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before construction, and other measures for maintaining high moisture content in the soil immediately around the building.

It will be observed that examination of the cracked buildings indicates an apparent downward movement of the corners of the structures. This would be the normal type of settlement to be expected if the shearing condition in the subsoil is the major contributing factor, and with small structures of the type considered, this condition would probably normally apply. The upward movements apparent from the levelling observations have however given the whole problem a new aspect and if, due to interference with the evaporation condition, a general rise in the line of saturation can be expected immediately under a newly erected building, it is most probable in the case of an expansive clay subsoil that

the structure will also exhibit a dome shaped rise. Cracking in these cases appears to have resulted from an apparent downward movement of the corners although the absolute movements may in general be upwards.

It will be our object to try and separate these two effects which show such apparently similar evidence. At the same time an attempt will be made to formulate simple rules for quick application by the practical builder. The investigation is necessarily a long term study and much of the work listed above has only just been started due to the recent establishment of the National Building Research Institute in South Africa. The Authors are particularly anxious to contact others who are faced with the same problems and exchange ideas and experience with those who have conducted similar work in this field.

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SOME SOIL RUPTURE AND SETTLEMENT PROBLEMS IN THE CONSTRUCTION OF A LOCK, AT DUFFEL

VI d 2

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INTRODUCTION.

The double lock of Duffel has been founded in the Boom Clay, which is a tertiary formation having in this region a thickness of more than 50 meters. Figure 1 gives a location plan of the lock, with its approach-walls. A cross-section through the lock and an other one through the approach-walls are shown in figures 2 and 3.

During the period of construction, abnormal movements occurred in some parts of the lock. For instance, the approach-walls indicated in figure 1 by shaded areas, and especially the walls 5L and 6L, have undergone important movements. At the joint between 5L and 6L, the following horizontal movements have been measured in March 1943:

at level	wall section	
	5L	6L
+ 7,00	21,3 cm	16,5 cm
- 0,20	2 cm	2 cm

These movements have reference to the vertical axis passing through the lower edge of the footing of the wall, which is located at the foundation-level - 4,00. It should be noted that against these walls the fill was completely placed.

Furthermore the walls and the floor of the locks have undergone very large vertical movements. After a general levelling of the top of the walls and of the floor of the locks, between the cross-sections $\alpha\alpha$ and $\beta\beta$ (large lock) and $\alpha'\alpha'$ and $\beta'\beta'$ (small lock), the following average vertical movements were found (see figure 1 and table I).

It may also be interesting to notice that some longitudinal cracks were observed in the floor of the lock; this floor consists of reinforced concrete beams which were shored up between the both walls of the lock. The intervals between these beams are protected with in-

TABLE I.

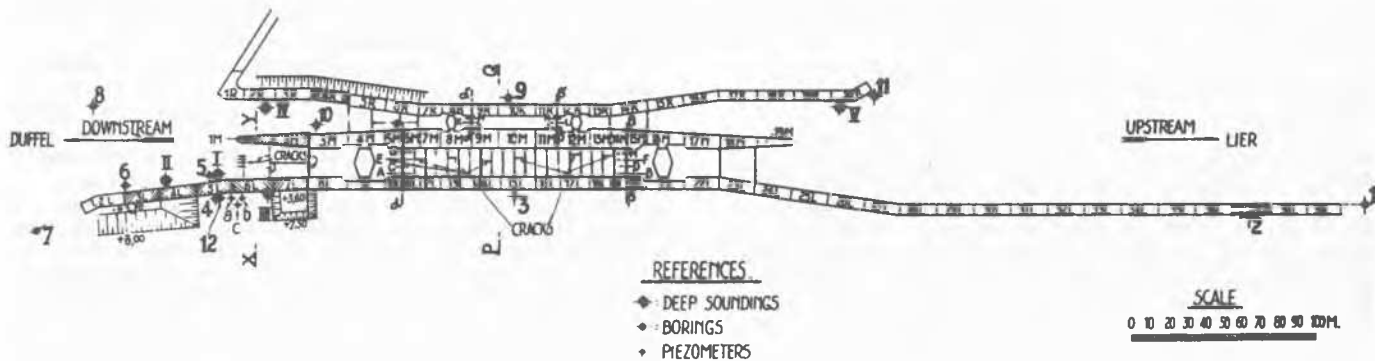
longitudinal sections	settlement cm	rising cm
<u>large lock.</u>		
left wall		
section AB	2	-
" CD	-	3,6
" EF	-	4,6
" GH	-	5,0
right wall	2,4	1,0
		-
<u>small lock.</u>		
left wall		
section IJ	2,3	-
" KL	-	0
" MN	-	1,0
right wall	2,0	1,6
		-

dependent concrete slabs, which had undergone an important upward movement in their median parts. Some of them show cracks. At the ends of the lock where the floor and the walls are constructed as one rigid body, no signs of movement were found. Cracks were also seen in the reinforced concrete beams of the apron on the downstream side.

After these difficulties, a thorough examination of the soil was ordered. A complete field investigation was carried out: a number of deep sounding tests and borings were made in the positions shown in figure 1. In the borings undisturbed samples were taken, and transported to the laboratory for further examination. Three piezometric tubes were placed in the backfill against the walls 5L and 6L, in order to permit the observation of the changes in the ground-water level.

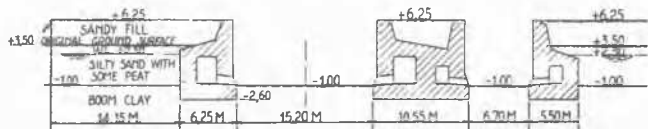
The profile of the soil, as revealed by the borings, can be approximated as follows:

- upper level of the fill : + 7,00
- natural ground surface and water level : + 3,50
- from + 7,00 to + 3,50 : sandy fill



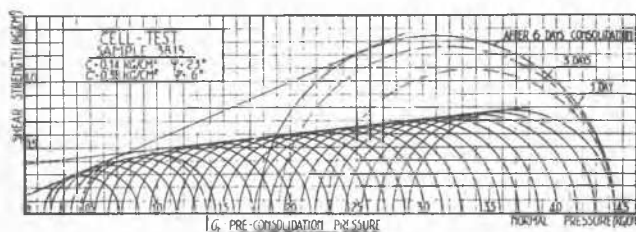
Plan of the lock at Duffel.

FIG.1



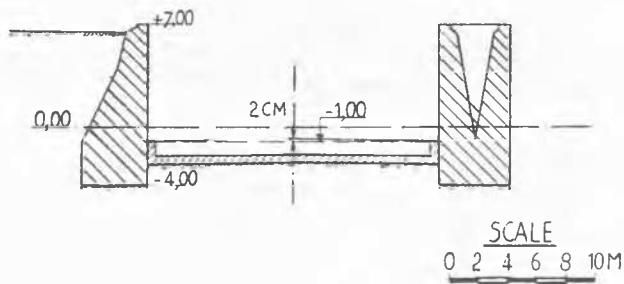
Cross-section P-Q through the lock.

FIG.2



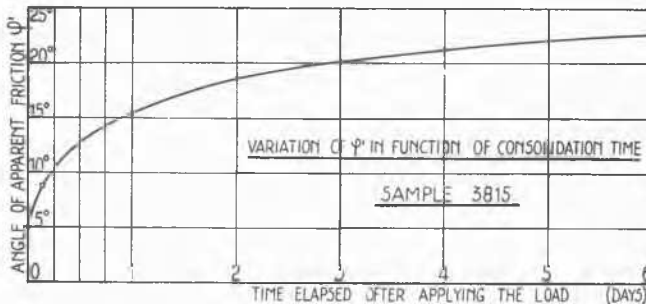
Cell - Test Sample 3815

FIG.5a



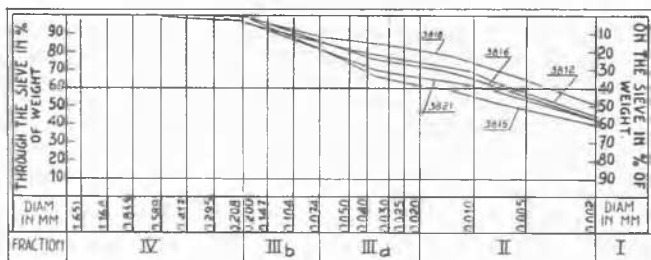
Cross-section X-Y through the approach-wall.

FIG.3



Variation of ϕ' in function of consolidation time. Sample 3815

FIG.5b



Granulometric size.

FIG.4

- from + 3,50 to - 1,00 : silty sand with some peat
- from - 1,00 to a great depth : very stiff tertiary clay (Boom Clay).

LABORATORY EXAMINATION.

The samples taken in the borings were submitted to a complete laboratory analysis. In the first place this analysis was directed

to a detailed study of the Boom Clay, as being the stratum in which the lock and its approach-walls are established. More than fifty undisturbed clay-samples, which were taken between the levels - 1,00 and - 4,00, were tested in order to determine the mechanical properties of the clay.

Some characteristics obtained for this clay are indicated in table II. In figure 2 some typical grain-size distribution curves are reproduced. The shear resistance of the clay was determined by testing the undisturbed samples in the triaxial compression apparatus. The result of such a test is expressed in figure 5a by Mohr circles of stress. From the shape of the normal pressure-shear strength curve, it was possible to estimate the "pre-consolidation pressure" σ_t , that is the pressure to which the clay had previously been exposed in the course of its geological history. The variation of the lateral pressure during the consolidation process was also observed. Figure 5b gives the relation between

the angle of apparent friction φ' and the time elapsed after applying the load. It can be seen that, owing to the low permeability of the clay, the consolidation progresses very slowly.

Consolidation tests were carried out, in the standard oedometer apparatus, on thirty-six undisturbed samples. A typical curve showing the volume reduction with increasing effective pressure is given in figure 6. Following Casagrande's empirical construction, the pre-consolidation load σ'_t has been determined. At last the results of a deep sounding test are given in fig. 7.

CORRELATION BETWEEN THE MECHANICAL CHARACTERISTICS OF THE BOOM CLAY.

The results of the different tests show that the Boom Clay had been pre-consolidated under a relatively considerable pressure. The triaxial compression tests indicate that the undisturbed samples taken between - 1,00 and -4,00, were subjected to a pressure of about $\sigma'_t = 1,4 \text{ kg/cm}^2$; in the consolidation tests a pre-consolidation pressure, with a mean value of $\sigma'_t = 1,5 \text{ kg/cm}^2$, was discovered.

On the other hand when using analytical formulas, (formulas 1 and 2) which give the

TABLE II

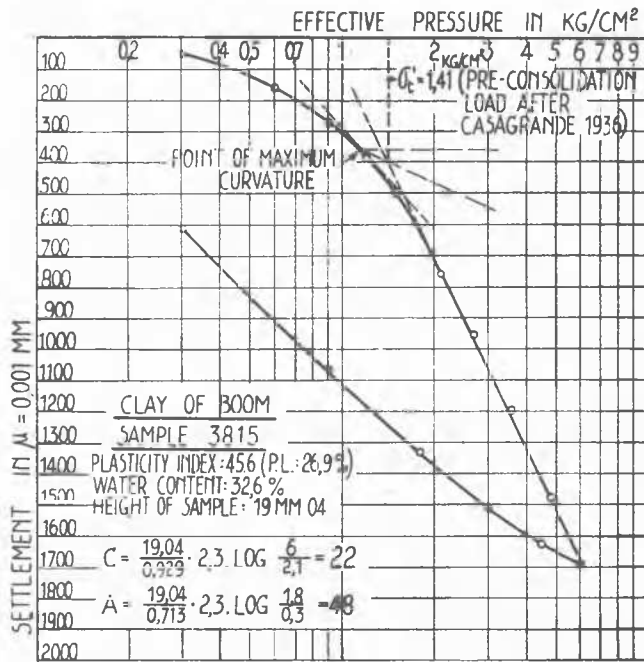
Property	Number of determinations	Maximum value	Minimum value	Mean value
<u>Physic properties</u>				
Liquid limit: % of dry weight	69	97,5	51	81
Plastic limit: % id	69	39,1	20,6	29
Plasticity index	69	66,6	27,5	52
Clay fraction (< 0,002 mm): % of total weight	59	57	31	44
Natural water-content: % of dry weight	62	39,7	17,25	29,9
Volume weight: t/m ³	62	2,040	1,765	1,933
Dry weight: t/m ³	62	1,661	1,266	1,497
Per centage of voids (for a specific gravity of particles: 2,65)	62	52,2	37,3	43,4
<u>Cone resistance</u> : in kg/cm ² .	51	29,8	6,3	14,6
<u>Shear constants</u> :				
angle of internal friction φ :	20	25°	18°	21°
angle of apparent friction φ' :	20	6°	3°	4°30'
true cohesion c: in kg/cm ² .	20	0,58	0,16	0,36
apparent cohesion c': in kg/cm ² .	20	0,24	0	0,12
preconsolidation pressure σ'_t : in kg/cm ² .	20	1,80	1,24	1,4
<u>Compressibility constants</u> :				
coefficient of compressibility C	36	27	12	17
coefficient of swelling A	36	88	38	46
pre-consolidation pressure σ' (after Casagrande) in kg/cm ²	36	2,20	0,99	1,5
<u>Coefficient of permeability k</u> :				
in cm/sec.	16	1,2x10 ⁻⁸	1,1x10 ⁻⁹	5,1 x 10 ⁻⁹

REMARKS.

- 1) As mean value of a serie of results, we consider the value that remains after eliminating successively the lowest and the highest values.
- 2) The values obtained for the clay fraction ($\% < 0,002 \text{ mm}$), the Atterberg limits, and the natural water-content, compared to their mean values present a deviation which, for the largest part of the tested samples is less than $\pm 10\%$.

penetration resistance of a cone placed at the surface or in the inner part of a soil mass and in which the characteristics c, φ and φ' resulting of the triaxial compression tests are introduced, it was found that in order to get a close agreement with the experimental results, it was necessary to admit a pressure, equal in all directions, which mean value was $p_{p,0} = 1,14 \text{ kg/cm}^2$.

The penetration resistance of a cone at the surface or at a certain depth below the



Consolidation test on Boom clay.

FIG. 6

surface ($C_{k,0}$ resp. $C_{k,d}$), can be found by the following equations:

$$C_{k,0} = 1,3 (v''_b \cdot p_{b,0} + v''_c \cdot c) \quad (1)$$

$$C_{k,d} = \pi_e \cdot \text{tg } \varphi' \cdot C_{k,0} \quad (2)$$

wherein: v'' and v''_c = coefficients which depend only upon φ and φ' , and of which analytical expressions have been established by Mr. De Beer.

Furthermore the pressure p_b which corresponds with the weight of the existing overburden, reaches for the examined zone of the clay (from -1,00 to -4,00) only values comprised between 0,5 and 0,8 kg/cm².

From this results it may thus be concluded that the Boom Clay, in the course of its geological history, has been subjected to a pressure which is notably higher than what may correspond with the actual topography of the site. In addition to this, it seems possible that the Boom Clay is actually submitted to pressures greater than the weight p_b of the overlying sediments. It is difficult to explain that the vertical pressure in the upper layer of the clay stratum may present values higher than p_b . Therefore, an explanation of the experiments can be sought in the existence of horizontal over stresses σ_h . In no case, these horizontal stresses σ_h can have a value greater than what is given by the following equation:

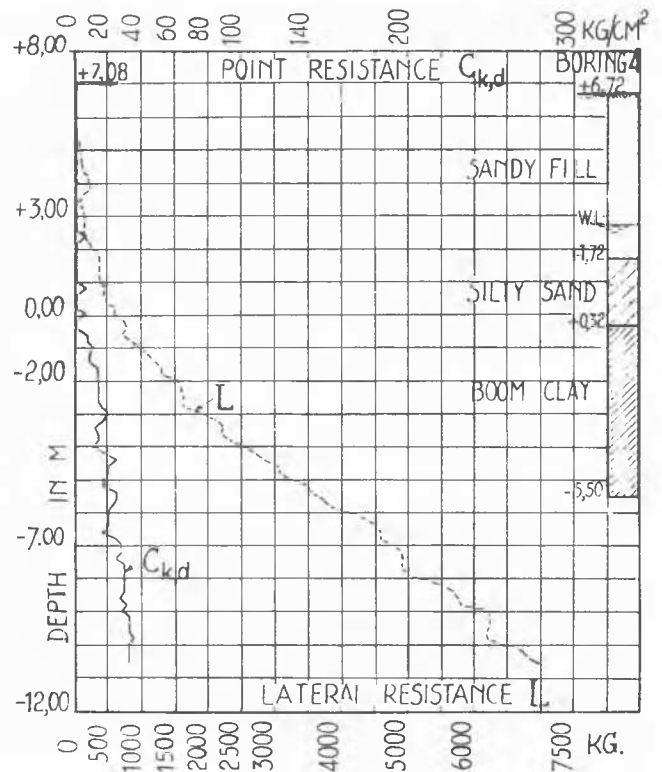
$$\sigma_h = p_b \cdot \text{tg}^2 \left(45^\circ + \frac{\varphi}{2} \right) + 2c \cdot \text{tg} \left(45^\circ + \frac{\varphi}{2} \right) \quad (3)$$

By means of this formula it was found that the mean maximum value of σ_h was about 1,92 kg/cm².

Finally the corresponding vertical pressure σ_v which may have consolidated the clay, can be expressed by:

$$\sigma_v = \sigma_h \cdot \text{tg}^2 \left(45^\circ + \frac{\varphi}{2} \right) + 2c \cdot \text{tg} \left(45^\circ + \frac{\varphi}{2} \right) \quad (4)$$

Regardless to the cohesion, a mean value of $\sigma_v = 4$ kg/cm² was obtained by equation (4). It must, however, be mentioned that this high



Deep sounding test III.

FIG. 7

value of the pre-consolidation pressure has not been pointed out by the performed tests.

In the assumption that this pressure has really been exerted, by unimmersed sediments, with a unit weight of 1,6 t/m³, these sediments may have been reached to a height of about $\frac{40}{1,6} = 25$ meters. When assuming that the overlying sediments were completely beneath the water level, we obtain a thickness of the sediments of $h = \frac{40}{2-1} = 40$ m.

Whatever may be the real explanation, it was admitted that, in relation with stability problems of constructions founded in the undisturbed Boom Clay, the allowable shearing resistance of this soil is directed by an average pre-consolidation pressure of $p_{b,0} = 1,14$ kg/cm².

Therefore, regarding to the mean values obtained by the shear strength determinations, the shear resistance s of the clay is expressed by the following equations:

$$s = 0,12 + \sigma \cdot \text{tg } 21^\circ \quad \text{when } \sigma < 1,14 \text{ kg/cm}^2 \quad (5)$$

$$s = 0,12 + 1,14 (\text{tg } 21^\circ - \text{tg } 4^\circ 30') + \sigma \cdot \text{tg } 4^\circ 30'$$

$$\text{or } s = 0,468 + \sigma \cdot \text{tg } 4^\circ 30', \quad \text{when } \sigma > 1,14 \text{ kg/cm}^2 \quad (6)$$

In the equations (5) and (6) s and σ are given in kg/cm².

THE ANALYSIS OF STABILITY OF THE APPROACH-WALLS.

In the downstream approaches to the lock, the walls are founded on continuous footings with a width of 4,50 meters. The foundation level is - 4,00. In front of the wall the ground was excavated from + 3,50 to - 0,80.

THE BEARING CAPACITY OF THE CLAY.

The ultimate bearing capacity d_r of the soil below these constructions has been computed, by using the Prandtl-Buisman formula modified by M. De Beer in such a way that, besides the constants c and ϕ , the angle of apparent friction φ' might be introduced. This formula can be written in the form:

$$d_r = V''_b \cdot p'_{b,o} + V''_c \cdot C + V'_g \cdot \gamma_k \cdot b \quad (7)$$

in which:

V''_b , V''_c and V'_g = functions which depend on φ and φ' .

$p'_{b,o}$ = an effective normal pressure existing beside and at the level of foundation.

γ_k = the unit weight of the clay, in correlation with effective pressures.

Introducing in the equation (7), the shearing resistance expressed by the equations (5) and (6), the following mean values of the ultimate bearing capacity d_r have been obtained:

- 1) For an effective pressure $p'_{b,o} = 0,3 \text{ kg/cm}^2$
corresponding with the actual ground surface at $- 0,80$: $d_r = 3,05 \text{ kg/cm}^2$.
- 2) For an effective pressure $p'_{b,o} = 0,8 \text{ kg/cm}^2$
corresponding with the original ground surface at $+ 3,50$: $d_r = 4,40 \text{ kg/cm}^2$.

COMPUTATIONS TO ESTIMATE THE SETTLEMENTS.

Settlements and risings are computed by means of the Terzaghi formula:

$$Z = \int_{\infty}^{\sigma'_t} \frac{dh}{A} \cdot 2,3 \log \frac{\sigma'_t}{p_b} + \int_{\infty}^{\sigma'_t} \frac{dh}{C} \cdot 2,3 \log \frac{p_b + \Delta p}{\sigma'_t} \quad (8)$$

$$R = \int_{\infty}^{\sigma'_t} \frac{dh}{A} \cdot 2,3 \log \frac{p_b}{p_b - \Delta p} \quad (9)$$

wherein:

Z = the final settlement of the construction.

R = the final rising of the construction.

C = coefficient of compressibility (mean value: 17).

A = coefficient of swelling (mean value: 46).

Δp = increase or decrease of the effective pressure at an arbitrary depth below the foundation.

p_b = original natural pressure.

σ'_t = pre-consolidation pressure (mean value: $1,5 \text{ kg/cm}^2$).

The results of these computations can be summarized as follows:

- 1) The final swelling R of the clay stratum in the median part of a cut 50 m wide, and with the bottom at the level $- 1,00$:

$$R = 34,5 \text{ cm.}$$

This value corresponds with the maximum rising of the bottom of the canal on the downstream entrance of the lock.

- 2) Settlement Z of a footing, 5 meters wide (width of the footing of the lock walls) whose foundation level is at $- 4,00$, and upon which a unit load of 2 kg/cm^2 is acting,

$$Z = 6,5 \text{ cm.}$$

This value gives some indication of the settlements to be expected under the load of the walls of the lock. In 1941, when this study was introduced, the observed values ranged from 2 to 2,4 cm.

- 3) Rising R in the median section of the large lock, in the assumption that this was of infinite length. (the floor of the lock at $- 1,00$):

$$R = 8,6 \text{ cm.}$$

Till 1941, the mean values of the risings measured in two longitudinal sections near the axis of the large lock were respectively 4,6 and 5 cm.

In observing that the calculated movements refer to a final state of equilibrium which, owing to the low permeability of the clay, cannot occur before a long period of years, a good agreement with the measurements was obtained.

CONTROL OF THE STABILITY OF THE APPROACH WALLS.

In order to find a mechanical explanation of the movements observed in the approach walls, and also to research measures to assure the stability of the walls in the future, a lot of stability calculations were performed. These calculations showed that in some conditions the Boom Clay was overstressed under the load of the walls. The following conclusions have been established:

- 1) In accordance with the shearing resistance of the Boom Clay as it results from the laboratory determinations, the equilibrium of the wall 5L, behind which the backfill was entirely achieved to the level $+ 7,00$, the ground water level being $+ 2,70$ behind and $- 0,80$ before the wall, can only be explained when the following conditions were satisfied:
 - on the back of the wall was exerted the active earth-pressure, inclined to the normal on the back of the wall, at its maximum value, namely the angle of internal friction of the fill. This involves the assumption that the wall had undergone a small movement directed to the canal excavation.
 - the walls 3L and 4L, where the load of the backfill was not yet applied, made one body with the wall 5L; the observed movements indicate that this has likely occurred.

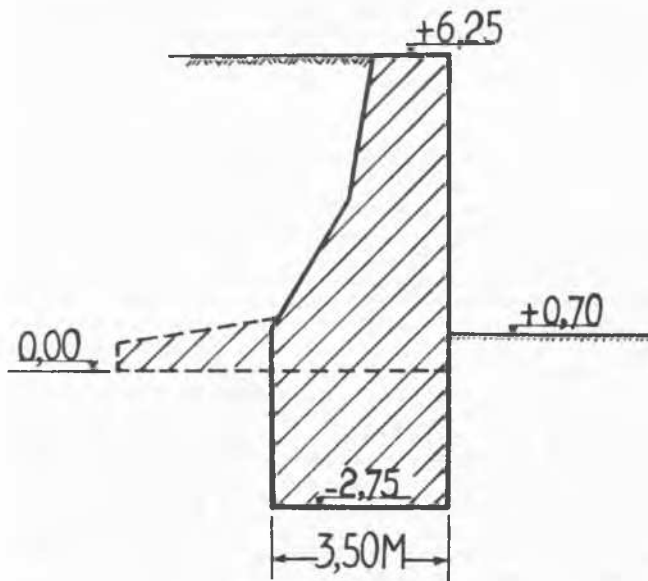
- 2) The measures of the changes of the ground water level in the piezometers a, b, c, have shown that, during some rainy periods of the year, the phreatic water level rises to $+ 4,00$. On the other hand in these periods an increase of the movements of the walls was observed. In order to explain the relative stability of the walls, in addition to the conditions indicated in 1), ante, it was necessary to introduce that the resistance offered to forward movement by the earth in front of the wall, reached its maximum intensity, namely the passive earth pressure which was inclined at the angle of internal friction, and was directed upwards. These limit conditions assume that the walls may have undergone a considerable forward movement.

- 3) Behind the walls 6L and 7L, the backfill was almost executed. Nevertheless these walls did not move to a great extent, of course they were held in equilibrium with the struts of the apron which exerted a considerable resistance at the lower part of the walls. These pressures, combined to the swelling reaction of the clay may explain the cracks which occurred in the floor extension of the lock on the downstream-side.

- 4) From calculations it may also be seen that, if the backfill was completely terminated behind the walls 2L, 3L and 4L, a general failure of the whole approach-wall would be probable.

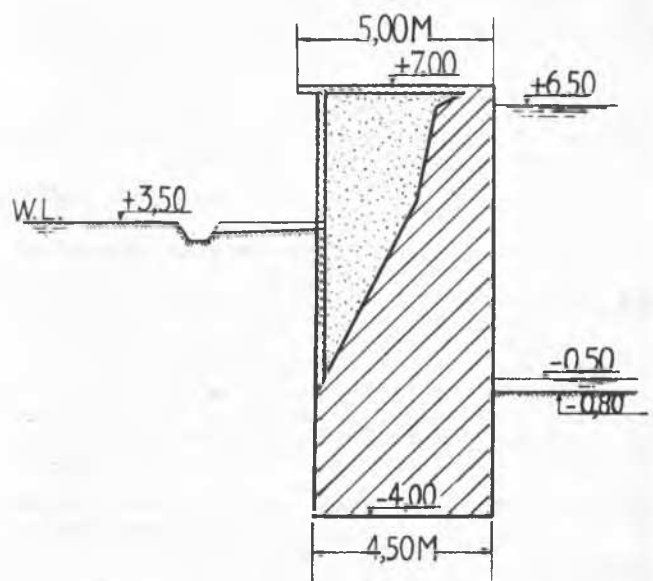
In order to increase the stability of the approach-walls, one or another of the following solutions have been considered:

- 1) When this analysis was introduced, some sections of the walls on the upstream side of the lock, were solely erected up to the level $+ 0,00$. Thus for these sections it was possible to modify the original profile of the wall, by widening the footing on the back side of the wall. This modified profile is



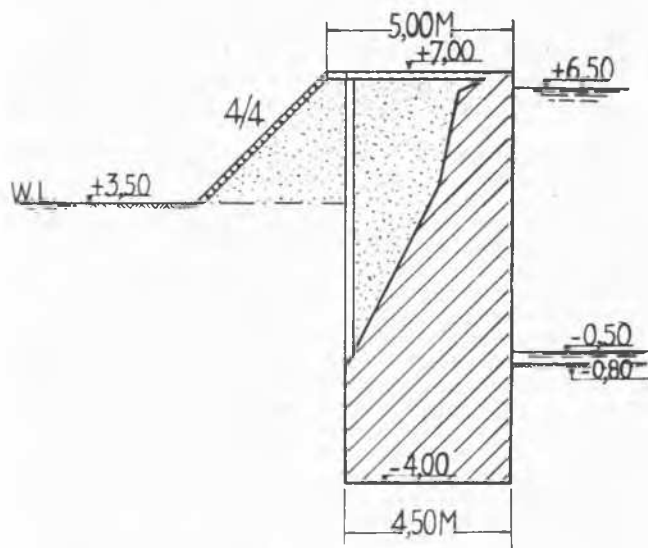
Solution a.

FIG. 8



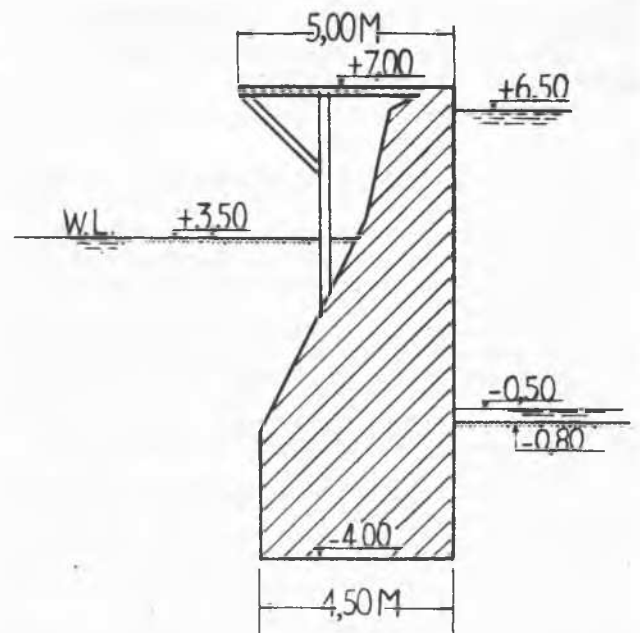
Solution c.

FIG. 10



Solution b.

FIG. 9



Solution d

FIG. 11

indicated as solution a, in figure 8.

2) For the finished walls on the downstream approaches of the lock, it has been decided to accommodate the profile of the backfill. A first solution (solution b) illustrated by figure 9, was presented by the engineer of the work. Two other systems - solutions c and d - were also examined; these are represented in figures 10 and 11.

When applying the classical method of stability computation, - horizontal active pressure on the back of the wall, and horizontal neutral pressure before the wall -, and when introducing in these calculations the mean lower limit of the bearing capacity of $3,05 \text{ kg/cm}^2$, it was found that not one of the three proposed solutions gives a safety factor greater than 1, if the water level in the canal is only at $-0,50$, and the ground

water level in the backfill is higher than $+3,50$. Only the solution c gives a little safety margin, if the canal level is at $+1,50$.

With the water-line at $+1,50$, and if small movements of the wall are allowed, the 3 solutions give a small factor of safety, the solution c being the most favorable.

In respect of the upper limit obtained for the mean bearing capacity of the clay, $4,40 \text{ kg/cm}^2$, the 3 solutions give a little safety, even when the water-line is at $-0,50$.

The preceding results refer to the stability of the clay underneath the lower edge of the footing near the canal. The stability conditions have also to be satisfied under the opposite edge, near the backfill, when

the water level in the canal will reach its upper limit, + 6,50. In this control, it was found that the solution c gives satisfaction.

For the two other solutions it was necessary to take into consideration a little movement of the wall toward the fill.

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VI d 3

SITE EXPLORATION FOR FOUNDATIONS AT PORTSMOUTH

J.W. HUNTER

1. INTRODUCTION.

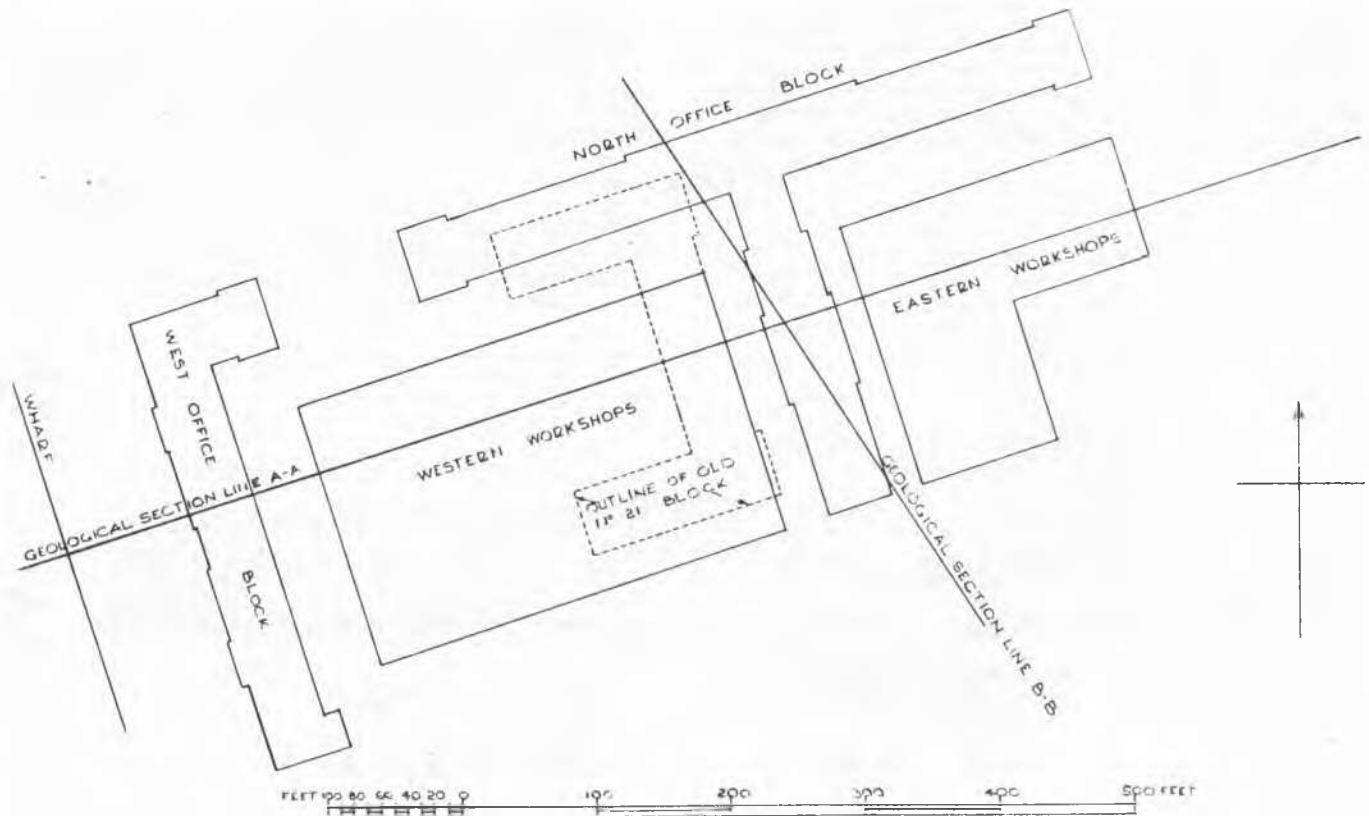
The site at which the examination described in this paper was made lies in the East side of Portsmouth Harbour near its entrance and was originally covered by the sea at high tide and around 1800 was reclaimed and built over. Many of the buildings then erected have now been demolished to enable modern buildings to be provided, the new buildings being mainly of two types:- multistorey offices and single storey workshops, the siting being shown in Figure "A". The old buildings showed considerable settlement and in view of the history of the site, a thorough investigation of the subsoil was considered necessary.

2. SITE INVESTIGATION.

Apart from the usual surface surveys the

investigation consisted of three main sections:-

- a) 35 trial bores were put down about 62 feet deep, except for four which were bored to 100 feet, so that possible long term consolidation of the material lying below the pile shoes could be considered. Ordinary disturbed samples were taken frequently from all the bores, undisturbed samples 4 ins. diameter by 18 ins. long being taken from suitable strata in 17 bores. The standing water level in the ground was also recorded.
- b) Two direct loading tests were carried out on the old filling lying just under the surface to determine its maximum bearing capacity irrespective of any long term effects, the position of the more important test being shown in Figure "A".
- c) The settlement of the main old building on the site was measured, the result being described in paragraph 5 below.



Plan of Redevelopment.

FIG. a