The formation of the bearing capacity of piles
Notion de capacité de charge des pieux

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ABSTRACT
In order to evaluate the bearing capacity of piles, it is important to understand the behavior of piles under growing pressure.

The best way to determine the bearing capacity is the method of characteristic points that enables the recognition of load levels that cause the changes in pile behaviour at growing pressure.

For estimating the side friction it is crucial to understand the friction between the side friction of the piles and the soil or the ability to transfer some of the load from the side of the pile to the soil. Consequently, the side friction of the pile depends on the geological section and that may either increase or decrease the bearing capacity of a pile in similar soil conditions. The bearing capacity of the pile end depends on the strength of the soil; determining the strength is complicated in large depths and may thus lead to different/inaccurate results.

For evaluating the soil settlements beneath the pile, it is important to determine correctly the deformation parameters. The method that is applicable close to the ground cannot be used in deeper levels and often the compressibility of soils beneath the pile end is predicted higher than the actual reality. The analyzes of 2000 pile tests and the calculations of the settlements of 60 houses have brought some clarity.

RÉSUMÉ
Afin d’estimer la capacité de charge des pieux, il est important d’appréhender la réaction des pieux à une pression accrue.

La meilleure façon de déterminer la capacité de charge est d’utiliser la méthode des points caractéristiques permettant de reconnaître les niveaux de chargement responsables de la variabilité des comportements des pieux dans des conditions de pression accrue.

Pour estimer le frottement latéral, il faut impérativement comprendre le frottement entre la surface de frottement latérale des pieux et le sol ou la capacité de transmettre une partie de la charge depuis la paroi latérale vers le sol. Par conséquent, le frottement latéral du pieu dépend de la section géologique et cela peut accroître comme diminuer la capacité de charge d’un pieu dans des configurations de sol égales. La capacité de charge de l’extrémité du pieu est fonction de la résistance du sol. Il s’avère compliqué de déterminer cette dernière à d’importantes profondeurs ce qui peut donner lieu à des résultats différents/incorrects.

Pour évaluer les tassements du sol sous le pieu, il est important de définir correctement les paramètres de déformation. La méthode appliquée près du sol ne saurait être utilisée à des niveaux plus profonds et souvent, la compressibilité des sols sous l’extrémité du pieu est présumée supérieure à ce qu’elle est réellement. Les analyses de 2000 tests relatifs aux pieux et les calculs de tassement de 60 maisons nous ont quelque peu éclairés.

The present work gives a survey of experience how to fix the bearing capacity of a single pile in Estonia.

The distribution of pile types in Estonia could be presented as follows:
1. Cohesion piles - CP. Piles that are driven deep into weak clay and the majority of their bearing capacity proceeds from the pile's side friction.
2. Friction piles - FP. Driven entirely into sand or till.
3. Hold piles - HP. Piles that have been driven through weak soil into dense sand or till.
4. Point bearing piles - PP. Driven through Quaternary sediments into over-consolidated clay or lime and sandstone.

1 PILE LOAD TEST
The dependency $S = f(N)$ is achieved by the use of pile load test.

The dependency $S = f(N)$ or the pile’s behaviour is in concordance with the growth of the load. A great number of experiments with tensiopiles and pile-sounds (60 piles in all) have enabled to evaluate the pile's behaviour in connection with the growth of the load and to stress the characteristic points which are important for evaluating the bearing capacity of piles and the settlement of pile foundations (see Fig. 1). These points are especially important because of their characterization of the changes of the division of strength between the pile's toe and pile's side [1]. To evaluate these points on common pile tests one may use the methods of EPT [2] worked out by the author. According to these methods the dependence between the settlement of the pile and time is estimated by the formula

$S = a(t/t_0)^b$ (S- the settlement of the pile in the selected time-range, $a$ initial settlement at the moment $t_0$ and $b$- the factor characterizing the change in the settlement).

By means of the mentioned dependence the settlement of the pile could be fixed at the required time range, or evaluated at the moment when the pile's settlement is practically finished.

In Figure 1 there are presented 2 curves characterising the settlement of the pile with curve 1 characterizing the settlement of the pile in the course of the test and curve 2 during the time-range under investigation. The second part
of the figure presents a typical dependence where the load is divided between the toe and the side of the pile; the third part - \( b \) changed \( t \) with the growth of the load:

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N_a = \text{characterizes the load that does not cause settlements; } b = 0 \text{ and they are connected with elastic deformations of the pile and the deformations of the extensions of the pile. As the relative precision of measurements in this sector is smaller, in this part the division of strength between the toe and the side of the pile is not clear.}
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\( N_a \) - this load is very apparent in all the tests and here the change in the division of load between the shaft and the toe of the pile takes place. The majority of tests (95%) gave the result that \( N_a \) was equal to the maximum bearing capacity of the pile's shaft at the pile's test.

\( N_a - n_i \) - the pile's side receives the majority of the load and the smaller part (15-30%) from the added load hits the toe of the pile. But when surpassing \( N_a \), the picture changes and the majority of the load is starting to affect the pile's toe up to the load \( N_i \). The factor \( b \) between the loads \( N_a \) and \( N_i \) is growing in linearity. The settlement of the pile's head \( N_i \) depends on many factors (the length of the pile, the geological cutting, the type of the pile etc.), but usually (in case of piles with diameter 10...40 cm) the settlement is 1.5...3.5 mm and 80% of the piles reach the settlement of 2...2.7 mm.

Our investigations and the settlement observations of the buildings built on piles have shown the great importance of the characteristic point \( N_a \), the proportionality limit. The dependence between \( S \) and \( N \) is practically rectilinear up to \( N_i \) and especially in the calculated curve of the settlements 2. The factor \( b \) between the loads \( N_a \) and \( N_i \) is constant. After \( N_i \), the pile's toe and the nonlinear dependence between the settlement of the pile and the load added to it will begin.

The formation of side friction was investigated by pile tests made by special pile sounds and tenso piles. The piles loaded in different engineering-geological conditions showed that the side friction of a pile does not practically depend on a pile's length, material and diameter. The lack of influence of a pile's diameter on the special side friction could be explained by the fact that the properties of the weak soil surrounding the pile are not affected by the driving. Thus the side friction of a pile depends only on the soil strength.

The minimal influence of the pile's length could be explained by the static nature of the depth of creep point that has been determined by undrained shear strength. The lack of influence of a pile's material can be explained by the formation of a surrounding "shirt" in the course of determining the size of the side friction of a pile.

In weak soil the special side friction does not depend on the static period of the pile as the made tests have shown. The lack of increment in a pile's side friction in weak soil during their static state could be explained by the fact that soil properties on the pile's side do not practically change in relatively small sensitiveness of the investigated weak soils.

In both plastic and hard clays the pile's resting-time has a strong influence on the pile's special side friction due to which it increases many times over. The special side friction of piles driven into morain was after the driving 0.02 MPa and during a month it increased up to 0.1 MPa. The special side friction of the piles driven into Cambrian blue clays immediately after the driving was 0.01 MPa, after a week's time it was 0.05 MPa and after a month's resting-time it was 0.1 MPa. The growth of the pile's special side friction in the time in plastic soils and in soils of hard consistency can be explained by a relatively large rigid cohesion of the last-mentioned and by the fact that while driving into these soils the soil structure is completely destroyed.

The comparison of special side friction measured in the course of pile tests to the creep threshold fixed at undrained shear test showed that in case of weak soils they practically coincide (figure 2). In varved clays, in marine silts and in sands containing organic matter, the coincidence of \( \tau_c \) and \( \tau_t \) is very good and usually the difference does not exceed 10 %. The deviation is greater in the case of marine clays, because their properties are very varied in a geological section. The pile's special side friction characterised by creep point in plastic till is a much more complicated issue, due to the structure of the test samples and perhaps also to the soil improvement on the pile's side in the course of driving.

The analysis of pile tests shows that the pile's special side friction is considerably less dependent upon soil's consistency as compared to soil's clay content and the geological history of the surface. The consistency of Pärnu varved clays and of marine clays spread in Tallinn is almost the same, but due to the older age, the creep point of Pärnu varved clays is greater and therefore their special side friction is also greater (fig. 2, groups I and II). The same can also be said about marine and glacial silts spread in Tallinn (fig. 2, groups III and IV); in glacial silt the piles' special side friction is nearly three times greater than in marine silts. In clays of the same age the special side friction depends on

![Figure 1](image1.png)

![Figure 2](image2.png)
clay content in soil. The marine sediments of Tallinn (fig. 2, groups II and III) having the same consistency, have a greater pile's side friction in silts than in clays. Piles' special side friction depends upon the engineering-geological section of the pile's base. During the tests in Tallinn, in Kadaka tee, the piles driven into the depth of 4.8 m of marine silt were loaded. Their special side friction was 9 kPa, not depending on the diameter or resting-time. After the loading the piles were countersunk into the glacial silt located 20...50 cm deeper, where-by the bearing capacity of the pile's tip increased nearly 2...3 times. That was accompanied by the increase of pile's special side friction up to about 16 kPa. This refers to the different working conditions of the pile, depending upon the fact whether the pile's tip has been driven into weak or hard soils. In the last case the origin of creep processes at the pile's side is made more difficult due to the smaller settlement of the pile conditioned by the greater bearing capacity of the tip and the value of the pile's special side friction becomes greater.

If the weak soils in the pile's base are located under the soils with a greater bearing capacity, the pile's special side friction depends on the creep threshold of the deeper weaker soil. In the centre of Tallinn there is a complex of soft clay at the 10...12 m depth under a layer of marine sand the thickness of which amounts to 20...25 m. The special side friction of the piles with the length of 8 m in marine sands was 16 kPa. After driving the piles into the depth of 16 m (in sands 12 m and 4 m in weak clayey soils), the special side friction of the pile decreased to 10 kPa, characterising the creep point of these clayey soils. In Pärnu the special side friction of piles in marine sands was 20...30 kPa; after driving the tips of piles into the yielding varved clays by 300...350 mm, the special side friction of piles diminished up to 10 kPa, responding to the creep point of varved clays. In weak soils the creep occurs at smaller shear tensions, bringing about the onset of creep deformations in the sands located above and the diminishing of the pile's side friction.

The data about side friction on pushing the piles inside and pulling them out is given in Figure 3. The graph shows that the side friction of $t_{s_{30}}$ pushing in is usually bigger than that of pulling out $t_{s_{30}}$ whereby the interval is as big as the side friction. The only exceptions are these piles (marked with crosses), driven through sands into weak soils. In their case the side friction of pulling out $t_{s_{30}}$ is bigger than the driving-inside friction.

The dependence between side frictions on the grounds of correlation calculations is

$$t_{s_{30}} = 0.5...0.8 t_{s_{30}}$$

The maximum side friction of a pile does not mean exceeding the maximum shear strength of soil, but the formation of creep processes on the side surface of the pile (depending upon the shear strength of the pile's base and the bearing capacity of the pile's tip). It is correct to consider the pile's side friction the maximum strength that the pile is able to bear to the soil through its side. After the onset of the creep processes on the pile's side, the whole load will be carried to the tip of the pile.

Pile toe. The pile tests made in different engineering-geological conditions showed that the bearing capacity of the pile's tip at the loads $N_t$ and $N_s$ practically does not depend upon the loading speed. The strength of the investigated soils does not diminish and they do not become very much denser under the pile's tip. Depending upon the filtration parameters of the soil, the settlement characterising the filtration consolidation will occur either in course of driving soft silt and moraine or there will be no densening together with the pile's driving or the pile will be driven on the account of pressing the soil; side upwards (soft clay and loam). After the loading test the pile's settlement will be caused by the skeleton's creep in silt in the dense prism formed under the pile's tip. On weak clays the process is influenced by the pore water pressure after the ramming and the time of loading is usually too short to bring about any remarkable consolidation.

Figure 3. $t_{s_{30}} = f(t_{s_{30}})$

The made tests showed that the bearing capacity of the pile's tip is influenced by the depth of the pile. The influence of the depth depends upon the angle of the internal friction of the soil under the pile's tip; for determining this, G. Meyerhoff's calculation scheme is the best choice.

In comparing the made pile tests with calculation formulae, it came out that both of them $N_t$ and $N_s$ were fixed by G. Meyerhoff's formula. In calculating $N_t$, the shear parameters should be the bases, characterising the shear proportionality limit and for fixing $N_s$ - the shear parameters characterising the maximum shear strength.

Driving piles into silt and till, a rather wide-spread condensed area of will appear under the pile's tip. In Figure 4 one can see a condensed area under the pile's tip, formed while driving 5 m long piles into soft glacial silt that in natural conditions is flowing and has the moisture of 25%. After the driving, the soil under the tip of the pile is in a super compact condition ($W_{N_t}=10\%$). The thickened zone reaches 1 m deep and there is a linear dependence between moisture and depth. Such thickened layer should increase the bearing capacity of a pile considerably, but as the calculations show, the bearing capacity of a pile is not determined by the strength of a compacted material, but by the strength of a natural material. Therefore the thickened cushion is a connecting link transmitting the load and does not increase the bearing capacity of a pile's tip by much. It does, however, diminish the deformation of the soil under the tip of the pile, because the active area of the pile is in the pile itself and further up to the proportionality limit (only the dry unit is represented here).strength they have similar parameters and when

Figure 4. Change $W_{N_t}$ under pile tip
The dependence between the named sizes gives us a good insight into the geological conditions. The data of the comparison is given in the figure. In Figure 6 we can see how the ultimate load $N_s$ of a pile's tip settlement depends on the geological section of the pile's base, on the geotechnical properties of the soil and on the pile's length. There is a definite relationship between $S_r$ and $N_s$ ($\eta=0.55$) and generally, in concordance with the growth of $N_s$, the settlement at this load also increases. Here we can see more clearly the influence of the geological structure of the pile's base on the aforementioned sizes; according to them, four groups can be distinguished in the figure. The group $I_b$ is formed by the CP piles driven into soft clay and silt. They reach their ultimate condition at the settlement of 10...15 mm. The group $I_s$ is also formed by CP piles, but they have been driven into soft silt; their settlement is considerably bigger – and the dependence between $N_s$ and $S_r$ can be clearly seen inside the group. Group $II$ is formed by hold piles, with the settlement at $N_s$ 15...35 mm, increasing simultaneously with the growth of the load. In the group $III$ there are piles of internal friction, driven into till and sands and they are characterised by a very big settlement - 35...64 mm at $N_s$.

Taking into consideration the fact that the groups $I_b$ and $III$ involve the soils which are characterised by a greater angle of internal friction at $\tau_{\max}$ it may be concluded that for the complete realisation of the power of internal friction in such soils, a considerably bigger settlement is needed (only if $Nu$ is accompanied by the driving of the pile point's base). Loam and clays belonging to group $I$ have small internal friction and the $N_s$ of piles' bases will arrive at considerably smaller deformations. The situation is more complicated for the CP piles belonging to group $II$, because the pile's side is in weak soil and its point in stronger soil and perhaps due to this, the settlement necessary for breaking the pile base is smaller than with piles with internal friction. The additional influencing factor may be the fact that the hold piles have been driven into the strong layer only with their points, weak soils are straight above the pressing-out prisms and therefore the trajectory of pressing-out prisms is shorter.

In most of the pile standard, the bearing capacity of a pile is estimated from the load $N_s$ by the factor of safety and for estimating its size the connection between $N_s$ and $N_r$ was compared with pile tests made in different engineering-geological conditions. The data of the comparison is given in Figure 6 and it shows that there is a good correlative dependence between the named sizes $N_{r}=0.6 N_s$ ($\eta=0.75$).

The group $I$ is formed by the piles driven into clay and loam, characterised by a relatively small interval between the named quantities ($N_{r}=0.8N_s$). In the second group there are the connection piles driven into silt; in their case the difference between $N_r$ and $N_s$ is considerably greater. The different behaviour of piles in clay and silt can be explained by the differences of shear strength at creep point and at the maximum shear strength that is small in clay and big in silt. In silts where the growth of shear strength at the maximum shear strength is based on internal friction bigger deformations are needed for realising it (as it could be seen earlier)- causing a greater growth of $N_s$. The 3rd group is formed (in the figure) by hold and internal friction piles.

It may be concluded from the above that the bearing capacity of the pile is best characterised by a proportionality limit that sets apart two different stages of the pile's behaviour: at a smaller load the soil becomes thicker, at a bigger load the soil will start pressing out from under the pile's point. The factor $b$ characterising the temporality of settlements makes a jump in this point. The exceeding of proportionality limit is always characterised by a considerably bigger final settlement. At the pile's proportionality limit the pile's side friction ($\tau_b = \tau_s$) has completely been realised and the pile’s point directs itself linearly in accordance with the regularities of the deforming environment. For describing the dependence between deformations and the load it is possible to use elasticity theory – but only up until this load. At bigger loads the use of elasticity theory is not justified and for describing the piles’ settlement it is necessary to use empirical methods taking into account the temporality of the piles’ settlement.

The above does not mean that $N_r$ is the maximum permitted load for a pile. The proportionality limit may be exceeded but the exceeding must be equal for all the piles under the building have been constructed with a view that all the settlements proceeding from it and fixed by some empirical methods, would be more or less equal. The most dangerous is the condition of a building when a part of the piles has been loaded by a load smaller than the proportionality limit and a part of the piles by a load bigger than the one which due to an increase in the intensity of settlement causes a different settlement of the building's foundations.

REFERENCES