Foundation on creep piles: Design parameters, graphical presentation by computer of resultant force systems as well as an analysis of test pile results

Fondation sur pieux sous charge de fluage: Paramètres de calcul, présentation grafique des résultats par ordinateur et analyse d'un essai in-situ

A. FREDRIKSSON, Ph.D., ADG Grundteknik AB, Stockholm, Sweden
R. ROSÉN, Civil Engineer, ADG Grundteknik AB, Stockholm, Sweden

SYNOPSIS According to a new principle for design of building foundations in soft cohesive soils, devoloped in Sweden, the building is founded on a piled raft. The use of this design principle is discussed in this report. A case record from design of foundations for dwelling houses using this design principle is also presented.

INTRODUCTION

The pile foundation on conventional friction piles is designed to carry the total load of the building with a safety factor against pile failure of about 3.

According to a new principle of design, developed in Sweden (Hansbo et. al.,1973 and Hansbo and Källström, 1983), the building is founded on a piled raft. The total load of the building is assumed to be carried partly by the piles and partly by direct contact pressure at the raft/soil interface, as shown in Fig. 1. The design load of the piles is equal to the "creep load", meaning the pile load that causes a state of creep failure. The piles are designed so that the load in excess of the preconsolidation pressure of the clay is assumed to be carried by the piles and the rest by contact pressure.

In cases where the average net load increase exceeds the preconsolidation pressure of the clay, the object of creep piling is to reduce consolidation settlements. This is achieved by the fact that the stress increment exceeding the preconsolidation pressure of the clay is transferred

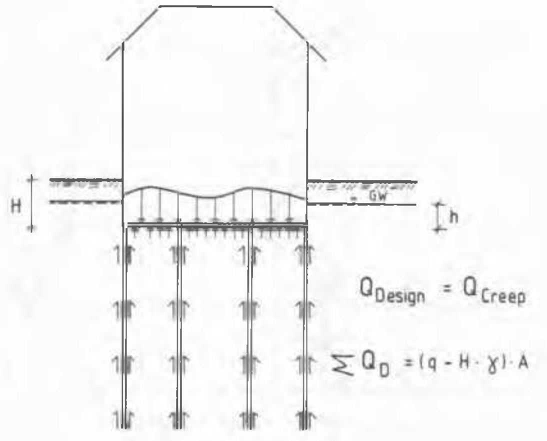

FIG. 1. New design principle for foundation on friction creep piles.

to greater depths. Moreover the piles can be distributed in such a way that the differential settlement of the building is minimized.

CALCULATION OF SETTLEMENTS

To be able to calculate the final settlements in any point at the base of a building, not only the compression properties of the soil, but also the stiffness of the building, including the foundation, must be known. If the superstructure is statically indeterminated, differential settlements will cause a redistribution of support reactions and section forces in the superstructure. This redistribution in its turn will affect the settlements.

A correct consideration of the soil-structure interaction requires very extensive calculations. In order to make such calculations computer programs have been developed to take soil-structure interaction into consideration. The superstructure is modelled by finite element methods and the calculation of the settlements in the soil is based on the oedometer modulus, (Beigler, 1976). The vertical stress increase within the soil from contact pressure is calculated according to Boussinesq stress distribution. The vertical stress increase from the load on the creep piles is determined using Geddes (1966) integration of the Mindlin equation for a pile with linear variation of skin friction.

In order to get consistency between distribution of contact pressure, distribution of settlements and the deformations of the building, an iterative method is used in the computer program.

CASE RECORD

The foundations of dwelling houses, in the blocks Sigurd and Edda, in the town Uppsala, have been designed by the authors, according to the new principle of design. In Fig. 2 is shown a plan over the block Sigurd. The buildings have 3 - 6 storeyes and a basement. The houses are constructed of cast in-situ concrete.

The subsoil at the site of the buildings consists of soft clay underlain by silt and sand.

TABLE I.

Pile	Dimension
B1	18 m wood + 10 m concrete 0.27 * 0.27 m
B2	32 m concrete 0.27 * 0.27 m
B3	35 m concrete 0.27 * 0.27 m
B4	18 m wood + 10 m concrete 0.27 * 0.27 m
B5	32 m concrete 0.27 * 0.27 m
B6	35 m concrete 0.27 * 0.27 m

TABLE II.
Results from load tests.

Pile	$Q_{Failure}$, kN	Q_{Creep}, kN	Q_C/Q_F
B1	560	490	0.88
B2	550	480	0.87
B3	670	560	0.84
B4	570	510	0.89
B5	460	390	0.85
B6	630	530	0.84
		Mean	0.86

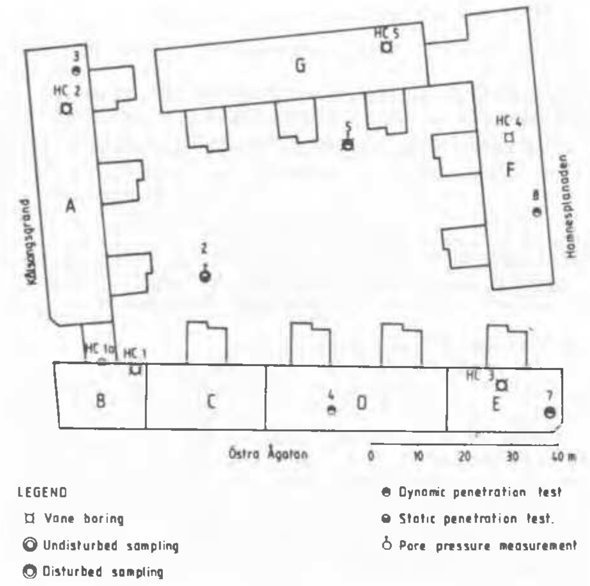

FIG. 2. Plan over the block Sigurd.

The thickness of the clay layer is about 40 m. The rock is about 100 m below the ground surface. Typical soil characteristics are presented in Fig. 3.

Determination of preconsolidation pressure and deformation characteristic of the clay.

The preconsolidation pressure has been determined in the laboratory by using oedometer tests with a constant rate of strain (CRS-test). The Modulus and permeability evaluated from a CRS-test are showned in Fig. 4. In the same figure is also showned the used variation of Modulus with the effective overburden pressure. This variation is expressed by four parameters, namely M_L, a, $\bar{\sigma}_L$ and m, according to Larsson and Sällfors, 1981. The variation of preconsolidation pressure with depth is showned in Fig.5. The variation of the modulus parameters with depth is showned in Fig. 6. In the figure is also marked the design values used for building G.

Load tests on friction piles.

In order to investigate the most economic pile type for the buildings in block Sigurd were 6 test piles loaded to failure. Three different

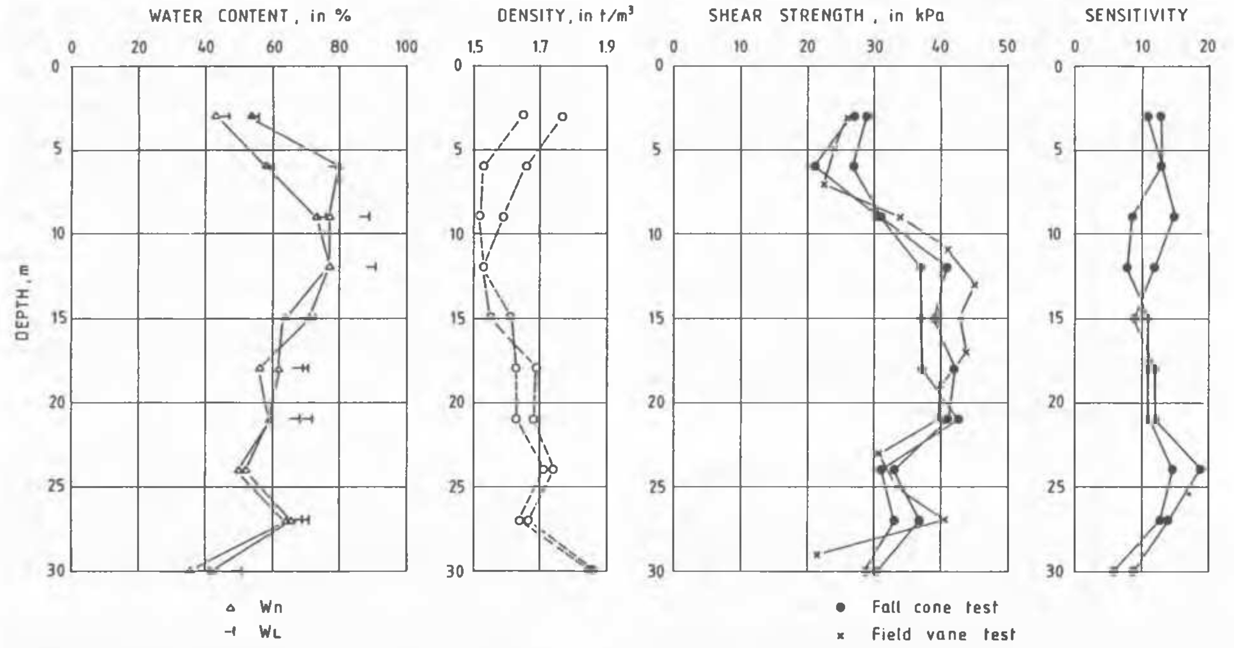

FIG. 3. Geotechnical characteristics of the subsoil at Sigurd.

pile types were tested, se table I. The piles were tested 112 days after pile driving. The piles were loaded stepwise and, in each step, a load of about 1/10 of the estimated failure load was applied. In each step, the load was kept constant for 18 minutes and the creep displacement during the loadstep was measured. The results from all load tests are summarized in table II. The relationship between the creep load and the failure load is about 0.86. This is consistent with other experiencies, see Bengtsson and Hansbo, 1979.

In Table III and IV the calculated failure loads based on shear strength determined by the field vane test are compared with the observed failure loads. As can be seen, the relationship between the calculated and observed load is 0.89 for the wood piles and only 0.49 for the concrete piles.

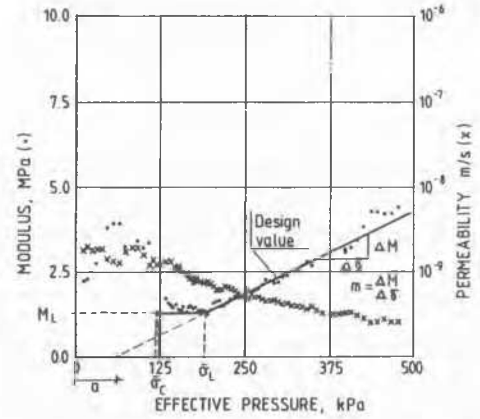

FIG. 4. Modulus and permeability evaluated from CRS-test.

TABLE III.

Pile	$Q_{Failure}$, kN	Q_{Vane}, kN	Q_F / Q_V
B1	434	471	0.92
B4	444	516	0.86
		Mean	0.89

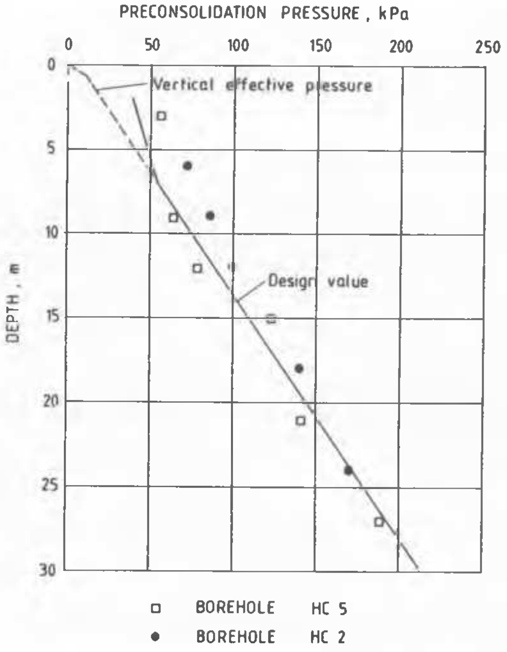

FIG. 5. Variation of preconsolidation pressure with depth.

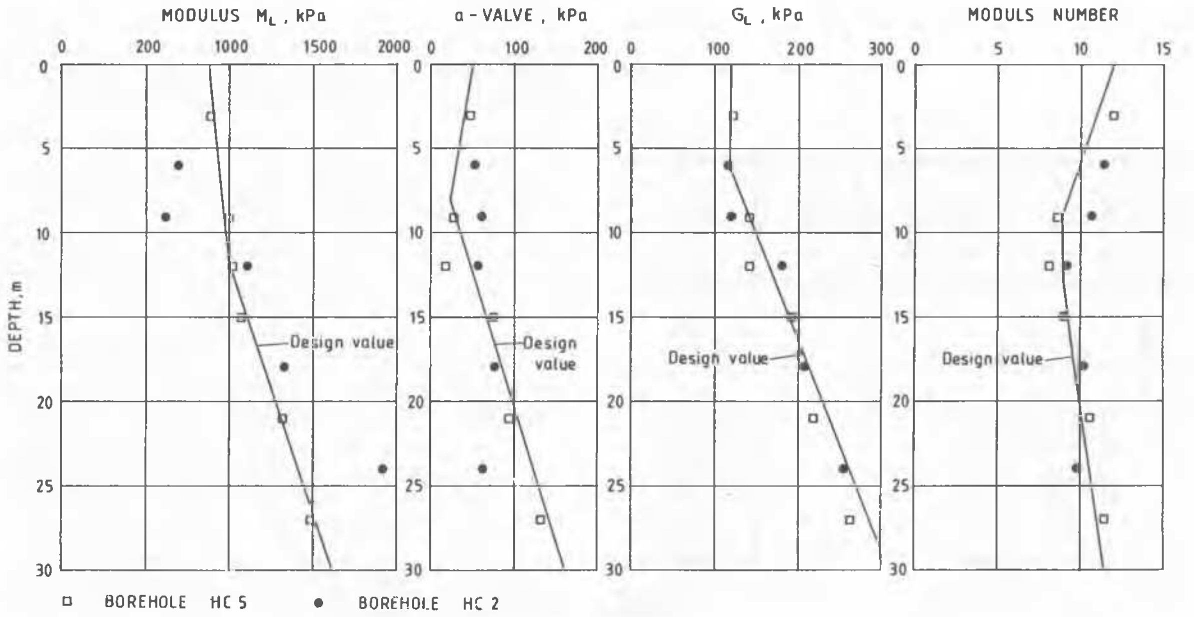

FIG. 6. Variation of settlement characteristics with depth.

TABLE IV.

Pile	$Q_{Failure}$, kN	Q_{Vane}, kN	Q_F / Q_V
B2	550	1176	0.47
B3	670	1289	0.52
B5	460	1176	0.39
B6	630	1289	0.49
		Mean	0.49

The choosen piles for the foundations consists of 18 m long wooden piles, spliced with 7 m long concrete piles. The creep failure load was estimated to 420 kN for this type of pile.

Calculated settlements.

In Fig. 7 is shown the computer model for the building G. In the figure is also marked the positions of the creep piles. The calculated settlements after 50 years are shown in Fig. 8. The consolidation settlements have been calculated under the assumption of onedimensional flow of the pore water. Calculated settlements, contact pressure , section forces in walls and plate were presented in diagrams by a computer. This made the design process easier and the results more accessible.

CONCLUSION.

The following conclusions have been drawned from using the new design principle for foundations on friction piles.

- The new principle of foundation design has turned out to be a cost-effective alternative to conventional friction piles.

- The number of piles can be reduced.

- Cheap wooden piles can be used.

- The bearing capacity of the piles is better utilized.

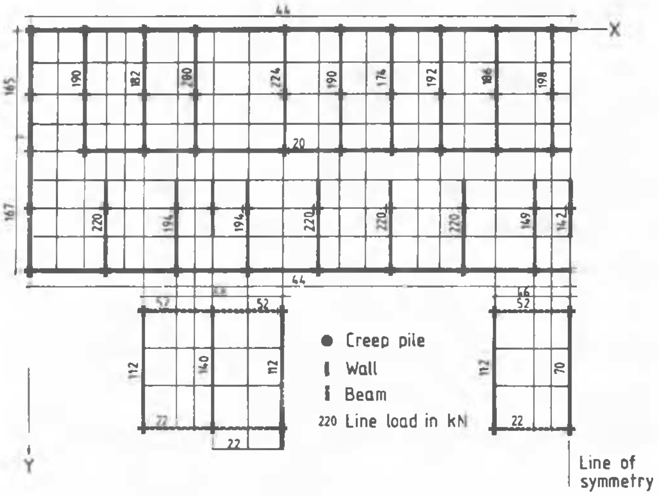

FIG. 7. Computer model for building G.

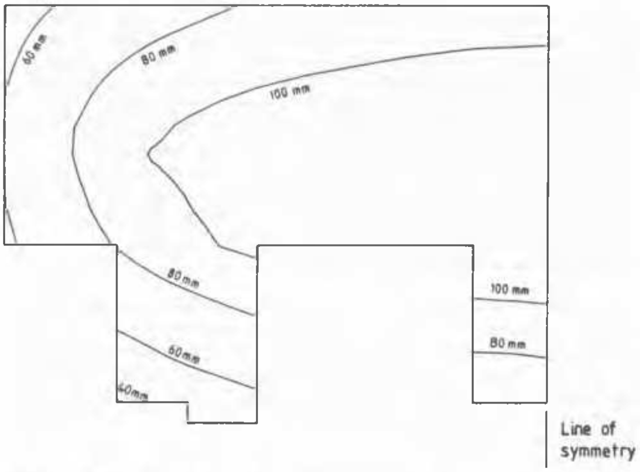

FIG. 8. Calculated settlements after 50 years.

- The differential settlements can be minimized using creep piles.

- Differential settlements between the building and the surrounding soil is also minimized.

- The time for construction of the foundations is reduced.

ACKNOWLEDGEMENT.

The foundations of the buildings in the presented case record were designed by order of A-projektering, Västerås, Sweden, when the authors were employed at J&W (AB Jacobson & Widmark, Lidingö, Sweden). The authors wish to express appreciation to J&W and A-projektering AB for the possibility of using the design material for this paper.

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