C. MEASURES TO PREVENT HORIZONTAL MOVEMENT.

1. The best method, but very often the most expensive, is to replace the soft layers, or at least part thereof by sand; this method is called "soil improvement". It is applied a.o. in highway and railroad construction near Rotterdam. Whenever the deep sandlayers are not far below the ground surface this method must be recommended.
2. Since movement of the soil may cause breakage of piles, (which movement cannot be resisted by a normal concrete pile) it may be advisable to use timber piles in some cases, as timber piles are far more elastic. The total length of the timber piles pulled out in the destroyed centre of Rotterdam amounted to more than 2200 km. Thousands of these piles have been used again for reconstruction purposes. A great many, however, were too much bent to be used again, but only very few piles proved to be broken.

The Vlaggemansbrug (see B2) has been replaced by a simple timber bridge with timber abutments (fig. 8). Built in 1939 it is expected that this bridge will last several years before collapse. The water pressure will then have decreased to such an extent that a permanent structure can be built.

3. A third method is to construct relieving floors. Behind the abutment and not connected to it, floors on timber piles are constructed for supporting the fill. With older structures the length of these floors (measured along the centre line of the road) is from 50 to 60 m. In course of time the most backward row of piles will fail, after that the next row and so on; until eventually the water pressure will have decreased sufficiently. For numbers of years this method has been applied successfully, especially for railroad embankments. In view of the latest experiences this old method deserves again our entire attention.
4. In order to prevent increase of water pressure a fill of a low specific gravity might be used, for instance debris or compressed peat. This peat, mechanically compressed into blocks of $\pm$ 11 cu.ft. is sold in Holland for agricultural purposes. It also has been applied successfully in constructing streets, where a sand fill, owing to its heavy dead weight, would cause more settlement than a peat fill. On being compressed the blocks are tied together with wire, so that they are already prestressed before being used as filling material. The bearing capacity of these blocks is low however.
5. Sometimes a steel sheet piling is driven behind the abutment in order to protect the piles against horizontal soil pressure. The bridge itself than acts as a strut. Technically and economically this method is only then advisable, when the soft layers are not too thick.
6. The piles have greater resistance against bending when they have a rectangular cross section with the longitudinal axis in the direction of the pressure.

However, when piles with a length of 18 m and more are used it will be almost impossible to handle them with the standard pile driving equipment, as they are far too heavy.

This method is therefore especially suitable for rather thin soft layers, requiring piles of comparatively short length.

7. Better results may be obtained by driving the piles in rows. The direction of the rows must correspond with the direction of the soil pressure. It will be advisable to choose the distance between these rows as wide as possible, so that the soil in between, may move freely.

With these types of foundations the most backward piles will fail first, then the next ones etc. This fact should be taken into account into account on designing a foundation. The water pressure will gradually decrease.

8. It will even be better to drive the piles in closed rows. This method is used with the lock "Leuvesluis" in course of construction in Rotterdam (fig. 9). First the layers of peat were excavated and replaced by sand. After the lock will be filled in, the height of the height of the total layer of sand will amount to 14 m. The layers of clay, from El. - 10 to El. - 17 under this sand fill will settle horizontally.

The piles have a cross section of 0.40 x 0.60 m and a length of 14 m.

9. If, in cases of bad soil conditions, absolute safety in connection with horizontal soil and water pressure is required, caissons may be considered. On the whole this method is more expensive than a pile foundation. Nevertheless for this purpose caissons have often been used with great success in Holland.

REVERENCE.

1)E.C.W.A.Geuze,Horizontal pressure against a row of piles.
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VII b 2

HORIZONTAL EARTH PRESSURE AGAINST A ROW OF PILES

IR. E.C.W.A. GEUZE

Head of the Research Department of the Delft Soil Mechanics Laboratory

INTRODUCTION.

The construction of the Vlaggemans-bridge, a road-bridge, was completed in December 1936. In February 1937 the groundworks including the digging of the canal, the sand fills and the embankments were finished.

During the winter of 1938-1939 large cracks were observed in the superstructure. An investigation of the cause of these cracks gave the following results:

a) One of the foundation piles of the row at the southern end of the bridge had subsided, others on the same side of the row apparently too.
b) Consequently one corner at the southern end of the bridge showed a considerable settlement.

The Delft Soil Mechanics Laboratory was

asked to investigate the causes of the rupture of the piles in a qualitative, and if possible in a quantitive sense. The author was charged with this investigation.

CONSTRUCTIONAL DETAILS.

A diagram of the longitudinal section of the bridge is given in fig. 1. This shows a flat parabolic shell of reinforced concrete with a span of about 30 m. Horizontal reactions of the shell are taken up by longitudinal reinforced concrete beams, acting as tension members. Vertical reactions are procured by two rows of cast-in-place concrete piles on either end of the bridge, with a distance of 1 m between the rows and a mutual distance of approximately 2 m between the piles, as measured from centre to centre of their respective cross-sections. The concrete beams were supported by four wooden piles per beam.

The sides of the bridge were constructed of concrete walls sheated with bricks. The space between the side-walls and the parabolic shell was filled with sand, extending over both ends of the bridge at a slope of appr.2%. The road giving access to the bridge was 21 m wide, the height of the road-embankment being approximately 3 m above soil-surface in the neighbourhood of the southern end of the bridge.

SOIL CONDITIONS.

The region round the bridge belongs to the Bergpolder. The ground surface originally (up to 1927) reached to 1,60 m below standard level (indicated by initials R.P.). In the years 1927 and 1928 the ground surface of this polder was raised in two stages by filling up according to the hydraulic method. In the first stage a level of 0,40 m below R.P. was reached; in the second stage the level was raised 1,40 m to 1,00 m above R.P. Thus the original polder surface was loaded with a layer of 2,60 m of wet sand. Some notion of the compressibility of the underlying soil layers can be obtained from the fact, that in the period between 1928 and mid 1936, when the building of the bridge was started, the original polder surface settled about 1 m.

The soil layers below polder surface consist of:
A top crust of 0,40 - 0,60 m thickness.
A peat-layer of 5 - 6 m thickness.
A mixed clay and peat layer of 7 - 8 m thickness.
Layers of fine sand alternating with layers of course sand (see diagram of fig. 1).

SCOPE OF PROBLEM.

From the beginning of the investigation it was clear, that the horizontal forces exerted on the piles by the soft cohesive soil layers were the principal cause of the ruptures in the row at the southern end of the bridge. So the chief problem was to find out:
1) The fundamental cause and the order of magnitude of these horizontal forces.
2) The resistance offered to this horizontal thrust by a row of cast-in-place concrete piles.

THE CAUSE OF HORIZONTAL PRESSURE ON THE PILES.

For the sake of simplicity the row of piles will be considered as a lattice-work composed of equidistant, vertical elements arranged along a vertical plane. Horizontal earth pressure exerted on the back side of the piles (i.e. horizontal forces directed towards the vertical plane of symmetry in fig. 1) will be taken up partly by the piles and partly by the soil layers at the front side of the piles. The first-mentioned part will be the most important one by far, because of the rigidity of the vertical constructional elements as compared with that of the soil in front. So one can expect the horizontal earth pressure to concentrate on the back side of the piles. This concentration will increase further by some effect of arching, acting along horizontal planes.

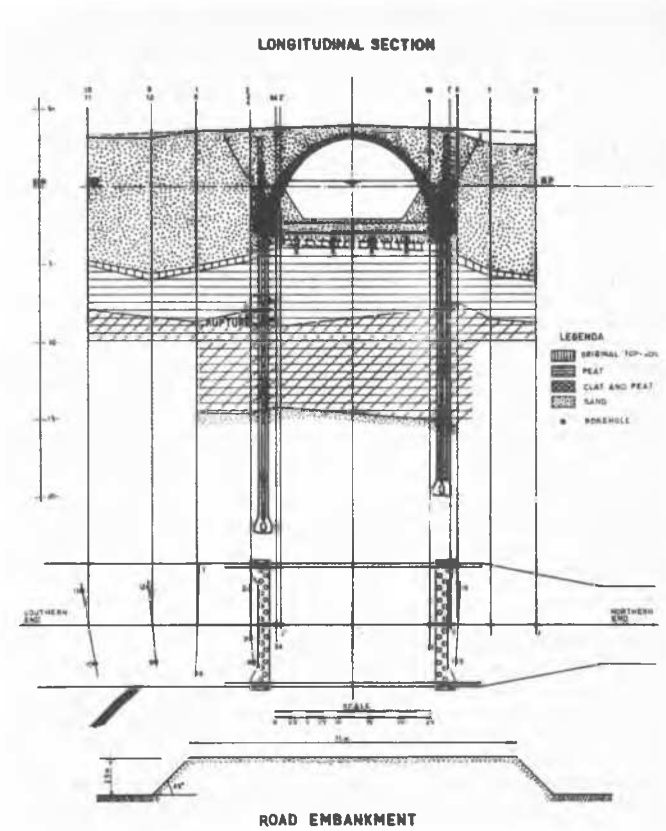

FIG. 1

The author proposes the term "grate-factor" in order to indicate the retaining effect of a row of piles as compared to that of a vertical flexible wall, with the same height and with the same factor of rigidity as the piles altogether.

One principal difference must however be borne in mind. Whereas the space between the piles permits the exchange of pore-pressure on both sides of the pile row to a large extent, this generally not the case with flexible retaining walls like bulkheads. So the term "grate factor" can be applied only to the pressure exerted by the solid phase of the soil.

The fundamental cause of horizontal pressure on the piles is the effect of the vertical load of the sand-embankment on the cohesive layers of soil. The mass of soil at the back of the pile row is enclosed in a vertical sense by sand layers. The effect of the vertical load on the layers of peat and of peaty clay, will be a substantial increase of pore-pressure, which will be largest in the middle of the mass of soil and then decrease to practically zero at the boundaries. This fundamental fact has been dealt with by Terzaghi a.o. and need not further be elaborated.

In the case, which will be considered now, this fact becomes especially important, as the difference of pore pressure in the soil mass at the back and in front of the piles is the driving force, which causes the soil to exert a certain amount of horizontal pressure on the row of piles.

This fact was considered before starting investigations and indicated the necessity of measuring pore-pressures in situ. The technique of measuring pore-pressure in the field has been discussed by Mr. Huizinga in one of the reports for the Conference and will not further be dealt with.

RESULTS OF MEASURING PORE-PRESSURES.

A) In order to determine the effect of loading by the embankment on the pore-pressures, a number of measurements were made in two verticals, each on one side of the canal and at some distance of the embankment, on the assumption, that these results would represent conditions before the filling up of the embankment. The situation of these points is indicated on the plan (fig. 5) by the numbers 14 and 13 respectively. Diagrams of the results are shown in figs. 2 and 3. Both diagrams prove, that considerable pore-pressure still existed 11½ years after the filling up of the polder with sand. With respect to the vertical no. 14 at the southern end of the bridge, the following facts may be stated. Pore-pressure in the top sand layer proved to be hydrostatic at a "normal" value of the piezometric level i.e. 0,20 m above R.P. In the deep sand layer hydrostatic conditions were met at a depth of 19 m below R.P., the value of the piezometric level being 3,60 m below R.P. Between both levels hydrodynamic conditions existed in the peat-layer, with a level of 3,5 m above R.P. at a depth of 6 m below R.P. and in the peaty clay layer, with 4 m above R.P. at a depth of 11,30 m below R.P. Though these results might lead to many interesting considerations from the point of view of consolidation-theories, only one fact will be stated (see fig. 4). The computation of the vertical "effective" stresses, which was possible now with the help of the volume weight of undisturbed soil samples and the pore-pressure measurements mentioned above lead to the conclusion, that after 11½ years only 40% of the vertical load of the sand fill could be considered as "effective" at a depth of 9 m below R.P.

With respect to the vertical no. 13 at the northern end of the bridge, conditions appeared to be more favourable (see fig. 3). In the top sand layer no measurements were made. In the deep sand layer hydrostatic conditions were met at 15 m below R.P. The piezometric level was at 0,90 m below R.P. The maximum of the hydrodynamic stress was found at a depth of 11 m at a value of 2,20 m above R.P. At a depth of 6 m the value of the piezometric level was found to be 0,75 m above R.P.

The "effective" vertical stress amounted to some 70% of the vertical load of the sand fill, at a depth of 11 m below R.P. The results of the measurements at point no. 14 will be regarded as representing normal conditions at the southern end of the bridge, before any construction work was started. Those at point no. 14 will be taken as normal for conditions at the northern end of the bridge. From the viewpoint of consolidation, conditions at the southern end seem to be much more unfavourable than those at the northern end. This is in full accordance with the fact, that ruptures of the piles occurred at the southern end.

B) A number of measurements were made in different points at two levels, 6 m and 11 m below R.P. respectively. The situation and the

DIAGRAM OF PORE PRESSURE AT POINT NR 14

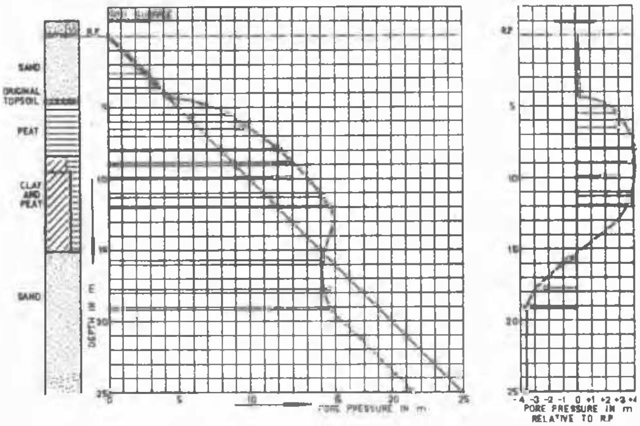

FIG. 2

DIAGRAM OF PORE PRESSURE AT POINT NR. 13

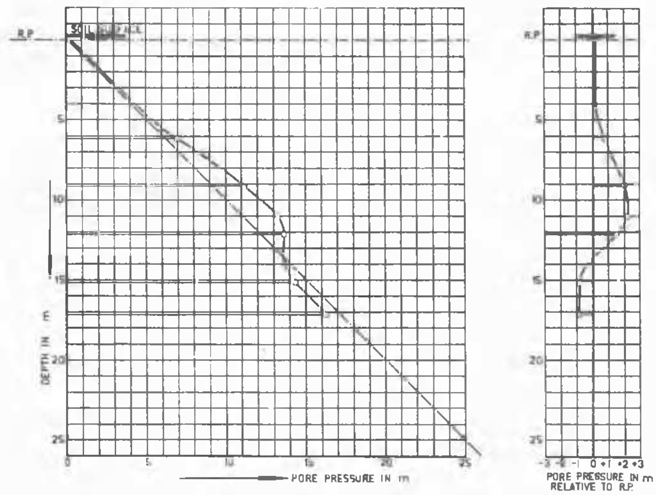

FIG. 3

DIAGRAM OF EFFECTIVE AND PORE PRESSURES AT POINT NR 14

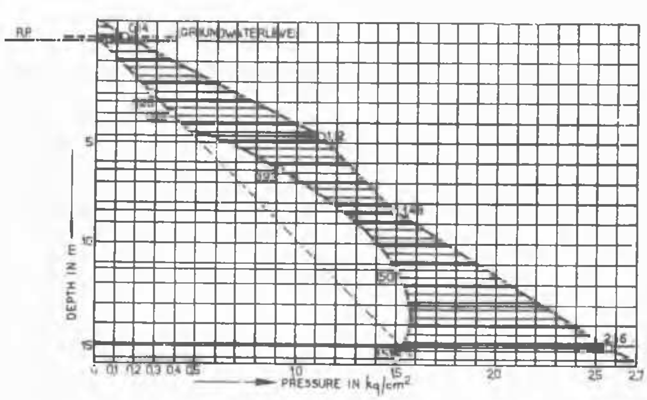

FIG. 4

elevation of these points are indicated in fig. 5. The piezometric levels are given in the top diagram of this fig. They are numbered according to the points of measurement; the heights of the level are expressed in meters with reference to "standard level": R.P. It could be expected, that the greatest rise of

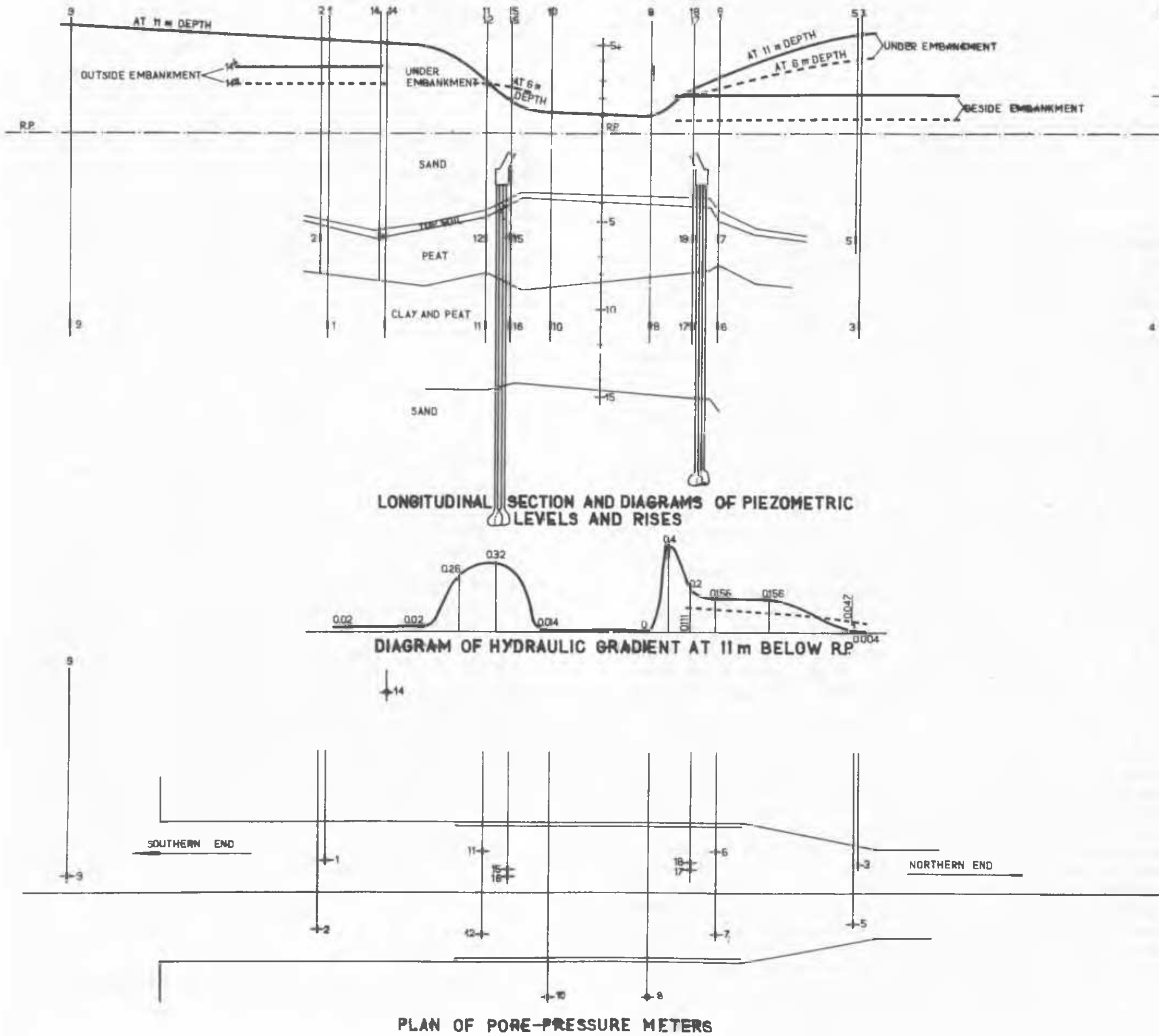

FIG. 5

the piezometric level-because of embankment loading-would be found at some distance from both ends of the bridge. This is demonstrated by the results of point 9 and 1 at the southern end. In comparison with the result of point 14 b, a rise of the piezometric level of 2,70 m and 1,50 m was found to be due to the vertical load by the embankment. As no loading took place over the stretch between the two rows of piles (or rather an unloading in digging the canal) a large difference of the piezometric levels will be the result of the distribution of the vertical loads along the axis of both embankments. This distribution is reproduced clearly by the results of the measurements as they are shown by the traced line, passing through the piezometric level points, which belong to the measuring points at a depth of 11 m below R.P. and situated at or near the longitudinal axis of the bridge.

The same considerations can be applied to the piezometric level line at the northern end, passing through the level points belonging to the measuring points 3, 6, 17 and 8 (at 11 m below R.P. too), though the path of this line differs from that belonging to the southern end. Measuring points in the peat layer at 6 m below R.P. (nrs. 12 and 15) at the southern end showed no marked rise of the piezometric level. Contrary to this the line of the piezometric levels at the northern end (points nr. 5, 7 and 18) showed the same path as for the points at 11 m below R.P. at the same end of the bridge, though with a smaller rise.

HORIZONTAL PRESSURES RESULTING FROM DIFFERENCE IN PIEZOMETRIC RISE.

With the help of the diagram described above, a computation of the horizontal forces acting on the soil masses on both ends of the bridge could be made. To that end a diagram of the hydraulic gradient was constructed with the help of the piezometric level line. This diagram is shown in the middle of fig. 5. The ordinates were computed according to the well known relation:

hydraulic gradient $= i = \frac{dh}{dx} = \frac{1}{\gamma}\cdot\frac{dp}{dx} = \frac{1}{\gamma}\,tg\alpha$ (1)

Where: h = height of the piezometric level in some point with reference to the standard level.
x = abscis of point considered.
p = pore-pressure in the same point.
γ = specific weight of pore-water.
α = angle between tangent to the piezometric level line and x-axis.

From the same theoretical consideration follows, that the magnitude of the horizontal force acting on a unity of volume of the soil mass is equal to the product of the specific weight of the pore-water and the value of the hydraulic gradient $\underline{i}$ in the centre of the volume considered.
Thus:

$$K = \gamma\cdot i = \frac{dp}{dx} = \gamma\cdot\frac{dh}{dx} = \gamma\,tg\alpha \qquad (2)$$

With γ practically equal to unity, the horizontal force acting on the unity of volume of the soil is numerically equal to the value of the tangent to the piezometric level line passing through the point defined as the centre of the volume considered. The diagram shows the fundamental difference between the results of computation obtained for the soil masses at the southern and the northern end of the bridge. This difference is closely connected with the distribution of the vertical stresses caused by embankment loading and the actual condition of piles of the bridge.

CAUSES OF DIFFERENCE ON BOTH SIDES.

As long as the rows of piles were intact on both ends of the bridge, the structure could be considered as fully rigid in the vertical direction. The considerable settlements of the cohesive soil layers could take place freely up to a certain distance of the pile rows. Near the ends of the structure however, the gradual sinking of the sand fill was counteracted by frictional forces acting along vertical planes coinciding with the pilerows. The influence of these frictional forces extends over some distance behind the pile rows, where it dies out gradually. Consequently the vertical stress on the cohesive soil layers, caused by the fill and embankment loading, will be equal to its full weight at sufficient distance from the bridge. The vertical stress will however diminish at increasing rate, nearing the pile rows. Close to the piles (at their back side) the vertical stress will have reached its minimum value.

The computation of this frictional effect is rather intricate, though one may arrive at an approximative solution, assuming only vertical planes of shearing deformation and a simple relation between shearing stress and the amount of shear in the sand fill.

Mr. E.E. de Beer, Director of the Gent Laboratory of Soil Mechanics, had the courtesy to send an informal mathematical study 1) of this special point, which can however not be discussed by the author for lack of space.

The influence of the decrease of vertical stresses is clearly demonstrated by the deformations of originally horizontal layers as follows from the diagram of borings in fig.5. The curvature of these layers near the pile rows is in full accordance with the above reasoning.

The rupture of the piles at the southern end of the bridge and its consequent settlement naturally lead to the abolishment of the frictional effects. As a consequence the vertical stresses near the pile row at the southern end gradually increased with increasing settlement of that end of the bridge and so did the pore-pressure. This is the cause of the fundamental difference between the path of the piezometric level line near the broken piles and of that near the sound piles.

COMPUTATION OF HORIZONTAL PRESSURE AGAINST PILE ROWS.

An approximative computation of the horizontal pressure acting on the pile rows necessitates the knowledge of the shearing properties of the cohesive soil layers. A study of these properties is discussed by the author in a report for this Conference, entitled: "Compression, an important factor in the shearing test". The results of the slow shearing tests described in this report, are shown in diagrams (l.c. figs. 6, 7, 8 and 9).

The shear along horizontal planes was found to be directly proportional to the logarithm of time since the application of the shearing stress. This relation holds up to a relatively large value of the shearing stress and was expressed in equation (14):

$$\gamma_{sh} = \Delta\tau\left\{(\gamma_p - \alpha_p) + (\gamma_s - \alpha_s)\log t\right\}$$

The values of γ_p and γ_s (direct and secular shearing constants) were represented in diagrams in the same figs. in relation to τ and τ/σ From the very start of the investigation of the present subject the author was troubled by the fact, that the piles broke some 2 years after the completion of the ground works. It was thus apparent, that <u>time</u> had to be an important factor in the <u>solution</u> of the problem. The results of the slow shearing tests gave the proper answer to the solution of this problem. It was apparent from these results, that the horizontal deflections of the piles would be accompanied by an equal deformation of the soil mass, which in turn would raise frictional resistances within the soil mass. Thus the bulk forces resulting from the action of pore-pressure on the soil would be counteracted by internal frictional forces. The magnitude of these forces depends on two factors: i.e. the horizontal deflections of the piles <u>and time</u>. For the sake of simplicity a "<u>grate factor</u>" equal to unity will be assumed, which means that the case of a bulkhead, with a permeability of that of the neighbouring soil layers has been dealt with. The deflection of the pile row will be practically zero at the top and increase to a maximum value at a level approximately half the distance to the deep sand layer. From this level to a relatively small depth in the sand layer it will decrease to practically zero deflection. The deflection line will be approximately a parabole. These deflections were taken as a base for the computation of the internal frictional resistance. The mass of soil was divided in horizontal layers and the part of the deflection line belonging to one of these layer was introduced in the computation as the resulting total shear γ at the "pile-end" of the layer. The resulting shear is the sum of the shears over the distance subjected to a hydraulic gradient. This distance can be taken from the hydraulic-gradient diagram of fig. 5.

Furthermore the period of 2 years was taken as a base for computation. Both values for γ and for τ were introduced in the equation (14). In this way the "retaining frictional forces" were computed over the whole length of the piles and naturally proved to be largest for both the top and the bottom of the cohesive soil mass and gradually decreased to zero - value somewhere near the half-distance level i.e. where the tangent to the deflection line of the piles was exactly vertical. The author wishes to state only the result of this computation, which - somewhat to his surprise - proved to be excellent.

The computation of the ultimate stresses in the concrete and its reinforcement proved to be a rather intricate problem. Besides the stresses resulting from the bending moment, caused by the horizontal bulk pressure, the influence of normal stresses resulting from the top loads on the piles (increased by the "negative" frictional forces of the top sand layer) proved to be considerable. By approximation the maximum bending moment the pile would withstand, was found to be 8,65 t.m. (= 865 000 kg.cm).

The largest bending moment as found by computation of the horizontal pressure (diminished by the internal friction-time effect) gave a result of 6,3 t.m. - 9,8 t.m., depending on the variation of some assumptions. So the same order of magnitude was found for both active and resisting forces, which in the author's opinion is an unexpected favourable result, considering the intricate nature of the problem.

CONCLUSION.

The investigations of the causes of rupture of a row of piles, as a result of horizontal soil pressure, appeared to be possible using the following tools and faculties:

a) borings, undisturbed sampling, volume weight of samples.
b) pore-pressure measurement in the field.
c) slow shearing tests.
d) applied mechanics.
e) reasoning.

The horizontal pressure on a row of piles was computed from:

1) The hydraulic gradient in the soil mass, derived from pore-pressure measurements.
2) The results of slow shearing tests on undisturbed samples.

These results showed the importance of the factor time. The shearing deformation increased directly proportional to the logarithm of time, at a constant value of the shearing stress. Consequently the shearing deformation speed is inversely proportional to time. The first relation is expressed by the equation:

$$\gamma = \tau\left\{(\gamma_p - \alpha_p) + (\gamma_s - \alpha_s)\log t\right\}$$

The second relation by;

$$\frac{d\gamma}{dt} = \tau(\gamma_s - \alpha_s)\cdot\frac{1}{t}$$

These relations hold for a lower range of values of the shearing stress τ. The values of γ_s depend to some extent on the magnitude of τ. With some soils γ_s is practically a constant independent of the magnitude of τ. With other soils γ_s shows a linear increase with increasing τ, whereas α_s always is found to be practically a constant. Thus the shearing deformation speed is either directly proportional to the shearing stress (in case of γ_s being practically a constant) or to the square of the shearing stress (in case of γ_s being a linear function of τ), besides being inversely proportional to the time elapsed since the application of the shearing stress.

On the other hand, a mass of soil subjected to a relatively high speed of shearing deformation, will offer an internal frictional resistance corresponding to the above-mentioned relations. This internal frictional resistance depends however on the time elapsed since the moment deformation started.

In all practical cases the amount and the speed of shearing deformation depends on the behaviour of the retaining structure or the retaining mass of soil. In these cases the speed of displacement of the retaining structure governs the magnitude of the internal frictional resistance and consequently the magnitude of the pressure exerted on the retaining structure.

In most cases the displacements of any retaining structure or soil mass will gradually come to an end, having shown the largest speed of diplacement shortly after the application of external loads. The first phase will be accompanied by ultimate deformation speeds and consequently by an ultimate value of the internal frictional resistance. Pressures exerted on any retaining structure will then be at their lowest level. They will gradually increase inversely to the decrease of the speed of displacement and accordingly reach their highest level, when a state of rest is attained. This process may take place at very different rates of speed, dependent on many factors like the nature of the soil, of the retaining structure, etc.

The author wants to stress the point, that no reference is made to consolidation effects, which follow up the above-described phase. In many cases however, as in the case dealt with in this report, the favourable effect of consolidation on effective stresses and the resulting increase of internal friction is not sufficient to out-balance the decrease of internal friction at decreasing shearing speeds. This is especially the case with thick layers of unconsolidated, cohesive soils with low permeability.

It is the author's firm conviction, that the above-mentioned theoretical and experimental facts and the considerations derived thereof will give the explanation of the fact - which has been observed in many similar cases of failure of retaining structures - that failures occurred a considerable time after the completion of structures and ground works. Thus, the "critical" phase - which is often supposed to be entered shortly after the completion of the structure - appears at a later stage.

The common practice of computation based on a definite resistance of cohesive soils against shearing, as it is applied in sliding planes methods, thus might lead to erroneous results. The author gladly wishes to state, that a similar conclusion was arrived at independently and by a different way by Mr. Gregory P. Tschebotarioff, Associate Professor of Civil Engineering at Princeton University, as a result of his findings with large-scale model tests on flexible bulkheads. 2) The author had the opportunity to study this synopsis, sent to him by Prof. Tschebotarioff thanks to the intermediary of Mr. E.E. de Beer, who paid a visit to Princeton.

Acknowledgement. The author wishes to express his thanks to the Rotterdam Municipal Technical Works for their courtesy to allow the publication of the facts described in this paper.

REFERENCES.

1) E.E. de Beer: "Drukverdeling achter een muur gefundeerd op palen" (confidential report).
2) Gregory P. Tschebotarioff: Synopsis of a lecture on: "Large-scale model tests on flexible bulkheads", January 1947.

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