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Time Dependent Increase of Bearing Capacity of Piles Driven into Cohesive Soils

Abstract

A time influence on bearing capacity increase of driven piles is presented. The field investigation results for different types of piles driven in various soil conditions obtained by various authors were analysed. In the paper the most significant quantitative relations and calculation formulae are given. The methodology and scope of own model tests for open-ended and closed-ended pipe piles driven in till was discussed. The comparison results of measured and calculated values of piles bearing capacities as a function of time was presented. It has been finally stated, that the total bearing capacity increase of piles is caused mainly by capacity increase of pile shaft.

Key words: piles, bearing capacity, time influence, bearing capacity increase

1. Introduction

Pile driving in cohesive, low and very low permeablee soil, induces excess pore water pressure, remoulds and partly alters the structure of soil around the pile shaft, and

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alters the stress state in the vicinity of the pile. As time elapses, dissipation of the excess pore water pressure and complex consolidation process take place. As the result of this phenomenon, the driving resistance of the pile may increase with time, which is described as "set up". Quantitatively this phenomenon can vary from an imperceptible one to a situation where a few minutes delay makes any further movement of the pile impossible.

The time effect is mentioned quite generally in the literature, although it is often incidental to the main point the particular authors are making. A fundamental understanding of this problem is necessary because the increase in bearing capacity can be safely used to support an engineering structure.

2. Review of Field and Model Test Results

Various field tests, model tests and theoretical solutions, concerning time influence on increase of the pile bearing capacity, have been carried out over the last 20 years, for different kinds of piles driven in different kinds of cohesive soils.

Soderberg (1962) informed that the first mention of this time effect in the literature was at the turn of the last century, when Wendel reported some test loading results he had obtained in Sweden. The bearing capacity of his piles increased rapidly with time elapse and reached its maximum in less than a month. An increase of the order of 10 times is apparent in these cases. The Scandinavian countries were among the first to recognize this effect and it has become common practice in Norway to load piles months after the driving in order to take advantage of this time effect.

Soderberg (1962) also explained the increase in the bearing capacity of a friction pile in clays and silts with time in terms of consolidation theory. The case of a single pile was examined in detail and with respect to the experimental data available to 1962. Additionally, problems related to large clusters were outlined and limits for the effect on a cluster were proposed. The final conclusions given by Soderberg are as follows:

a) The bearing capacity of a friction pile is dependent on the dissipation time of the hydrostatic excess water pressure that is created by pile driving. The time required for this to reach a specified state is proportional to the square of the horizontal dimension of the pile and inversely proportional to the horizontal coefficient of consolidation.

b) A static load test on an isolated pile should never be relied on to give the bearing capacity of the pile in cluster. Consolidation must be given to the dissipation time of the group.

c) In order to predict this dissipation phenomenon without extensive field tests, a large backlog of information on the coefficients of consolidation and pressure distribution will be necessary. This information will necessarily have to be collected from field construction projects.

Nowadays, thirty years after the formulating of Soderberg's conclusions, there exists significantly more comprehensive data base from field and model investigations and analytical considerations. The most essential results of both foreign and own model tests will be presented in the next parts of the paper.

Skov and Denver (1988) point out that the control methods, such as static loading tests, stress wave measurements and driving formulae, determine the bearing capacity at the time of testing, and if they are applied at the time of initial driving, when the conditions in the surrounding soils are unstable, the long-term bearing capacity of the piles will often be miscalculated. In practice, it is only for piles driven in coarse sand that the driving resistance is comparable to be the long-term capacity. Pile foundations in which these conditions are fulfilled both at the pile toe and along the pile shaft are rare.

A static loading test on a restrike with dynamic measurements performed at a certain time after the initial driving is the most reliable method for evaluating the long-term capacity.

Soil and water are displaced from around the pile during the initial driving, and the internal bonds in the soil are disturbed near the pile. During restriking the soil will be disturbed again, especially beneath the pile toe, and the balance will also be disturbed around the pile shaft, but not nearly as much as during the initial driving.

Depending on the soil conditions, the bearing capacity of piles usually increases with the time elapsing after driving. The main reasons for this "set up" effect are assumed to be:

- equalization of the pore water pressure (reconsolidation),
- re-establishment of the internal bonds in the soil (regeneration in cohesive soil).

It is obvious that the pore water effect will dissipate faster the more permeable the soil is. In coarse sand, for instance, the pore water pressure dissipates rapidly. Pile driving formulae can often be applied with satisfactory accuracy during initial driving under these conditions. For piles in stiff clay, reconsolidation takes longer and it is not unusual for capacities to increase even in terms of years. Piles driven in chalk constitute a special case, for that often seem to have very little resistance during initial driving, but their capacity increases considerably after a period of time.

Skov and Denver (1988) described four case histories for prefabricated concrete piles (0.25 × 0.25 m cross section and 19.0 or 21.0 m in length) and for steel pipe piles (0.762 m in diameter and 33.7 m in length). In order to study the time dependent development of capacities on this site, dynamic measurements, CAPWAP – analysis and static loading tests were carried out in different time intervals. The main results of these measurements are presented in Figs. 1 and 2, together with soil conditions.

The time-dependent increase in capacity observed stabilizes after initial driving as a logarithmic function of t/t_0 where t_0 is a function of the soil type. Results from loading tests and CAPWAP analysis leads to the following empirical formula:

$$\frac{N}{N_0} = 1 + A \log \frac{t}{t_0} \tag{1}$$

where:

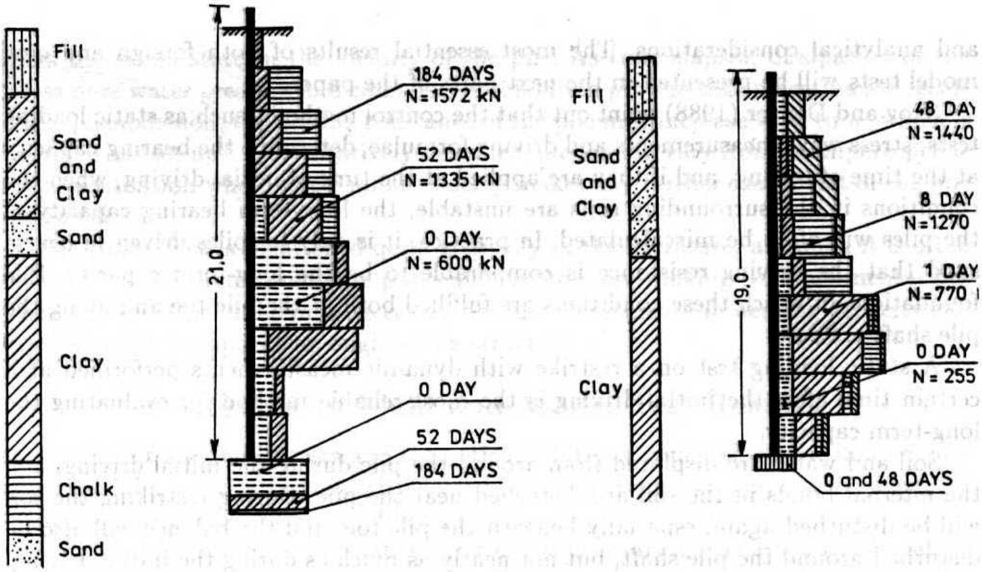
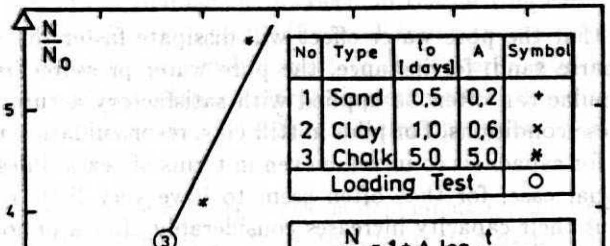


Fig. 1. Distribution of shaft and toe resistance from CAPWAP analysis on driving and restriking (Skov and Denver 1988)



No	Type	t_0 [days]	A	Symbol
1	Sand	0,5	0,2	+
2	Clay	1,0	0,6	×
3	Chalk	5,0	5,0	✱
Loading Test				○

$$\frac{N}{N_0} = 1 + A \log \frac{t}{t_0}$$

Fig. 2. Empirical formula for time-dependent capacities for different subsoils (Skov and Denver 1988)

N_0 – pile bearing capacity for time t_0,

N – pile bearing capacity for time $t > t_0$,

t_0 – time elapsed after initial driving from which the increase in capacity is linear in logarithmic time scale (function of the soil type Fig. 2),

t – time after initial driving,

A – factor which is a function of the soil type (Fig. 2).

The analyses of case histories for prefabricated concrete piles and closed-ended steel piles proved the following:

a) After a certain time interval (t_0) which is a characteristic of the soil conditions at the site, the time-dependent increase in bearing capacity can be considered linear with logarithm of time after initial driving.

b) The long-term capacity of piles driven in cohesive soils can be calculated from formula (1) for sandy, clayey and chalk subsoil. This formula may be used in other cases with caution.

c) Experience shows that capacities of piles nearly always increase during the time after driving. This is a phenomenon of importance for the economy of pile foundations.

d) Formula (1) is recommended if the dynamic measurements (using FPDS equipment) are carried out on piles in different time intervals, and CAPWAP or TNOWAVE analyses are used.

Radugin (1969) performed the results analyses of Russian dynamic capacity tests (over 150 piles) and static capacity tests for driven piles in various cohesive soils. It was the basis for determining approximate limit times T_1 for particular kinds of soils. The limit times T_1 after which a pile reaches the maximum of its capacity are presented in Table 1.

Table 1

Limit time T_1 [in days] for different soil types

Type of soil	Limit time T_1
Sand	2 – 3
Slightly clayey sand	6 – 10
Clayey sand	20 – 25
Clayey sandy silt	30 – 35
Clay	50 – 60

Maximum capacity of pile $N_{\max}$ after time T_1 from the moment of pile driving in cohesive soil can be calculated from the following formulae (Radugin 1969):

$$N_{\max} = N_0 + \frac{N_t - N_0}{F} \tag{2}$$

and

$$N_{\max} = N_0(1 + S_u) \tag{3}$$

where:

$N_{\max}$ – maximum pile capacity after $t = T_1$,

N_0 – pile capacity immediately after driving for $t = 0$,

N_t – pile capacity during loading test for $0 < t < T_1$,

T_1 – limit time depending on kind of soil (Table 1),

F – coefficient depending on t/T_1 ratio, given by Radugin (Table 2),

S_u – the degree of pile capacity increase after $t = T_1$, depending on kind of soil (Table 3),

t – time of loading tests.

Table 2

Values of the F coefficient

$\frac{t}{T_1}$	F	$\frac{t}{T_1}$	F	$\frac{t}{T_1}$	F
0.05	0.100	0.40	0.683	0.75	0.940
0.10	0.197	0.45	0.730	0.80	0.954
0.15	0.290	0.50	0.790	0.85	0.960
0.20	0.383	0.55	0.830	0.90	0.976
0.25	0.465	0.60	0.866	0.95	0.980
0.30	0.546	0.65	0.890	1.00	0.988
0.35	0.585	0.70	0.920	–	–

Formulae (2) and (3) are recommended for the case of loading tests of piles before the limit time is reached.

Table 3

The degree of pile capacity increase

Type of soil	States of consistency	S_u
Slightly clayey sand	hard	0.5
	firm	1.2
	liquid	2.5
Clayey sand and Clayey sandy silt	hard	0.8
	stiff	1.5
	soft	2.8
	liquid	4.0

Hamza (1991) described the short and long-term shaft resistance of concrete piles driven into clay. A special testing programme was carried out for concrete piles of length 36.0 m and 0.35 m in diameter driven through soft, very soft or medium consistency clay to rest finally in dense sand (3.0 m). The instrumented pile was statically tested twice, for short and long-term evaluation, one month and eighteen months after driving. The pile was instrumented at six levels with two strain gauges of each level, capable of sensing and recording the value of the pile load at the installed level. The measurements allowed the field load transfer profile and shaft resistance to be determined for static loading condition.

The interaction between the pile and the soil during axial loading of the pile, controls the adhesion developed on the pile shaft. In the long term, when reconsolidation is completed, the new increased undrained shear strength shall be operating in the soil around the pile shaft. The adhesion τ between pile shaft and surrounding soil, is commonly expressed as a factor of the undrained shear strength when behaviour is presented in terms of total stresses. The notation α is conventionally used to describe the adhesion factor relating τ, used in pile carrying capacity evaluation, to the initial in situ undrained shear strength c_{u0} as:

$$\alpha = \frac{\tau}{c_{u0}} \tag{4}$$

Commonly recommended values of α vary from 0.6 to 1.0 and in some cases it was found to be more than one. Measurements of average shaft unit friction described by Hamza (1991) yield different values of α depending on test times as follows:

- short term $\alpha = 0.45$,
- long term $\alpha = 1.25$.

The variation reflects the increasing nature of the undrained shear strength of the soft clay.

Analysis and comparison of the short and long-term load transfer profiles indicated the time effect in increasing shaft resistance after pile driving. The long-term pile shaft resistance in the soft clay was 2.75 times larger than the short term value.

Preim at al. (1989) also described two case histories concerned with evaluation of the static bearing capacity of piles driven into soils that exhibit time-dependent strength changes. They consider steel and concrete piles 27.0 m in length, driven into soils consisting generally of silty and clayey fine sand. Dynamic measurements and analyses were performed on both piles during the initial installation and also during the restrike one week later. The results are presented in Table 4.

Restrike values represent a doubling in the total pile capacity and 6.7 times or 9.8 times increase in shaft resistance from those at the end of driving for concrete and steel piles respectively. This information was used to predict each pile's long-term capacity, defined 14 days after the initial drive and compared with that measured in a static load test. The agreement between predicted and measured capacities was remarkably good, with an average error of only 4.5%. Two weeks after driving the total pile capacities were three times those at the end of the drive. The increase

Table 4

Summary of CAPWAP results for concrete and steel piles

Pile capacity	Initial driving		Restrike after 7 days	
[kN]	concrete pile	steel pile	concrete pile	steel pile
Shaft resistance	205	80	1375	795
Toe resistance	930	600	935	435
Total	1135	680	2310	1230

appeared to occur almost entirely in shaft resistance and was found to be linear over the time period considered.

Tretner and Burt (1989) present results of static loading tests carried out directly after driving and in different time periods after that on four open-ended steel pipe piles, 54.5 m and 43.3 m in length and 0.4 m in diameter. Piles were driven into firm and stiff silty clay (about 34 metres) and very hard silty clay (about 18 metres). The results are presented in Table 5.

Table 5

Ultimate bearing capacity increase [kN] with time for open-ended steel pipe piles driven in silty clay

Pile Number	No. 3 $D = 0.4$ m $L = 54.5$ m			No. 4 $D = 0.4$ m $L = 43.3$ m			
Time from driving to testing [days]	2.3	3.0	4.2	1.7	10.3	20.5	32.5
Bearing capacity [kN]	1555	1615	1670	1225	1555	1670	1670

Analyses of these results show a considerable increase in the bearing capacity with time, for both piles.

The shaft resistance was analysed in terms of total and effective stress using the following well-known expressions:

$$\tau = \alpha c_{u0} \tag{5}$$

$$\tau = \beta p_0 \quad \text{Burland (1973)} \tag{6}$$

$$\tau = K p_0 \tan\delta \quad \text{Chandler (1968)} \tag{7}$$

where:

τ – unit shaft resistance,

α – average adhesion factor in terms of total stress,

β – corresponding factor in terms of effective stress,

δ – average angle of shearing resistance between the soil and the pile,

K – average coefficient of lateral earth pressure (assumed to be $1 - \sin \phi'$ or about 0.6),

p_0 – vertical effective stress.

The calculated values for these parameters are given in Figure 3. It will be observed that a reasonably smooth curve be drawn through the points to a projected equilibrium value some 30 to 50 days after driving in both cases.

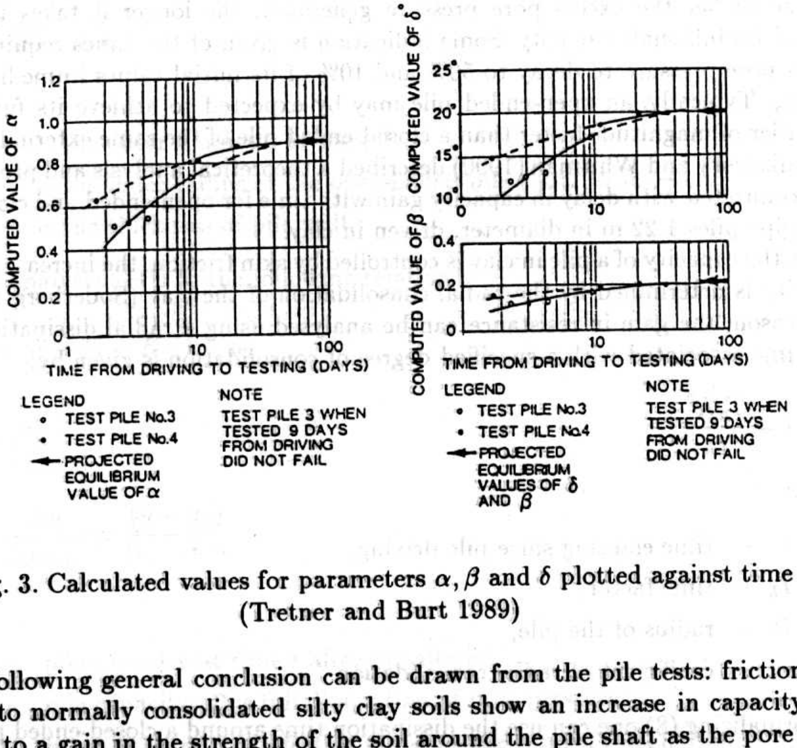


Fig. 3. Calculated values for parameters α, β and δ plotted against time (Tretner and Burt 1989)

The following general conclusion can be drawn from the pile tests: friction piles driven into normally consolidated silty clay soils show an increase in capacity with time due to a gain in the strength of the soil around the pile shaft as the pore water pressure generated during driving is dissipated. An increase of up to about 50% in carrying capacity was measured 30 to 50 days after driving.

Carter at al. (1979) described a theoretical study of stress and strength changes in clay due to installation of driven open-ended and closed-ended steel pipe piles and when examined at different time intervals after driving. The stress changes will depend on the type of pile driven. In particular, there are likely to be differences in the long-term shaft resistance of open-ended and closed-ended steel pipe piles used in the construction of offshore oil platforms (pile diameter 1 to 2 m, penetration over 50 m).

Clay has been idealized as a work hardening elastoplastic material and pile installation has been modelled as the expansion of a long, cylindrical cavity. Numerical results showing how stress and length changes in a soil are affected by its consolidation history prior to pile driving, and by the amount of soil displaced by the pile. In particular, it has been shown that the size of the disturbed region increases as the net cross-section area of the pile increases. The magnitude of the excess pore pressure generated during driving increases the more the soil is disturbed. This leads to greater stress and strength changes in the soil adjacent to the pile shaft as the area ratio of the pile increases. A comparison of the cases of a closed-ended and open-ended pile with $\rho = 0.1$ ($\rho =$ net volume/gross volume) shows that the final strength of the soil adjacent to the pile may by some 25% higher in the former case.

The higher the excess pore pressure generated, the longer it takes the pile to achieve its full shaft capacity. Some indication is given of the times required for the excess pore pressure to decay to 50% and 10% of its initial values immediately after driving. Typically, an open-ended pile may by expected to achieve its full capacity one order of magnitude faster than a closed-ended pile of the same external diameter.

Paikowsky and Whitman (1990) described a theoretical analysis and provided field data connected with delay in capacity gain with time for open-ended and closed-ended steel pipe piles 1.22 m in diameter, driven in clay.

As the capacity of a pile in clay is controlled by skin friction, the increase in bearing capacity is determined by the radial consolidation of the clay (Soderberg 1962). For this reason, the gain in resistance can be analysed using a radial dissipation theory. The time associated with a specified degree of consolidation is given by:

$$t = \frac{T_h R^2}{c_h} \tag{8}$$

where:

t – time elapsing since pile driving,

T_h – time factor,

R – radius of the pile,

c_h – coefficient of radial consolidation.

By normalizing (8) one can use the dissipation time around a closed-ended pile of one size (R_1, t_1) to estimate that of another size (R_2, t_2) such as:

$$\frac{t_1}{t_2} = \left(\frac{R_1}{R_2}\right)^2 \tag{9}$$

The above findings can be summarized as follows:

a) equation (9) can be used to estimate the time required for dissipation of excess pore pressure around one closed-ended pile when given the dissipation measurements of another pile of a different size in the same soil,

b) equation (9) can also be used to predict a gain of pile bearing capacity with time,

c) the volume of the displaced soil is proportional to the square of the pile size.

An analysis of soil displacement around open-ended vs. closed-ended piles may therefore be used to determine the ratio between the gains in bearing capacity of the two types of piles. The volume of soil displaced during open-ended unplugged pile penetration depends on the pile diameter and wall thickness. A calculation of the ratio of the soil volumes displaced by closed-ended vs. open-ended piles of the same outer diameter leads to

$$\frac{t_1}{t_2} = \frac{R_0^2}{R_0^2 - R_1^2} \tag{10}$$

where:

R_0 – is the outer diameter of the pile,

R_1 – is the inner radius of the open-ended pile equal to $R_0 - t$,

t – is the thickness of pile wall.

Finally an analysis leads to the following relations:

$$\frac{N_t}{N_f} = K + (1 - K)(1 - \bar{u}) \tag{11}$$

$$K = \frac{\sigma'_r}{\sigma'_{v0}} \tag{12}$$

$$\bar{u} = \frac{\Delta u}{\Delta u_{\max}} = \frac{(u - u_0)}{(u_i - u_0)} \tag{13}$$

where:

N_t – pile capacity at time t after installation,

N_f – pile capacity after full dissipation of pore pressure,

σ'_r – horizontal (radial) effective stress at the time of installation,

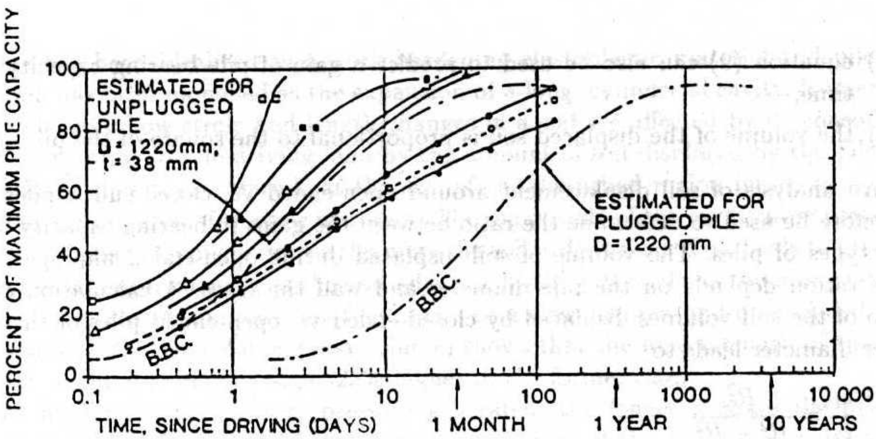
σ'_{v0} – original vertical effective stress,

u – pore pressure measurement with time,

u_i – pore pressure at time of installation,

u_0 – pore pressure after full dissipation.

The use of (11) – (13) to estimate the gain in capacity with time for a 1.22 m pile in two modes of penetration (plugged and unplugged) is shown in Figure 4 along with measurements complied by Vesic (1977). Figure 4 shows the following:



	TYPE	DIA.	LENGTH [m]	SOIL TYPE	LOCATION
■ □	STEEL H	0,35	58.0 66.0	SILT	TAPPAN ZEE,N Y
△	CLOSED ENDED STEEL PILE	0,15	6,7	SOFT CLAY	SAN FRANCISCO
▲	STEEL PIPE	0,30	18,0	SOFT CLAY	MICHIGAN
• • ○	STEEL PIPE	0,60	74,0 96,0 92,0	SOFT TO STIFF CLAY	EUGENE ISLAND

Fig. 4. Comparison of predicted set-up time for a 1.22 m diameter pile, installed in Boston Blue Clay, to field data calculated by Vesic (Paikowsky and Whitman 1990)

a) The calculated gain in capacity with time fits in with the separate trend in the actual data measured.

b) For the 1.22 m friction piles driven in Boston Blue Clay considered here, the time required for the closed-ended or plugged pile to achieve its maximal capacity is one order of magnitude greater than the time required for the unplugged open-ended pile.

c) An unplugged pile 1.22 m diameter in BBC requires 2.5 days to reach 50% of its maximum capacity and 25 days to reach 90%. A plugged pile of the same diameter requires much more time, so that 50% of its maximum capacity is reached after 50 days and 90% after about 500 days.

d) All the analyses assumed that a plugged pile is analogous to a closed-ended pile. While this assumption is entirely correct for the end-bearing consideration, it

> is conditional for the time-dependent pile capacity. If the pile penetrates in plugged mode along a length that is significant in relation to its penetration depth in the unplugged mode, these approximations are valid. If, however, the pile penetrates in a plugged mode for only a short distance at the end of its driving, then the effect on the gain in capacity with time will be less than that described above, despite the fact that the contribution of a given pile length to the total capacity of friction piles increases with depth.

The review of various investigation results, field measurements and analytical considerations presented above show that the bearing capacity increase with time may be significant for the piles driven in cohesive soils.

To determine quantitative relations for piles driven in till the author had performed special model tests at the semi-technical scale. The characteristics and most essential results are presented in the next section.

3. Own Model Test Results

The programme of model test related to plugging of soil in open-ended steel pipe piles, was carried out by the author in 1991/1992 in the Geotechnical Laboratory of Tampere University of Technology in Finland. Scope, characteristics and procedure of those tests is presented in the paper by Zadroga (1993). The special series of tests were carried out to compare the quantitative increases in the bearing capacities of the piles as a function of time.

Open-ended and closed-ended steel pipe piles, 5.0 m long were used, with the characteristics given in Table 6.

Table 6

Pile characteristics in [mm]

Type of pile	Open-ended		Closed-ended
Outer diameter D_0	90	170	90
Inner diameter D_i	80	150	80
Wall thickness t	5	10	5

The piles were driven vertically to the depth of 4.0 m into homogeneous saturated till with the grain-size distribution shown in Figure 5, and main geotechnical parameters presented in Table 7.

In order to study the time dependent development of capacities of tested piles, dynamic measurements using FPDS-2 equipment, TNOWAVE analysis and static loading tests were carried out in different time intervals. The main results of this measurement are presented in Table 8 and Figs. 6 – 10.

The calculations were based on formulae (1) – (3) given by Skov & Denver (1988) and Radugin (1969). The values of particular parameters for calculation were taken from Figure 1 and Tables 1 – 3. Calculation results are shown in Table 8 and Figure 7. Analyses of the results presented in Fig. 6 and Table 8 show that:

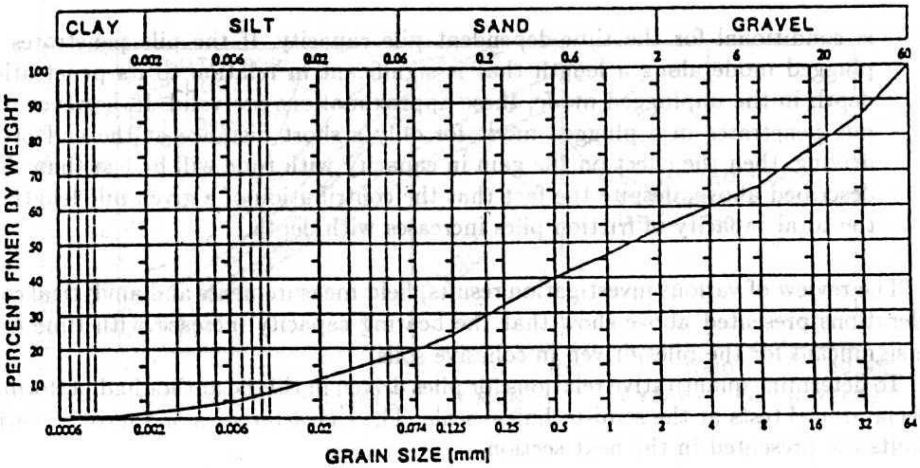


Fig. 5. Grain-size distribution of till used in model tests

Table 7

Geotechnical parameters of till

Geotechnical parameter	Unit	Value
Angle of internal friction, ϕ	[deg]	36.6
Cohesion, c	[kPa]	0.0
Unit weight, γ	[kN/m^3]	18.3
Natural water content, w_n	[%]	9.5 – 15.7
Optimum water content, w_{opt}	[%]	8.3
Coefficient of permeability, k	[m/s]	$1.37 * 10^{-7}$

a) The load-settlement curves for different times after pile driving are similar in form and the measured settlement obtained for the same loading values in the static loading tests performed later is considerably smaller,

b) An increase in the total ultimate bearing capacity is observed for all the piles in static loading tests, being of the order of 20-40% for open-ended plugged piles to about 40% for closed-ended piles,

c) Similar increases in total bearing capacity were obtained by dynamic FPDS measurements during pile driving,

d) One could obtain good conformity between experimental and calculated results of time influence on piles capacity increase.

TNOWAVE analyses, employing a CAPWAP-like program which can be used to simulate stress-wave measurements, were also performed on all the steel pipe piles for comparison purposes. Results of TNOWAVE analyses during initial driving and redriving of piles (11 or 12 days later) are presented in Figs. 8 and 9.

If follows from the analysis of quantitative dependencies presented, that:

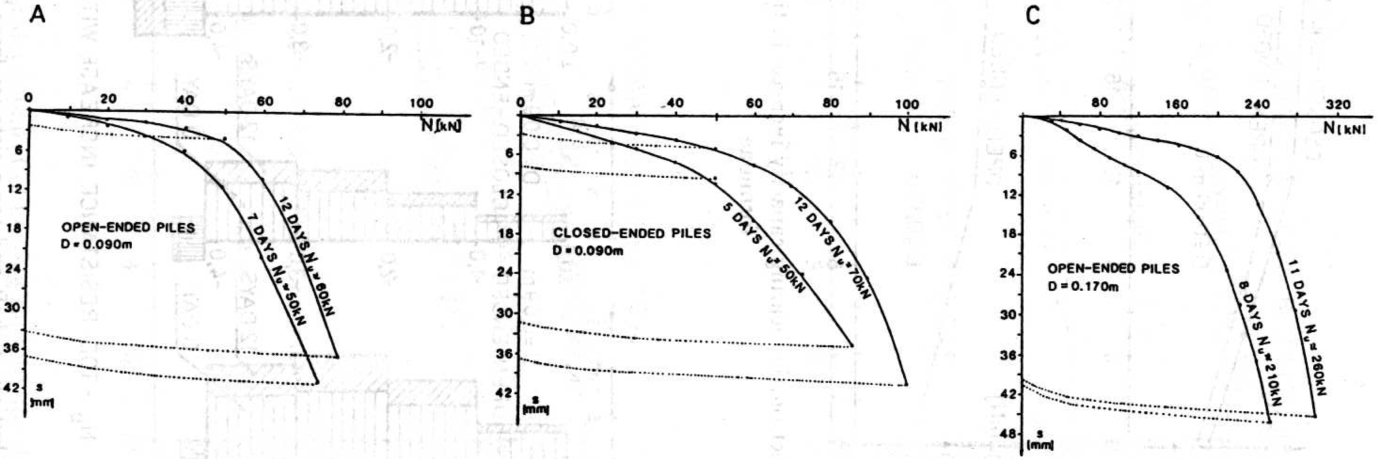


Fig. 6. Influence of the time effect on the bearing capacity increase of closed-ended and open-ended plugged piles

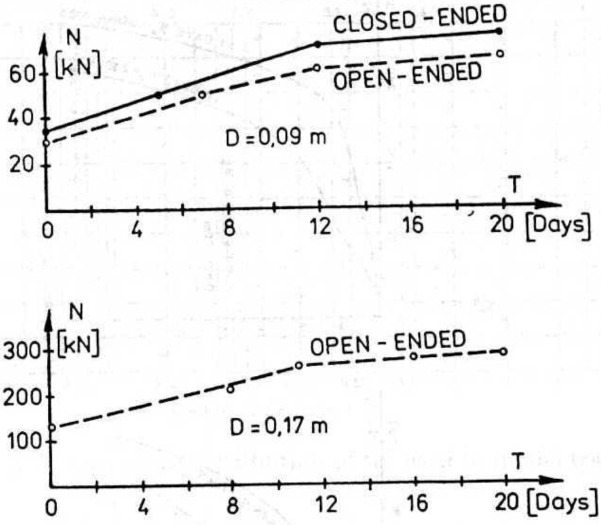


Fig. 7. Time effect on the bearing capacity increase in till according to Radugin formulae

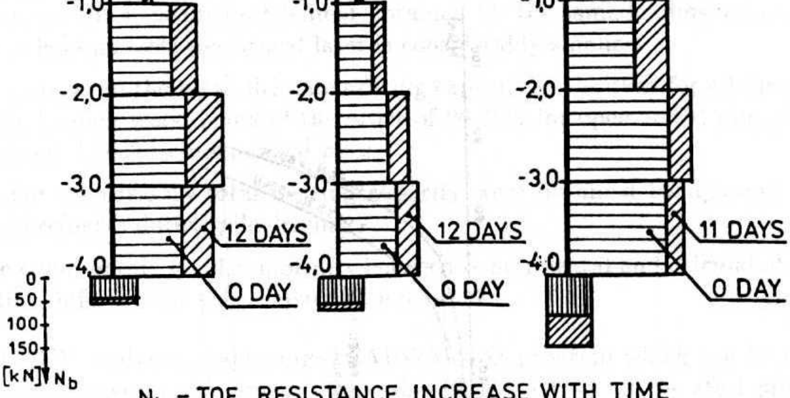
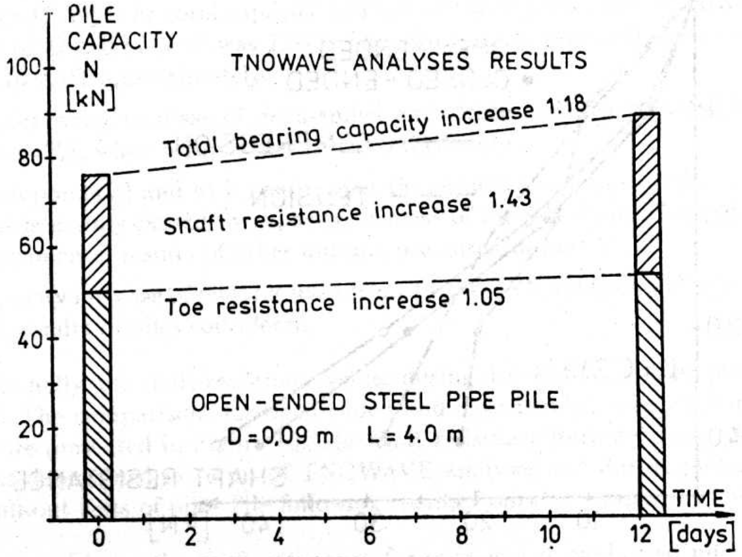


Fig. 8. Distribution of shaft and toe resistance from TNOWAVE analysis



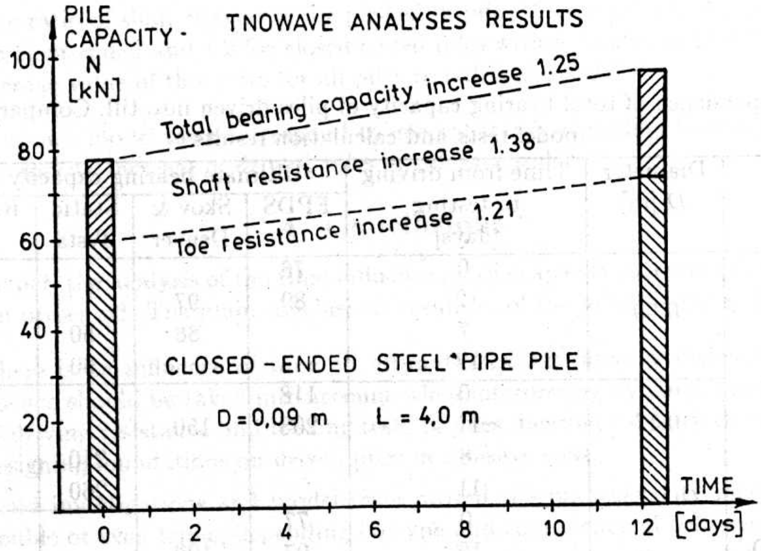


Fig. 9. Time-dependent development of bearing capacity for closed-ended and open-ended plugged piles driven into till

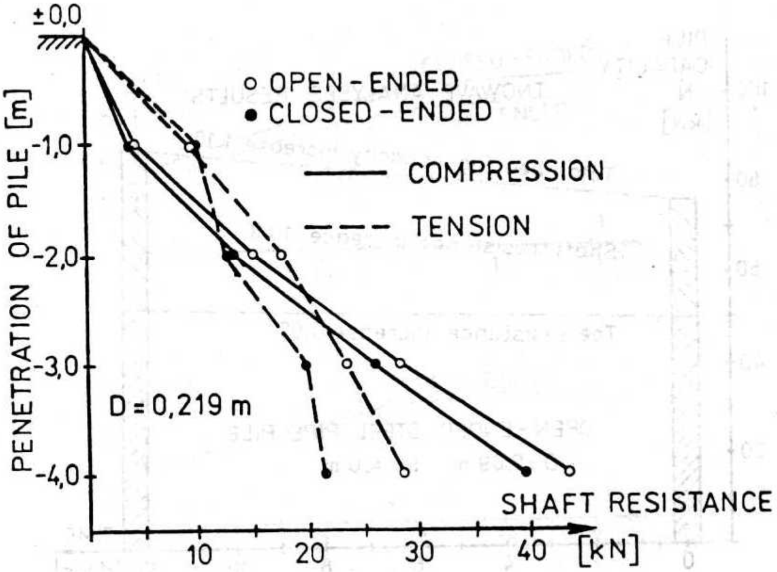


Fig. 10. Comparison of shaft resistance during driving and pull-out tests for open-ended and closed-ended piles

Table 8

Time dependence of total bearing capacity of piles driven into till. Comparison of model tests and calculation results

Type of pile	Diameter D [m]	Time from driving to testing [days]	Ultimate bearing capacity [kN]			
			FPDS	Skov & Denver	Static Tests	Radugin
OPEN-ENDED	0.09	0	76			
		12	89	97		
		7		88	50	
		12			60	65
	0.17	0	118			
		11	203	150		
		8			210	
		11			260	286
CLOSED-ENDED	0.09	0	77			
		12	97	100		
		5		87	50	
		12			70	76

a) After 12 days the total capacity increase of tested piles amounts to 18–25% (for pile of $D = 0.17$ m it was 72%). This gives an average daily increase equal to 1.5 to 2.1% approximately.

b) The capacity increase of open-ended and closed-ended pile shafts varied from 31 to 42%, while for pile toes it was 5–21% only.

c) From points a) and b) it results that the total increase in capacity of piles with time is mainly caused by capacity increase of the pile shaft. This conforms with experimental results of other authors presented in part 2.

d) Capacity increase of piles received from TNOWAVE analysis is similar to loading test results of piles considered.

Additionally, the shaft resistance values during driving and pulling piles out were analysed. The comparisons for open-ended and closed-ended piles of diameter $D =$ 0.219 m are presented in Figure 10. The shaft resistance during driving at particular levels was determined in terms of TNOWAVE analyses and during pulling in terms of full pull-out tests of pile. The following results from the comparisons presented:

a) During driving the shaft resistance for open-ended unplugged piles is on final depth of penetration about 10% higher than for closed-ended one.

b) During pull-out tests, shaft resistance for open-ended unplugged piles is about 15 to 30% higher than for closed-ended ones.

c) The ratio of shaft resistance for pushed-in piles to that for pull-out piles is 1.5 for open-ended and 1.9 for closed-ended piles with diameter of $D = 0.219$. The average value of this ratio for all piles tested is about 2.1.

Results of own model tests correspond to general trends regarding time influence on bearing capacity increase of driving piles in cohesive soils.

4. Conclusions

In the paper, the analysis of the time influence on the capacity increase of driven piles has been presented. The main conclusions resulting of the analysis are as follows:

1. There is an influence of time on ultimate bearing capacity increase. This influence should be taken into account when interpreting dynamic measurements of driving resistance and loading tests of piles, bearing capacity calculations of designing foundations on driven piles in cohesive soils.
2. Field investigations and model tests proved the piles capacity increase to be double or even triple, depending on type and consistency of cohesive soils.
3. The maximum bearing capacity of piles after limit time may be with sufficient accuracy determined in terms of dynamic measurements and CAPWAP or TNOWAVE analyses using formula (1) or in terms of loading tests of piles using formulae (2) and (3).

The detailed relation presented in this paper regarding time influence on the bearing capacity increase of particular types of piles (i.e. concrete, steel open-ended and closed-ended) driven in various types of cohesive soils (i.e. soft and stiff clays, silty clays, silty and clayey sands, till) should be of practical importance and may be useful for initial estimation of pile bearing capacity.

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Summary

Phenomena occurring during the driving of piles in cohesive soils result in the pile bearing capacity after driving varies, gradually increasing with time. This increase may be quite significant. In mainly results from pore water pressure dissipation being increased while pile driving and progressing soil consolidation atound the pile shaft. For more complete recognition of the phenomena described as "set up" the results of various authors field investigations were analysed. In addition individual model tests were carried out. Special attention was paid to quantitative dependencies for different types of soils and to bearing capacity increase for pile shaft and toe with time, for various types of piles. Appropriate bearing capacity calculations of piles were performed to verify the reliability of formulae recommended in literature. Detailed quantitative relations given in this paper may be useful for estimation of pile bearing capacity increase.

Wpływ czasu na wzrost nośności pali wbijanych w grunty spoiste

Streszczenie

Zjawiska zachodzące w trakcie wbijania pali w grunty spoiste powodują, że nośność pala po wbiciu nie jest stała, lecz wzrasta stopniowo wraz z upływem czasu. Wzrost ten może być dość znaczny i wynika głównie z dysypacji, zwiększanego w trakcie wbijania pali ciśnienia wody w porach gruntu oraz stopniowej konsolidacji gruntu spoistego wokół pobocznicy pala. W celu pełniejszego poznania tego zjawiska przeanalizowano wyniki obcych badań terenowych. Dodatkowo wykonano własne badania modelowe. Szczególną uwagę zwrócono na zależności ilościowe dotyczące wzrostu nośności pobocznicy i podstawy różnych rodzajów pali wbijanych w różne grunty spoiste. Przedstawiono także odpowiednie obliczenia nośności pali w celu weryfikacji wzorów zalecanych w literaturze. Szczegółowe zależności ilościowe przedstawione w pracy mogą być praktycznie wykorzystywane do określania wzrostu nośności pali wbijanych w grunty spoiste.

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