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sibility of mud flow through the tracks into the suspension.

Scaling pressure. For the square plates on both soils Figs. 16 (b), (d) and 19 (b) show that there is a narrow pressure range for the three plates where scaling holds. This is the region where the three curves intersect and is about 7.3 lb./sq.in. (about 8 lb./sq.in. if the results are corrected for machine inertia) for the clay and about 7.7 lb./sq.in. for the brick-earth. The curves of sinkage against plate pressure (see Figs. 16 (a), (c) and 19 (a)) are roughly parallel so that within the range of pressures and plate sizes studied, model tests could be used to reveal qualitative differences in plate performance.

The scaling pressure appears to be less well defined for the plates carrying a spud as the presence of spuds adversely affects the consistency of the results. Figs. 20 and 21 show that although in general the sinkage is less with the plate carrying a spud than with a plain plate, in some cases it is greater. A further inconsistency appears in the curves for 4 lb./sq.in. loading (Fig. 20) where the curve for the 8-in. plate with a 4-in. spud crosses that for the 6-in. plate with a 3-in. spud.

Estimated bearing capacity. A comparison of the estimated basic bearing capacity and perimeter shear values for the two soils with their measured cohesions and angles of internal friction shows that the basic bearing capacity is predominantly influenced by the angle of internal friction while the perimeter shear is predominantly influenced by the cohesion. This suggests that the effectiveness of open-work tracks and spuds would be expected to be greater in cohesive than in non-cohesive soils. However, there is an unknown factor - "arching", to be considered. This factor would be expected to offset the perimeter shear deficiencies of an openwork track in non-cohesive soils by giving a pressure distribution almost as uniform as that for a plain plate.

Conclusions. See Summary.

ACKNOWLEDGEMENTS.

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SOME LABORATORY TESTS ON CHALK

GUTHLAC WILSON, S.M., B.Sc.

A site exploration was undertaken in 1946 at Messrs. Reckitt & Colman, Ltd., 's Carrow Works at Norwich. As a result of this exploration, it was found possible to divide the site into five areas, which differed from one another from the point of view of the construction of foundations. These five areas are outlined in figure 1. The sequence of strata in the respective areas is given below:

Area A, along the river Wensum

	Stratum	Average thickness
Sub-Area (i)	Fill	10 ft.
	Peat	6 ft.
	Gravel	8 ft.
	Medium Chalk	- at least 100 ft.

Sub-Area (ii)

As sub-area A (i), but the chalk is very soft to a considerable depth.

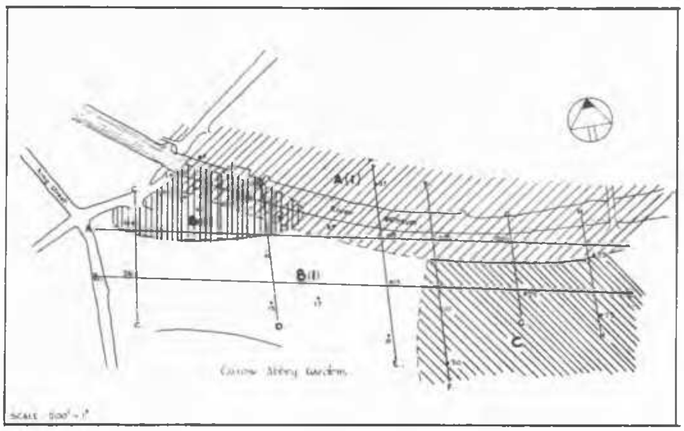
Area B

Sub-Area B (i) Fill av. 6 ft.
Thin layer of gravel in some places.
Medium chalk - at least 100 ft.

Sub-Area B (ii)

As sub-area B (i) but the chalk is very soft to a considerable depth.

Area C Fill av. 6 ft.
Gravel 20 + ft. to 50 + ft.
Presumably medium chalk below gravel, but not confirmed by boreholes.



Foundation conditions at Carrow works,Norwich

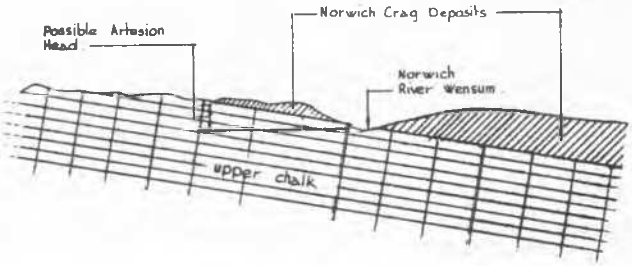
FIG.1

The chalk varied in colour from pure white to orange yellow and was lumpy, the interstices between the lumps being completely filled with a smooth, plastic, chalk paste. The size of the "lumps" varied from quite large pieces down to grains of the size of coarse sand.

In this area the chalk is overlain by a deposit which varies considerably, containing beds of sand, laminated clays and pebbly gravels. This deposit is known locally as Norwich crag. Its full extent is about 40 miles in a north-south direction and 20 miles east-west.

At Norwich the River Wensum has cut into the crag and exposed the chalk locally, at a point on its dip slope where it disappears under the crag. Consequently, slightly artesian head of water might be expected in this chalk. Since Norwich is so near to the main chalk outcrop the artesian head is not likely to be very great.

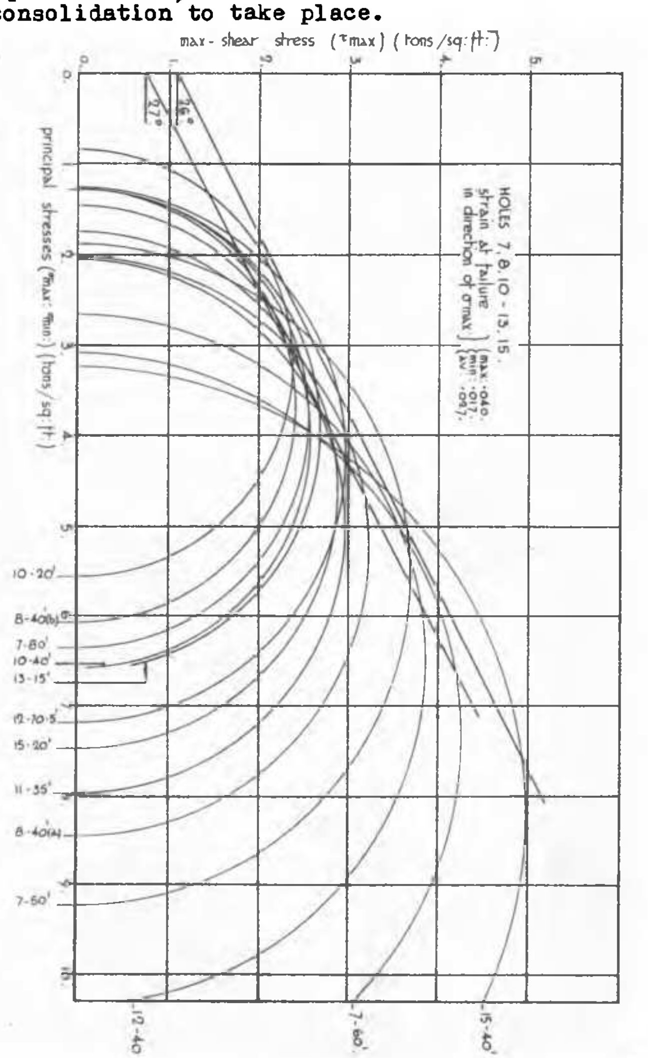
A sketch of the geological conditions is given in figure 2.



Sketch of geological conditions at Norwich.

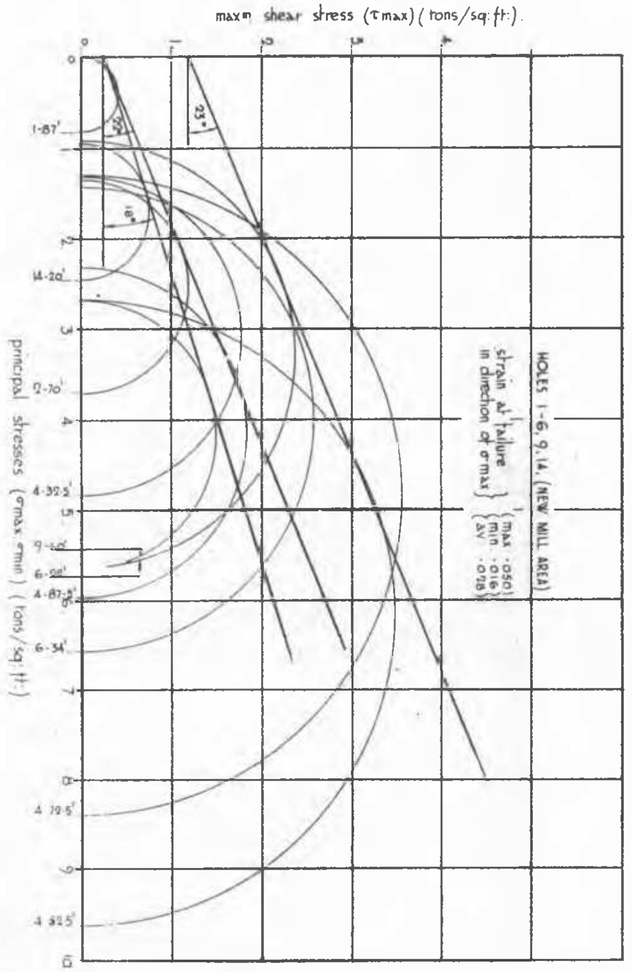
FIG.2

A fairly extensive series of tri-axial compression tests was carried out on undisturbed samples of the medium and soft chalks. The results of these tests are shown in Table I and on figures 3 and 4 respectively. All tests were "quick" tests, i.e. time was not allowed for consolidation to take place.



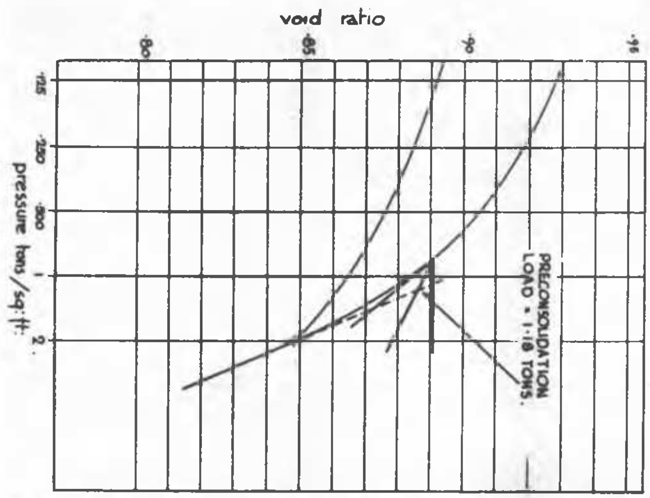
Mohr circle diagrams, showing results of tri-axial tests.

FIG.3



Mohr circle diagrams showing results of tri-axial tests.

FIG.4



Void ratio-pressure curve of sample from borehole no. 1. Depth 87'-6"

FIG.5

As a result of these tests it was concluded that the mechanical properties of these two grades of chalk were as given in Table II.

The low value of the strain at failure indicates that the ultimate bearing capacity will be determined by general shear failure. 1)

TABLE I

RESULTS OF TRIAXIAL TESTS ON CHALK

Hole No.	Depth	Conditions at Start of Test			Conditions at Failure					Angle of Internal Friction
		Vert. pressure σ_1	Lateral Presuure σ_{III}	Water content	Vert. Pressure σ_I	Lateral Pressure σ_{III}	Max. Shear Stress τ_{max}	Water content	Strain	φ
	ft.	T/ft.^{2}	T/ft.^{2}	%	T/ft.^{2}	T/ft.^{2}	T/ft.^{2}	%	%	
1	87	0	0	30.5	0.79	0	0.40	-	-	18°
14	20	0.94	0.94	-	2.48	0.94	0.77	-	-	
9	60	2.67	2.67	29.4	5.67	2.67	1.50	29.8	2.0	
2	70	1.35	1.35	29.9	3.7	1.35	1.18	31.8	4.1	
4	32.5	1.31	1.31	33.0	4.86	1.31	1.77	31.8	2.6	22°
4	87.5	2.34	2.34	31.2	5.94	2.34	1.80	28.4	2.0	
6	22	0.92	0.92	-	5.64	0.92	2.36	-	-	
6	34	1.45	1.45	33.0	6.57	1.45	2.56	31.8	-	
4	72.5	1.31	1.31	30.0	8.39	1.31	3.54	28.2	1.6	23°
4	52.5	2.67	2.67	31.6	9.63	2.67	3.48	30.3	2.1	
7	50	1.87	1.87	28.8	9.21	1.87	3.67	27.5	2.0	
7	60	3.11	3.11	29.7	11.59	3.11	4.24	29.1	1.9	
7	80	1.31	1.31	26.8	6.33	1.31	2.51	29.9	3.0	
8	40(a)	2.05	2.05	29.6	8.43	2.05	3.19	28.9	3.7	
8	40(b)	1.31	1.31	30.6	6.05	1.31	2.37	29.4	2.0	
10	20	0.87	0.87	30.8	5.53	0.87	2.33	28.2	2.2	
10	40	1.45	1.45	31.75	6.59	1.45	2.57	30.5	4.0	26-27°
11	35	2.05	2.05	31.1	7.97	2.05	2.96	28.6	3.0	
12	40	2.67	2.67	33.2	10.35	2.67	3.84	33.4	2.7	
12	70.5	1.31	1.31	27.6	7.17	1.31	2.93	28.9	3.6	
13	15	1.31	1.31	24.1	6.57	1.31	2.63	23.5	2.6	
15	20	1.75	1.75	33.5	7.49	1.75	2.87	31.1	2.4	
15	40	3.25	3.25	32.3	13.15	3.25	4.95	30.5	1.7	

TABLE II

Mechanical Properties of Chalk from Norwich

Type of Chalk	c, cohesion, tons p.s.f. (min.)	φ, Angle of internal friction, degrees (min.)	Strain at failure
Medium	0.75	26°	.017 to .040 av. .027
Soft	0.25	18°	.016 to .050 av. .028

RESULTS OF CONSOLIDATION TESTS ON CHALK.

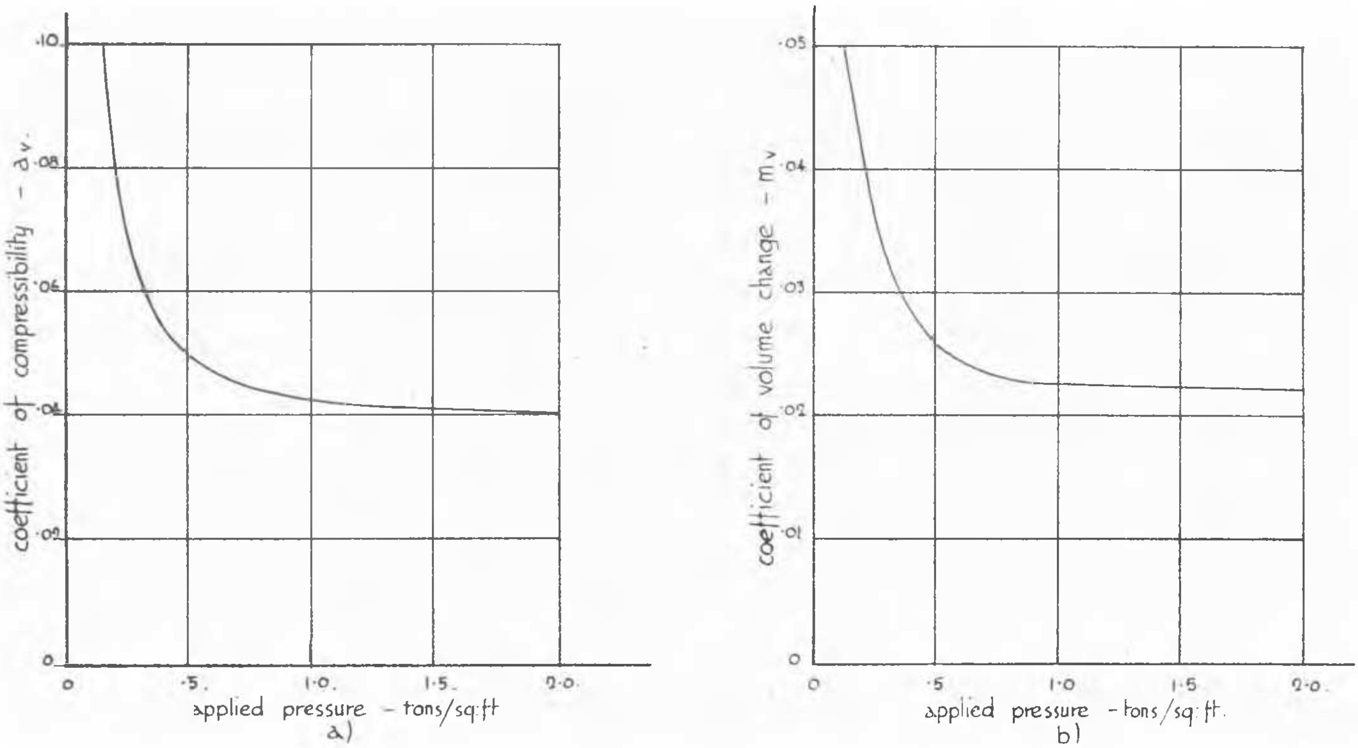


FIG.6

Atterberg Limit tests were carried out on the sample taken at a depth of 87.5 feet from borehole No. 1 and on a sample from a depth of 80 feet in borehole No. 7. The results were:

Plastic Limit 20.8% to 24.4%
Liquid Limit 27.4% to 27.7%

It will thus be seen that the mechanical and consolidation properties of these chalks are similar to those of medium to weak, but short, clays.

The specific gravity of the solids was found to be 2.78.

It is interesting to note that 13 inch square precast concrete piles were driven to depths of up to 69 ft., i.e., 46 feet into the chalk, in the vicinity of borehole No. 7.

A consolidation test was carried out on a specimen cut from the sample taken at a depth of 87'6" in borehole No. 1. The (p, e) curve is shown in figure 5 and the values of the coefficients of compressibility and volume change are shown in figure 6.

The effective pressure due to the weight of overburen above the sample was 2.3 tons p.s.f., whereas the pre-consolidation load, as determined by Casagrande's construction on figure 5, is only 1.2 tons p.s.f. This fact would appear to indicate that there is an excess pressure in the intersticial water due to the artesian head mentioned above.

The excess pressure in the intersticial water has, of course, an effect on the bearing capacity. The effective pressure between the grains of the soil is reduced by the seepage pressure.

The seepage pressure acting upwards on the elementary volume ABCD, figure 7, is:

$$J = h_2\gamma_w \cdot 1 - h_1\gamma_w \cdot 1 = (h_2 - h_1)\gamma_w$$

if γ_w is the unit weight of water.

If the hydrostatic excess pressure at

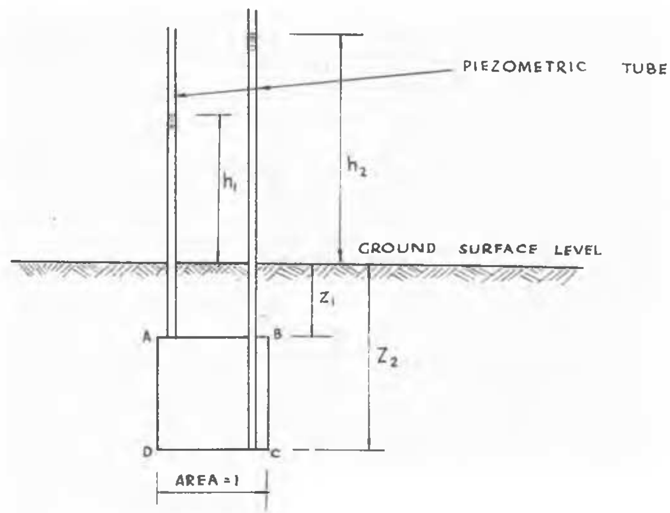


FIG.7

depth Z_1 is w_1 and that at depth Z_2 is w_2, we have:

$$w_1 = h_1\gamma_w \quad \text{and} \quad w_2 = h_2\gamma_w$$

$$\therefore \quad J = w_2 - w_1$$

If we assume that $\frac{w_1}{w_2} = \frac{Z_1}{Z_2}$ i.e. if we assume that the excess pressure varies uniformly as the depth from the surface, we have:

$$J = \frac{w_2}{Z_2}(Z_2 - Z_1)$$

The effective weight of the elementary volume ABCD is $\gamma_m'(Z_2 - Z_1)$

where γ_m' is the submerged unit weight of the soil.

Therefore, the nett increase of intergranular pressure between AB and CD is

$$\left(\gamma'_m - \frac{w_2}{z_2}\right)\left(z_2 - z_1\right)$$

and the effective unit weight of soil to be used in calculations of bearing capacity is:

$$\left(\gamma'_m - \frac{w_2}{z_2}\right)$$

i.e. the effective unit weight of the soil is reduced by w_2/Z_2.

In our case the reduction in effective unit weight is (1.1 x 2240) : 87.5 = 28 lbs. per cubic foot.

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THE INVESTIGATION OF DECOMPOSED GRANITE IN HONG KONG FOR USE AS A STABILISED BASE COURSE MATERIAL

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INTRODUCTION.

During the Japanese occupation of Hong Kong the only existing airfield, situated at Kai Tak, had been extended and improved by the construction of two concrete runways. However, on its re-occupation by the Royal Air Force in September, 1945, it was realised that the site, restricted in size and half-ringed by mountains, was unsuited to further development, and that a new site must be found if Hong Kong was to be provided with an airfield complying with P.I.C.A.O. standards. It was therefore decided to select and survey a new site, to prepare designs for a new airfield and to proceed with such ancillary works in connection with the project that could be executed without hindrance to the local population. It was also decided to improve the facilities at Kai Tak by providing additional hardstanding areas and taxi-tracks before the advent of the wet season in the following summer.

The authors, then on the Headquarters Staff of the Royal Air Force Airfield Construction Unit stationed in the Colony, were instructed to proceed with the design of pavements for the above works. They had at their disposal a mobile soils laboratory and a trained staff of assistants. The Construction Unit had adequate engineer plant for earthmoving and stabilisation work, but few concrete mixers, while supplies of cement and bitumen were then limited and their costs high. Accordingly it was decided to investigate the possibility of constructing stabilised base courses using decomposed granite, of which large deposits occurred in the Colony.

This paper summarises the results of this investigation, the authors' subsequent work being described in the paper entitled "The Incorporation of Decomposed Granite in the Design and Construction of Pavements in Hong Kong."

INVESTIGATION OF DEPOSITS.

Hong Kong is a mountainous region composed of igneous rocks, which in general lie within the range of granites, syenites and granodiorites. Most of these rocks decompose readily under the conditions of high temperature and rainfall, while others appear to be almost indestructable. The decomposition takes place 'in situ', and the decomposed material occurs generally as a carpet covering the mountains, pierced here and there by outcrops of sound rock. Large numbers of boulders of sound granite occur embedded in deposits of otherwise decomposed material. This indicates that the original rock varies in composition, the greater part being liable to decomposition while small portions are practically indestructable. These latter remained as boulders when decomposition of the main mass of rock took place. This theory is borne out by the local building tradition of using only granite from boulders when it is desired to obtain a facing stone which will not stain with age. However, the authors have found no complete explanation for the varying resistance of the rocks to weather-

Aerial view of decomposed granite foothills

PHOT. 1