RESULTS OF A SETTLEMENT MEASUREMENT

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Abstract

The paper discusses results and conclusions of the settlement measurements of a five level panel building block, constructed with shallow foundation. The results of measurements, their analysis, the semi-empirical method can be applied for settlement prediction and determination of deformation parameters. The method is advantageous in case of buildings having similar structure and similar soil conditions.

1. Introduction

The paper summarizes the results of a settlement measurement made on a panel building founded on shallow foundation. The intention of the paper is to contribute to the analysis of expectable settlements in similar cases.

According to the experiences, the calculated and measured settlements often differ significantly [4, 5, 11]. The reliability of the calculations is affected by many circumstances: the quality of sampling and laboratory test method [1, 2, 16], the analogy or deviation between the soil-structure interaction model and the reality [4, 9, 13, 14, 17], just mentioning the most important factors.

To examine stability of different buildings, to determine modern movement-elements, there would be a need on much more accurate and reliable settlement calculation method [6, 7, 17]. In spite of this, no considerable progress can be seen in more fields [4, 15]. Among circumstances like these, analysis of former settlement measurements or semi-empirical methods can also be used to the preparation of "settlement prediction" or to determine certain data [3, 4, 8, 9, 10, 12].

It is obvious that this method can be applied successfully only if more buildings are constructed with the same structure and load on a field of relatively same geological structure.

The aim of this paper was to give a more reliable settlement prediction based on measurements and under the given conditions.

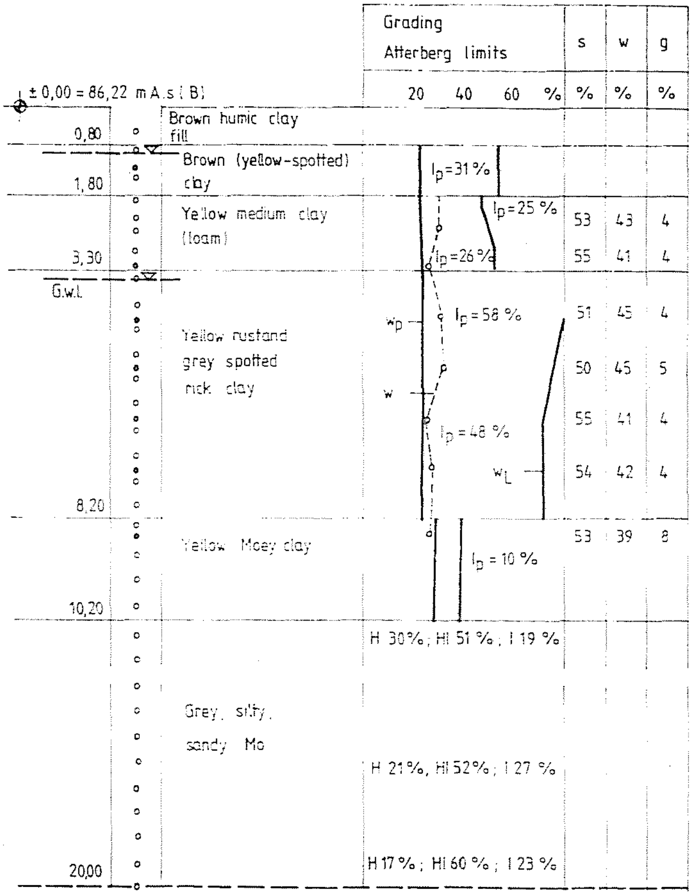

Fig. 1. Soil profile representing the soil layers below the building

2. Construction conditions and data

2.1 Soil and ground water conditions

The tested construction site situates at the Eastern part of the Hungarian Plain, where, above the 1000—2000 m thick Pannonian strata complex Pleistocene and Holocene cover layers can be found.

From the aspect of soil mechanics and foundations, only the Holocene and Pleistocene layers have importance.

A typical soil profile can be seen in Fig. 1 as a representation of the data of large diameter drillings and test data.

According to the soil exploration data, the layers were deposited very uniformly and almost horizontally. The typical five layers in the original order up to the depth of drilling (20 m) are the following:

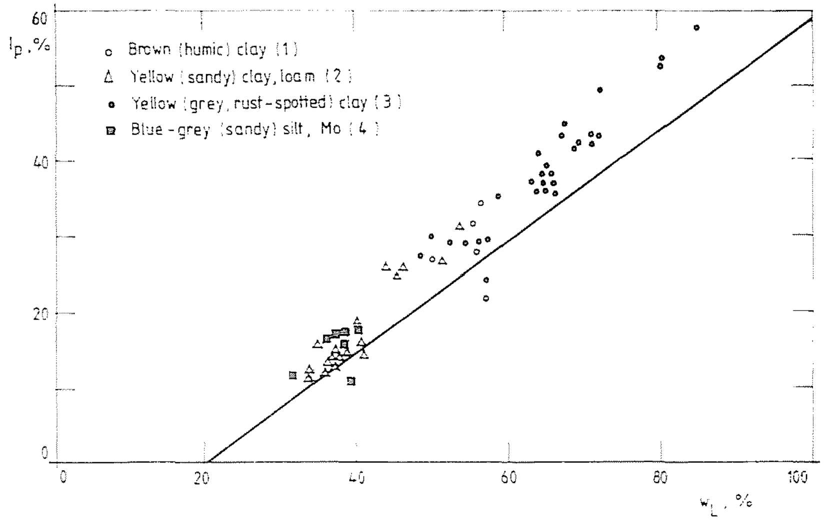

Fig. 2. Soil classification by means of the plasticity graph

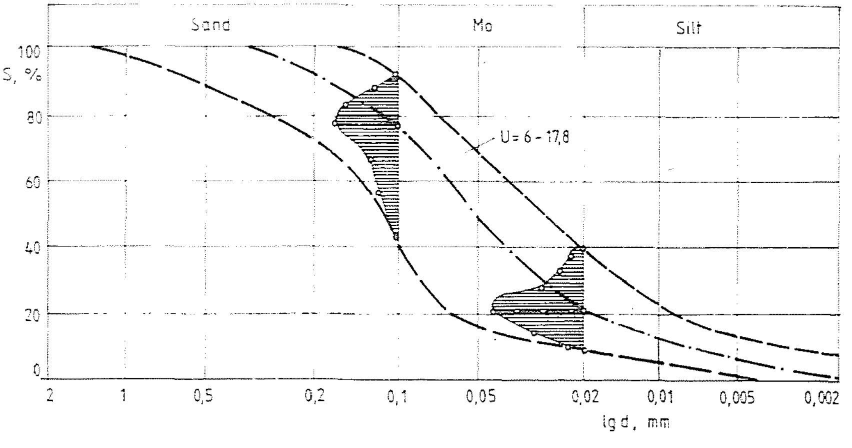

Fig. 3. Grading of the low-cohesion and transition subsoils

— brown humic clay;
— yellow, yellow-brown sandy clay (loes, loam);
— yellow, grey-veined, rust-spotted clay;
— yellow, grey-yellow, Moey silt;
— grey, blue-grey, sandy-loamy Mo.

Fig. 2 summarizes the classification characteristics of the 8—10 m thick cohesive layers close to the ground; the various soil types form well separable groups. The grading, the range of distribution and the relative frequency of the grey silty sandy Mos can be seen in Fig. 3.

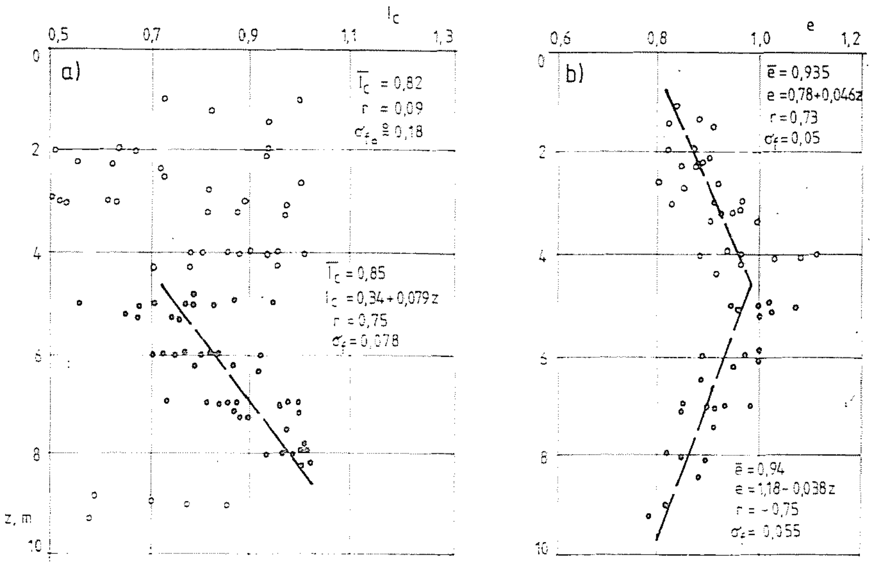

Fig. 4. Statistical evaluation of the variation of soil characteristics v. s. depth

The normal ground water level at the site is 0.9—1.0 m below ground. Due to the cohesive upper layers, internal water can occur frequently at some lower places. For these reasons it can be understood that the variation of water level affects significantly the soil conditions close to the surface.

Data about the soil conditions can be seen in Fig. 4. Figs 4a and 4b represent the variation of the relative index of consistency (I_c) and the void ratio (e), respectively versus the depth. From the results the following tendencies can be determined:

— between ~1 and 4.5 m the relative index of consistency changes sharply, no correlation can be found either with depth or soil conditions;

— between ~4.5 and 8.0 m the relative index of consistency increases with depth;

— between ~1 and 4.5 m the void ratio increment is proportional to depth;

— between 4.5 and 9 m the void ratio decreases gradually.

The results of statistical analysis (mean, regression line, correlation index (r) and the values of the residual scatter (σ_f) are given in Fig. 4.

2.2 Strength and deformation characteristics

To determine load bearing, CU tests were carried out. The test results can be seen in Fig. 5a.

The deformation characteristics were measured in oedometer tests made on samples taken from bore samples and from a 3.2 m deep open pit.

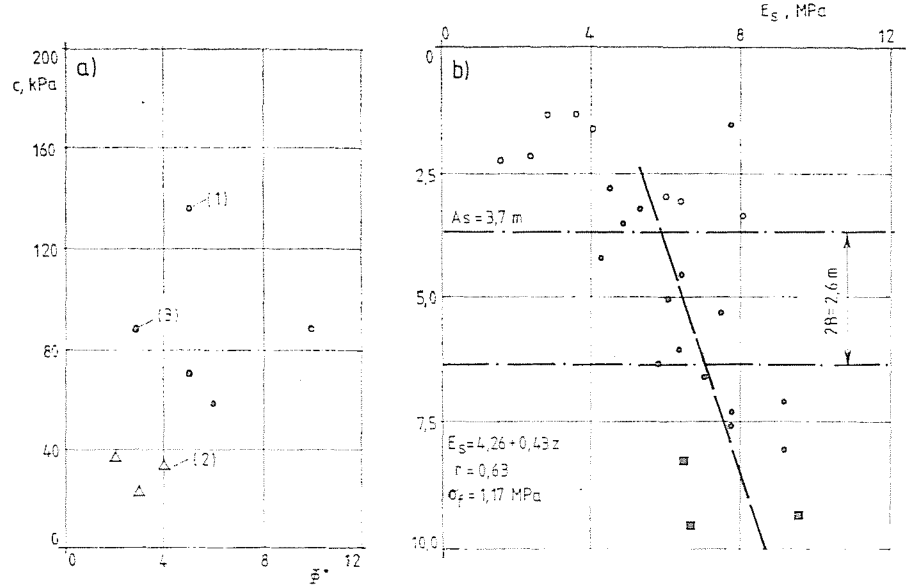

Fig. 5. Results of strength and deformation tests

The summary of the variation of modulus of compressibility (E_s) versus depth can be seen in Fig. 5b. On the basis of test result two groups can be separated:

— group of humic clays between 1 and 2.5 m;

— group of soils between 2.5 and 10 m.

As for the building settlements, the properties of the last group are determinant. The values of E_s were determined with the help of the pressure range between the geostatic pressure and 100 kPa overpressure.

The mathematical-statistical analysis provided the following data:

— equation of the regression line: $E_{s1} = 4.26 + 0.43$ MPa where $z =$ the depth below the ground;

— coefficient of correlation: $r = 0.63$;

— residual scatter: $\sigma_f = 1.17$ MPa.

The mean modulus of compressibility of the $2B = 2.6$ m thick soil below the foundation level is $E_{31} = 5.1 \pm 0.8$ MPa with a probability of 95 percent.

2.3 Building data

The sketch of the plan and the section of the tested four-sector building can be seen in Fig. 6.

The 5-level building has no cellar level and was constructed with PEVA tunnel-formwork technology. The size of one sector is 11.04 m to 17.25 m. The

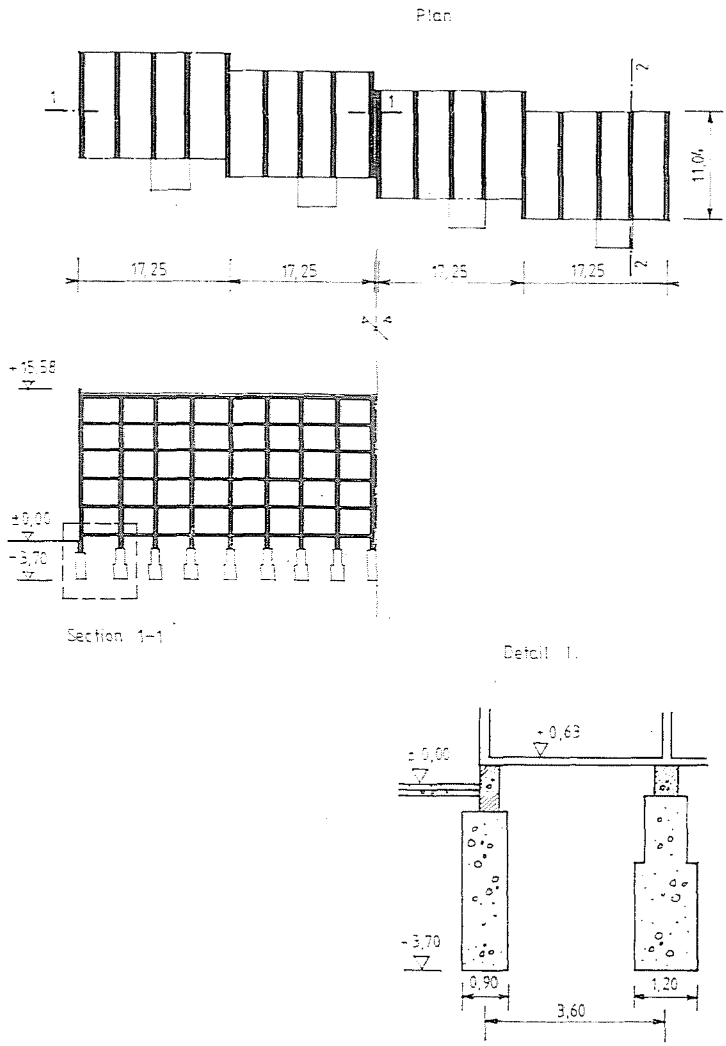

Fig. 6. Plan of the building, main dimensions

total length of the four-sector building is 60 m that is devided in the middle with a dilation gap. The load bearing walls are transversal, the distance between their axes is 2.7 m at the staircases and 3.6 m at the living area. The concrete stripe footings and the transversal walls are joined with a r.c. raft slab that is also the fundament of the ground floor pavement.

The width of the footings and the calculated actual loads are listed in Table I. The foundation level and the most important data are given also in Fig. 6.

Table I

Width of footing B (m)	Load on foundation level q (kN/m)
0.9	192.7
1.2	297.4
1.3	341.7

3. Results of settlement measurement

To observe settlements of the building measurements were performed at 45 places and at 8 occasions by the staff of the Department of Higher Geodesy, Technical University of Budapest. From the data of survey drawings with the contour of settlements were made. Fig. 7 represents the lines of settlement levels measured in the first and in the last survey. Some interesting conclusions can be drawn from the results.

Fig. 8. shows the displacement of the footings of the transversal walls. The settlement graphs show a slight tilt of the building to one side. The value of tilt ranges between:

$$\frac{\Delta y}{L} = 2\times 10^{-4}—6\times 10^{-4}.$$

Thank to the high transversal stiffness, practically no inflexion can come into being below the footings.

Settlements measured on walls parallel to the building axis — contour walls — proved deformations characteristic of the case of stress concentration, since the longitudinal stiffness is much less than the transversal one. It can be determined from the settlement graph that the maximum inflexion is different at the different sectors (see Fig. 9). The value of calculated inflexion ranged between 6×10^{-5} and 9×10^{-5}.

The consolidation process is demonstrated with the help of data measured at some characteristic points.

Fig. 10. represents the settlement process of a longitudinal wall. The four sectors were built in couples with level steps. At first sectors A and B were completed.

According to the time-settlement curves, the consolidation did not end even 12—14 months after the completion of the 5th level. Parallel with the rapid load increase only 50—60 percent of the total settlement took place. At the border of sections $A—B$ and $C—D$ — at the dilation joint — a 2—4 mm excess settlement came into being in consequence of the stress superimposition.

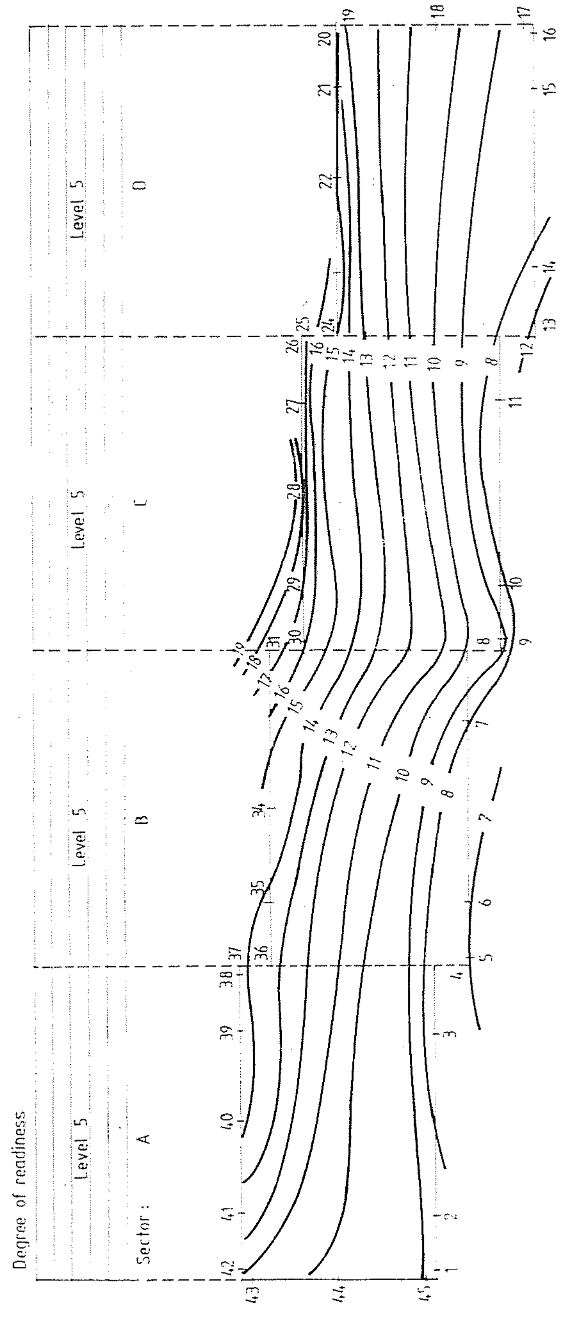

Fig. 7. **Graph of the max. settlements measured on the building**

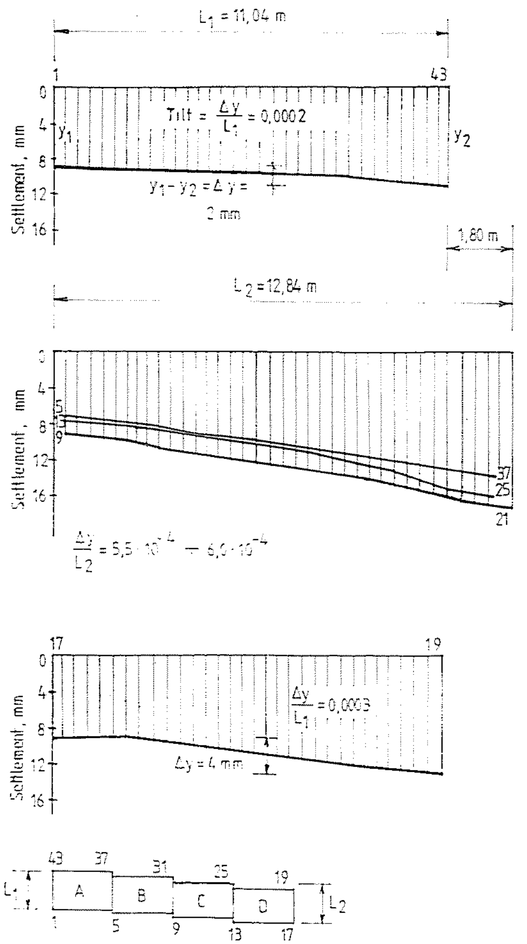

Fig. 8. Transversal building settlements

The time of consolidation can also be determined from the settlement measurements. A characteristic result is represented in Fig. 11. according to which the degree of consolidation was between 95 and 100 percent at the time of the last survey.

The mean value, the scatter of expected settlements were calculated from the observed data by means of the method of mathematical statistics.

The mean value of the *measured* settlements was:

$$Y_{(95)} = 12{,}0 \pm 1{,}0 \text{ mm with a probability of 95 percent.}$$

The maximum settlement can be estimated to be 25 ± 5 mm.

We put emphasis on expression "measured" because some parts of sectors A—B were already loaded when the first survey took place.

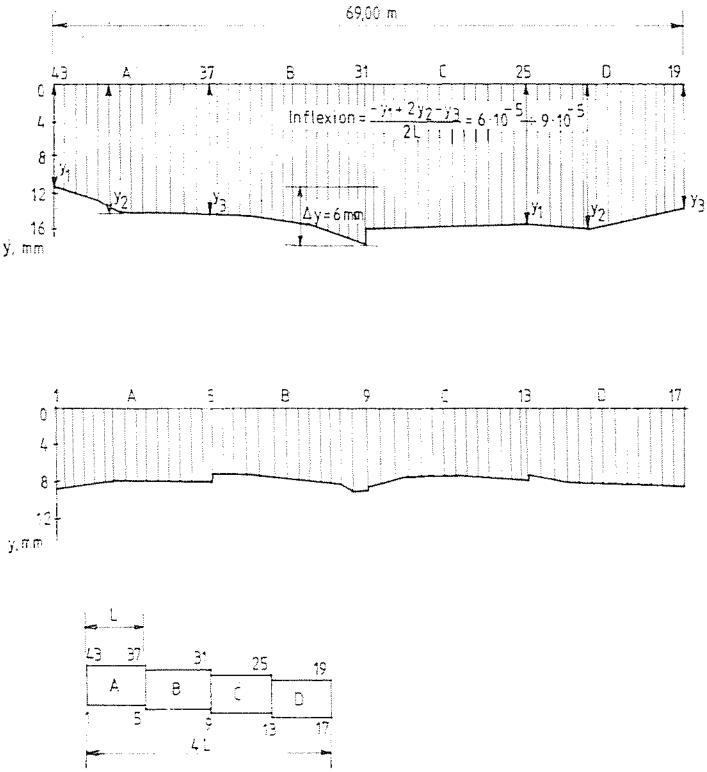

Fig. 9. Longitudinal deformations

The estimated settlements included initial compression and 95—100 percent of primary consolidation. The value of secondary consolidation is not discussed here since no further settlements were observed.

Making use of measured data, the real deformation characteristics of soil can also be calculated. From the well-known basic equation of settlement analysis the modulus of compressibility can be expressed:

$$E_s = \frac{1}{Y}\int_0^z \sigma_z dz = \frac{1}{Y} F(\sigma)$$

where: Y = the actual settlement

$F(\sigma)$ = the area of stress distribution graph between foundation depth and z limit depth.

The minimum and maximum values of the modulus of compressibility, taken by the load data of Table I, are listed in Table II.

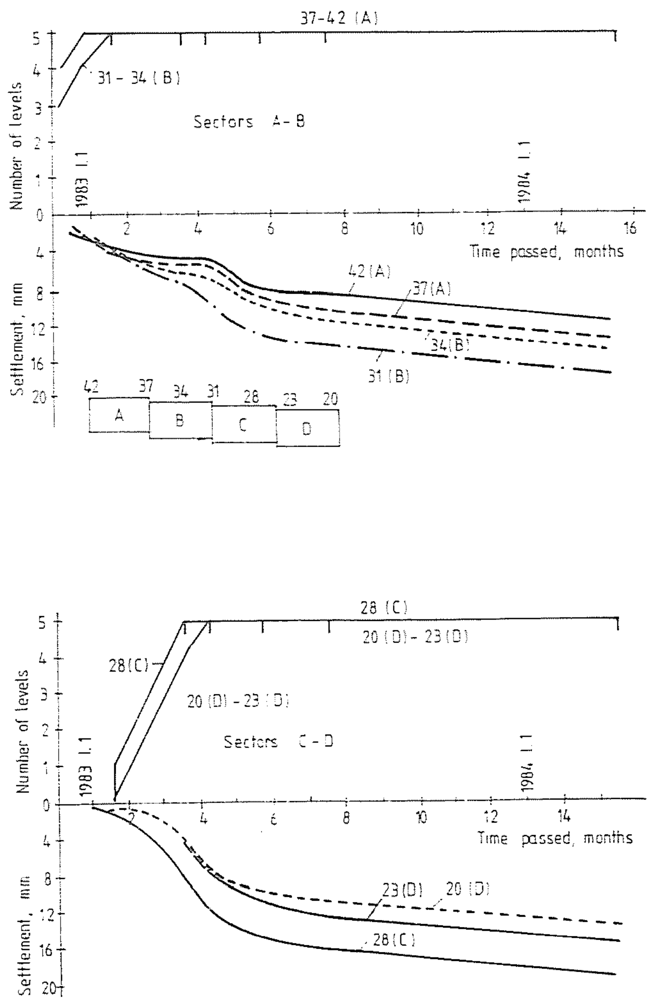

Fig. 10. Process of consolidation

Table II

Width of footing B (m)	Observed settlement Y (m)	Modulus of compressibility E_{s_2} (MPa)
0.90	0.009—0.013	23.6—31.4
1.20	0.007—0.019	24.4—65.7
1.30	0.007—0.017	38.7—94.0

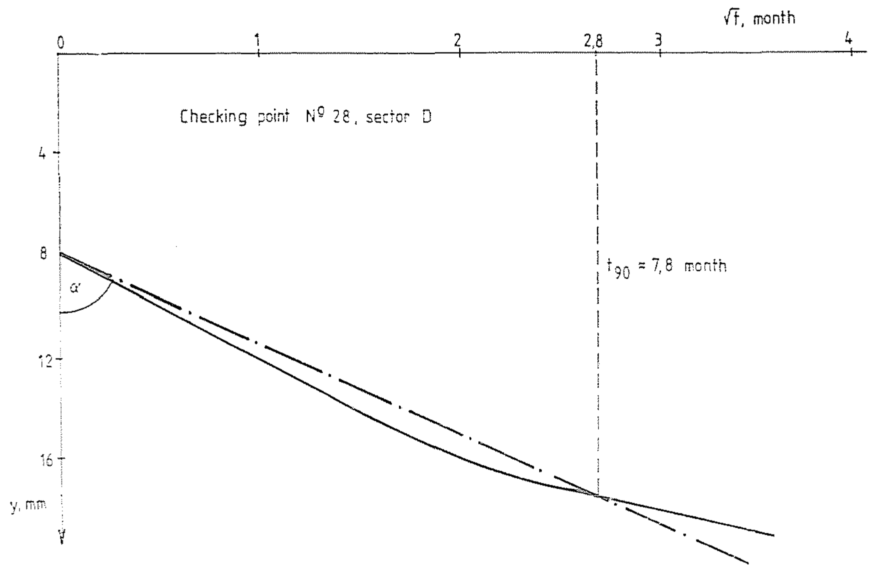

Fig. 11. Determination of consolidation time

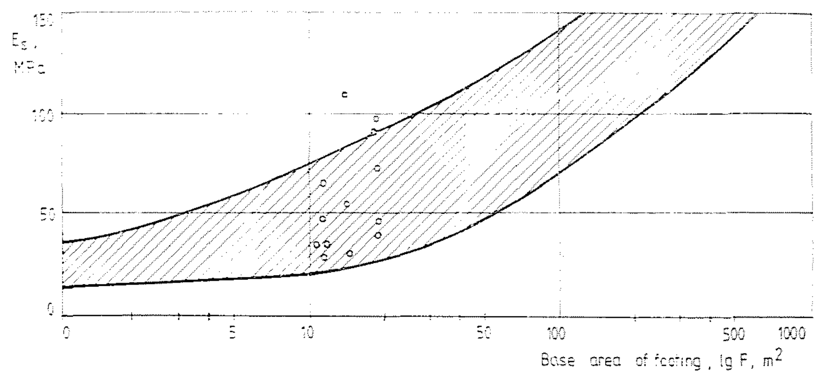

Fig. 12. Relation between the width of footing and the recalculated E_s values

These calculated values of the modulus of compressibility were also analyzed by means of the method of mathematical statistics. With a probability of 95 percent the value of E_s can be estimated to be:

$$E_s = 44.8 \pm 0.3 \text{ MPa}.$$

From labor test data (oedometer tests) the modulus of compressibility is much less than the reanalyzed value.

$$E_{s1} = 5.1 \pm 0.8 \text{ MPa} \ll E_{s2} = 44.8 \pm 0.3 \text{ MPa}.$$

Owing to the existence of neighbouring footings their interaction should be also treated. At 1—1.5 m below the foundation level a $\sigma_z \approx 86$ kPa, approximately uniformly distributed stress can be calculated from the spreading of stresses. Taking this excess stress into account, $E_{s3} \simeq 70.3$ MPa average modulus of compressibility was calculated. This result confirms again the contradictions of the determination method of E_s and the use of calculation model. The very high value of E_{s3} is a consequence not only of the stress spreading, but of the tendency of E_s increase with depth also, as it is proved by the test data.

4. Summary

E_s moduli calculated from settlement measurements are higher with 0.5—1 order than those measured in laboratory. These recalculated moduli agree well with published experiences [10] as it is proved by the points plotted on the border curves of Fig. 12.

Laboratory test results-oedometer tests, except on the soft saturated clays, are often heavily loaded by errors. Calculations based on such data are contradictory. There are many unanswered questions in this field, in spite of the abundant literature.

Test data may contribute to the design work, make easier the understanding the behaviour both of the soil and of the structure.

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