Diaphragm wall performance in soft clay excavation

La performance des parois moulées dans l'argile molle

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SYNOPSIS The performance of a retaining structure is dependent on the actual behavior of each of the three main structural elements: the retaining wall, the support system, the soil; and of the interaction between those three elements. The present incomplete knowledge of design parameters describing the stress-strain relationship of the soil when interacting with structures is the reason for presenting this paper which describes measurements of performance parameters for a diaphragm wall supporting an excavation in soft clay. Records of earth pressures, pore pressures in the soil, anchor loads, stresses in reinforcement steel, concrete strain, wall deflection and settlements are presented. The observations demonstrate discrepancies between the design earth pressures based on classic theory and the actually acting earth pressure. The load bearing capacity of the total soil structure system proved to be greater than assumed by the design. Vertical prestressing of the wall gives a considerable positive effect.

INTRODUCTION

Case histories reported in the litterature and experiences from various deep excavations show that the commonly used design methods based on classic earth pressure consepts lead to an underestimation of the bearing capacity and discrepancy between the predicted behavior and actual performance of retaining structures in soft clay. Methods of adjustments (moment reduction consepts) applicable for silty/sandy soils may on the other hand lead to an overestimation of the bearing capacity when applied in soft plastic clay excavation design. Design methods taking care of the interaction between soil and structure, such as finite element methods, are sensitive to stress-strain parameters for the materials involved, especially those of soil. The correlation between the common wellknown soil parameters and the modified soil parameters adequate for finite element methods is uncertain.

The foundation work for the new head office building for The Bank of Norway provided an opportunity to measure the performance of a cast in situ slurry-trench wall and to give a desireable contribution to further studies of the soil-structure interaction problem. The studies have so far resulted in the development of a handy computer program for design of retaining structures through all construction stages in a rapid and practical manner. This paper summarizes the results from the measurements performed on one of the instrumented panels, and compares originally computed values and backcalculated values with the actual performance data for the retaining structure observed during various phases of the construction.

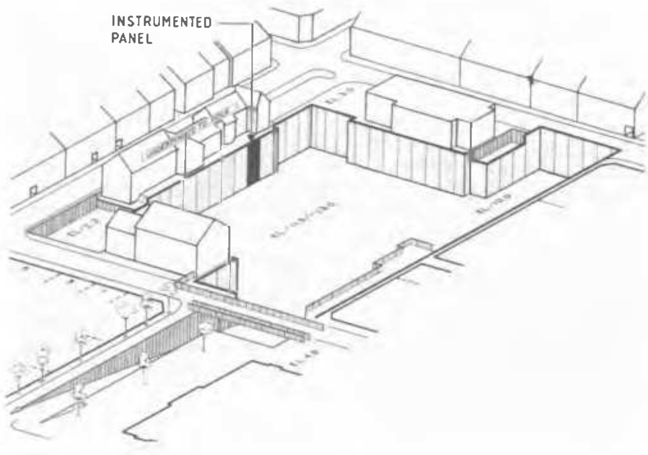

Fig. 1 View of the construction site

SITE CONDITIONS

A view of the construction site located in the center of Oslo is shown in Fig. 1. Previous and excisting older buildings at or near to the construction site was originally built on shallow raft foundations and shallow basement. The soil conditions are typical for the lower part of central Oslo. The ground surface is nearly level at approx. elevation + 3 m. Underneath a 2-3 m thick layer of fill material is a thick layer of soft clay. At this particular site the bedrock, which varies in depth from 10-29 m, is overlain by a thick deposit of normally consolidated marine clay

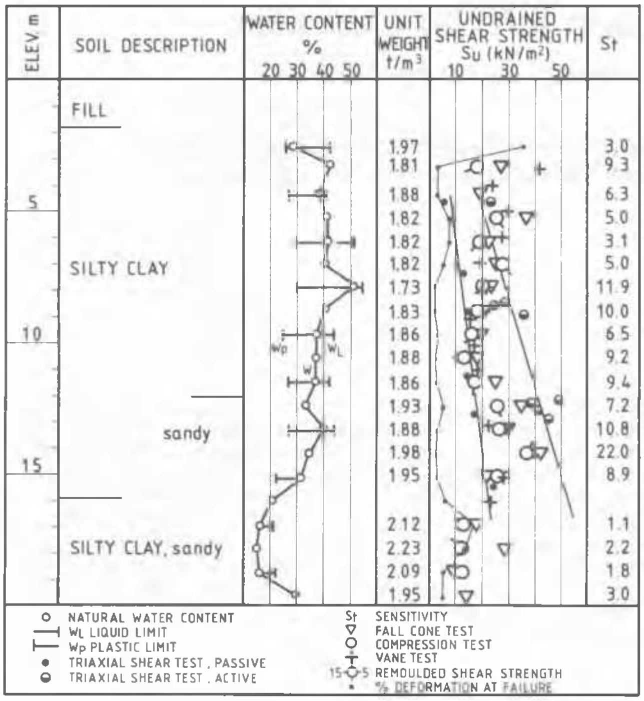

Fig. 2 Characteristic soil data

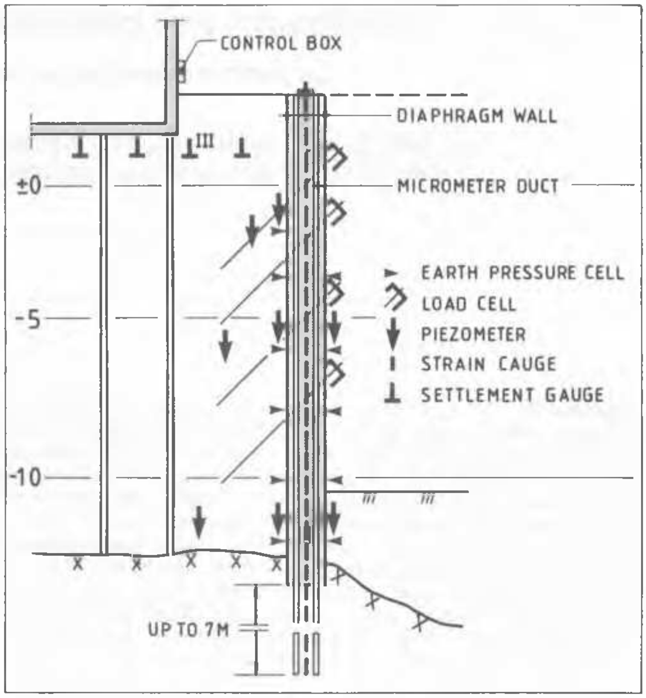

Fig. 3 Instrumentation in panel 116

of low sensitivity and with an undrained shear strength of 15-40 kPa. The lower part of the clay is slightly sandy. Bedrock at the site is alun slate which is crossed by frequent hard Permian dikes. The rock is also crossed by some waterbearing cracks. Right over bedrock is a thin waterbearing layer of sand and gravel. Characteristic soil data are summarized in Fig. 2. At the selected instrumented section of the diaphragm wall reported in this paper the depth to bedrock is nearly 20 m. The adjacent by law protected 17th century masonry buildings were underpinned to bedrock using bored steel piles before the diaphragm wall was constructed.

DESCRIPTION OF THE PROJECT

The Bank of Norway New Head Office Building is founded on bedrock by means of a peripheral load bearing wall and interior columns. The about 350 m long diaphragm wall which is 1 m thick and varies from 10 to 29 m height (total wall area approx. 6000 m^2) was constructed using the bentonite slurry trench method. The wall was keyed into the rock and prestressed by permanent vertical tendons anchored in rock to establish horizontal support at the bottom between the rock and the wall. During excavation of some 120.000 m^3 clay to a maximum depth of 16 m inside the diaphragm wall, a temporary support was established by anchor cables drilled through the wall and tied in the rock outside the wall. The anchors were prestressed to a load that allowed for an optimum displacement and structural behaviour during the various stages of excavation works.

The bottom slab at elevation - 12 m and a slab at elevation - 1 m form the permanent main support of the diaphragm wall.

INSTRUMENTATION

The measurements performed on various instrumentated wall panels were part of the construction control program, planned and carried out by the geotechnical consultants on the project. The observations and the continous check of the performance data allowed adjustments of the stresslevel of the anchors to obtain a safe and optimum behaviour of the retaining structure. Research funds were allocated for this part of the project allowing for a more extensive instrumentation and post evaluation of the wall behaviour than deemed necessary for the construction control purpose. The observations reported in this paper consentrates on and is limited to measurements on the most complete instrumented wall panel, no. 116. The measurements consists of: total earth pressures, porewater pressure at the transition zone between the wall and the soil, and in the soil 2-5 m outside the wall, settlements, anchor loads, stresses in reinforcing steel, wall deflections and strains in concrete on both sides of the wall. The types and positions of the instrumentation are shown on Fig. 3.

Earth pressure

The objective was to determine the magnitude and the distribution of the earth pressure on both sides of the diaphragm wall before, during and some years after the excavation work. Eleven total earth pressure cells of the vibrating wire type, as reported by (Øien 1958) were installed in vertical rows on each side of the wall to record active and passive earth pressures respectively. The cells were positioned in contact with the clay in the excavated slurry filled trench using inexpensive hydraulic jacks mounted in the reinforcing steel cage. The jacks were kept under pressure during panel concreting.

Concrete strain

The strain in the concrete wall were measured in both sides of the wall using a sliding micrometer of type ISETH. The method and device are described by (Kovāry & Amstad 1983).

The sliding micrometer records the distance between fixed heads every meter through a vertical micrometer duct. The ducts were placed and grouted in casings attached to both sides of the reinforcing steel cage. Measuring at various times gives differences of the distances, and thus the strain. The accuracy of the device in field is about 3 μ. Measurements of strains at two positions in the cross-section allow a calculation of the curvature of the wall. The high accuracy of the recordings give reliable wall deflection observations.

Deflections of the wall

The horizontal movement of the diaphragm wall was determined for several stages of excavation by means of both the above mentioned sliding micrometer and inclinometer measurements performed in a casing attached to the reinforcing steel. The micrometer measurements gave the more accurate and reliable observations.

Stresses in reinforcing steel

Stresses in the reinforcing steel were determined at four locations by means of vibrating wire load cells welded in series with vertical steel bars.

Anchor loads

A number of anchore heads on the wall were equipped with load cells in order to record the anchoring loads at various stages of the construction work. The load cells were of a type using the vibrating wire.

Pore pressure

Five vibrating wire pore pressure cells were mounted in the wall similar to the earth pressure cells. Additional fast reacting vibrating wire type piezometers were placed at various levels in the clay outside the wall. Hydraulic piezometers were placed in permeable layers just over the bedrock.

Ground settlements

A large number of settlement reference points were established around the site. Behind the instrumented panel reported here, the buildings were underpinned, and special settlement anchors were placed into the ground to allow records of ground settlements observations independent of the building settlement.

RESULTS

The most significant results of the measurement program related to time and construction work progress are shown in Fig. 4. Recorded earth pressures and wall deflections related to depth for various construction stages are shown in Fig. 5. Teoretical computed at rest pressure, active pressure and passive pressure are shown in Fig. 5 as well.

With regard to the recorded deflections, the first reading could unfortunately, due to construction progress, not be taken before the upper three meter of the soil was excavated. The wall deflection for the very first stage was therefore computed by a back-calculation based on the recorded behaviour of the wall during later construction stages. The corresponding adjustments on the recorded wall deflection are indicated in Fig. 4 and 5 as adjusted "0-lines". These adjustments have to be taken into account when comparing predicted versus observed behaviour.

The earth pressure recordings show that the initial pressure is "prestressed" by the fresh concrete pressure higher than the teoretical at rest pressure at the upper part of the wall, decreasing with depth. The records also show that the pressure during excavation were considerably higher than the active pressure, especially in upper part of the wall. It should be noted that optimum performance in particular with regard to deflections and outside ground settlements, was the criterion for the prestressing level of anchor loads.

The records of concrete strains and the reinforcement stresses show that the prestressed load of the vertical tendons, and the vertical component of the temporary anchor load reaction give a considerable contribution to the bending moment capacity of the wall.

The observed settlements have contributions of two different causes. The first cause is the displacement of walls due to shear stress mobilization in the interacting adjacent soil. The second cause is a temporary lowering of the pore pressure in the soil just above the rock due to leakages from the permeable parts of the rock through the anchor borings and through the rock to the excavated pit. The groundwater problem was met by means of a grouted curtain in permeable rock below the wall, by making anchor borings waterthight by grouting and by water infiltration in bedrock outside the wall. Separating the two settlement contributions is difficult. The total settlement is about 0.3 % of excavation depth, and it is considered that approx. 0.2 % is related to the ground water problem.

PREDICTED VERSUS OBSERVED PERFORMANCE

The design was carried out by a combination of a finite element method and a simplified method assuming a linear soil reaction modulus. The simplified method was used since significant variations in depth to bedrock and weight of buildings on the surface made it necessary to design a large number of different wall sections. The method was calibrated

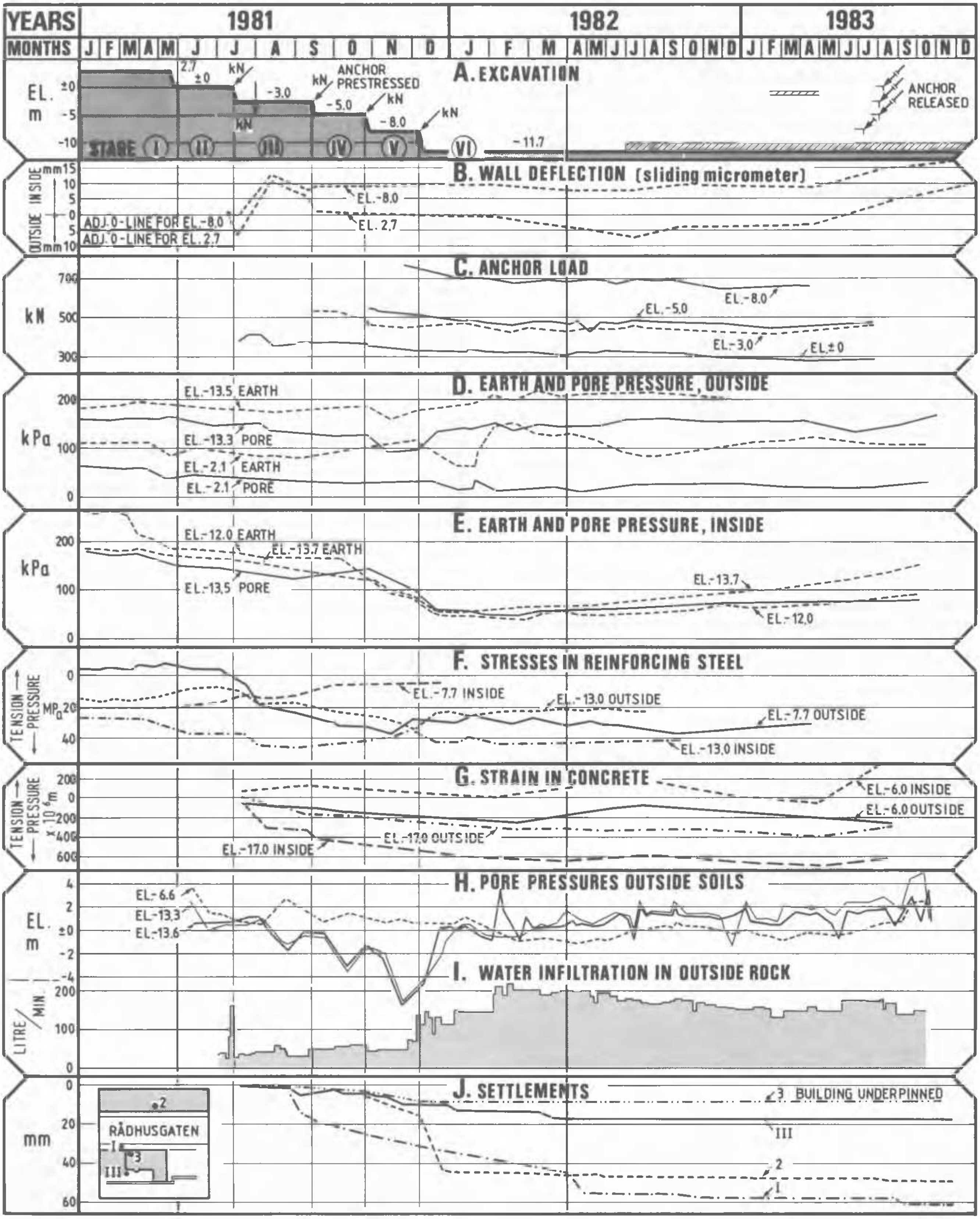

Fig. 4 Results of the measurements

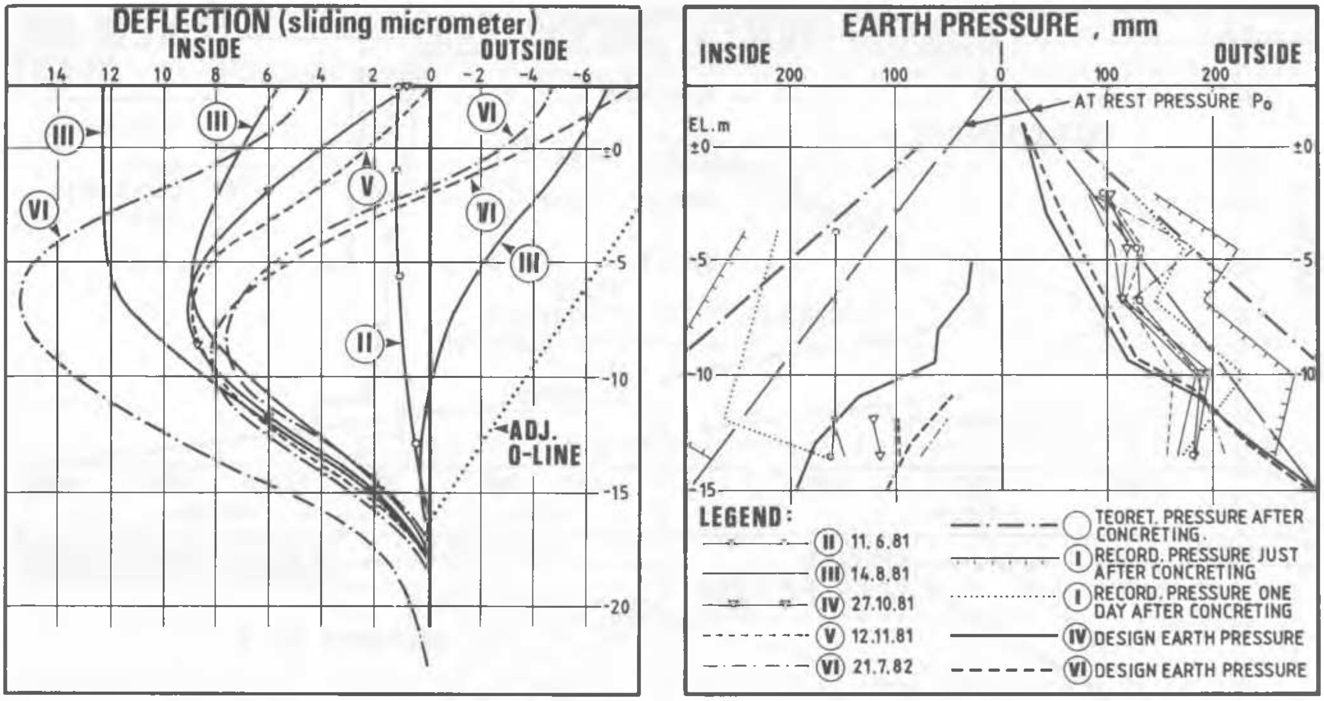

Fig. 5 Wall deflections and earth pressure for various construction stages.

to the finite element method for two sections. Both methods were also checked against performance records from previous projects, reported by (DiBiagio & Roti 1972), (Roti 1978) (Mørk & Roti 1979).

The proper design of a retaining structure consisting of various interacting elements such as soil, wall and anchors requires knowledge of parameteres describing the stress-strain relationships of the interacting elements. Using well-known structural design parameters for the anchors and the wall, it remains to select a parameter for the soil. The stress-strain relationship of the soil is not linear and is dependant on the displacement of the wall. The parameter used in the simplified design method, "C-modulus", is defined as a coefficient of soil reaction with dimension (kPa/m) valid for a 1 m wide wall section. Requirements for maximum mobilized shear stresses in the soil for active and passive earth pressures give limitations for the range of validity of the "C-modulus". The design method includes an iteration process starting with an initial at rest earth pressure prior to excavation work. During excavation the earth pressure decreases on the active side, and increases on the passive side of the wall. The "C-modulus"has to be adjusted in such a way that the earth pressures always stay within the defined design limits for active and passive states, respectively.

The original design-prediction of the wall deflection is shown in Fig. 6, in comparison with the actual observed deflections and with backcalculated deflections after adjustments on the "C-moduli". The "C-moduli" used in both calculations are shown as well. The observed behaviour is more favourable than predicted. The extensive back-calculations varying the soil-structure interaction parameter "C" were carried out best possible observed performance at all construction stages. The results recorded in Fig. 6 show an underestimation of the bearing capacity of the interacting system in spite of the use of previous experience from similar projects. Explanations as to the more favourable performance may be found in two factors. The vertical prestressing of the wall by vertical rock anchors and by the vertical components of the temporary tie-back anchors have given a general compression and thus little resulting tension in the wall. This is evident by the records of reinforcing steel stresses and of the concrete strains. This has led to a smaller deflection of the wall than predicted. The second important factor for the favourable performance is the restraining of the foot of the wall into the rock. This favourable factor was not taken into consideration at the design stage, because of uncertainty as to the degree of rigidity which could be achieved.

The results from the measurements of the described project give us opportunity to study stress-strain parameters for soil and soil-structure interaction problems using finite element design methods.

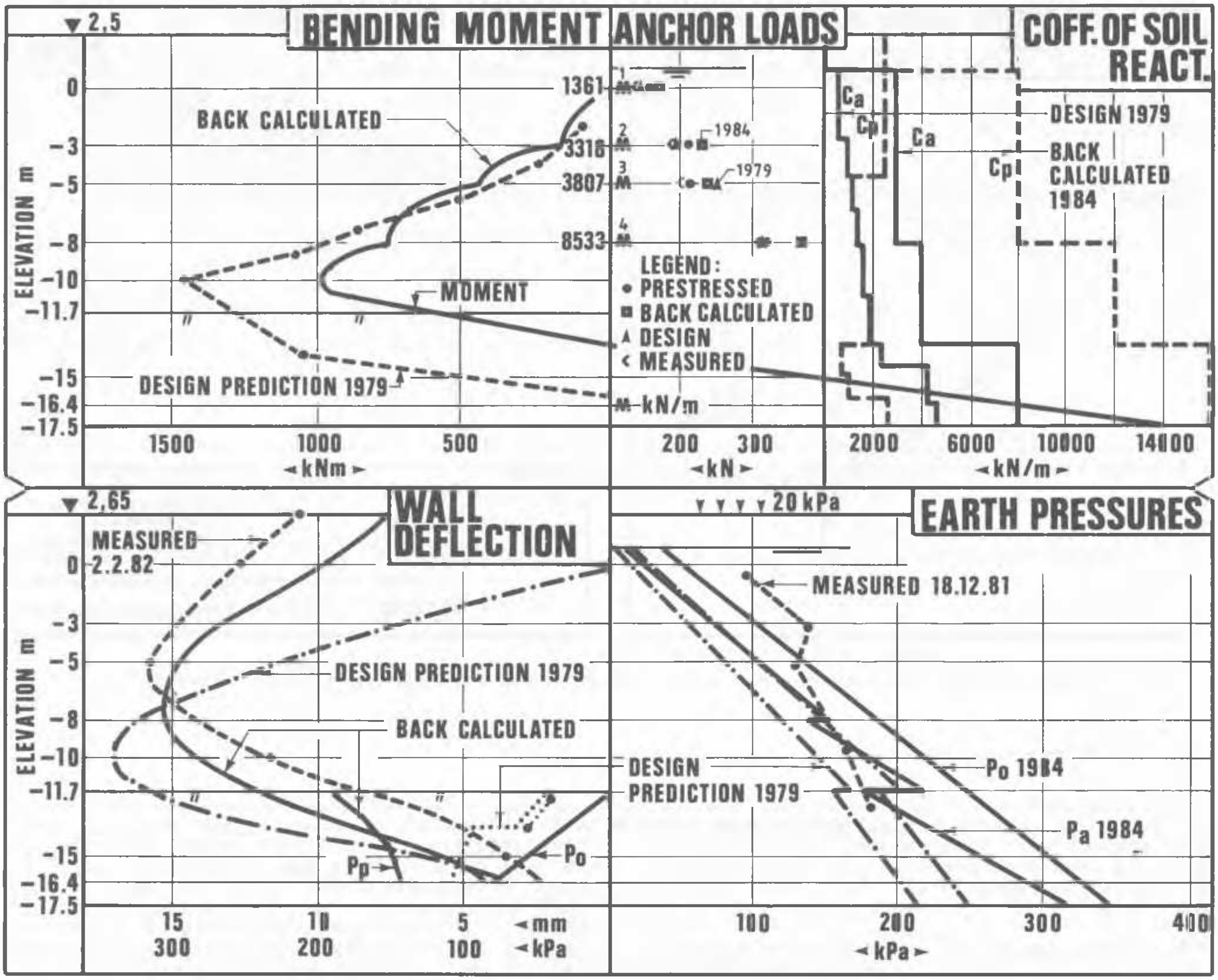

Fig. 6 Comparison between predicted, measured and back calculated behaviour.

References:

DiBiaggio, E. & Roti, J.A. (1972) Earth pressure measurements on a braced slurry-trench wall in soft clay. European Conference on Soil Mechjanics and Foundation Engineering, Madrid 1972. Proceedings, Vol. 1, p.473-483.

Kovary, K. & Amstad, Ch. (1983) Fundamentals of deformation measurements in geomechanics. International Sumposium on Field Measurements in Geomechanics, Zürich 1983, Proceedings, Vol. 1 p.219-239.

Mørk L. & Roti, J.A. (1979) Instrumentation and measurements on slurry-trench wall in soft quick clay in Drammen. (Norwegian) Nordiska Geoteknikermøtet 1979, Helsinki, Finland. Proceedings, p.67-76.

Roti, J.A. (1978) Slurry trench walls as structural element, illustrated by measurements and calculations (Norwegian). Seminar Norwegian Society of Chorterel Engineers, Proceedings, Fagernes 1978.

Øien, K. (1958) An earth pressure cell for use on sheet piles, Oslo Subway, Brussels Conference on Earth Pressure Problems, 1958, Proceedings, Vol. 2 p.118-126, also as Norwegian Geotechnical Institute Publication NO. 33.