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Foundations on friction creep piles in soft clays

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Foundations on Friction Creep Piles in Soft Clays
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SYNOPSIS. In this lecture, a new principle for design of building foundations on friction piles in soft, cohesive soils is presented. Four case records on buildings designed according to the new principle are analyzed in respect of total and differential settlement. In two cases, a comparison has been made with buildings on conventional friction piles. The savings in foundation cost of the buildings designed with creep piles is evaluated in relation to buildings designed with conventional friction piles. The lecture is a synthesis of research and development carried out together with Leif Jendeby, M.Sc., Chalmers University of Technology and Rolf Kilström, M.Sc., AB Jacobson & Widmark, Gothenburg.

INTRODUCTION
In Sweden, in a number of regions, the subsoil consists of soft, highly plastic clay to very great depths. For example, in the Uppsala and Gothenburg regions, clay deposits reach depths of over 100 m. Foundation on piles driven to refusal in underlying bearing strata will in these cases be very expensive and therefore friction piles have been extensively used for foundation purposes. Important research and development of this foundation technique was carried through by two outstanding pioneers in this field of geotechnical engineering: Wendel (1900) and Hultin (1928). Their work increased the understanding of friction pile behaviour and formed a basis for friction pile design. The present design of conventional friction piles, according to the Swedish Building Code, is based on the assumption that the piles should carry the total load of the building with a factor of safety against pile failure under working conditions of at least 3. The failure load of the piles is generally estimated on a theoretical basis from the undrained shear strength determined in situ be means of the field vane test or on laboratory samples by means of the fall-cone test (Hansbo, 1957). Full-scale loading tests are exceptional.

Due to the extensive lengths of the friction piles, required to reach sufficiently high bearing capacity, these often have had to be spliced twice which, of course, increases the foundation cost. Therefore with the present decline in the economy, more cost-effective solutions than those relating to conventional friction piles have become increasingly desirable.

One of the first modifications of the principle for the design of friction piles, used in order to reduce the cost of piling, was made in connection with the reconstruction of Ostra Nordstan, Gothenburg (Hansbo et.al., 1977). Here old foundation principles, sprung from sound engineering judgement and long experience, were revived and modified, on the basis of modern geotechnical science, to fulfill present foundation demands.

NEW PRINCIPLE OF DESIGN
Encouraged by the positive experience of the foundation of Ostra Nordstan, which was designed by J&W (AB Jacobson & Widmark), the same firm took another step forward in the utilization of friction piles by introducing a new design principle. According to this design principle the building is founded on a piled raft. The load of the building in excess of the preconsolidation (maximum past) pressure of the clay is assumed to be carried by the piles while the remaining load (i.e. the stress increment up to the preconsolidation pressure) is assumed to be carried by direct contact pressure at the raft/soil interface. The design load of the piles chosen is equal to the so-called "creep load", meaning the pile load that causes a state of creep failure.

In cases where the average net load increase exceeds the preconsolidation pressure of the clay, the object of creep piling is obviously not to eliminate but to reduce consolidation settlement. The latter is achieved by the fact that the stress increment exceeding the preconsolidation pressure of the clay is transferred to greater depths where the preconsolidation pressure is higher. Yet another advantage is attained. Since the piles are in a state of creep failure, the design of the raft with regard to the influence of the piles is made simpler: the piles can be considered as upward point loads equal to the creep load. Moreover, the piles can be distributed in such a way that the differential settlement of the building is minimized which, in turn, generally also represents the most cost-effective solution.

THE CREEP FAILURE LOAD
The ideas behind the creep failure load emanate from modern research into the effect of strain rate on the undrained shear strength of clay (e.g. Bjerrum, 1971; Hansbo, 1972; Torstensson, 1973; Hansbo, 1975).

For example, it has been shown that the shape of the stress-strain curve observed in the field vane test is very much dependent on the rate of rotation of the vane. It has also been shown that creep failure of clay will take place in the undrained triaxial test at maintained loads well below the conventional failure loads. A synthesis of the results obtained indicates that there is a certain lower limit below which shear failure in creep will not take place. This lower limit is named here the "creep strength" of the clay.

A suitable procedure for determination of the creep
strength, in practice, is to carry out field vane tests with different rates of rotation of the vane (different times to failure). In theory, only three different rates are required, but due to the natural variation in soil characteristics which may give rise to unacceptable values, four to five rates of rotation are preferable (Fig. 1). From the values thus determined, the shear strength \( \tau_f \) at a certain time to failure, \( t \), can be obtained from the relation (Hansbo, 1971 and 1975).

\[
\frac{\tau_f}{\tau_{fu}} = A + B \ln \left( \frac{t_0}{t + C} \right)
\]

where \( A, B \) and \( C \) are empirical parameters

\( \tau_{fu} \) = conventionally determined undrained shear strength (shear stress leading to failure after time \( t_0 \) approximately 1 minute).

The creep strength \( \tau_{cr} \) is obtained at infinitely low strain rate, i.e. when \( \frac{t_0}{t} \to \infty \) in Eq. (1).

Hence,

\[
\frac{\tau_{cr}}{\tau_{fu}} = A + B \ln C
\]

An alternative to Eq. (1) was given by Torstensson (1973) through the relation

\[
\frac{\tau_{cr}}{\tau_{fu}} = a \left( \frac{t_0}{t} \right)^{-B}
\]

where \( a \) and \( B \) are empirical parameters, but this relation obviously does not permit calculation of what is defined here as creep strength.

The problem of finding the parameter values \( A \) to \( C \) is easily resolved. Thus, experience has shown that the influence of \( C \) on the results obtained by Eq. (1) is negligible (at least for Scandinavian clays) when \( t_0/t \) is above about 0.05. Therefore, first \( A \) and \( B \) can be solved for two values of \( t_0/t > 0.05 \) and then \( C \) for some value \( t_0/t < 0.05 \). The values of \( A, B \) and \( C \) that give the best fit between calculation and observation are chosen.

The correlation between Eqs. (1) and (3) and observational data for Bäckebol clay is demonstrated in Fig. 1. We find that the normalized creep strength in this case is 0.72.

![Fig. 1](image)

Normalized undrained shear strength \( \left( \frac{\tau_{cr}}{\tau_{fu}} \right) \) determined by the field vane test in Bäckebol clay vs. time to failure. Vane allowed one day's rest in the clay before test was carried out. \( \tau_{fu} \) is shear strength determined by conventional field vane test. Soil characteristics: natural water content \( w = 80-90\% \); fall-cone liquid limit \( w_l = 70-60\% \); plastic limit \( w_p = 30-40\% \). Parameters used in Eq. (1): \( A = 1.17 \); \( B = 0.125 \); \( C = 0.15 \times 10^{-3} \) in Eq. (3); \( a = 1.17 \); \( B = -0.05 \).

The vane tests whose results are given in Fig. 1 differ from conventional tests in that before the tests were carried out, the vane was allowed one day's rest in the soil. This explains why the normalized shear strength at 1 minute's time to failure is not equal to unity. Whether or not this will affect the creep strength is, to our knowledge, not yet investigated.

However, the results obtained can be compared with another method for determining the creep failure load of a pile, namely to carry out full-scale pile loading tests. Then, each load step should be kept constant during, say, 16 minutes (with readings after 2, 4, 8 and 16 minutes). The creep settlement of the pile from 2 to 16 min is plotted vs. pile load. The creep load corresponds to the point on the curve where it has its minimum radius of curvature and after which creep is increasing drastically (Fig. 2).

![Fig. 2](image)

Fig. 2 Result of pile loading test carried out at the Bäckebol test site. Soil characteristics given in Fig. 1. Pile length 6 m. Pile diameter 0.14 m.

Studying the results shown in Fig. 2, we find that the creep load is 16.5 kN and the failure load - evaluated according to Mazurkiewicz (1972) - is 26.5 kN. This gives \( P_{cr}/P_f = 0.62 \). This value is very nearly equal to the normalized creep strength value obtained from Fig. 1 if this is divided by the normalized shear strength obtained at 1 minute's time to failure: 0.72/1.2 = 0.60.
The creep failure load of a pile can now be calculated under the assumption that the creep strength of the clay is fully mobilized along the pile/clay interface. On the calculation, regard must be paid to the influence of the shape of the piles (full values used for conical piles; 80% of the values for cylindrical piles).

Although the determination of the creep strength is very simple in principle, it is time-consuming and costly and, therefore, in reality only a quite restricted number of such tests can be carried out, generally only one per building site. It is then assumed, by experience, that the ratio $T_f/T_u$ remains constant throughout the soil profile.

**CREEP STRENGTH INVESTIGATIONS AT SITES OF OBSERVATION**

Four case histories including buildings situated in the town districts Stampen, Gullbergsvass, Olskroken and Aby, in the Gothenburg-Mölndal region, will now be presented.

At these sites field vane tests with different strain rates were carried out (Fig. 3).

![Fig. 3 Normalized undrained shear strength ($T_f/T_u$) determined by the field vane test and the theoretical correlation according to Eq. (1).]

From Eq. (2), we find for the four sites Stampen, Gullbergsvass, Olskroken and Aby that $T_f/T_u$ is equal to 0.81, 0.79, 1.00 and 0.80 respectively.

Due to the restricted number of long-term field vane tests and the scattering observed, the values of $T_f/T_u$ to be used for calculating the creep failure load of the piles will have to be chosen conservatively. One plausible assumption, based upon the results of the previously mentioned pile loading tests, is that the respective ratios $T_f/T_u$ given above should be corrected by dividing their values by the value obtained at a time to failure of 1 minute. This would give the following creep strengths: 0.66 $T_u$ at Stampen; 0.60 $T_u$ at Gullbergsvass; 0.67 $T_u$ at Olskroken and 0.78 $T_u$ at Aby.

**OFFICE BUILDING, STAMPEN**

The soil consists of 0.5-1.5 m fill underlain by 30-40 m of soft, highly plastic clay on cohesionless sedimentary soil or till. Typical soil characteristics are presented in Fig. 4. The clay is lightly overconsolidated. At the time of the soil investigation, the preconsolidation pressure exceeded the effective overburden pressure by about 30-40 kPa. The virgin compression ratio $CR=C/(1+e_0)$ varies between 0.4 and 0.55. The building, Fig. 5, which was constructed in 1974-75, is an office that is 7 storeys high on part of the area and 1-3 on the rest. The lower parts are connected to an old building founded on traditional friction piles. The whole building area (2300 m²) is provided with a car park in the basement. The basement floor is 2.5-3 m below street level. The average load of the building and the net load increase (total load minus load of excavated soil) is given in Fig. 6. As can be seen, an average net load increase is obtained only below the part of the building with 7 floors. Therefore, this is the only part which is founded on a creep piled raft.

The piles are of wood and are 12 m in length. Their creep failure load has been calculated on the assumption that the creep strength of the clay is 60% of the conventional undrained shear strength. This gives a creep failure load of 100 kN. Now the design of the piled raft is made under the assumption that the total average net load increase is carried by the creep piles. According to this design principle, one pile is required per 4 m² of raft area.
The settlement distribution, 8 years after the completion of the building, is presented in Fig. 6. We find that the settlement varies from round 15 to 30 mm. The maximum differential settlement amounts to about 1:800.

The average settlement vs. time is given in Fig. 7.

**FACTORY, GULLBERGSVASS**

The building, Fig. 8, is situated in an area which was reclaimed by in-filling of the river Gota Alv in the middle of the 19th century and built on in 1880-1890. The soil consists of 2.5-3 m fill underlain by soft, highly plastic clay to a depth of at least 50 m. The clay contains gyttja (necron mud) to a depth of round 15 m. It also contains shells. Typical soil characteristics are presented in Fig. 9. The preconsolidation pressure exceeds the effective overburden pressure (prevailing at the time of the site investigation) to a depth of approximately 6 m by about 20 kPa and between depths 20 and 25 m by about 50-60 kPa. At other depths the clay can be considered as being normally consolidated. The virgin compression ratio, CR, varies between 0.4 and 0.5.
The building, which was constructed in 1979-1980, consists of a factory built on an area of 2750 m² and a company dwelling on a site area of 1300 m².

The factory is in concrete. It is provided with a basement (with the floor 4.4-5 m below street level) and with 2-3 storeys above street level, see Fig. 10. As can be seen from Fig. 10, no average net load increase is obtained. Therefore, the factory is founded on an essentially unpiled raft foundation and creep piles are used only for the purpose of taking the pressure peaks caused by the column loads and front walls. The piles below internal columns are of wood and 18 m in length. Under the front walls, 18 m long wooden piles, spliced with 8 m long concrete piles, have been used. The creep failure load is calculated on the assumption that the creep failure strength of the clay is 60% of the conventional undrained shear strength. This gives a calculated creep failure load of 190 kN for the unspliced piles and 330 kN for the spliced piles. The direct upward pressure at the soil/raft interface is assumed to be equal to 35-60 kPa while the creep piles are assumed to carry the rest of the total average load, 10-15 kPa. This gives one pile per 20 m² of raft area.

The raft is cast in watertight concrete. A drainage layer has been placed underneath the raft so as to make possible a controlled and evenly distributed water pressure of 30 kPa. Thus, the effective clay pressure at the raft/soil interface will be reduced to only 5-30 kPa.

The company dwelling is a 1-storey building of wood, partly without and partly with a basement.

The part of the dwelling without a basement is founded on a grillage of prefabricated concrete elements supported by concrete beams that are cast in-situ and rest on conventional friction piles (18 m long wooden piles spliced with 5 m long concrete piles; one per 12 m² of raft area). The part of the dwelling with a basement is founded on a piled raft. Of the total average foundation pressure, 25 kPa is assumed to be carried by direct contact pressure at the raft/soil interface, while 15 kPa is assumed to be carried by 18 m long creep piles. In this case the creep failure load of the piles is 170 kN. This gives one pile per 12 m² of raft area.

As can be seen in Fig. 10, the settlement 3 years after the completion of the foundation varies from about 20 to 45 mm. A maximum differential settlement of around 1:400 is obtained at the stair-well in the factory, close by the company dwelling. For the low part without piles (seen in the front of the factory, Fig. 8) a differential settlement of around 1:300 is observed. However, this part is just provisional.

The average settlement vs. time observed for the factory (founded on creep piles) and for the dwelling (founded on conventional friction piles) is presented in Fig. 11.

RESIDENTIAL BUILDINGS, OLSKROKEN

Two nearby houses, Fig. 12, one founded on creep piles and the other on conventional friction piles, were selected for a case study made possible through financial support from the National Board for Technical Development (STU).

The foundations of the two buildings were carefully monitored with piezometers at different depths beneath the buildings, settlement gauges, hydraulic load cells (flat jacks) for measuring the pile loads and Glötzl cells for measuring the contact stresses at the soil/raft interface, Fig. 13.

The subsoil at the site of the buildings consists of soft, highly plastic clay underlain by sand and gravel on till or rock. The thickness of the clay layer under the buildings varies between 35 and 55 m. The clay depth below the instrumented parts of the buildings is between 45 and 50 m. Typical geotechnical characteristics of the clay are presented in Fig. 14. With regard to possible variations in groundwater level, the clay can be considered normally consolidated. The virgin compression ratio CR varies between 0.4 and 0.65.

Before the clearance, this sector of the town was densely built over with 3-storey houses constructed with the
first storey of stone and the upper storeys of wood. The lighter parts of these houses were founded on wooden mats, the heavier parts (chimneys, fire walls and front walls) on 1-2 m thick stone walls on short, closely spaced wooden piles.

Building on conventional friction piles

This is a 4-storey concrete building with a bottom area of 50 m by 14 m. Space for water pipes and sewers is located below the bottom floor. The foundation beams are cast in-situ. The rest of the house is constructed of prefabricated concrete elements. The average weight of the house is approximately 6.5 t/m², which gives a total load of 46.4 MN. The house is founded in full accordance with the rules given in the Swedish Building Code, SBN 75. This means that the pile foundation is designed so as to carry the total load of the house (no support by direct contact pressure of the soil) with a safety factor of 3 against short-term pile failure. The piles utilized consist of 18 m long wooden piles spliced with 10 m long concrete piles, 270 mm in side length. The total number of piles then required is 211.

The "grillage" of foundation beams, Fig. 15, can be considered to be very rigid.
Fig. 14 Geotechnical characteristics of subsoil, Olskroken.

Fig. 15 View of foundation structure of the residential building, Olskroken, on conventional friction piles.

Building on creep piles

As in the former case, the building on creep piles is a 4-storey concrete building but its bottom area is larger - 75 m by 12 m. The building is completely cast in-situ. The basement is mostly utilized as a space for sewers and water pipes but it also contains an air-raid shelter. The raft consists of a 0.4 m thick plate of watertight concrete, and the basement as a whole can be considered as being quite rigid.

The average weight of the house is approximately 6 t/m².

Since the clay is normally consolidated, the net increase in vertical pressure, caused by the weight of the building, will have to be carried by the creep piles while the load that corresponds to the weight of the soil, excavated for the basement, is carried by direct contact pressure at the soil/raft interface. However, for the sake of safety against possible variations in groundwater level, the design contact pressure is assumed to be equal to 23 kPa whereas the excavated soil corresponds to a pressure decrease of 30 kPa at the foundation level.

The piles used for the foundation consist of 18 m long wooden piles spliced with 8 m long concrete piles, 0.3 m in diameter. The creep failure load of these piles, calculated on the basis of the creep strength of the clay (in this case 70% of the conventional undrained shear strength) is 330 kN. This gives a total number of 104 piles.

Precoring was used to reduce as much as possible disturbance effects on the clay due to pile driving.

Results

The amount of observations assembled during the period of study is very extensive (Jendeby, 1983), and, therefore, in this paper only the most interesting facts have been selected for presentation. These will give a general picture of the correlation between the basic assumptions used in the design of the foundation (pile loads; contact stresses at the soil/raft interface) and the settlement of the building on creep piles as compared to the settlement of the building on conventional friction piles.

Pile loads and contact stresses

The results of the pile load and contact stress observations are given in Figs. 16 through 18.

Obviously, the design assumptions used in the two cases are justified.

In the case of the building on conventional friction piles, we find that the contact stresses are negligible except for those at the beginning of the construction period, when the rigidity of the concrete foundation beams had not yet been established (the casting period). After the rigidity of the foundation was established the contact stresses were more or less reduced to zero. Consequently, the load of the building is carried completely by the piles, as assumed.

In the case of the building on creep piles, the picture is quite different. The contact stresses are found to carry the part of the load that causes a stress increase up to the preconsolidation pressure, while the other part is carried by the piles. The real behaviour is thus in full agreement with the assumptions behind this new principle of design.

Settlement

The settlement distribution 14 years after the completion of the frameworks of the buildings is presented in Figs. 19 and 20. We find that the settlement varies between 17 and 27 mm for the building on conventional friction piles and between 15 and 20 mm for the one on creep piles. The maximum differential settlement is 1:900 in the former case and 1:1500 in the latter case.

From the geotechnical point of view, the main object of using creep piles in the foundation technique, namely to reduce differential settlement, has doubtless been achieved. As regards the total average settlement, the difference between the two buildings is more or less negligible and the period since the buildings were completed is too short to permit a prediction of the long-term consolidation settlement (Fig. 21). The rate of
settlement seems at present almost equal for both build-
ing. Some indication of the consolidation settlement, yet
to come, is obtained from the excess pore pressure
readings (Fig. 22). The average excess pore pressure
from 10 to 40 m depth is round 20 kPa for the conven-
tional building one year after completion of its framework
and 14 kPa for the building an creep piles six months
after completion of its framework. With reservation for
the influence of the differences in excess pore pressure
distribution as well as variations in compressibility with
depth, we have reason to believe that the final consoli-
dation settlement in both cases will be approximately
equal.

STORAGE BUILDING, ABY

The soil consists of lightly overconsolidated, soft, highly
plastic clay to at least 40 m depth. The clay has a 1-1.5
m thick dry crust. Typical soil characteristics are pre-
sented in Fig. 23. The virgin compression ratio, CR, of
the clay varies between 0.5 and 0.65.
he building (Fig. 24) which was constructed in 1980-
81, has 2 storeys and no basement. It is constructed
of prefabricated concrete. The front walls are made of
grey concrete. The building covers an area of
1,000 m².

The house is founded on a creep piled raft 150 mm
thick. Of the total foundation pressure (in this case
equal to the net load), 15-20 kPa is assumed to be
transmitted by contact pressure at the raft/soil interface
while 5-10 kPa is assumed to be carried by the piles.

The creep piles are placed under concrete beams cast
long the column rows. They are 18 m long wooden piles
placed with 3 m long concrete piles. The creep failure
ad has been calculated on the assumption that the
creep strength of the clay is 75 % of the conventional
undrained shear strength. This gives a pile load of
0 kN. Thus, one pile per 30 m² of raft areas is re-
dired.

The settlement distribution, 2 years after the comple-
tion of the foundation, is presented in Fig. 25. The settle-
ment varies from about 10 to 20 mm. The maximum
differential settlement is around 1:3000.
The differential settlement observed in, for example, the factory, Gulbergsvass, may seem very close to acceptable limits, but in reality a large percentage of these differential settlements have taken place before the structural rigidity of the building is established (Fig. 11). Therefore, most probably, a large tolerance still remains for a possible increase in differential settlement before any damage will be caused to the structures. As a matter of fact, most of the settlement represents "elastic" settlement due to preloading during the construction period.

The case records presented show clearly that from a technical standpoint foundations on creep piles as compared to those on conventional friction piles are of equal or even higher merit. Above all, the new design principle of creep pile foundations seems to offer very good possibilities for minimizing differential settlement.

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