Experiences of Tapered Friction Piles

Expériences des Pieux de Frottement en Forme Coince

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SYNOPSIS Friction piles of steel or precast concrete with a tapered lower portion have to some extent been used succesfully to compete both tecnically and economically with traditional, point bearing piles or straight sided friction piles.

In the article various efforts to compute the bearing capacity of friction piles is discussed. The main objective has been to give a number of case descriptions of the use and static load testing results of precast concrete friction piles, with a tapered lower end of 1.0 to 3.0 meters of lenght.

INTRODUCTION

The predetermination of the bearing capacity of tapered friction piles based on the boring and sampling data of a given site still seems to meet invincible difficulties. The traditional bearing formulas built on the values of various soil parameters and driving energy etc. have often obviously given greatly erroneous and misleading information on the bearing capacity of friction piles. Costly and time consuming loading tests are generally unavoidable to obtain reliable bearing capacity values.

According to the case descriptions given in the reference litterature, the influence of the tapered shape on the bearing capacity of friction piles has a dominating share among the factors involved. However, the loading tests performed by the writers have shown that the effect of the tapered shape is greatly influenced by circumstancial factors i.e. soil conditions in general, the pile driving method and most of all by the degree of compactibility of the soil stratum, which is penetrated by the tapered portion of the piles. If the silt content of this stratum exceeds certain limits and cohesion begins to dominate the strength parameters of the material, the effect of tapered shape of a pile may remain negligible.

DEVELOPMENT OF FRICTION PILING IN FINLAND

In geological areas dominated by deposits of postglacial, highly compressible clays, often overlain by still younger strata of mud and peat, piling with wooden piles has been the traditional solution for building foundations until the 1940:s. The slightly tapered shape of the wood trunks was an obvious advantage in cases, where a considerable lenght of the pile was driven through loose or medium dense friction material, whereat the bearing capacity is built merely on the skin friction and not the tip bearing principle.

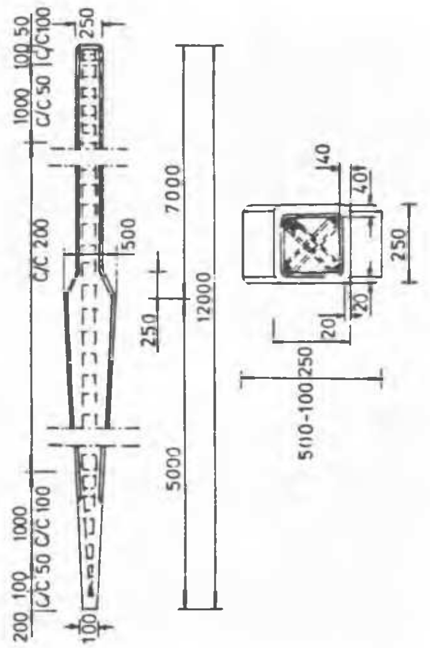

Fig. 1 First type of tapered precast concrete pile 1955

Lately, tip bearing precast concrete piles have been most common practice, where piling is needed. In certain considerable areas in Finland the strata of sand, silty and gravel certain extend to a great depth and consequently unreasonable driving effort through these strata is at hand, to obtain an adequate bearing capacity. In these conditions, in order to limit the pile lengths, a desire to simulate the advantageous shape of wooden piles lead in 1955 to an experiment with spear shaped concrete friction piles, Fig. 1. Since that time, the use of similar or modified piles has been developed to a routine, mostly using a standardized pile shown in Fig. 2. The lenght of the tapered portion varies from 1.5 to 4.0 meters and the cross section of the pile shaft from 250 by 300 mm^2 to 400 by 500 mm^2.

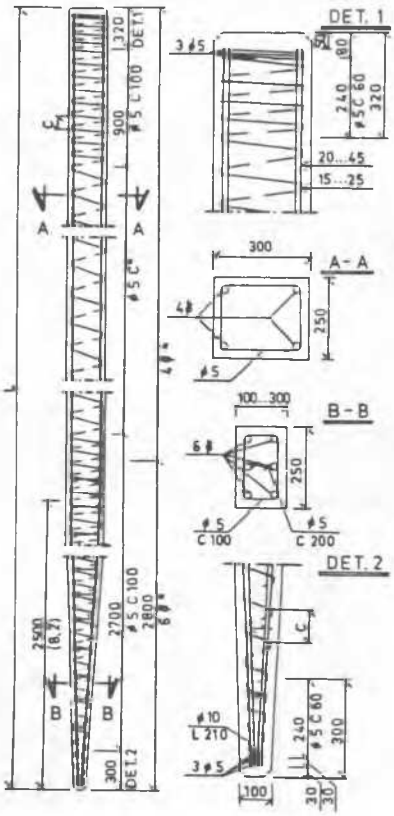

Fig. 2 Finnish standard tapered precast concrete pile

THE BEARING CAPACITY OF FRICTION PILES ON THE BASIS OF FIELD EXPLORATION DATA

The ultimate bearing capacity at failure P_u consists of the point resistance P_p and side resistance P_s, i.e.

$$P_u = P_p + P_s \quad (1)$$

The point resistance may be expressed using the bearing formula of spread footings:

$$P_p = N_q \cdot A \cdot p_D \quad (2)$$

where

N_q = bearing coefficient (Fig. 3),

A = area of the pile tip,

p_D = effective overburden pressure at the elevation of the pile tip.

N_q = a function of the angle of internal friction $\emptyset$ as shown in Fig.3.

The accuracy of the bearing capacity values obtained by using the formula (2) is rather poor, while the friction angle of a soil is not accurately known, and in addition, is subject to a change due to the compaction effect of the pile driving.

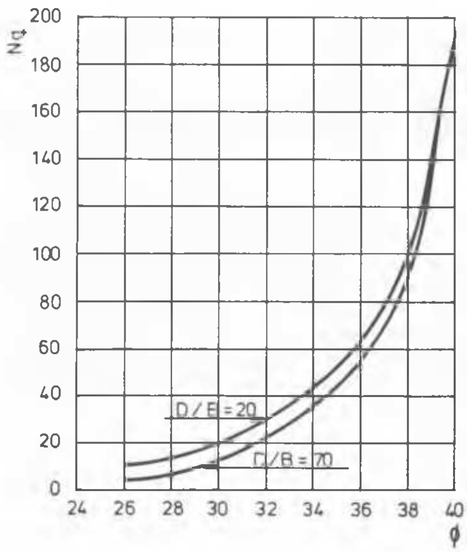

Fig. 3 Bearing coeffient N_q vs. $\emptyset$

The side resistance by straight sided (non-tapered) friction piles is obtainable from the well known formula:

$$P_s = \sum_{z=0}^{D} K_\delta \cdot \tan\delta \; p_z \, \Delta z \quad (3)$$

On tapered portions of the pile the increase of the side resistance can be 2-to 4-fold, Norlund (1963), Hartikainen (1976). Using the parameters given in Fig. 4, a formula of the side resistance can be written:

$$P_s = \sum_{z=0}^{D} K_\delta \, p_z \cdot \sin\delta \cdot u_z \cdot \Delta z \quad (4)$$

where

u_z = the pile perimeter.

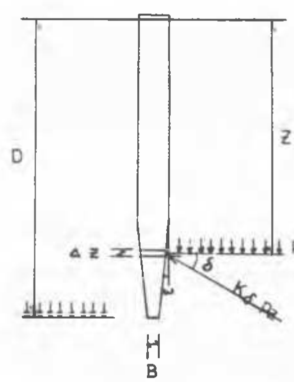

Fig. 4 Side resistance of a pile

Thus, the side resistance depends on the following factors:

1. The friction angle of soil, Ø
2. The friction angle on surface of sliding, δ
3. The taper of the pile, ω
4. The effective overburden pressure in soil, p_z
5. The dimensions of the pile.

As claimed by Norlund (1963), K_δ is directly influenced by ω, and so is the bearing capacity, Fig. 5

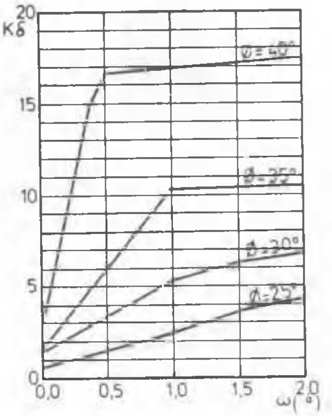

Fig. 5 The relationship between taper ω and K_δ .

The curves are valid only, when δ is equal or greater to Ø. Values of the angle δ are greatly affected by the smoothness of the pile facets i.e. the rougher the surface the higher the value of δ. On the other hand, with increasing cross section area of the pile the density of the soilmass in the vicinity of the pile, as well as Ø, is increasing. It is obvious, that the ratio Ø/δ is not constant. To a given type of pile, Ø is growing with increase of δ near the pile, but not at proportionally same rate.

The accuracy of the values of bearing capacity obtained using the formulas (1)...(3) depends mostly on the accuracy in evaluating the friction angle. Best results can be reached if Standard Penetration Tests have been run at the site. The dependence between the SPT resistance N and friction angle Ø has, among others, been given by Peck, et al (1953).

However, the common practice in Northern Europe for evaluation of the angle of internal friction at a soil stratum is mostly based on the results of Weight Sounding Tests, ISSMFE, 1979 and/or on laboratory tests on disturbed samples, whereat the ultimate rate of accuracy lays somewhere $\pm 2^{\circ}$. This scatter of Ø-values has an influence of -60 to + 250 % to the breaking load computed from the piling formulas. Thus, these formulas have very little or no value at all at the stage of final design of pile foundations. They may be useful at a preliminary stage of planning, merely for the choice of the type of piles.

The Norwegian Piling Code, Den Norske Polekomite (1963), presents some diagrams and empiricial formulas, based on the driving resistance, to determine the bearing capacity of friction piles. However, these formulas are not applicable for tapered piles, mainly because the compaction effect caused by pile driving in the soil mass around the piles is not considered.

DETERMINATION OF THE BEARING CAPACITY BASED ON THE DRIVING PERFORMANCE

All North European piling codes include piling formulas based on the driving performance, in Finland the modified Krüger formula:

$$Q_n = 0.8 \frac{k \cdot W \cdot h}{s + 1/2 \cdot c} \left(1 - 0.1 \frac{W_p}{W} \right) \qquad (5)$$

where
W = weight of hammer,
h = height of blow,
W_p = combined weight of pile and pillow,
s = remaining penetration,
c = elastic rebound,
k = 1 by free fall hammer and
k = 0.8 by cabled drop hammer.

The fact that on tapered piles a considerable part of the driving energy is consumed to accomplish a dislocation of a soil mass around the pile has not been considered at any of these piling formulas for driven piles. Especially for tapered piles the presumption that the entire driving resistance should be concentrated to the pile tip, is erroneous.

Only in very dense strata of friction material, where the pile penetration at the end of the driving doesn't exceed more than 3-to 4-fold the penetration of a tip bearing pile, the piling formulas may give reliable values of bearing capacity. These conditions of penetration seldom prevail for friction piles in loose or medium dense silt and sand strata.

DETERMINATION OF FRICTION PILE SETTLEMENT

According to the Finnish Standards for Foundation Planning, the design of foundation structures is based on Limit State Analysis.

The applicable design load has to be chosen based on the bearing capacity determined at the Ultimate State Limit, or on the settlement of the pile derived at the Service State Limit.

The breaking load of friction piles is not unambiguous, while the bearing capacity is usually increasing unlimited with the increase of pile settlement. If test load data are available, there is no need to determine the breaking load. In these cases the applicable design load can be directly considered to be the load at permissible settlement divided by the factor of safety.

Displacement ε caused by an unit force in a semi-elastic solid substance can be presented in z - r coordinate space in accordance with the strain distribution model by Boussinesq,

$$\varepsilon = P\frac{(1+\nu)}{2\pi E}\left(\frac{r^2}{R^3} + \frac{2(1-\nu)}{R}\right) \qquad (6)$$

where

E = Young´s modulus and
ν = Poisson´s modulus
R = r_o = radius of a pile or a footing

The unit friction force along the surface of a straight sided pile can be expressed according to the symbols of Fig. 6 as follows

$$P = R_\zeta \cdot 2 \cdot \pi \cdot r_o \, d_\zeta \qquad (7)$$

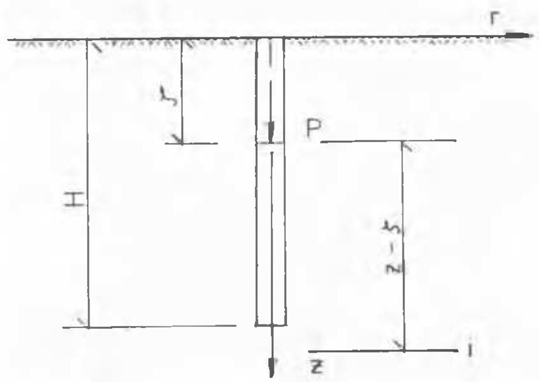

Fig. Displacement caused by Force P at point i

By inserting P (7) into formula (6) and by integrating along the length of the pile, following equations can be written according Przystanski, (1963)

when $z \le H$

$$\varepsilon_{zr} = \frac{r_o R_i (1+\nu)}{E} \cdot \left\{ \frac{z}{\sqrt{r^2+z^2}} + (2-2\nu) \ln\left(\frac{z}{r} + \sqrt{\frac{z^2}{r^2}+1}\right) + \frac{2(1-\nu)}{P}(H-z)\right\} \qquad (9)$$

and when $z > H$

$$\varepsilon_{zr} = \frac{r_o R_i (1+\nu)}{E} \cdot \left\{ \frac{z-H}{\sqrt{r^2+(z-H)^2}} - \frac{z}{\sqrt{r^2+z^2}} - (3-2\nu) \ln \frac{z-H+\sqrt{r^2+(z-H)^2}}{z+\sqrt{r^2+z^2}} \right\} \qquad (10)$$

If the compaction effect is not considered, the settlement of a pile group always exceeds that of a single pile, because the pressure bulb under a pile group covers a larger area than the bulb underneath a single pile.

The settlement of a group of friction piles is often estimated using the same principle, Fig.7, as by a group of cohesion piles. It is presumed, that the settlement of a pile group responds that of a stiff spread footing, loaded with the total load of the pile group and situated at the elevation of the lower 1/3-point of the length of the piles.

The increase of density in the soil mass is taken into account by using soil compressibili parameters increased by a rate, which is estimated based on sounding data or experience gained by loading tests in similar soil conditions. In pure friction soil, where the compacting effect extends underneath the elevation of pile tips, that elevation is presumed to be subjected to the loading.

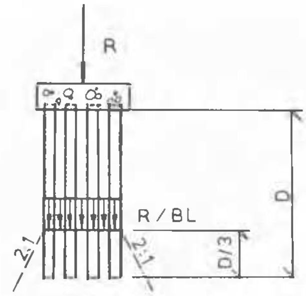

Fig. 7 Load transfer at a pile group

COMPACTION EFFECT OF PILING

Few theoretical experiments with scale models have been performed to determine the compaction effects in soil strata subjected to piling. A.A. Grigorjan, (1971) described measurements of compaction of a soil mass in a laboratory test basin by driving concrete wedges of various shapes into the soil mass. Determinations of the relative densities before and after the compaction were performed.

As most feasible and yet sufficiently accurate means to control the degree of densification of a soil mass around and underneath a pile or pile group at an actual piling site, in situ penetration tests of a kind or another are recommended, ISSMFE, 1979.

In an actual case of compaction piling of very loosely stratified, uniform sand deposits (silt content 5 to 30 %) with wedge shaped, 3 meters long precast concrete piles, Weight Sounding Tests (WST) were performed at various stages of the piling.

In Fig. 8 sounding A was performed in the natural state, sounding B when piles No. 1 to 4 had been driven, C after pile 5 and D after the driving of the entire pile group.

MISCELLANEOUS LOADING TESTS

Parsons (1963) and Hartikainen (1976) have presented extremely informative pile test loading data, including comparisons between bearing capacities of regular straight sided and tapered piles. Both studies end up to a conclusion, that considerably higher bearing capacities are achieved with friction piles of a tapered shape.

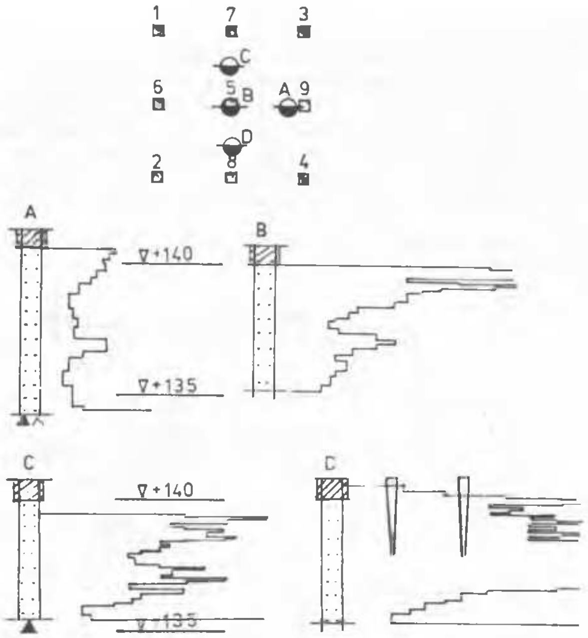

Fig. 8 Compaction effect of piling in a sand deposit. Diagrams of WST tests. Uppermost the pattern of piles with driving sequence.

CASES OF TAPERED FRICTION PILES IN FINLAND

General

In Finland, tapered, wedge shaped precast concrete friction piles have been used in hundreds of cases of pile foundations for residencial and industrial buildings,machinery as well as for miscellaneous other structures. The distribution of pile compression stress at allowed design loads in a large number of various cases is shown in Fig. 9.

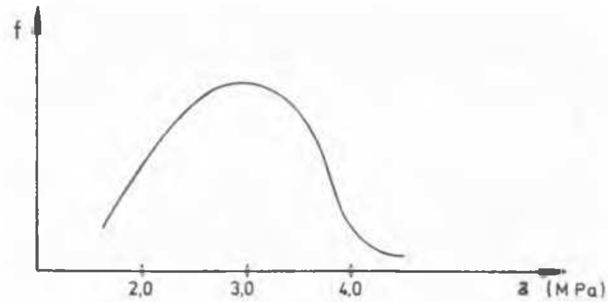

Fig. 9 Compression Stress Distribution of piles

Through the history of the cases known to the writers, there has not been reclamations of a construction damage, caused by an unsatisfactory behaviour of the tapered wedge shaped piles. The presumption is though, that a careful foundation planning has been performed on basis of a profound site exploration. In critical cases test piling and loading tests have been performed.

In situ test loading data are available only of a limited number of piling sites. Four actual cases, where best information on pile driving methods, soil conditions on the site and test loading data have been available, are presented.

1. Cardboard and paper mill, Kemi, Northern Finland, 1969
2. Sawmill, Eno, Eastern Finland, 1979
3. Mechanical Wood Industry warehouse, Jyvaskyla, Central Finland, 1968
4. Office and Warehouse Building, Helsinki, 1959

Typical grain size distribution curves of the pile bearing strata at the sites above are given in Fig. 10.

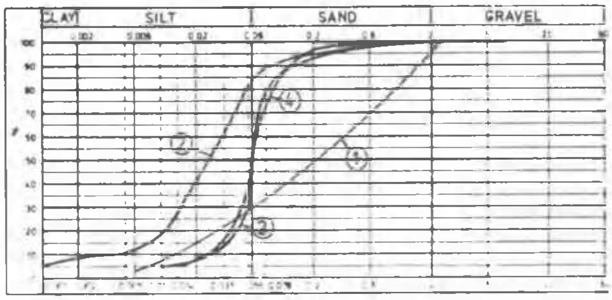

Fig. 10 Typical Grain Size Distribution of the Pile Bearing Strata at the Sites 1 through 4

In all cases in question the piles have been placed into close groups of at least 5 piles, in the Kemi case even 14 and 28 piles.

In the following the computed breaking load is considered to be the load resulting to a settlement twice the settlement value received with 90 % of the same load.

Any comparisons of computed values of bearing capacities vs. test loading results are not presented herein on the grounds, that Ø-values of the soil strata have not been determined with a necessary accuracy in due time.

Cardboard and Paper Mill, Kemi

The piles used at this case had a shaft cross section of 300 by 400 mm^2, length of the taper 2.5 m and the tip cross section 300 by 150 mm^2 The pile lengths varied from 7.5 to 9.5 m. The ultimate design loads were 400 kN at building foundations and 300 kN under the machinery with dynamic loading effects and extremely strict allowable limits of differential settlement.

Using a free fall 4000 kg hammer with a drop height of 0.6 m, and penetrations of 4 to 30 mm with series of 10 blows were obtained.

The diagram of WST (weight sounding test) at the site and a pile at a typical elevation is shown in Fig. 11.

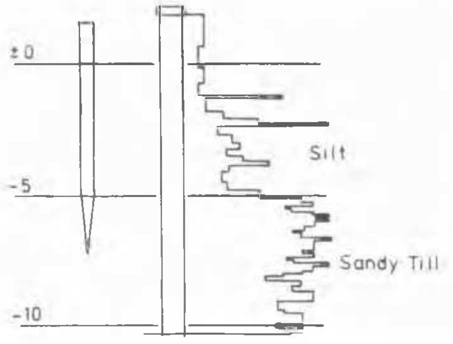

Fig. 11 WST-diagram and pile position at Kemi site

Altogether 12 piles were test loaded with an apparatus having a maximum vertical loading capacity of 750 kN, i.e. near to the double design load. The settlements at maximum load varied from 1.5 to 9.0 mm. The load-settlement curves representing maximum, minimum and average displacements are shown in Fig. 12.

Breaking loads from Krüger formula in this case have a scatter from 0.95 to 1.85 MN. Tension test for one pile with a length of 8.0 m gave a displacement of 1.0 mm with a tension of 200 kN.

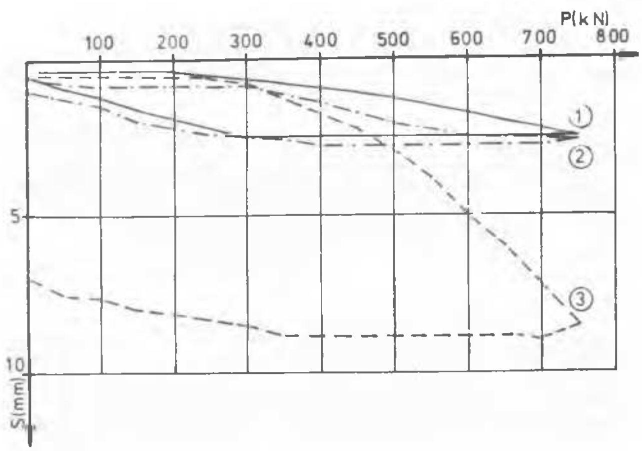

Fig. 12 Test Loading Data. Kemi
1. Minimum
2. Average of 12 and
3. Maximum displacement

Sawmill, Eno

In this case piles with 250 x 300 mm^2 shaft 250 x 100 mm^2 tip area and 2.5 m taper (Fig. . were used. The pile length was 7.5 m and the preliminary chosen design load 200 kN. The WST-diagram representing the soil conditions and a typical pile position are shown in Fig.1

The test loading results of all 7 loaded piles are shown in Fig. 14.

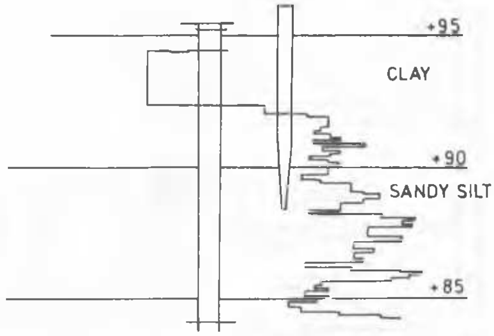

Fig. 13 WST-Diagram and Pile Position, Eno

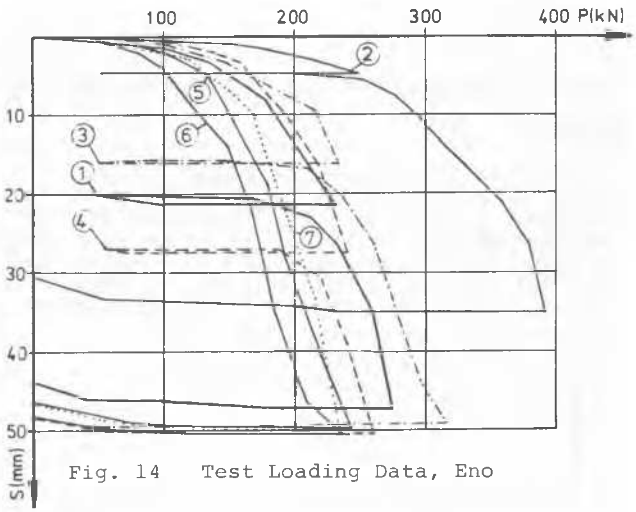

Fig. 14 Test Loading Data, Eno

The breaking loads and correspondent settlements of the piles determined in accordance with the definition in chapter "General" are shown in the Table I. On the grounds of test loading results a final ultimate design load of 160 kN was decided.

At the same site a group of test piles were driven into a pattern shown in Fig. 15. Each pile was driven in two stages to a final depth. During the piling, movements of an adjacent building as well as of neighboring piles driven before were observed. The data of these observations are given in the Table II.

TABLE I

Breaking Loads and Correspondent Settlements, Eno

Pile Nr.	P_u (kN)	s (mm)
1	260	31
2	(390)	(35)
3	260	30
4	250	26
5	180	25
6	180	35
7	185	20
Average	220	28

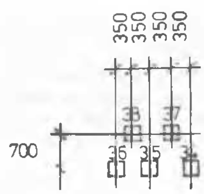

Fig. 15 Test Piling Pattern, Eno

TABLE II

Test Piling, Eno

PILE NR	34		35		36		37		38		CONTROL POINT
DRIVING SEQUENCE	PENETR 10 BLOWS	ELEV. HEAD	PENETR 10 BLOWS	ELEV. HEAD	PENETR 10 BLOWS	ELEV. HEAD	PENETR 10 BLOWS	ELEV. HEAD	PENETR 10 BLOWS	ELEV. HEAD	SETTL (mm)
PILE 36					120	95.88					+3
35			115	95.89		95.86					±0
34	165	95.80		95.90		95.87					±0
37		95.84		95.88		95.87	120	95.04			±0
38						95.84		95.01	110	95.17	±0
36					70						
35			80								
34	110	94.63		94.61		94.66					±0
37							100				
38		94.60		94.60		94.59		94.61	80	94.60	±0

The breaking loads computed with the Krüger formula, using parameters from test piling, were from 450 to 600 kN. An ultimate design load of 160 kN was chosen. All piles were driven in two stages principally following the same procedure as in the test piling (Table II).

Mechanical Wood Industry Warehouse, Jyvaskyla

Finnish standard tapered piles (Fig. 2) were used, the pile lengths being 10 to 14 m. Soil conditions and pile position are shown in Fig. 16.

A loading test was performed on the grounds that by pile driving some surrounding piles already driven showed considerable displacements, up to 600 mm downwards. The test loading data in Fig. 17 show settlements of the piles of 22 to 24 mm by first loading. By a repeated loading of 250 kN, the increase of settlement was small. An allowable load of 250 kN for piles in groups with not less than 5 piles was chosen.

Pile driving was continued in two stages for every pile, first stage to an elevation 0.5 m above the aimed level and second stage to the final level, where the tapered part of piles had penetrated into the bearing silty sand layer. No damaging settlements of the building have been observed.

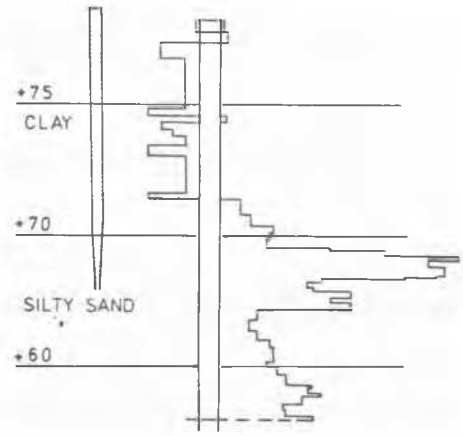

Fig. 16 WST-diagram and Pile Position, Jyvaskyla

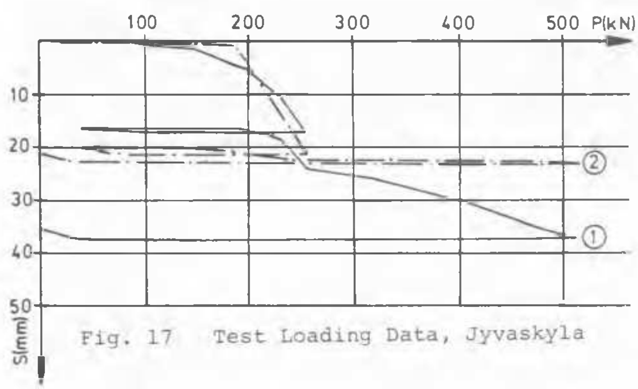

Fig. 17 Test Loading Data, Jyvaskyla

Office and Warehouse Building, Helsinki

At this site, under a soft deposit of post clacial organic clay s stratum, 2 to 3 m thick, of medium dense silty sand was considered to become effectively compacted with wedge shaped piles. To avoid perforation through this "bearing" sand layer with piles to an underlaying loosely stratifield sandy silt deposit, piles with only 1 m long tapered portion were chosen. The pile tip was 250 x 150 mm^2, the "neck" of the tapered part 250 x 500 mm^2 and the head of the piles 250 x 250 mm^2. The soil conditions and typical elevation of a pile is shown in Fig. 18.

Test loadings of three piles in a group of 5 piles in a square pattern with one pile in the middle were performed. The test loading data are presented in Fig. 19. The curves numbered 1 and 2 represent piles on opposite corners of the square foundation and nr. 3 the middle pile, driven as the last of the group of 5. The ultimate design load was 250 kN.

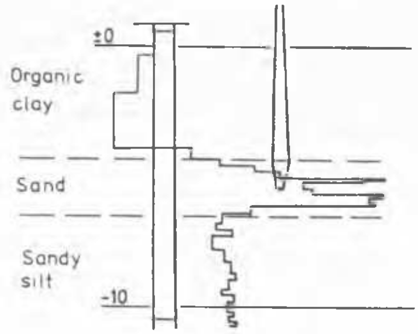

Fig. 18 WST Diagram and Pile Position, Helsinki

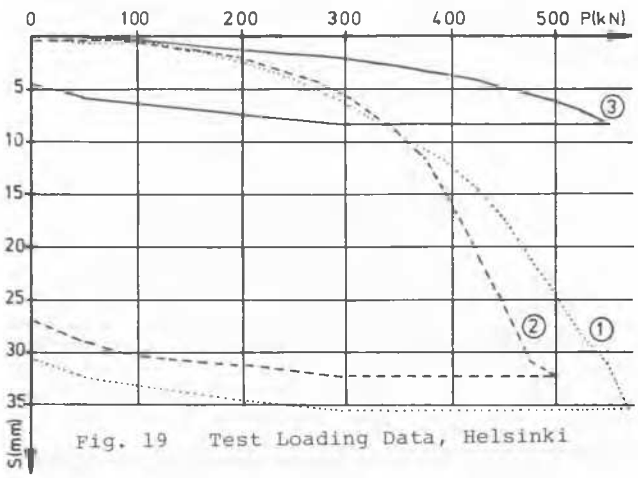

Fig. 19 Test Loading Data, Helsinki

SUMMARY

The design load of friction piles is usually determined on the basis of the allowable total and differential settlements of the structures in question. The actual displacements of piles are greatly depended on the rate of compaction of the bearing strata around and underneath the pile tips.

If the silt content of these strata is high, as in the case of Eno above, or if the pile spacing is too sparce, the densification of soil strata may not be achieved and consequently the gain in bearing capacity remains negligible. Therefore, the use of single piles is unadvisable. Increase of the number of piles in a tightly spaced group is found to cause an increase in the bearing capacity of the individual piles of the group.

Furthermore, an artesian groundwater pressure prevailing in the bearing friction soil stratum is apt to diminish the friction along the surface of piles, even if a considerable compaction effect has been achieved.

In all cases where the ultimate bearing valuea determined by loading tests have been small, suggestions of this state of the matter have been received already at the time of pile driving, by following the development of driving resistance and behaviour of neighbouring piles etc.

The writers are willing to dispute that in most cases possibilities to the use of considerable higher ultimate bearing capacities exist than in the common practice of the friction piling today, provided that

- experience and test loading data of friction piles on the same geological area exist,
- it has been secured that the silt and/or clay content of the proposed bearing layer is not too high, and no artesian ground water pressure exists,
- test pilings with loading test are performed and/or behaviour of the piles and the rate of soil compaction during the driving is properly observed and
- the specifications and instructions for piling are given individually, considering the specific soil conditions, type of piles and adjacent structures at the particular site.

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