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1973
москва

PROCEEDINGS OF THE EIGHTH INTERNATIONAL CONFERENCE ON SOIL MECHANICS AND FOUNDATION ENGINEERING

ТРУДЫ ВОСЬМОГО МЕЖДУНАРОДНОГО КОНГРЕССА ПО МЕХАНИКЕ ГРУНТОВ И ФУНДАМЕНТОСТРОЕНИЮ

COMPTES RENDUS DU HUITIEME CONGRES INTERNATIONAL DE MECANIQUE DES SOLS ET DES TRAVAUX DE FONDATIONS

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COMPTES RENDUS DU HUITIEME CONGRES INTERNATIONAL DE MECANIQUE DES SOLS ET DES TRAVAUX DE FONDATIONS

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SYNOPSIS

For the construction of a silo complex with elevator building at the Beneluxharbour in the Rotterdam harbour area, a special method of soil improvement has been applicated. In this area structures are usually founded on piles. For the abovementioned structures the subsoil has been improved by means of the deep compaction method. For this purpose special provisions have been made to the compactors to make effective compaction possible also in thin clay- and peat layers, occurring in this area. The settlements have been continuously checked during the construction and filling of the silo's with approx. 100.000 tons of grain. A filling programm has been set up in order to have the settlements of the silos take place as uniform as possible. The dimensions of the silo complex amount to 37 x 120 m, and the maximum average soil pressure 35,0 t/m fig.1

INTRODUCTION

On behalf of the grain transhipment concern "Royal Bunge" a grain elevator complex has been constructed in the Europoort Area (Benelux port) at Rotterdam. The complex consists of three units:

1. The elevator building, approximately 60 metres high, founded on a reinforced concrete slab, 1,0 m in thickness and with dimensions of 36 by 18 sq.m. The foundation pressure at a level of 4,50 m above N.A.P. amounts to 2,08 kgf per sq.cm. Under extreme circumstances this pressure may increase to 3,8 kgf per sq.cm locally and for short periods mainly as a result of wind loads.
2. First elevator unit, at a distance of 6,0 metres from the elevator building, containing fifteen silos with a diameter of 10 metres and 45 metres high, constructed in reinforced concrete. The silos are founded on a concrete slab, 1,0 m thick and with dimensions of 38,0 by 62,0 sq.m. The foundation pressure at a level of 4,50 m above N.A.P. amounts to 0,86 kgf per sq.cm. for empty silos and an average of 2,7 kgf per sq.cm. when full. Under extreme conditions this pressure may increase locally to 1,7 and 3,9 kgf per sq.cm. respectively.
3. Second elevator unit. The dimensions and design of these silos are similar to unit no.1.

The foundation slab is located at 9,5 metres from the crestline of the harbour embankment. The slope of this embankment is 1 : 1,7 above water to 0,8 metre minus N.A.P. and 1 : 3,5 under water to the bottom of the harbour basin at 10,65 m minus N.A.P. The horizontal berm at 0,8 metre minus N.A.P. has a width of 3,0 metres. The ground surface is at 5,5 metres above N.A.P. A lay-out of the terminal

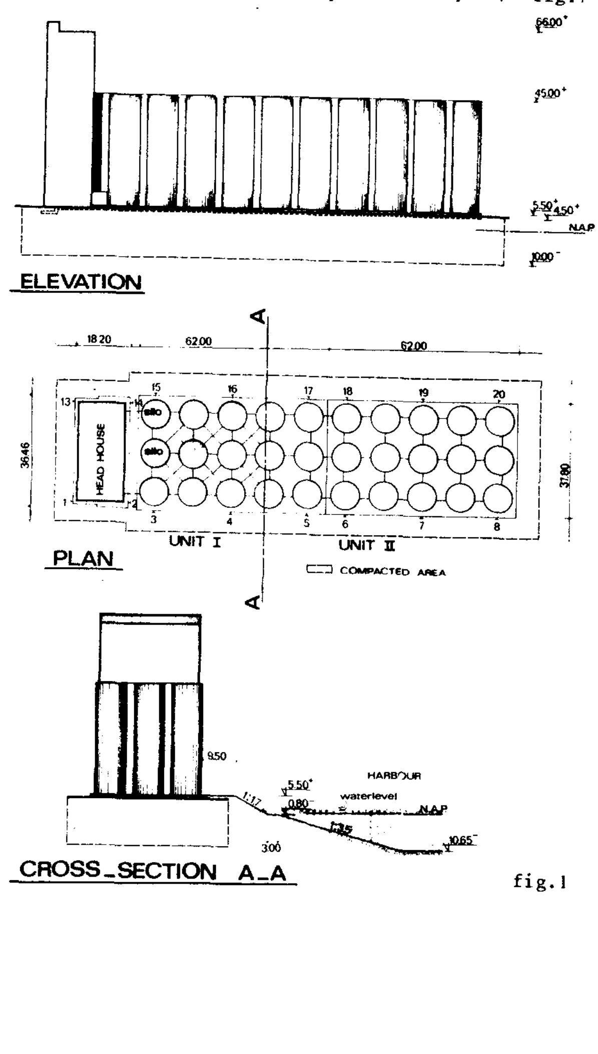

fig.1

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complex with section is shown in figure 1.

SUBSOIL CONDITIONS.

Subsurface investigations consisted of the execution of 47 static cone penetrometer tests and three borings.
The compressibility and shearing resistance of the subsoil layers were estimated based on the results of laboratory investigations carried out for the construction of the adjacent harbour basin.
The results of the penetrometer tests and borings indicate a sand deposit consisting of hydraulic fill, occurring from the ground surface, located at 5,80 metres above N.A.P., till approximately 5,00 metres below ground level.
Below this sand fill placed upon the original ground surface, a clay layer, approximately 1,00 to 3,00 metres in thickness, is found.

This clay layer is overlying a tidal flat area formation reaching until approximately 18,00 metres minus N.A.P.
This formation is characterized by sand layers of widely varying properties interspersed by thin clay layers, clay lenses, traces of peat and thin layers of shells.
Below the tidal flat area formation a marine clay layer of 0,50 to 2,00 metres in thickness is located, followed by a 10,00 to 15,00 metres thick pleistocene sand layer of good quality.
A stratum containing sand and clay was found below the pleistocene sand deposit.
Results of some characteristic penetrometer tests, a schematic boring log as well as the estimated soil properties are shown in figure 2.

fig 2.

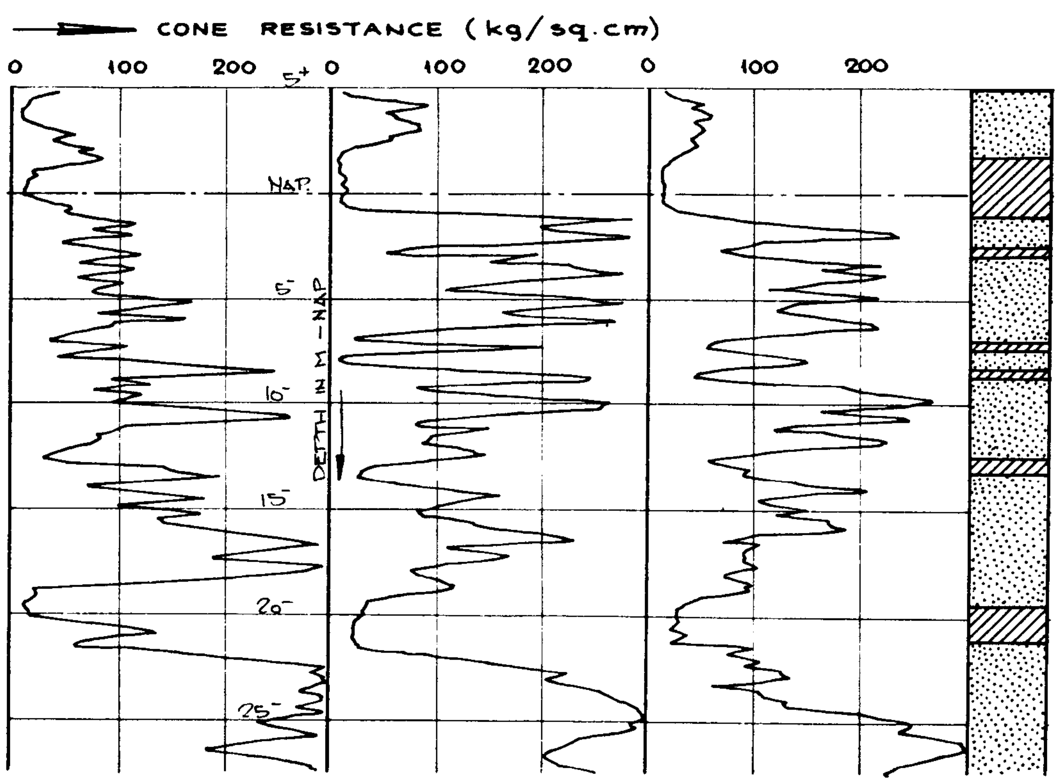


FIG. 2
RESULTS OF A FEW CHARACTERISTIC CONE PENETRATION TESTS AND A SCHEMATIC BORING PROFILE

SAND

CLAY

FOUNDATION POSSIBILITIES.

Under the circumstances the foundation method usually selected in this area consists of a pile foundation transferring the loads to the pleistocene sand layer.
In these cases the pile length amounts to 25 to 27 metres.
With regard to a footing foundation, the settlements and in particular the differential settlements of structures of any importance, will be of an inadmissible magnitude. This is a result of the presence of the top clay layer and the tidal flat area formation.
For that reason 95% of all structures in this area is founded on pile foundations driven into the pleistocene sand.
For the design of these pilesfoundations two particular aspects should be taken into account:

a. negative skin friction;
b. driving stresses.

In view of the above reasons it was decided in the first instance to apply pre-cast concrete piles for the foundation of the elevator complex, which were to be installed by means of driving.
For piles with a length of 27 to 28 metres, a square section of 0,45 x 0,45 metre and an enlarged point with a surface of 3000 sq.cm, a net bearing capacity of 100 tons would be admissible.
The extremely hard driving conditions to be expected made such demands upon the concrete quality, that these piles could not be supplied in the available period.
After this, the possibility to apply cast-in-place piles (type Vibro) was studied. Piles, approx. 28 metres long with a point surface of 3000 sq.cm, would meet the requirements. (100 tons design load).
Also due to the limited time-schedule this foundation method would be relatively costly.
The above mentioned reasons necessitated the study of the possibilities for a spread foundation.
It appeared that, combined with soil replacement, a spread foundation would be possible and even very attractive. This in view of the very short execution period required and the relatively very low foundation costs.

FOUNDATION METHOD.

The design of the foundation was based upon:

a. the type of the structure which can be classified as very rigid (eleminating differential settlements);

b. the allowable settlements and differential settlements whereby the allowable differential settlements (approximately 5 cm at 10 metres distance) were the controlling factor;

c. the stability of the harbour embankment and the consequent deformations.

Since an important part of the occurring settlements would be caused by consolidation of the clay layer located at approximately N.A.P. level (varying considerably in thickness), removal of the clay and replacement by clean sand locally available (originating from the hydraulic sand fill) was necessary. Mechanical stabilization of this newly installed 5 to 6 metres thick sand deposit, now immediately resting upon the tidal flat area formation, was also essential.

In order to change the erratic structure of the tidal flat area formation, the properties of which vary considerably at short distance, it was decided to apply an adjusted method of vibroflotation.
The subsoil was compacted by means of a vibrator to a depth of approximately 10,00 metres minus N.A.P.
In principle a more or less uniform density was aimed at.
The vibrator which was lowered into the ground by means of jetting, was provided with a short cylindrical cutting edge with a diameter of 65 cm.
Through this the thin clay layers present in the tidal flat area formation were removed at each compaction point.
After the vibrator had reached the required depth it was set into action and withdrawn (at a rate of 4 to 6 metres per minute), while constantly coarse sand was added. Thus a densely compacted sand column with a diameter of 0,70 metre and reaching to approximately 10 metres minus N.A.P., was built up at each compaction point (one point per 1,6 square metre).
This compaction scheme was carried out under the complete elevator complex until 6 metres from the outer perimeter in each direction. (based on a load distribution of two vertical to 1 horizontal).
Due to the relatively high rigidity of these sand columns (as compared with the subsoil in between the sand columns) it is to be expected that a stress concentration will occur at each of these columns resulting in a relatively uniform deformation pattern.
The compaction was carried out in two phases (see the section on construction).
The vibroflotation method utilizing vibrators provided with a cutting edge, has been developed for a soil stabilization project at The Hague, Holland.
This project concerned the foundation of the 17-story office building of the Royal Shell Group.
The foundation pressure amounted to 3 to 4 kgf per sq.cm in this case.
Settlement observations were made during construction and these agreed reasonably well with the predetermined values. The maximum settlement amounted to 2,6 cm, the differential settlement was 0,7 cm.

THEORETICAL CONSIDERATIONS CONCERNING STABILITY AND SETTLEMENTS OF THE ELEVATOR COMPLEX.

For the determination of the stability of the elevator complex four starting-points are to be mentioned.

a. The ultimate bearing capacity of the foundation slab or parts of it, evaluated by the method of Terzaghi (ref. 1)

b. The stability of the elevator complex or parts of it, especially with a view to the location quite close to the edge of a port (approximately 9,50 metres).

c. The stability of the compacted sand columns in the natural sand strata.

d. The settlements and differential settlements that are to be expected.

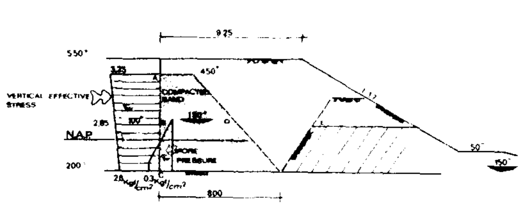
a. **The ultimate bearing capacity of the foundation slab or parts of it.**

For the evaluation of the ultimate bearing capacity an arbitrary slab width of 1,50 m was adopted with a length equal to that of the elevator complex.
This is in agreement with the calculation method normally applied in Germany for large foundation slabs.
The shear stresses occurring in the floor slab are not taken into account in the computations but naturally contribute to the bearing capacity.

It has been assumed that the angle of internal friction of the compacted sand would be not less than 40°. Further the cohesion of the sand was fixed at 0.
The position of the ground water table was taken at the base of the foundation slab.
Based on these assumptions the factor of safety amounted to approximately 5,1.
This figure, regarded from the point of view of safety of the footing with respect to breaking into the ground, can be considered as amply sufficient.
In case the arbitrarily chosen slab width is taken as more than 1,00 metre, the factor of safety will increase.

b. **The stability of the elevator complex or a part of it, especially in view of the location close to the edge of the Benelux port.**

The elevator complex is located at a distance of 9,50 metres from the edge of the Benelux port. This brought on the necessity to take into account the possible occurrence of slides along either circular or plane surfaces.
A large number of circular sliding surfaces has been calculated, resulting in a minimal factor of safety with respect to the maximal shear stress of 1,30 (ref. 2).
When this procedure for investigating the stability of the slopes is applied for normal slopes of dikes or excavations, a factor of safety of 1,05 to 1,10 is required, and consequently this factor of 1,3 indicates an ample safety margin.
In the stability computations an excess water pressure of 3,00 metres on the surface of sliding has been taken into account.
On account of the presence of the clay layer at the level of the old ground surface, which layer could not be removed under the harbour embankment, also a horizontal surface of sliding through this clay layer had to be considered.



(fig. 3)

The results of these computations have lead to an extension of the compaction in the direction of the edge of the port in order to remove the clay layer locally and replace the clayey material by well compacted sand. Due to this measure the factor of safety of this surface of sliding could be increased to 1,5 and consequently the application of steel sheet piling could be omitted, resulting in a considerable financial saving.

The stability computations were made for two horizontal surfaces of sliding, i.e. one at 2,00 metres minus N.A.P., and one at 1,00 metre above N.A.P.
In the surface ABC the active soil pressure is taken into account as well as an excess water pressure of 3,00 metres.

The angle of internal friction of the compacted sand amounts to 40°. (see fig. 3).
The factor of safety against sliding in the surface at 2,00 metres minus N.A.P. was determined at 1,5 and in the surface at 1,00 metre above N.A.P. at 2,34.
In these computations the contribution of the clay layer to the total shearing resistance has not been included in view of the considerable difference in deformation characteristics. If the latter would be taken into account in the case of ultimate failure, the factor of safety in the surface at 2,00 metres minus N.A.P. amounts to 2,16 and in the surface at 1,00 metre above N.A.P. to 2,98.

c. **The stability of the compacted sand columns in the natural sand strata.**

The various forces acting on a cylinder with a diameter of 0,70 metre and 0,50 metre high, are shown in fig. 4.

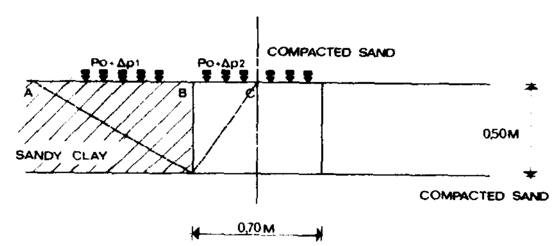


fig.4

The clay layers of any substantial thickness are found at a depth of approximately 5,50 metres minus N.A.P. Taking into account the pressure distribution, the vertical stress at this depth amounts to 2,0 kgf per sq.cm. The method of stress-distribution in the system sand column - clay layer can be schematized as follows:

The distribution of the effective load ΔP between clay layer and sand column can be considered in relation to stiffness and surface.
The distribution is as follows:
ΔP_1 = 0,37 kgf per sq.cm. and Δp_2 = 7,0 kgf per sq.cm. Assuming that p_o amounts to 1,42 kgf per sq.cm., a pressure on the clay layer of 1,79 kgf per sq.cm. and on the sand column of 8,42 kgf per sq.cm. is arrived at. The clay layer has been subjected to an excess pressure of approximately 1,50 to 1,60 kgf per sq.cm. for a considerable period as a result of the hydraulic fill.

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It is assumed that in the sand column an active state of stress occurs.
The angle of internal friction in the sand can be estimated at 40^o and consequently λ_a can be figured at 0,18.
The horizontal active pressure on the clay layer then amounts to 8,42 x 0,18 = 1,51 kgf per sq.cm.
It is further assumed that for a required λ_p equal to 1,0 no horizontal consolidation will occur. (Terzaghi). The supporting pressure of the clay then comes to 1,0 x 1,79 = 1,79 kgf per sq.cm.
The supporting pressure is therefore higher than the active pressure exerted by the sand column.
In case the earth pressure at rest is taken as supporting pressure on the sand column and the pressure is calculated with $\lambda\eta$ = (1- sin ϕ),then it appears that the earth pressure at rest is almost sufficient to ascertain equilibrium with the active pressure in the sand column; this supporting pressure is approximately 1,35 kgf per sq.cm.
Consequently no large deformations of the sand column are to be expected.
However, a more thorough investigation on the mechanism of failure of a similar system, both in the laboratory and in practice, will be necessary.

d. Evaluation of the settlement and differential settlements.

Computations of the settlement and differential settlements have been based on a one-on-two pressure distribution (two vertical on one horizontal). Further the formula of Terzaghi-Keverling Buisman is used, reading as follows:

$$z = \frac{1}{c} \times h \times \ln \frac{p_o + p}{p_o}$$

The c-values of the subsoil required for the equation have been chosen as 450 for the compacted sand. This value has been determined from settlement observations made on structures founded on subsoil stabilized by means of the vibroflotation process.
Compressibility coefficients of the remaining subsoil strata were estimated based on data available from extensive subsoil investigations, carried out by the Municipality of Rotterdam in this portarea. For the clay layer overlying the pleistocene sand with a c-value of 30

Introducing these values in the equation, a settlement of 15 to 20 cm is found.
The differential settlements that are to be expected will amount to approximately 7,5 cm at 15,0 metres; the rigidity of the superstructure, especially of the elevator units, will of course have a stabilizing effect.

EXECUTION OF THE VIBROFLOTATION COMPACTION.

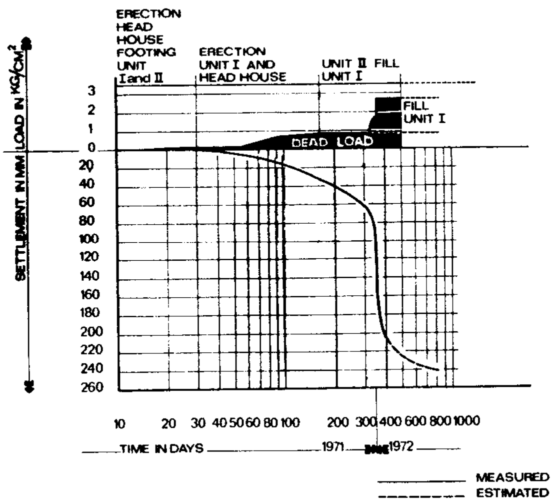
The Vibroflotation has been carried out by "Nederhorst Grondtechniek", a subsidiary of the International Foundation Group I.F.G. at Gouda, Holland.
The stabilization was done with vibrators, in accordance with the system of Johann Keller, equipped with a cutting edge of 70 cm diameter, located at approximately 1,50 metre from the point of the compactor.

The clay layer located at the level of the old ground surface, was too thick (2 to 3 metres) and too close to the base of the foundation, to allow an economic stabilization by means of vibrator with cutting edge.
In that case the distance between compaction points should have been reduced to such an extent, that also from a technical point of view the removal of this layer and replacement by sand appeared attractive.
For this purpose an excavation was made combined with dewatering by the vacuum method, the so-called well-point-system, allowing the clay layer to be replaced by sand. The excavation was backfilled to approx. 2,00 metres above N.A.P. From this level the first phase of the compaction was carried out to a depth of 10,00 metres minus N.A.P.
The effective upper limit of compaction was taken at 1,00 metre above N.A.P.
Next the second layer of backfill was placed to appr. 5,00 metres above N.A.P. From this level the second phase of the compaction was carried out.
During the first phase of the work coarse concrete sand was added in order to render the compaction as effective as possible also in sand layers o poorer quality.
The backfilling, and therefore also the compaction, in two phases has been chosen for economic reasons. Thus the depth of compaction could be reduced, permitting the use of a standard type crane.
The complete job consisted of 5100 compaction points with a total length of 70.000 metres.
The construction period amounted to 26 weeks. The compaction has been carried out with eight vibrators.

SETTLEMENT OBSERVATIONS.

After placement of the foundation slab for the elevator complex, measuring bolts were installed at 16 locations. Thus it was possible to observe accurately the settlements during construction, but especially also during filling of the silos.
A filling programm was made up in order to ascertain as much as possible an uniform settlement in particular during filling operations.
Continuous day and night observations were made during filling with grain.
Based on the results of these observations the loading of the silos during filling has been adjusted to the settlement-pattern.
The settlements as measured are shown in fig. 5.

fig. 5



AVERAGE SETTLEMENT REFERENCE POINT 4 AND 6

CONCLUSIONS

1. It is possible in more cases to use the deep compaction method. It is necessary to use a special type of vibrator in case of sand interspersed with thin layers of peat or clay.
2. By using a filling program it is possible to get settlements and differential settlements under control, for structures and foundation method as described in this article.

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   Soil Mechanics in Engineering Practice.
2. T.K.Huizinga, Grondmechanica

PREDICTION OF SETTLEMENTS OF FOUNDATIONS ON SAND

PREDICTION DES TASSEMENTS DES FONDATIONS SUR SABLE

ПРОГНОЗ ОСАДОК ФУНДАМЕНТОВ НА ПЕСКЕ

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SYNOPSIS. The subject studied hereafter is the settlement of superficial foundations on sand. The traditional empirical formula of Terzaghi-Peck (1948), based on the bearing plate test of 1 ft square, is inquired into. It is demonstrated that its applicability is restricted and unsatisfactory. On the other hand, the importance and validity of the equation of Housel-Burmister is evidenced. Dealing with the latter, the results of settlement measurements (collected by Bjerrum-Eggested, 1963) described by several authors are analyzed. It is shown how to determine the characteristics of deformability of sandy soils subjected to loaded plates and footings of varied dimensions.

INTRODUCTION

In this paper, the Author shows that experimental and field results (collected by BJERRUM EGGESTAD, 1963) of settlements of plates, footings and fills on the surface of sandy grounds, coincide quite adequately with the results obtained by the application of the theoretical-experimental method of HOUSEL-BURMISTER. Besides, it is demonstrated that the traditional formula of TERZAGHI-PECK (1948) has restricted applicability, while the expression of HOUSEL-BURMISTER presents a much more general character.

Also, the Author contests some hypothesis of Bjerrum-Eggestad (op.cit.) related to the influence of sand density over the settlement ratio of different foundation's size.

THE EXPRESSION OF TERZACHI-PECK

It is sufficiently known and largely divulged the empirical expression of TERZAGHI-PECK (1948):

$$\delta = \delta_o \cdot \left(\frac{2 \cdot B}{B + B_o}\right)^2 \qquad (1)$$

where:

δ = the settlement of a square plate (B x B) subjected to pressure p;

δ_o = the settlement of a "standard" square plate (B_o = 1 ft $\cong$ 30 cm) subjected to the same pressure p.

The expression (1) may written:

$$\frac{\delta}{\delta_o} = \frac{4}{\left(1 + \frac{B_o}{B}\right)^2} = \frac{4}{\left(1 + \frac{1}{\frac{B}{B_o}}\right)^2} \qquad (2)$$

BJERRUM and EGGESTAD (1963) worked out the curve corresponding to (2) in a log-log graph, correlating δ/δ_o with B/B_o (see Fig. nº 1).

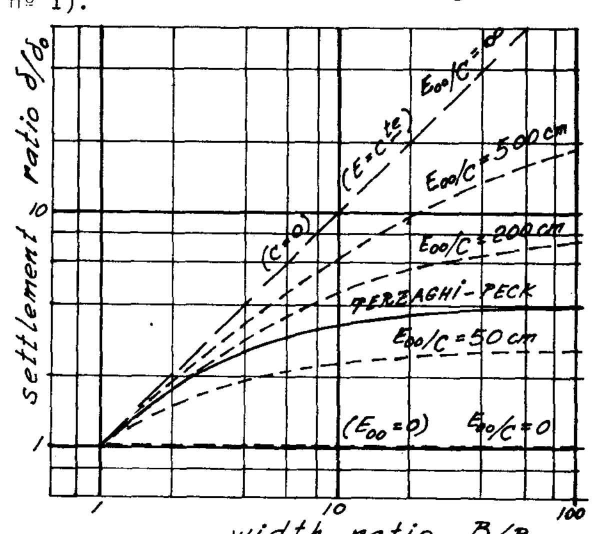

Fig. 1 - Curves correlating δ/δ_o to B/B_o in a log-log graph.

In Fig. nº 2, in a natural scale graph, the same correlation is presented, with the advantage of more precision and detail in the range of usual dimensions of foundations (up to ratio B/B_o = 30, i.e., $B \cong 900$ cm = 9 m).

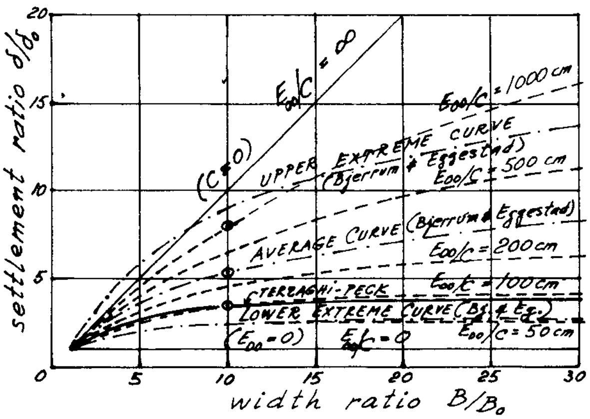

Fig. 2 - Curves correlating δ/δ_o to B/B_o in a natural graph.

THE EXPRESSION OF HOUSEL-BURMISTER

According to HOUSEL (1929), in case of square plates (side B = 2.b) on the surface of the ground, the settlement is produced by the pressure

$$p = n_o + m_o \cdot \frac{P}{A} \qquad (3)$$

where P is the perimeter and A the area of the plate, n_o and m_o characteristic coefficients of the ground.

In another form:

$$p = \frac{C \cdot \delta}{C_\delta \cdot (1-\mu^2)} + \frac{1}{4} \cdot \frac{E_{oo} \cdot \delta}{C_\delta \cdot (1-\mu^2)} \cdot \frac{2}{b} \qquad (4)$$

according to BURMISTER (1947), who adapted the empiric expression (3) of HOUSEL to the Theory of Elasticity, considering that in the soils the Modulus of Deformation E_z may vary (to increase, in general) with the depth.

Comparing (3) and (4) Burmister obtained:

$$n_o = \frac{C \cdot \delta}{C_\delta \cdot (1-\mu^2)} \qquad (5)$$

$$m_o = \frac{E_{oo} \cdot \delta}{4 \cdot C_\delta \cdot (1-\mu^2)} \qquad (6)$$

being $\frac{2}{b} = \frac{4}{B} = \frac{P}{A}$ and:

E_{oo} = Modulus of Deformation at the surface of the ground;

C = Increment of Modulus with the depth ($E_z = E_{oo} + C.z$)

μ = Coefficient of Poisson

C_δ = Coefficient dependent of the shape and rigidity of the plate.

The expression (4) will be referred to, from here further, as Expression of HOUSEL-BURMISTER (for superficial plates). We do justice to Terzaghi recalling that he had already arrived at an expression similar to (6), concerning m_o, but without setting forth the corresponding expression (5) to n_o - see TERZAGHI (1943), art. 142, page 402.

It is important to bear in mind that the expression (4), in spite of having been originated from the Theory of Elasticity, does not demand for application that the material be of elastic behaviour; it may be applied to soils, when loaded by plates, since there has been proportionality between pressures and settlements - see BARATA (1967).

The Author (BARATA, 1966) demonstrated theoretically that the expression (4) is general and valid for any dimensions of plates, since it deals with plates on the surface of the ground. In case of plates at depth, there will be corrections needed (derived and divulged by the Author in his former papers), which do not concern with the scope of the present paper.

THE GRAPHIC REPRESENTATION OF HOUSEL-BURMISTER'S EXPRESSION

The expression (4) may be written in the classical form:

$$\delta = C_\delta \cdot p \cdot \frac{B}{E_{oo} + C \cdot B} \cdot (1-\mu^2) \qquad (7)$$

according the modification suggested by BURMISTER (1947).

For a "standard" plate (B_o = 30 cm)

$$\delta = C_\delta \cdot p \cdot \frac{B_o}{E_{oo} + C \cdot B_o} \cdot (1-\mu^2) \qquad (7\text{-a})$$

Graphs are presented in Figs. nº 3 to 6, in order to solve graphically the expression (7) where the case be, respectively, B = 30 cm, 100 cm, 150 cm and 300 cm, for the conditions:

C_δ = 0,85 (rigid square plate)

μ = 0,30 (average value for soils)

p = 1,00 kg/cm²

(NOTE: Those graphs may be used for p or C_δ other than the original; it will be necessary, only, to correct proportionally to the other values).

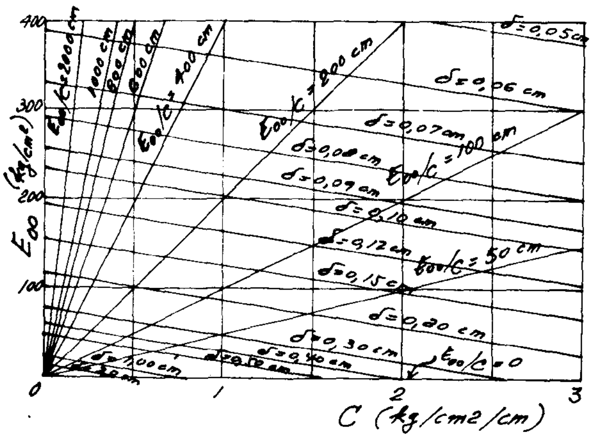

Fig. 3 - Settlements of Rigid Square Superficial Plates (expr. 7) with $B = 30$cm ($\mu = 0,30$ and $p = 1,0$ kg/cm^2).

From the expression (7) and (7-A) we obtain:

$$\frac{\delta}{\delta_o} = \frac{B}{B_o} \cdot \left(\frac{E_{oo}+C.B_o}{E_{oo}+C.B}\right) = \frac{B}{B_o} \cdot \theta \qquad (8)$$

Where

$$\theta = \frac{E_{oo}+C.B_o}{E_{oo}+C.B} = \frac{E_{oo}/C+B_o}{E_{oo}/C+B} \qquad (9)$$

<u>In the particular case of $C = 0$</u> (soil of Modulus of Deformation constant with depth):

$$\theta = 1 \qquad \frac{\delta}{\delta_o} = \frac{B}{B_o} \qquad (10)$$

equation which in the Fig. nº 2 (and also in Fig. nº 1) is represented by a straight line of 45°.

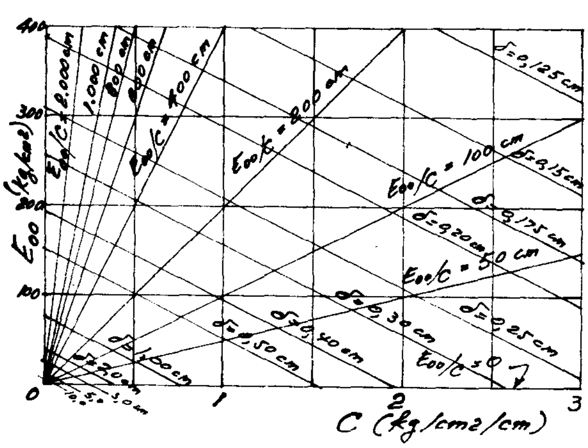

Fig. 4 - Settlements of Rigid Square Superficial Plates (expr. 7) with $B = 100$cm ($\mu = 0,30$ and $p = 1,0$ kg/cm^2).

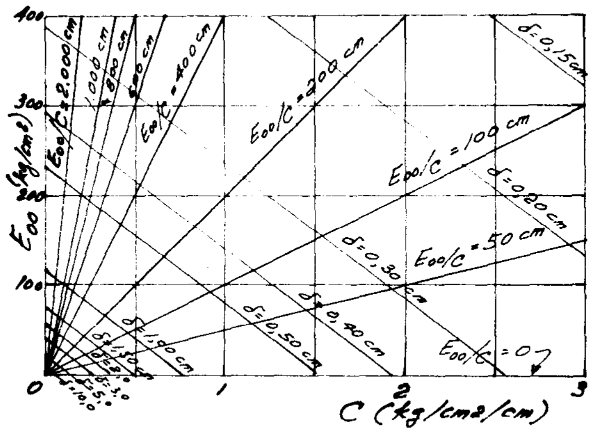

Fig. 5 - Settlements of Rigid Square Superficial Plates (expr. 7) with $B = 150$ cm ($\mu = 0,30$ and $p = 1,0$ kg/cm^2).

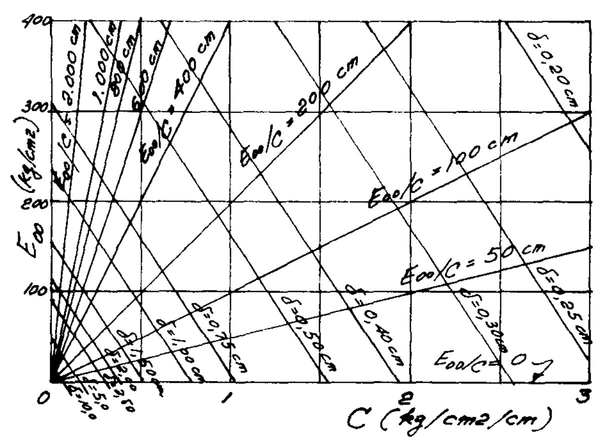

Fig. 6 - Settlements of Rigid Square Superficial Plates (expr. 7) with $B = 300$ cm ($\mu = 0,30$ and $p = 1,0$ kg/cm^2).

<u>In the particular case of $E_{oo} = 0 \quad C > 0$</u>:

$$\theta = \frac{B_o}{B} \qquad \frac{\delta}{\delta_o} = 1 \qquad (11)$$

which corresponds to a straight line parallel with the axis of B/B_o.

The general expr. (8) can be represented, therefore, in Fig. nº 1 or nº 2, by a Family of E_{oo}/C curves, varying between the limits $E_{oo}/C = 0$ (expr. 11) and $E_{oo}/C = \infty$ (expr.10)

There is also the less common case (which is possible, although, to occur, at least to limited depth) <u>which is the one where $C < 0$</u> (soil of decrescent Modulus):

$$E_z = E_{oo} - C.z$$