Raft foundation on floating micropiles in soft soils

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ABSTRACT: The paper introduces system of raft foundations on floating micropiles in normally consolidated soft soils. Settlement measurements on projects with raft foundation on floating micropiles show a reduction of settlement compared with raft foundations. Laboratory model tests on a segment of a micropile show an improvement of the surrounding soft soil along the injection horizon after injection with cement grout.

1 INTRODUCTION

There are a lot of experiences on foundations of multi-story buildings in soft soils in Mexico City (Mexican clay), Norway (Oslo clay), Asia (Bangkok clay) etc. These are reported in the literature for e.g. in Auvinet (2002), Hansbo/Jendeby (1983), Broms/Hansbo (1981), etc. In theses countries piled raft foundations, raft foundations or a combination of both systems are used predominantly.

In recent years, the new foundation system of a raft foundation on floating stabilizing grouted micropiles (in german: **s**chwimmende **s**tabilisierende **V**erpresspfahl-**P**lattengründung, SSVP) are used successfully in southern Germany. The system has been proven very effective in areas close to the lakes, where the underground consists of a deep lacustrine soft layer with a thickness partly greater 30 to 60 m. The lacustrine clay is postglacial deposit with varying silt and fine sand seems and it is known as normally consolidated or partially underconsolidated (Gudehus et al. 1987).

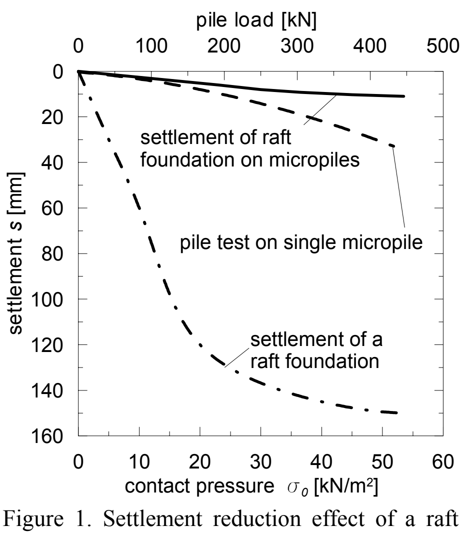


Figure 1. Settlement reduction effect of a raft foundation on floating micropiles (SSVP)

Settlement measurements on several projects with SSVP foundation system show a large settlement reduction and stabilisation of the soft soil (Fig. 1) (Kempfert 1986; Kempfert/Böhm 2003).

Figure 1 shows the effectiveness of the raft foundation on floating micropiles (SSVP) compared to a raft foundation resting on soft soils. A settlement improvement factor β has been calculated for several projects with SSVP foundation system according to the following equation:

$$\beta = s_{raft}/s_{SSVP} = 3 \text{ to } 10. \qquad (1)$$

where s_{raft} = Settlement of the raft foundation
s_{SSVP} = Settlement of SSVP-foundation system

2 MICROPILE-SYSTEMS AND MICROPILES SUITABLE FOR THE SSVP

Micropiles are originally developed by the Italien Lizzi and patented in 1952 to support and safeguard foundations at risk. Since then micropiles have been used world-wide for distinct buildings measures. According to Bruce/Juran (1997), micropiles are classified internationally in four categories based primarily on the type and grouting pressure. *Type A* includes those micropiles, where grout is placed in the pile under gravity head only. The pile is cast without reinforcement, with a monobar, a cage or a tube. The drilling casing itself can be also used as reinforcement. The loading capacity of the pile can increase when the equilibrium of the surrounding soil remains undisturbed (Lizzi 1997).

The neat cement grout of the micropile *type B* is injected as the temporary steel casing is withdrawn. Monobar(s) or tube are used as reinforcement. In these type of piles the grout is injected over the hole length while it is fresh and fluid. The injection pressures range from 3 to 10 bar.

The micropile *type C* is installed in two steps. In the first step neat cement grout is placed in the hole and after

15 to 25 minutes later similar grout is injected with preplaced grout pipes at a pressure of at least 10 bar before hardening of the primary grout. Single or several monobars and also tubes are used as the reinforcement of type C.

The installation of *type D* follows in two steps similar to t*ype C*. Cement grout is placed in the hole under gravity only or injected into the hole while the drill casing is withdrawn. After hardening of the grout, i.e., after approximately 6 hours, the semi-hardened grout body is burst open by fresh grout under high pressure through preplaced grout pipes. The grout is injected with a pressure between 20 to 80 bar. If necessary the injection can also be repeated several times after flushing the pipes. As reinforcement a single or several monobars as well as tube are used.

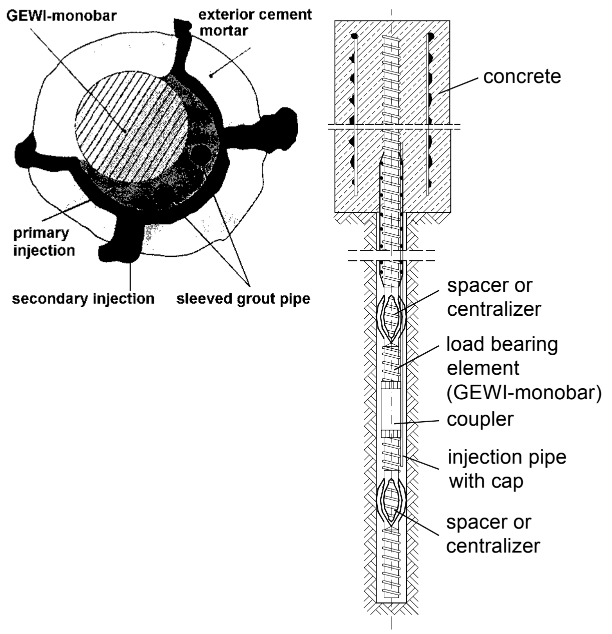


Figure 2. GEWI-micropile (system Dywidag);
a) Profile of monobar-micropile after injection,
b) GEWI-micropile

In the European countries micropiles are called cast-in-place piles with a diameter between 0.15 and 0.3 m and composite pile with a diameter greater 0.1 m to 0.3 m. The boreholes for micropiles are made either by drilling, driving or vibration methods. As reinforcement, monobars as round or other profile steel bar, or tubes can be used. The primary injection of the micropile is effected through the top of the pile, whereas the secondary injection is made through injection pipes or tubes (a single use) or bands of tube (multiple use) at different horizon along the depth of the pile. Figure 2 shows a GEWI-micropile, which is used in the practical project described in the following.

3 SSVP FOUNDATION SYSTEM

The essential design features of SSVP foundation system is illustrated in figure 3 and consists of the following:

- Micropiles as composite piles with a steel profile (GEWI-pile), length between 15 to 25 m, diameter between 15 und 30 cm, a uniform square grid at a spacing of 1 m to 3 m and one to three injection horizon distributed in the lower half length of the micropile,
- Reinforced raft foundation in which the pile heads are structurally connected and fixed,
- Depending on the boundary conditions of the form of the foundation layout, the soil conditions and pile grid, an additional stiffened reinforced basement as compensated foundation may also be necessary.

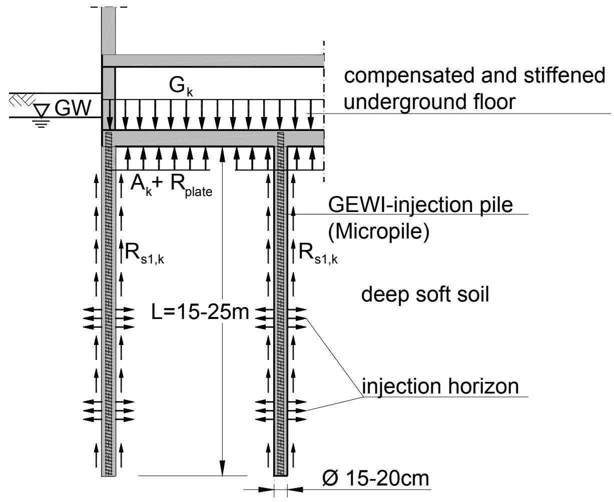


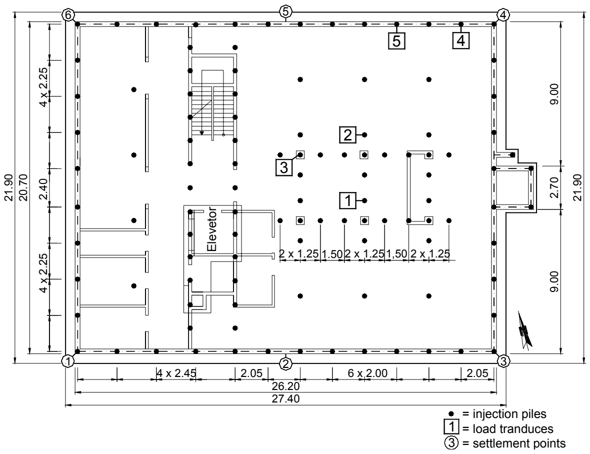
Figure 3. Sketch of the mode of function of the construction elements of the SSVP-foundation system

4 MEASUREMENT RESULTS OF A PROJECT WITH SSVP-FOUNDATION SYSTEM

In the following a project with SSVP-foundation system is introduced. The 4-story building rests on a stiffened basement (compensated foundation) over floating micopiles. The size of the building is 26.2 x 20.7 m (Fig. 4) The raft is located 4.2 m below the ground surface. The raft foundation overhangs 0.6 m over the outer dimension to account for buoyancy. The groundwater level fluctuates according to the seasonal variation of the water level of the nearby river and it is located on average about 1.4 m above the raft foundation. In case of high and low water level of the river, the groundwater is located at 0.3 m below ground surface and at the raft foundation level respectively (Fig. 4b).

The project is founded on SSVP-foundation system with 111 micropiles (GEWI-micropiles, System Dywidag, BSt 500 S/Øbar = 50 mm). The raft is rested on the top of the micropiles and 0.5 m thick compacted gravel layer.

a)



b)

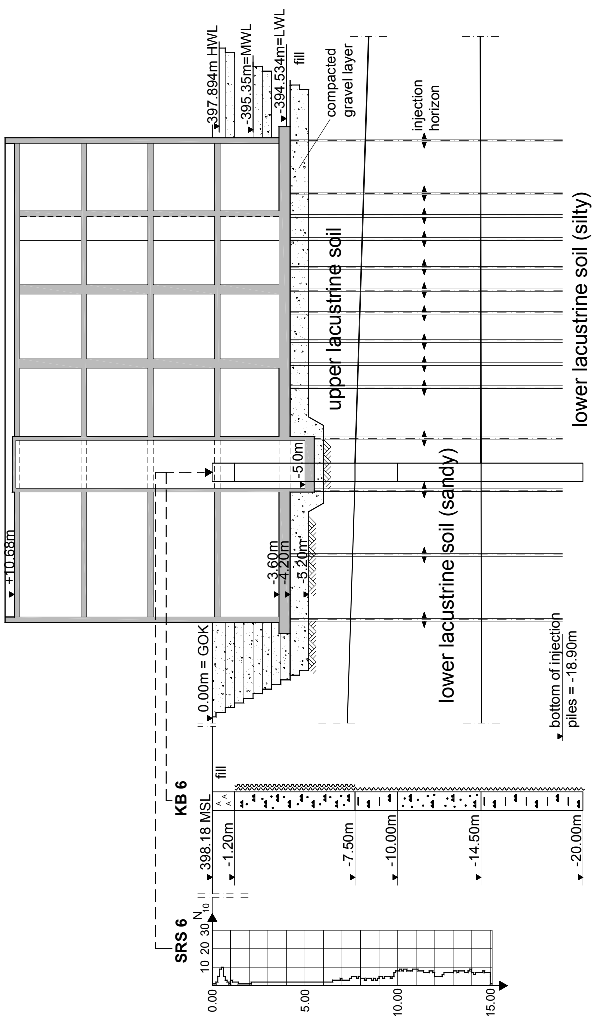


Figure 4. Example of a SSVP foundation; a) Ground plan, b) Section plan

The micropiles have a length of 15 m, 4 m of which is embedded in the lower lacustrine clay. They are arranged at center to center distance of 4 m in the middle of the foundation and at 2.25 m at the external walls. At location of staircases, elevators and others, the spacing is reduced to 1.25 m. The radius r_c of the influence area of a single micropiles lies between 1.4 and 4.5 m.

The boreholes for the installation of the micropiles were made using auger drilling with casing. First, the boreholes had been filled with C25 concrete under gravity head only. After one day hardening time, a cement grout (w/c-ratio = 0.6) was injected once or sometimes twice through the preplaced sleeved pipes under a pressure of about 35 bar. The injected grout volume was about 63 litres per pile and injection horizon.

a)

Figure 5. Exposed micropiles with plates at head

The underground investigations using drilling and sounding revealed up to 36 m thick layer of soft lacustrine clay overlain by about 2 m thick fill (Fig. 4b). The lacustrine layer is divided into upper and lower layer according to the consistency of the clay at a depth of 15 m below surface. Inclusions of thin horizontal silt and fine sand seems had also been observed in the lacustrine layer. Beneath the lower lacustrine clay layer, a moraine layer was encountered consisting of clayey silts with large proportion of sand and boulder. The range of soil parameters for the lacustrine clay is summarised in table 1.

Table 1. Range of soil parameter for lacustrine clay layer

specific weight γ	[kN/m³]	19 - 20
water ratio w	[%]	20 - 60
stiffness E_s	[MN/m^2]	2 - 8
shear strength φ'	[°]	20 – 27,5
cohesion, drined c'	[kN/m^2]	0 - 5
cohesion, undrained c_u	[kN/m^2]	10 - 30

The contact pressure of the foundation lies between 75 and 87 kN/m². The pressure relief due to excavation was about 35 kN/m² and the uplift pressure was estimated to be 10 kN/m² at the middle of the foundation.

Foundation settlements were monitored at different locations of the raft as shown in figure 4a. The result of settlement measurements is shown in figure 6a. An end settlement between 8.5 mm and 22.2 mm were extrapolated from the measured settlement according to Sherif (1973) at measuring points MP1 and MP4. On the other hand, the settlement of the raft foundation without the SSVP system was estimated to be 90 mm. Hence, settlement improvement-factor for this particular project becomes $\beta = 4$ at a point with maximal settlement (MP4).

From a pile load test a maximum pile resistance of 800 kN was measured. After the completion of the building a maximal force of 300 kN was measured at the head of pile No. 5 (Fig. 6b).

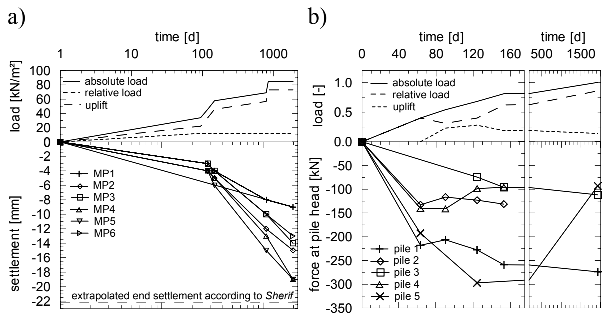


Figure 6. a) Settlement measurement and b) Measured force on pile head

The following conclusions can be drawn for the described project with the SSVP-foundation system:

- There exists a mutual interaction of the adjacent micropiles, because an increased injection pressure was observed for micropiles in a group than single micropiles,
- radial deformation of the surrounding soil was measured as a result of the injection, and
- deeply located injection horizon requires a higher injection pressure than shallowly located. Hence, the injection pressure is directly dependent on the stress level and stiffness of the surrounding subsoil.

The settlement improvement factor β has also been calculated for several practical projects with SSVP-foundation system in southern Germany and it lies between 3 and 10.

Measurements on similar projects that rest on raft foundations on floating driven reinforced concrete piles with a diameter of 0.3 m showed no significant settlement reduction.

5 MODEL TEST

5.1 *General*

To examine the load bearing behaviour of SSVP foundation in normally consolidated soft soils, a series of model tests had been conducted at the University of Kassel. The injection can improve the bearing capacity of a micropile and an optimum bond can be achieved between the grouted body and the surrounding soil. Investigation on stabilisation effect of micropiles in sandy soils can be found in the literature, for example, Schwarz (2002), Ripper (1984). However, the state of knowledge of micropiles in soft soils has barely been reported and needs further intensive investigations.

5.2 *Model concept and test procedure*

A section of a micropile was simulated at a scale of 1:1. Figure 7 illustrates the arrangement of the model test. The model consisted of a 2 m high and 2 m diameter hollow cylinder made of steel.

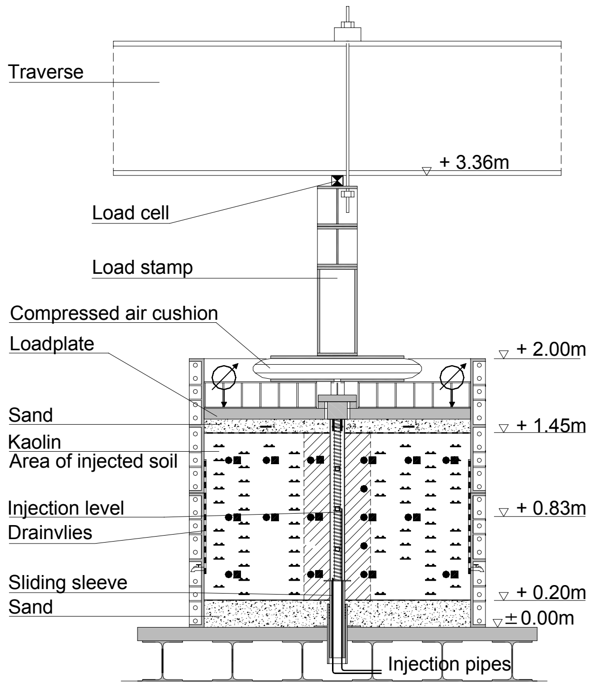


Figure 7. Model test on a segment of a micropile in soft soil

The soft soil is simulated kaolin powder prepared at a water content of 55%. To minimise the pore air, the kaolin was compacted using a rod vibrator. A sand layer was also placed at the bottom and the top to accelerate the consolidation of the kaolin layer. At the middle of the model a hollow perforated pvc pipe was placed to create a space for the micropile. This had also provided an additional drainage possibility for the kaolin. A 1.2 m long GEWI bar was then inserted into the hollow. At the bottom end of the hollow pipe, a special sliding sleeve was provided so that the micropile can sink freely during load test at the end of the test. In other words, only skin resistance could be measured in the load test.

A rigid steel plate was placed at the top to transfer the load from the pressure air bag. As a reaction to the pressure through the air bag, a rigid plate connected to a fixed frame by a steel rod was place at top of it. For further detail, see figure 7.

After the kaolin had been fully consolidated under a pressure of 40 kN/m², the hole was filled with concrete while withdrawing the hollow perforated pipe. Immediately after withdrawal of the pipe, a cement suspension was injected under pressure up to 7 bar into the fresh concrete through pre-placed tubes. Few hours later, after the concrete started to harden, the pile was post-grouted with a cement suspension under pressure of 12 bar, which results in bursting of the concrete and form cracks. All in all 25 litres of cement suspension were injected.

After the excess pore pressure due to the grouting pressure had been dissipated, the micropile was loaded incrementally to determine its carrying capacity.

5.3 *Results of the model test*

The model was equipped with pore pressure and earth pressure transducers as well as deformation gauges at different locations. Hence, horizontal stress and pore pressure was measured within the kaolin at different depths and radial distances. Settlement was also measured at the top of the load plate at four positions.

Figure 8 shows, for example, the development of the horizontal stress and pore pressure with time at the injection horizon and radial distance of about 0.3 m from the pile axis. The decrease in pore pressure and horizontal stress before grouting is due to the rearrangement of the load temporarily and withdrawal of the perforated pipe. As it can be seen from figure 8, the first grouting resulted no significant change in pore pressure and horizontal stress. However, the excess pore pressure reached its maximum value during the post-grouting but dissipated very fast within one day and reached its steady conditions. This indicates that the soil could not retain the additional stress due to injection pressure for a long time. Results at other measurement points also show similar course of pore pressure and horizontal stress development but with lower maximum values.

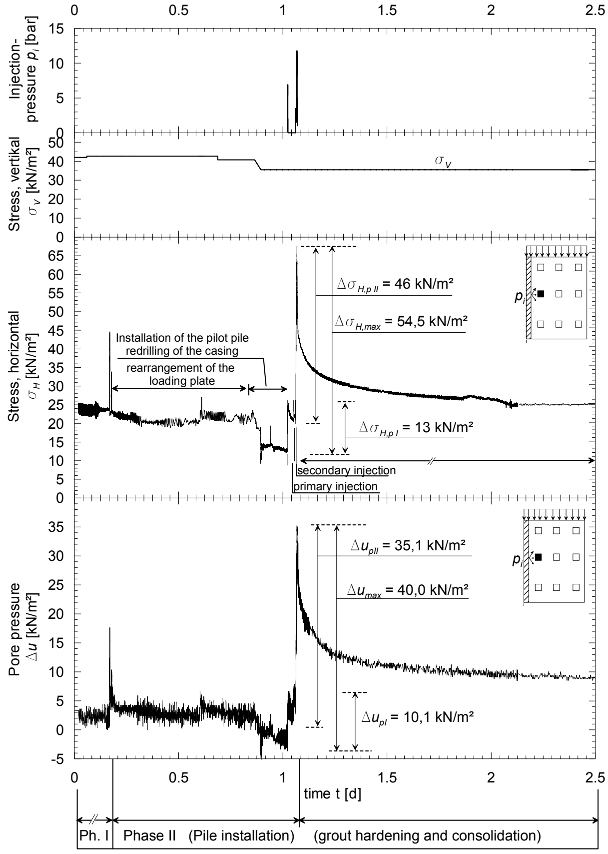


Figure 8. Example of test results: development of pore pressure and horizontal stress

Figure 9 shows results of further laboratory tests on specimens taken from the model. It appears from this figure that the shear strength and stiffness of the soil directly near the pile increase while the water content decreases.

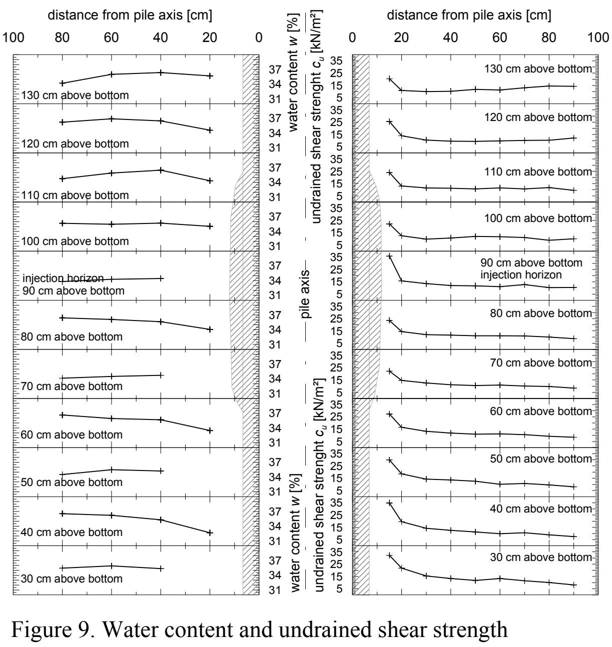


Figure 9. Water content and undrained shear strength

a) b)

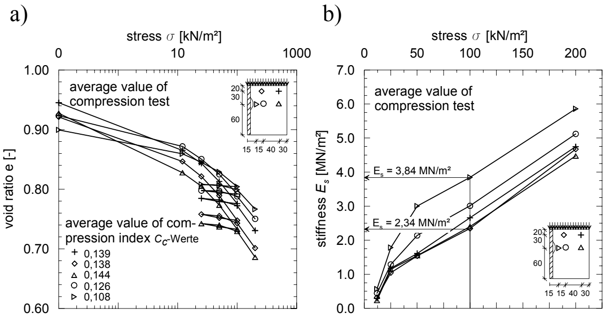


Figure 10. Stiffness of the model soil at different radial and vertical positions

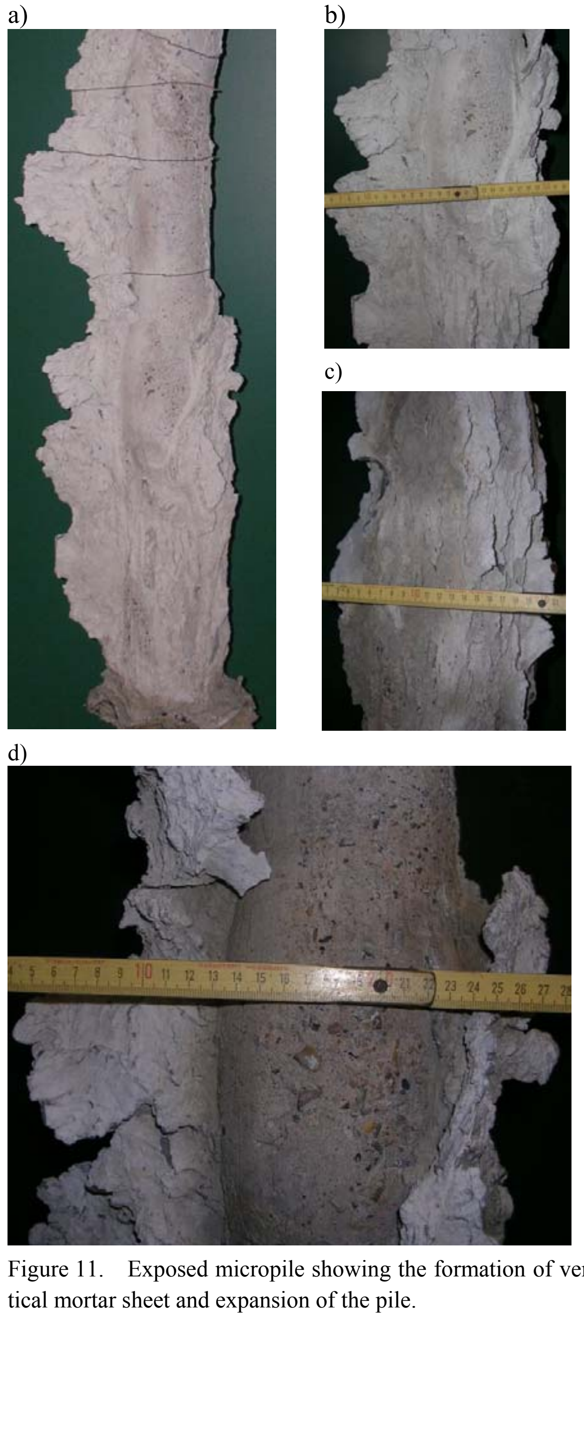


Figure 11. Exposed micropile showing the formation of vertical mortar sheet and expansion of the pile.

On the other hand, results by similar model test without injection show no significant change by water content and shear strength in radial direction.

The stiffness of the soil surrounding the micropile increased by $\Delta E_s = 1.5$ MN/m² at a stress level of 100 kN/m². The average value of the compression index C_c lies between 0.108 and 0.144 (Fig. 10).

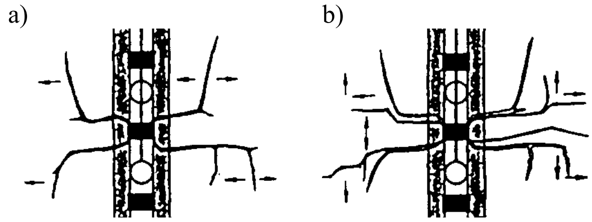
Figure 11 shows the exposed model pile after demounting. The formation of a vertical mortar sheet penetration the surrounding soft soil and expansion of the pile at injection horizon can be clearly seen from the figure.

6 THEORIES TO THE FORMATION OF CRACKS

Grouting of micropile can improve the bond between the pile and the surrounding soil. Measurements on micropiles showed that the pile load capacity can be increased, if additional injection horizons can be arranged.

Derived from injection techniques, the injection or grouting in soft soil can be classified into compacting grouting and fracture grouting.

In fracture grouting, the injection process includes two phases. In normally consolidated soft soils ($K_0 < 1$) where the vertical overburden stress σ_V is higher than the horizontal stress σ_H, cracks develop predominantly in vertical direction following small resistance (Fig. 12a). The soil then starts to displaced laterally and developed horizontal strain (*Raabe/Esters 1986)*.



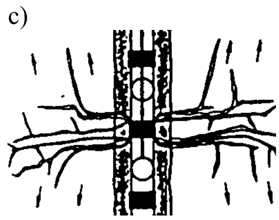


Figure 12. Developments of fracture after Raabe/Esters (1986): a) First injection (contact grouting); b) Secondary injection (compensation grouting); c) Continued injection

In subsequent injections, the soil developed more and more horizontal cracks (mortar sheets) and gradually attains a new state of stress with $K_0 > 1$ (Fig. 12b and c). Such phenomena had also been observed in the model test described in section 5 (Fig. 11). Hence, predominately vertical mortar sheets can be developed in normally consolidated soils in the first grouting phases, however, additional horizontal mortar sheets can also be developed during the subsequent grouting.

During injection of the micropile, the soil surrounding the pile develops a higher excess pore pressure as a result of injection pressure, which is also recorded in the model test (Fig. 8). Similar observations are also reported by Soga et al. (2004), Au et al. (2003), Au (2001). It was asserted that soil can be consolidated as a result of the injection pressure. The general tendency is that the normally consolidated soil ceases to settle after it undergone heave during the injection (Komiya et al. 2001). Based on model tests on normally consolidated kaolin, Soga et al. (2004) showed that a simultaneous injection at one time is more effective than incremental injections provided that the amount of grout remains the same (Fig. 13).

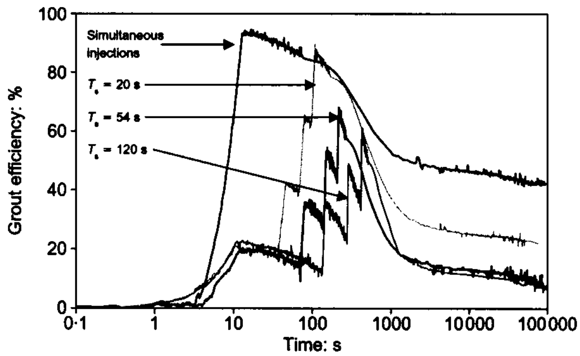


Figure 13. Grouting efficiency in normally consolidated soils (Soga et al. 2004)

In connection with grouted micropiles, the way how the injection pressure effect the surrounding soil is of importance. Several approaches to estimate the injection pressure P_f required for the development of fracture in soil can be found in the literature. Some of them are summarised in table 2.

Table 2. Approaches to estimate fracture pressure (contents of the table are partly adopted from Au 2001), terms adapted

Reference	Equation of fracture pressure	Failure mechanism/ Theory or approach
Hassani (1983)	$P_f = \sigma_H + \frac{\sigma_t}{2}\left(\frac{b^2-a^2}{b^2}\right)$	tensile failure/ elastic
Jaworski et al. (1981)	$P_f = m\sigma_H + \sigma_t$ with m = 1 to 2	tensile failure/ empirical
Lo/Kaniaru (1990)	$P_f = c \cdot \cos\varphi + (1+\sin\varphi)\ \sigma_t/2$ $+(1+\sin\varphi)\ \sigma_H$	shear failure/ empirical
Mori/Tamura (1987)	$P_f = \sigma_H + q_u$	shear failure/ empirical
Murdoch (1992)	$P_f = \frac{K_{Ic}}{\sqrt{\pi d}} + \sigma_n$	shear failure/ Mohr-Coulomb
Panah/ Yanagiswa (1989)	$P_f = \frac{(1+\sin\varphi)\sigma_H + c\cdot\cos\varphi\left(1-(a^2/b^2)\right)}{1+(a^2/b^2)\sin\varphi}$	shear failure / elastic
Soga et al. (2005), Andersen et al. (1994)	$P_f = 2\sigma_{Hi} - u_0 + \sigma_t'$ total stress $P_f = 2\sigma_{Hi}' + u_0 + \sigma_t'$ effective stress	tensile failure/ elastic
Soga et al. (2005)	$P_f = \sigma_H + nc_u$ total stress $P_f = \sigma_H' + u_0 + nc_u$ effective stress	shear failure/ elastic

In all cases, a distinction is made between shear failure and tension failure.

The pressure required to crack the concrete or grout of micropiles depends on the degree of hardening of the grout, the diameter of the pile and the stress surrounding the injected area of the pile. *Wawrzyniak (2002)* suggests an approach (Eq. 2) to calculate the necessary injection pressure to crack a hardened grout surrounding a tube. In his calculation, he also considers the possible loss of grout on the way to the injection points.

$$p_{eff} = p_i + p_s - p_\eta - p_{\tau_0} - p_w \qquad (2)$$

Where p_i is the injection pressure measured at injection pump, $p_s = \gamma_s \cdot h_s$ is the pressure due to head difference between pump and injection point, p_η is the loss of pressure resulting from change of viscosity, p_{τ_0} is the loss of pressure to overcome the fluidity limit of the suspension and p_w is the water pressure at location of the injection point.

7 FURTHER WORK

The model test of grouted micropiles described in section 5 have been analysed using finite element method. The main focus is the modelling of the injection horizon. It will include the numerical simulation of the load bearing behaviour of the SSVP-foundation system and development of an optimised design approach.

8 SUMMARY AND CONCLUSIONS

The new raft foundation with floating grouted micropiles (SSVP) in normally consolidated soft soils have been introduced on the basis of practical project. The SSVP-system is subjected to a low settlement compared to a pure raft foundation. The settlement improvement-factor β calculated from several practical projects lies between 3 and 10 and for the project presented in this paper is about 4.

The effect of injection and the load bearing behaviour of floating grouted micropiles in soft soils have been investigated using a model test at a scale of 1:1. It can be shown that the mortar sheets developed due to injection contribute much to the bearing behaviour of the tested micropile similar to compensation grouting in soft soils reported in the literature. It have also been proved that the injection have a soil improvement effect as a result of straining in the soil. An increase of the shear strength and stiffness and a decrease of water content had been observed in soil surrounding the pile.

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LIST OF SYMBOLS

a	inner radius of injection needle
b	outer radius of injection needle
c_u	undrained cohesion
d	half-width of initial fracture slot
K_{Ic}	Fracture toughness
m	slope of the linear function of fracture pressure with horizontal stress
P_f	fracture pressure
q_u	unconfined compression strength
u_0	excess pore pressure
β	settlement improvement-factor
ν	Poisson´s ratio
σ_H	total horizontal stress
σ_n	confining stress normal to the fracture
σ_t	tensile strength