and consisted of 50 - 70% Illite and 30 - 50% Halloysite. There was a general and low-angle scattering due either to organic matter or amorphous inorganic material. No montmorillonite was found.

7. SPECIFIC GRAVITY.

Specific gravity determinations were carried out on seven samples. The average value was 2.67 with a range of 2.62 to 2.70.

8. OVERBURDEN PRESSURE.

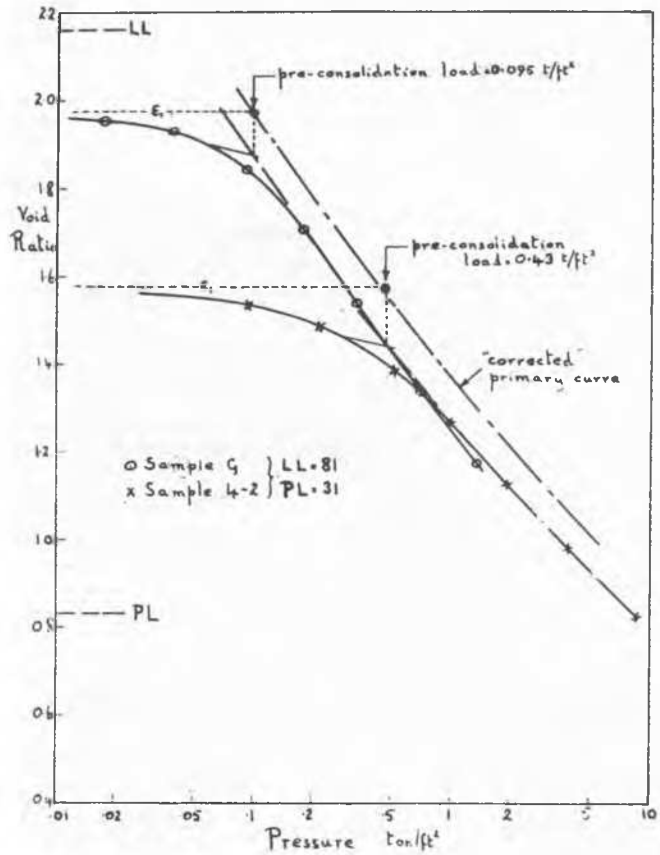
In Figs. 3, 4 and 5 the effective overburden pressure before placing the fill is shown as a dotted line. The effective overburden pressure, if consolidation under the fill had been complete, is also shown. In calculating these pressures the following densities have been adopted.

fill above G.W.L.		105 lb/ft^3
fill below G.W.L.		65
black clay and stones	)below	47
grey clay	)G.W.L.	42

Consolidation under the fill, at the time of sampling, cannot be complete, however, and the effective overburden pressure has therefore been calculated from the theory of consolidation (Tarzaghi and Frohlich 1936). Tests showed a reasonably constant coefficient of consolidation equal to 0.004 cm^2/min, and using this value it is an easy matter to estimate the effective pressure at any depth. The results are shown by the full lines in Figs. 3, 4 and 5.

9. PRE-CONSOLIDATION LOAD.

Consolidation tests were carried out on five samples from the boreholes and on the two foreshore samples. Representative p - ε curves are shown in Fig. 6. These are typical of undisturbed soft clays with a moderately high initial water content and an estimate of the pre-consolidating load can be found by the graphical construction suggested by Casagrande (1936). In Table III these values are compared with the calculated effective overburden pressure; and also in Figs. 3. 4 and 5, they are plotted on the same graph as that relating overburden pressure to depth. It will be seen that a reasonable agreement is obtained, probably within the range of accuracy of the construction and the inevitable slight disturbance in sampling.



Consolidation Test Results

FIG. 6

The two foreshore samples gave definite pre-consolidation loads, which are assumed to be an indication of the capillary pressures existing in the clay at a depth of about 8 ins.

The correlation between calculated and experimental pre-consolidation loads for the bore-hole samples is good evidence for the conclusion that the stratum is a true normally consolidated clay.

10. DECREASE IN VOID RATIO WITH OVERBURDEN PRESSURE.

Owing to the variations in liquid limit

TABLE III

Sample	Depth	Consistency				Calculated Overburden Pressure	Estimated Overburden Pressure (Casagrande)
		W	LL	PL	$\frac{LL - w}{LL - PL}$		
F	7 ins.	77	75	29	- .05	-	0.093 t/ft^2
G	9 ins.	74	81	30	+ .14	-	0.095
1 - 2	21 ft.	54	69	25	.34	0.37 t/ft^2	0.28
1 - 6	61	60	100	36	.62	1.24	1.22
3 - 2	19	52	71	28	.44	0.34	0.30
3 - 7	56	60	86	33	.49	1.05	0.84
4 - 2	17	59	81	32	.45	0.32	0.44

with depth a plot of water content (or void ratio) against overburden pressure would be of no value. Consequently the consistency index (LL-w)/(LL-PL) was calculated for each sample and the water content was then found which gave this same consistency index with average liquid and plastic limits of 80 and 30 respectively. In this way the variations were smoothed out.

The void ratio (water content x 2.67/100) calculated in this manner is plotted against overburden pressure in Fig. 7. As would be expected there is a considerable scatter, but the general decrease in void ratio with increasing pressure is quite definite.
The results are in good agreement with data on other clays of a similar type (Skempton 1944).

Fortunately sample G from the foreshore and sample 4-2 both had a liquid limit almost exactly equal to the average value of 80, and results of the consolidation tests on these samples can therefore be compared directly with the data in Fig. 7.

Referring to Fig. 6 it will be seen that a "corrected" primary p - ε curve has been drawn through the two points corresponding to the natural void ratio and pre-consolidation load, and parallel to the experimental primary curve. This represents an attempt to obtain, as far as a laboratory test is concerned, a true primary p - ε curve for the clay. The "corrected" curve is plotted in Fig. 7 and it is, rather surprisingly, found to agree well with the observed data. It is considered that this result constitutes a second indication that the clay is normally consolidated.

The curves in Fig. 6 are also of interest in that they show that the primary curve of Sample 4-2, which was taken from a depth of 17 ft., is an almost exact continuation of the primary curve of sample G. A comparison of this nature is not often possible, owing to the rare chance of finding two samples of the same stratum with the same liquid limit and with such widely different pre-consolidation loads.

In contrast a p - ε curve obtained from a consolidation test carried out on the clay, remoulded with an initial water content equal to the liquid limit, lies well below the point.

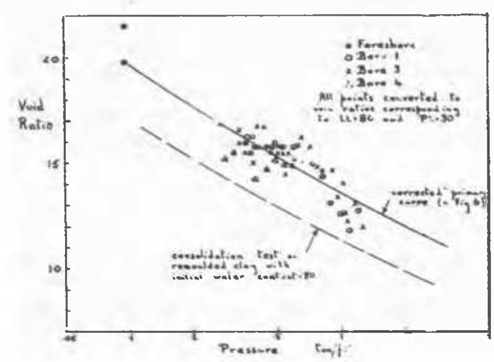
These results show that a test made on the undisturbed clay, in the natural state in which it was deposited, corresponds closely to the actual consolidation under pressure in the ground, and that its structure is not seriously affected by the test.

11. SHEAR STRENGTH.

Triaxial compression tests carried out under different lateral pressures with no water content change during the test showed an angle of shearing resistance equal to zero. Typical results are given in Table IV.

TABLE IV

Sample 3 - 2 LL = 82 PL = 29			Sample 4 - 3 LL = 107 PL = 36		
σ_{III}	$\sigma_I - \sigma_{III}$	w	σ_{III}	$\sigma_I - \sigma_{III}$	w
0 lb/in^2	2.90 lb/in^2	61	0 lb/in^2	4.7 lb/in^2	70
20	2.94	58	10	4.3	68
40	2.88	59	20	5.1	73
			30	4.8	68



Pressure-Void Ratio Relationship.

FIG.7

Shear strength is therefore equal to one-half compression strength and it was determined by carrying out unconfined compression tests on at least two specimens from each sample. In about half the samples the compression strengths of the clay after remoulding was also measured. The results are given in Table V. Remoulding index equals the ratio of remoulded to undisturbed strength.

In Fig. 8(a) shear strength is plotted against calculated overburden pressure and, in Fig. 8(b) against pre-consolidation load. In both cases there is an approximately linear relationship, represented by the equation

$$c = p. \tan 15^\circ = 0.27\ p.$$

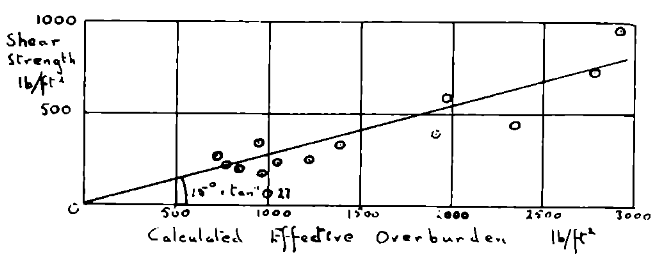
where 15° is the angle of shearing resistance of the clay.

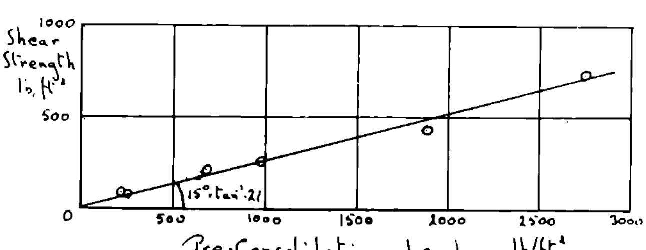
This linear relationship is a third indication that the clay is normally consolidated. So far as the author is aware, Fig. 8 is the first published information on the increase in strength with overburden pressure obtained from field data on a thick stratum of clay.

The variation of strength with depth is shown in Figs. 3,4 and 5. The increase with depth is quite obvious, and is especially apparent when it is remembered that the foreshore samples had a strength of only about 100 lb/ft^2.

TABLE V

Sample	Consistency				Calculated Overburden Pressure lb/ft²	Pre-con-sold Load lb/ft²	Shear Strength lb/ft²	Remoulding Index
	w	LL	PL	$\frac{LL-w}{LL-PL}$				
F - 1	75	73	27	-.06	-	-	70	-
F - 2	77	75	29	-.05	-	210	90	-
F - 3	72	77	27	+.10	-	-	120	0.23
G	73	74	28	+.02	-	210	85	0.24
1 - 2	55	73	29	.41	830	630	190	0.47
1 - 3	49	66	27	.44	1050	-	225	0.51
1 - 4	57	76	30	.41	1390	-	325	0.41
1 - 5	51	82	34	.65	1970	-	595	0.45
1 - 6	62	98	40	.62	2780	2730	735	0.51
3 - 2	58	82	29	.45	760	670	210	-
3 - 3	55	77	27	.43	960	-	160	-
3 - 4	62	94	33	.52	1230	-	245	-
3 - 6	50	71	28	.49	1910	-	390	-
3 - 7	55	86	33	.58	2350	1880	440	-
3 - 8	70	127	50	.74	2920	-	975	-
4 - 2	63	96	35	.54	710	980	260	.43
4 - 3	69	107	36	.53	940	-	340	.40





Relation between Shear Strength and Consolidation pressure.

FIG. 8

The effect of remoulding seems to decrease with increasing strength. For the very soft foreshore samples the remoulding index is about 0.25, while for the deeper samples taken in the borings the index is about 0.45. It will at once be asked whether this is not simply due to disturbance during sampling from the borings. Some slight disturbance is, of course, inevitable but the stability calculations on the embankment failure, described in the following section, show that the drop in strength in the borehole samples is unlikely to be more than 10 per cent.

In the author's opinion the true relation between shear strength and overburden pressure allowing for this possible disturbance may be

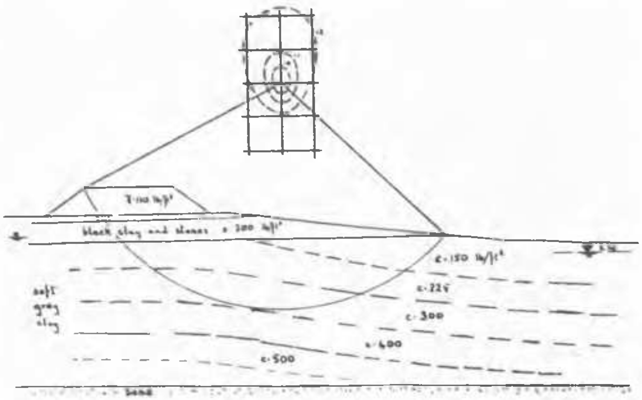
$$c = p.\ \tan 17^{\circ} = 0.30\ p.$$

but is it improbable that the angle of shearing resistance could be appreciably higher.

12. EMBANKMENT FAILURE.

The embankment shown in Fig. 1 and in Fig. 9 was built in 1910 to a height of 8 ft. But immediately on reaching this height it subsided catastrophically. It was rebuilt successfully by forming a gravel toe simultaneously with the formation of the bank.

In Fig. 9 the contours of shear strength have been drawn from the data in Fig. 8, and an analysis of stability has been made on the



Ø = 0 Analysis of Embankment Failure
Min. Factor of Safety = 0.93
γ for strata above L.W. = 105 lb/ft^3

FIG. 9

Ø = 0 assumption (Skempton 1948). It will be seen that the minimum factor of safety is 0.93. The correct value should be 1.00 and it is therefore possible that the average strength of the clay has been underestimated by about 7 per-cent. This can probably be attributed to slight disturbance of the samples, as mentioned above, but in general the evidence provided by this failure lends support to the values of shear strength as measured in the laboratory, and the maximum effect of disturbance is probably not in excess of 10 per-cent.

13. SHEET PILE WALL FAILURE.

The analysis of the sheet pile wall was rather complicated but the result is in agreement with this conclusion.

14. CONCLUSIONS.

a) The following results show that the deep stratum of soft, post-glacial, clay at Gosport is normally consolidated.

1) the experimentally determined pre-consolidation loads are of the same order as the effective overburden pressures throughout the depth of the stratum

2) the primary p-ϵ curve derived from consolidation tests on the clay as deposited in its natural state agrees with the p-ϵ relation deduced from overburden pressures and water contents on samples taken from the boreholes.

3) shear strength is directly proportional to overburden pressure, and hence also to pre-consolidation load.

b) Shear strength c, corrected for possible disturbance on sampling, and overburden pressure p are related by the expression $c = p \tan 17^\circ = 0.30\, p$, where 17° is the angle of shearing resistance of the clay.

c) The clay is moderately sensitive to remoulding, the sensitivity decreasing with increasing strength.

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