Dynamic pre-loading of large diameter bored piles

Préchargement dynamique des pieux forés de grand diamètre

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SYNOPSIS Foundation on large diameter bored piles in loose and medium dense non-cohesive soils is not economic in Sweden. If the non-cohesive soil beneath the pile is made more dense and if the stiffness of the soil could be measured the situation for bored piles would be better. A method to pre-load non-cohesive soil beneath bored piles dynamically has been described by Eresund (1972). Several field and laboratory tests have shown positive results, Berggren (1981). In 1983, the Swedish contractor Stabilator AB decided on development of equipment for production of bored piles. Development work was supported by the National Swedish Board for Technical Development (STU).

This paper describes the theoretical background for dynamic pre-loading and the results from a field test. The test shows that the analysis of the dynamic pre-loading gives a good estimate of the static behaviour of the pile tip. The test also shows that the pile can take considerably more load, 5-10 times, after dynamic pre-loading than if it was cast in an untreated soil.

1. RELATIONSHIP BETWEEN DYNAMIC AND STATIC LOADING

The relationship between the dynamic and the static force giving the same displacement of a weightless foundation on soil may according to Lysmer & Richert (1966), be expressed by:

$$\frac{P_{dyn}}{P_{stat}} = k_1 + c_1 \frac{D}{2v_s} \cdot \frac{\dot{s}}{s} \qquad (1)$$

where

P_{dyn} = dynamic force
P_{stat} = static force
k_1 = coefficient dependent on the elastic behaviour of the soil
c_1 = coefficient dependent on the viscous behaviour of the soil
D = diameter of the foundation
v_s = shear wave velocity
s = displacement
$\dot{s}$ = displacement rate

The assumptions for equation (1) are a weightless foundation and that the dynamic loading is a continous sinusodial one. Now introduce the additional assumptions that the force induced in the soil by the blow from a free-falling weight follows a sinusodial course and that the time to reach maximum load is t_0. The mean value of the displacement rate is then

$$\dot{s} = s/t_0 \qquad (2)$$

which, introduced into equation (1), gives

$$\frac{P_{dyn}}{P_{stat}} = k_1 + c_1 \frac{D}{2v_s t_0} \qquad (3)$$

The coefficients k_1 and c_1 are dependent on the frequency factor a_0 which may be expressed by (Lysmer & Richart, 1966)

$$a_0 = w \frac{D}{2} \sqrt{\frac{\rho}{G}} \qquad (4)$$

where

w = angular frequency of the loading
ρ = density of the soil
G = shear modulus of the soil

It is stated that

$$v_s = \sqrt{\frac{G}{\rho}} \qquad (5)$$

$$w = 2\pi \frac{1}{T} = 2\pi \frac{1}{4t_0} \qquad (6)$$

where

T = period

The expression (4) of the frequency factor a_0 can thus be rearranged

$$a_0 = \frac{\pi D}{4 v_s t_0} \qquad (7)$$

An analysis of the equation (3) shows that only one graph is needed to obtain the relationship between the dynamic force and the static force at the same displacement for different foundation diameters D. Figure 1 is valid for the diameter D_0 = 1.0 m. If another diameter D_1 is needed the graph in Figure 1 can be used by changing the parameter v_s to v_s/α where

$$\alpha = \frac{D_1}{D_0} \qquad (8)$$

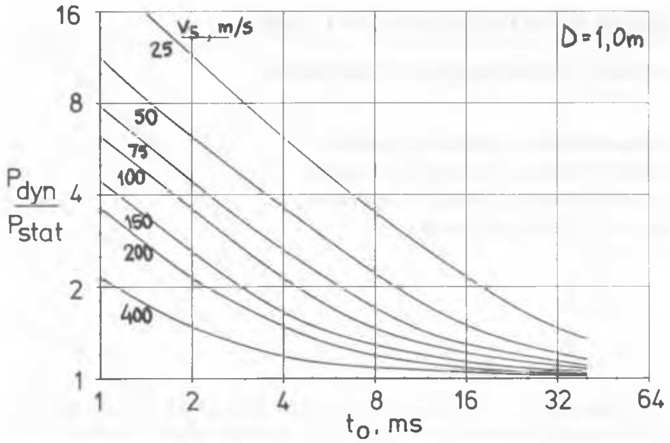

FIG. 1. The relationship between the dynamic and static force P_{dyn}/P_{stat} at the same displacement, the shear wave velocity v_s and the time t_0 from zero-force to maximum force.

2. MEASUREMENTS DURING DYNAMIC PRE-LOADING

The dynamic loading can be transferred to the soil by a heavy ram that is allowed to fall freely on a foundation standing on the soil. The contact pressure between the foundation and the soil can be evaluated by measuring the stress in the foundation and the acceleration of the foundation according to the idea described by Eresund (1972). The steel stress and acceleration are measured at a certain level, x, of the foundation.

$$\sigma_x A - F = m_x \ddot{x} \qquad (9)$$

where

A = cross section of the foundation
m_x = the weight of the foundation below the level x

The contact pressure between the foundation and the soil will be

$$p = \sigma_x - \frac{m_x \ddot{x}}{A} \qquad (10)$$

The displacement of the foundation during the blow is evaluated by double integration of the acceleration curve, i.e.

$$x = \int\int \ddot{x} dt \qquad (11)$$

3. EARLIER EXPERIENCE FROM DYNAMIC PRE-LOADING

Laboratory dynamic pre-loading tests on dry sand were made by Eresund (1972). The positive effect of the pre-loading was very evident. One of the factors that describes the effect of the pre-loading is the plastic ratio s_{pl}/s_{max}, the ratio between the residual and the maximum displacement caused by a blow. The plastic ratio s_{pl}/s_{max} is a measure of the plasticity created by the dynamic loading. A decreasing ratio indicates a hardening of the soil and the smaller the ratio the more elastic the soil. The agreement between calculated and measured force is best at a low plastic ratio. Eresund's conclusion was that the hardening effect of the dynamic pre-loading was low after the plastic ratio had reached 0.1. He also stated that the pre-loading should not be stopped until the platic ratio was 0.1.

Later experiments, Berggren (1981), in the field and in the laboratory on saturated fine sand show that the plastic ratio can not reach a value as low as 0.1 even at relatively low loads. Realistic final values are instead 0.15 to 0.25. The measured increase in stiffness of the soil before and after dynamic pre-loading reached 5 to 10 in Eresund's and Berggren's tests.

Eresund (1972) found that the loading rate must be limited so that the pressure in the soil beneath the foundation increases at the same rate as the pressure on the contact base. Later laboratory tests show that if the loading rate is too high shear deformations spread deeper in the soil than the accumulating displacement of the foundation. A low dynamic loading rate has an effect positive on the displacement of a foundation during a later static loading. At high dynamic loading rates, deterioration can also occur compared to the original situation.

One other important factor is development of pore pressure, especially in fine, non-cohesive soil, where the rate of pore pressure dissipation is low. What happens during a pore pressure increase is not entirely evident. However, it is essential that the load cycles are slow enough to ensure pore pressure dissipation.

The conclusion that can be drawn is that dynamic pre-loading ought to be performed in steps of increasing load levels and so that the plastic ratio s_{pl}/s_{max} decreases during each step.

4. EQUIPMENT FOR DYNAMIC PRE-LOADING

The equipment developed by Stabilator AB consists of five parts:

1. Weight 3.8 t, length 4 m
2. Guide line for weight
3. Steel foot foundation 1.4 t, diameter 850 mm
4. Measuring unit, diameter 300 mm
5. Measuring equipment, station for data acquisition and analyzing equipment.

In order to register acceleration and steel stress a measuring unit is placed directly above the steel foot foundation. Accelerometers and resistance strain gauges are mounted inside this measuring unit. The signals from the strain gauges and the accelerometers are distributed by a cable to the station, where the signals are recorded on a digital oscilloscope. If required the signals can be stored on diskettes. A microcomputer with plotter is used for analyses and plotting.

5. PERFORMANCE OF DYNAMIC PRE-LOADING

Macadam is placed at the bottom of the bored hole to a thickness equal to half the diameter of the hole. The risk of suction is thereby decreased in the event of up-lifting of the equipment. The suction otherwise loosens the bottom and must be avoided.

The dynamic pre-loading should be started with a low fall height, 20 cm. Continuous registration of residual axial displacement of the foundation shows whether the soil is hardening or not, i.e. whether the residual displacement and the plastic ratio decrease with the number of blows. When the residual displacement and the plastic ratio are lower than certain criteria a larger fall height (the double) is ordered and the procedure is repeated.

6. EVALUATION OF DYNAMIC PRE-LOADING

In order to calculate the relationship between the dynamic and the static force according to Chapter 1 above, the shear modulus of the soil must be determined. This is achieved interactively, starting with an empirical value G_0.

The shear modulus G for a stiff foundation on the surface of an elastic half-space may be expressed by:

$$G = \frac{p}{s} \cdot \frac{D}{4} \cdot \frac{1-\nu}{2}$$

where

p = contact pressure (static)
D = diameter of the foundation
ν = Poisson's ratio

Since the displacement is the same in the dynamic and static cases the shear modulus should be the same. If the calculated value of G differs from G_0 the calculation is repeated with another value of G_0.

The dynamic pre-loading is finished when the required values of load and displacement are reached. The required safety margin to bearing-capacity failure of the soil is achieved if

- the soil is hardening
- the residual axial displacement follows a linear path with increasing fall height
- the displacement at maximum contact pressure is less than 2% of the diameter of the foundation.

7. FIELD TEST

7.1 Soil conditions

The soil layers at the test site consists of

0.5 m fill
3 m clay
silty, sandy moraine.

7.2 Dynamic pre-loading

When the boring reached a "firm" moraine layer at 6 m depth the dynamic pre-loading equipment was installed in the hole and an extensive test was performed. The measured residual displacement and a "shear modulus" G_{dyn} are shown in Figure 2 (ν=0.3)

where $$G_{dyn} = \frac{P_{dyn}^{max}}{s(\text{at } P_{dyn}^{max})} \frac{\pi D}{4} \frac{1-\nu}{2}$$

Figure 2 shows that the residual axial displacement decreases within every drop series (series with constant fall height) as long as the fall height does not exceed 2.0 m. At the fall heights 2.0 m and 4.0 m no hardening effect can be noticed. Similarly, the "shear modulus" is constant, in contrast to an increasing course at lower fall heights. The observation indicates that the soil is still elastic but that the bearing capacity has not been reached, even at a fall height of 4 m.

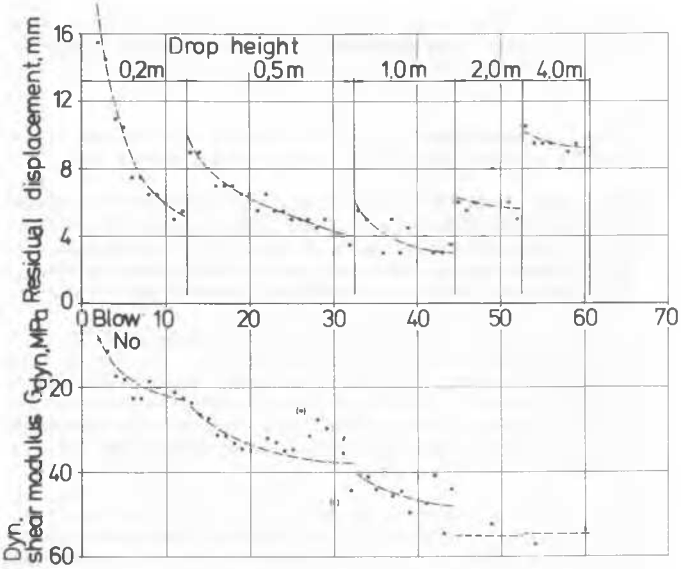

FIG. 2 Measured residual settlement and calculated dynamic "shear modulus".

7.3 Analysis of dynamic pre-loading

The calculation of the load displacement relationship of the pile tip (according to Chapter 6) shows that at 20 mm displacement a static pressure of 2.85 MPa is required on a foundation with a diameter of 0.85 m (dynamic pressure 4.4 MPa). The analysis also shows that the shear modulus of the soil is 40 MPa. The static pressure, 2.85 MPa, corresponds to the load 2.36 MN at 20 mm displacement of a pile tip with a diameter of 1.0 m.

The skin friction capacity of a pile is normally reached after a relatively small displacement, 5-10 mm. The calculated skin friction capacity for the test pile is 0.53 MN.

The total calculated load supported by the test pile is 2.89 MN at 20 mm displacement.

7.4 Static load test

After dynamic pre-loading of the pile bottom the pile itself was cast.

The static pile load test arrangement is shown in Figure 3.

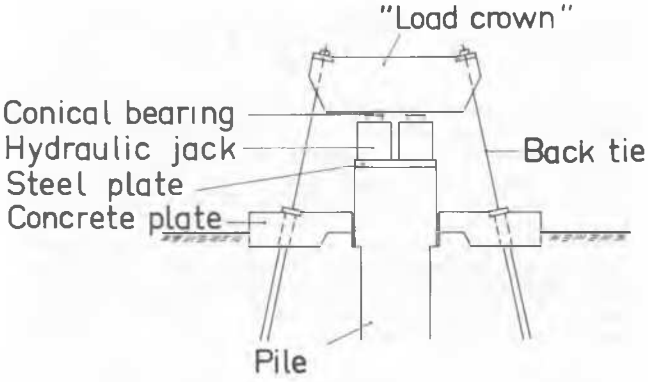

FIG. 3 Test arrangement for the static load test.

Axial displacement of the pile cap and two other points along the pile length were measured.

The load test was performed as loading in steps of the same duration. The load steps were basically 250 kN, with a duration of 16 minutes. Unloading and reloading for a short duration (4 minutes) was performed on some occasions.

During the static load test the displacement of the concrete plate (see Figure 3), followed that of the pile. At a pile-top load of 8.5 MN the displacement of the concrete plate was 50 mm. The ultimate strength of the connection between the pile and the plate was calculated to be 1300 kN.

The skin friction capacity of the pile part in clay was calculated to be 300 kN and that of the pile part in non-cohesive soil to be 300 kN.

Figure 4 shows a calculated load-displacement curve for the pile/plate connection and for the skin friction of the pile in clay and in non-cohesive soil.

The general load-displacement relationship for pile skin friction in clay given by Torstensson (1973) has been used for all given loads in Figure 4.

The total bearing capacity of the pile is 8.5 MN of which that of the pile point is 5.6 MN.

The non-linear load-displacement relationship given by Torstensson (1973) has also been used to describe the function of the pile point.

Addition of the parts in Figure 4 gives a load-displacement curve according to Figure 5, which shows the measured load-displacement curve. The load-displacement curve calculated according to Swedish quidelines for large diameter bored piles (not dynamically preloaded) is also shown in the figure.

Figure 5 shows clearly that dynamic pre-loading can be used to considerably increase the soil stiffness and that analysis of the pre-loading will give a good estimate of the static behaviour of the pile point.

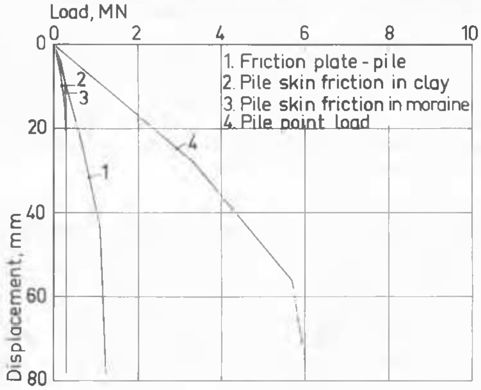

FIG. 4 Calculated load-displacement relationships.

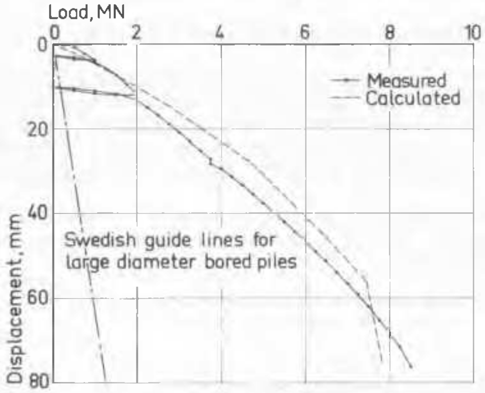

FIG 5. Calculated and measured load-displacement curves of the pile load.

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