PERFORMANCE OF MICROPILES SUBJECTED TO REPETITIVE LOAD AND UNLOAD CYCLES OF 1500KN LOAD

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ABSTRACT: Since early 1980s cast in place micropiles have been implemented in various part of Indonesia. Its main application is to provide support for machine foundation with limited headroom inside a factory, to strengthen a building foundation by underpinning, or to support a structure under a special site condition where the use of traditional driven piles or bored piles was not possible. Three types of cast-in-place micropiles have been implemented elsewhere, i.e. non grouted, globally grouted, and repetitive grouted micropiles. This paper describes the design, construction and performance of Repetitive Grouted Micropiles (RGMs) subjected to loading and unloading cycles of 1500 kN load. The RGMs, which served as a gantry crane foundation, were designed to bear 1000 kN working load each. To check the capacity and the load-unload behavior of the piles, 6 out of 195 used piles were subjected to 20 cycles of loading-unloading between zero and 1500 kN test-load. One unused pile was constructed and loaded to failure. It was found that the piles performed satisfactorily.

Keywords: Micropiles, Repetitive Grouted Micropiles, Repetitive Loading, Load and Unload Cycles.

INTRODUCTION

The term micropiles generally refers to the piles having a diameter smaller than 30 cm which can be installed by light weight equipment under a limited headroom or by heavy equipment in an open air project site. As in normal pile foundation practices, micropiles are classified into two broad categories i.e. cast-in-place and precast micropiles. By the end of 1980s both precast and cast-in-place micropiles have been implemented in various projects all over Indonesia. To the authors' knowledge, the cast in place micropiles were introduced by Bachy Soletanche Indonesia. Frankipile Indonesia pioneered the early generation of precast micropiles with their 280mm equilateral triangle shaped reinforced concrete piles. Many manufacturer and contractors then followed with the 200mm to 300mm width precast piles. They were all driven into the ground. By mid of 1990s, PT. Vipalindo introduced the push-in or jack-in precast piles installation techniques. Since then, these precast micropiles which commonly known as minipiles by the public, either driven or jack-in, have been gaining popularity. By year 2000s, practically all of light structures such as one to three stories residential buildings, factories, and shop houses were built on these minipiles.

Depending on how grouting is carried out in the construction of the cast-in-place micropiles, they can be classified into three categories, i.e. non grouted micropiles (NGM), globally grouted micropiles (GGM) and repetitively grouted micropiles (RGM).

NGM is generally made by drilling a 150mm borehole, then a one inch tremie pipe is inserted down to the bottom of the boreholes, from where cement grout is pumped to push out the muddy slurry inside the boreholes and to form the body of the pile. The grouting pipe is then pulled out. No pressure grout is applied in forming the pile. An 18 mm to 32 mm deformed bar is then pushed into the grouted hole as pile reinforcement.

For GGM and RGM a permanent grouting pipe is inserted together with the reinforcement once the body of the pile is formed by pumping cement grout through tremie pipe into the borehole. The permanent grouting pipe is generally made of 1.5–2.0 inches steel pipes or high strength PVC pipes; 4 holes of about 10 mm diameter are drilled at every 50 cm interval along the pipes, the holes are then covered by rubber sleeve at the outer perimeter of the pipe. This rubber sleeves act as valves that open when pressure grout is applied from inside the pipe to let the grouting material flow out into the ground and when the pressure is released the rubber sleeves shrink back to close the holes at the pipes. This type of grouting pipe is known as Tube a Manchette, commonly abbreviated as TAM, grouting system (Fig.1). The TAM rubber sleeves ensure only outwards flow of cement-grout is made possible. The advantage of using TAM is that it allows re-injection whenever necessary.

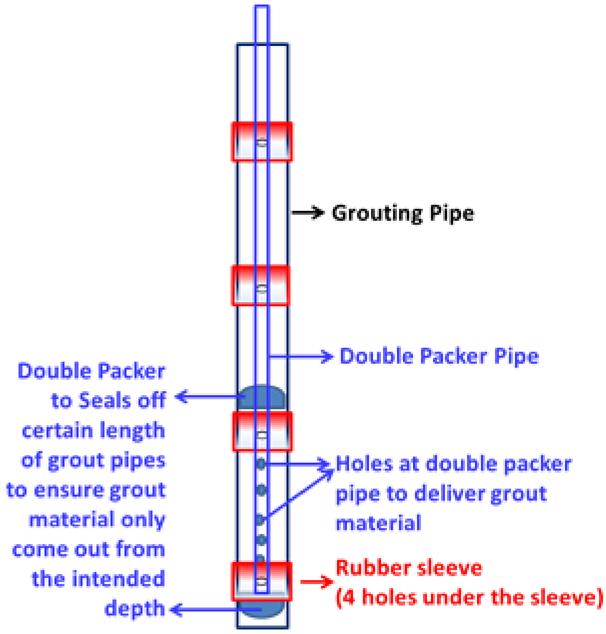


Fig.1 Tube a Manchette (TAM) Grouting System

Once the borehole of the GGM or RGM has been formed, a one inch tremie pipe is inserted up to the bottom of the hole, and high strength cement grout is pumped into the hole to push the drilling mud and form the body of the pile. This is generally known as primary grouting. The pile reinforcement and the TAM are then inserted into the hole. High pressure grouting is carried out 24 hours later to pump in cement-grout to increase the capacity of the piles; this second stage grouting is known as secondary grouting. In GGM the grouting is carried out in one stage, that is the top of the TAM is sealed and cement grout is pumped into the TAM with high pressure to crack the body of the piles where the TAM holes are located, then the grout material is pumped into the ground up to a certain pressure or up to a certain grout volume. In RGM the grouting is carried out in stages from bottom up or from top to bottom, each time for a certain length of the pile, usually at 1 m interval.

From the injection techniques involved it is clear that the NGM has the lowest bearing capacity, the GGM has mid-range bearing capacity and the RGM has the highest bearing capacity among the three cast-in-place micropiles. Generally, the allowable bearing capacities of a single NGM, GGM, and RGM are up to 150 kN, 400kN, and 1000 kN, respectively.

This paper elaborates the performance of 1000 kN working load RGM applied to a load test of 20 cycles of loading and unloading from zero load to a maximum of 1500 kN where the first author was involved in the design, execution, and load testing of the piles.

THE PROJECT

To accommodate the increasing production activities, a ship yard located in Surabaya, East Java, Indonesia, had to build a high capacity gantry crane. The gantry crane was designed to be moveable along the wharf. The existing wharf was constructed some 50-60 years ago on an open box type reinforced concrete caisson filled with loose silty fine sand. Analysis showed that the caisson would not be able to bear the 3000 kN load imposed by the operation of the gantry crane and pile foundation system was required to transmit the load down to the soil stratum below the caisson. The problem was the piles had to be installed through the caisson base slab of 40 cm thickness which laid 14 m below the ground surface (Fig.2). It was required that the installation of the piles should cause only limited damage to the caisson, otherwise the whole caisson had to be rehabilitated. Conventional driven pile or bored pile would certainly induce serious damages to the existing caisson. This situation led to the use of high capacity cast-in-place micropiles, i.e. the RGM. The advantages of the RGM over driven pile and bored pile were that no vibration was induced during construction and only small diameter, i.e. 200 mm, coring through the caisson slab was required. Aside from that, the part of the RGM above the caisson base slab could also be designed in such a way, as elaborated later, so that relatively no load was transferred to the soil inside the caisson and to the caisson slab. Thus, it would limit the interaction between the piles and the caisson which in turn would also limit the deformation of the bottom of the caisson.

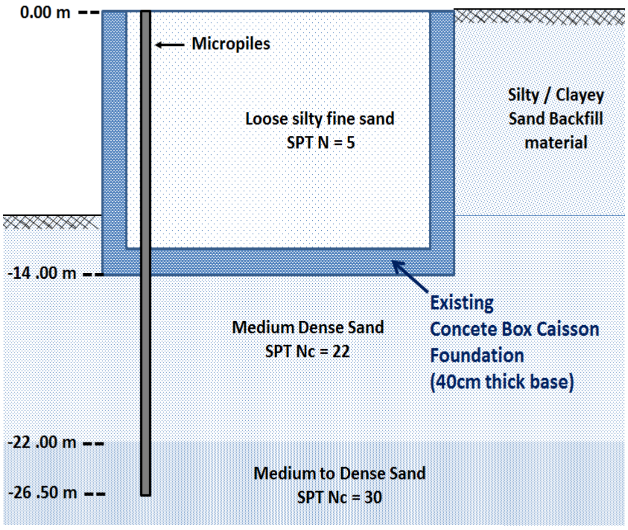


Fig. 2 Cross Section of the Site

SUBSOIL CONDITION

The soil inside the box caisson was loose silty fine sand with SPT N value of 5 blows/ft. Below the caisson, up to 26.5 m depth below the ground surface, the soil consist of medium to dense sand. From 14 to 22 m depth the corrected SPT blow count, N_c, was 22 blows/ft and below 22 m depth was 30 blows/ft. The corrected SPT was calculated by the TERZAGHI & PECK (1948) equation, i.e. N_c = 15+0.5*(N-15).

THE MICROPILE DESIGN

Keeping in mind that limited damage to the existing caisson was required, it was decided to transfer the load exerted by the gantry crane through a continuous reinforced concrete beam, 90 cm x 90 cm in size, to a row of micropiles which transferred the load to the soil below the caisson level. 195 micropiles, with 200 mm drilling diameter spaced at 2.45 m center to center, each designed to bear 1000 kN working load, were required. To resist the tension load induced by the gantry crane, 34 prestressed ground anchors (each subjected to 500 kN tension load) were installed at 22 degree inclination to the vertical. These ground anchors would not be elaborated further.

Due to its small diameter, i.e. 200mm, the end bearing capacity of a micropile generally contributes less than 5% of its ultimate capacity. Therefore, micropiles are often regarded as friction piles and the end bearing is neglected. The ultimate friction capacity of a single micropile in granular soil, Q_{ult}, is calculated as follows:

$$Q_{ult} = \tau_{ult} * A_s \quad \text{..........................(1)}$$

$$\tau_{ult} = k * \sigma_v' * \tan \phi' \quad \text{..........................(2)}$$

$$k = k_i * k_d * k_o \quad \text{..........................(3)}$$

where A_s is the area of the pile shaft, σ_v' is the effective overburden pressure, ϕ' is the effective angle of internal friction, k_i is the grouting or injection factor, k_d is the drilling factor and k_o is the coefficient of earth pressure at rest. It can be seen that the formulas are very similar to the formulas for conventional driven pile or bored pile. The only difference is on in the k factors as shown in equation 3. The grouting factor, k_i, varies from 1 to 7 depending on the type of the granular soil and the grouting technique. The drilling factor varies from 0.5 to 0.9 depending on the skill of the driller. At this site, k_i = 7and k_d = 0.9 were adopted, for a 200 mm drilling diameter the RGM ultimate capacity was in the order of 2000 kN. This means a safety factor of 2 was obtained against the design load of 1000 kN.

To ensure minimal or no load was transferred to the loose sand inside the caisson, the section of the RGM inside the caisson was encased with a PVC pipe and no pressure grouting was carried out along this section. This part of the RGM, named as free length, was 12.05 m length. Below the caisson level, staged and repetitive grouting was carried out up to 26.5 m depth to form a load bearing element or the bond length of 12.5 m length. The total length of the pile was 24.55 m as shown in Fig. 3a. For a factor of safety of two, structural design of the pile required the use of 6 numbers of 32 mm diameter deformed bar with 400 MPa yield stress. The bars were tied together around the perimeter of a 2 inches diameter PVC TAM pipe. The pile was connected to the upper girder

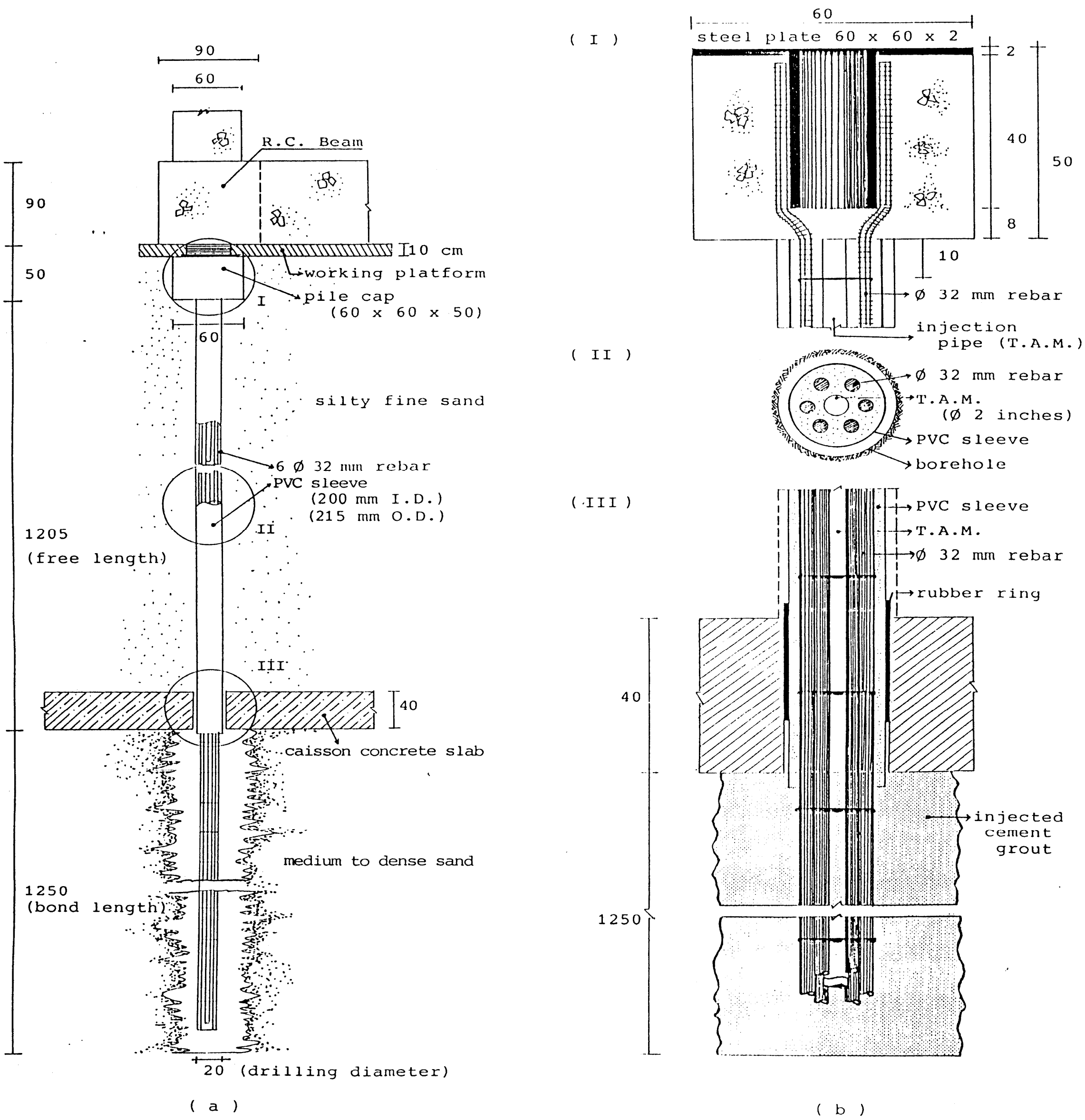


Fig. 3 Detail Construction of the Repetitive Grouted Micropiles (RGM)
(unless specifically mention, all dimension are in cm)

through a pile cap in a hinge type connection. The details of the pile cap, the body of the RGM and the connection to the bottom of the caisson are shown in Fig. 3b.

THE CONSTRUCTION METHOD

The constructions of the RGMs were carried out in the following sequence (see Fig. 4) :

1. Drilling through the caisson backfill material down to the top of the caisson base slab. A temporary steel casing of an inside diameter of 250 mm was used to prevent the borehole from collapsing. To make sure that the hole was clean enough for coring out the caisson slab, airlift technique was used to clean out the soil debris inside the borehole.
2. Coring out the caisson reinforced concrete slab by using a special Tungsten Carbide bit with a diameter of 228.6 mm.

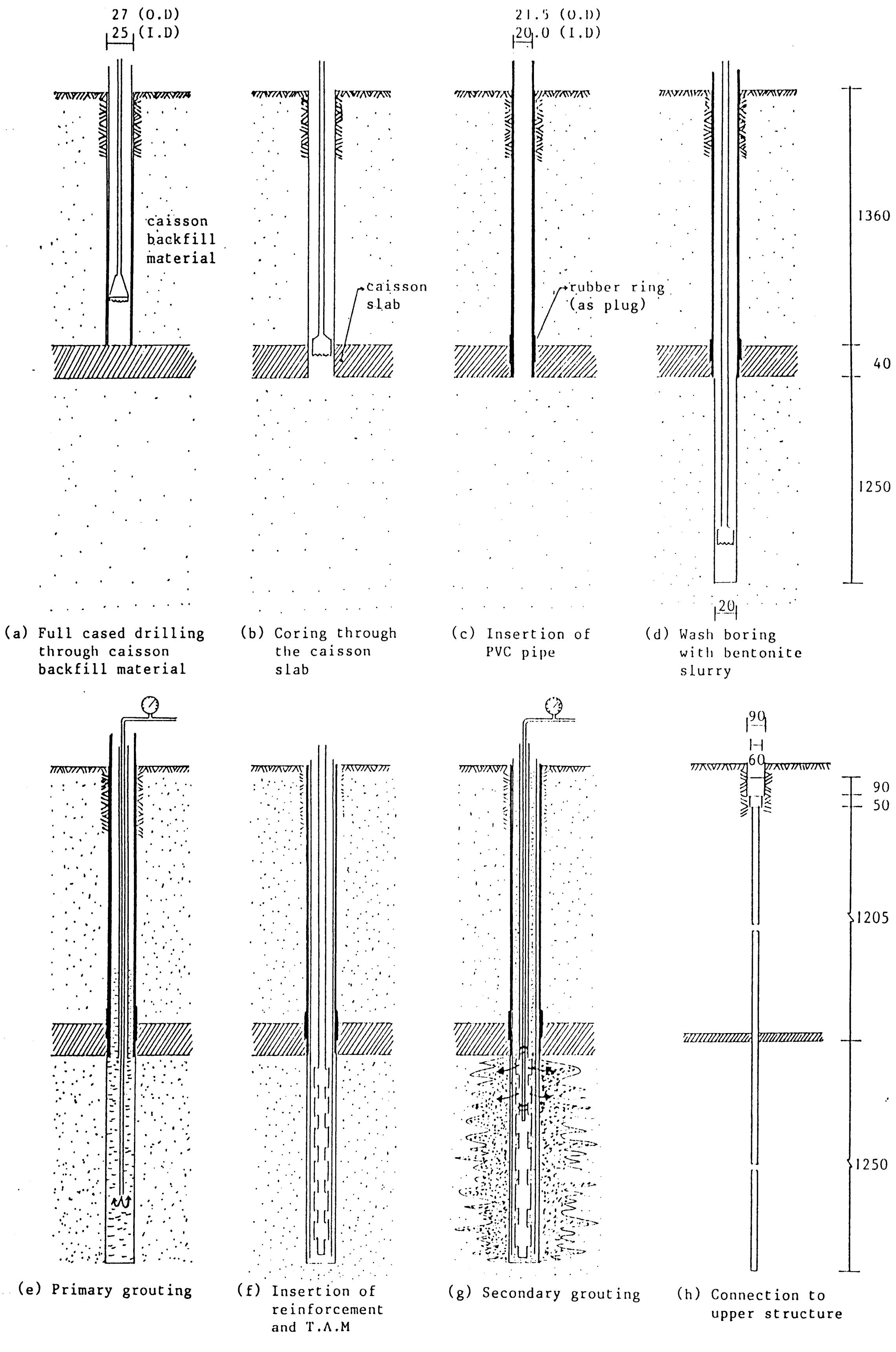


Fig. 4 Construction Method (Not for Scaled – All Measurements are in cm)

3. Insertion of the PVC pipe (200 mm I.D. and 215 mm O.D.), followed by extraction of the temporary steel casing. The PVC was inserted up to the bottom of the caisson slab. To close the annular space between the PVC and the caisson slab, the bottom tip of the PVC was wrapped around by a rubber ring. This was intended to prevent the extrusion of sand from inside the caisson. To ensure the part of the pile inside the caisson was friction free, the PVC was also coated with bitumen.
4. Drilling through the sand beneath the caisson with 200 mm tricone roller-bits up to the designed level. Bentonite drilling mud was used to stabilize the borehole.
5. Forming of the body of the RGM. This was done by pumping high strength cement-grout through a small tremie pipe. When clean cement-grout had come up to the mouth of the borehole, the tremie pipe was extracted. This process was named as primary grouting. The cement-grout which formed the body of the pile was named as sleeve grout. The sleeve grout was intended to prevent leakage during the process of injection/grouting.
6. Insertion of the reinforcement cages and the grouting pipe (TAM). This was done right after the primary grouting.
7. When the sleeve grout had hardened, pressure grouting (injection) of up to 20 bars was carried out in stages through the TAM, each at one meter interval until the whole bond length was formed. Mechanical double packers were used for carrying out the multi-phase injection. This process was named as secondary grouting. The cement-grout had a minimum compressive strength of 22 MPa after 28 days curing. Cement type 5 was used for both primary and secondary grouting.
8. Upon completion of the secondary grouting, the TAM pipe was cleaned out by flushing it with fresh water. The intention was to allow re-injection if the pile failed to carry the design load during proof tests.
9. Casting the pile cap. Connection to the upper structure was allowed only after all the loading tests on the used piles were approved by the Engineer, otherwise re-injection was required (Once the pile loading tests were successful, all the TAM pipes were grouted, and connection to the upper structure was begun).

THE LOADING TEST

As in normal practice, loading tests on the working piles, known as proof tests, were carried out. Six piles, selected arbitrarily by the engineer, were tested up to 1500 kN with 20 loading and unloading cycles. The load testing was done in the following procedure:

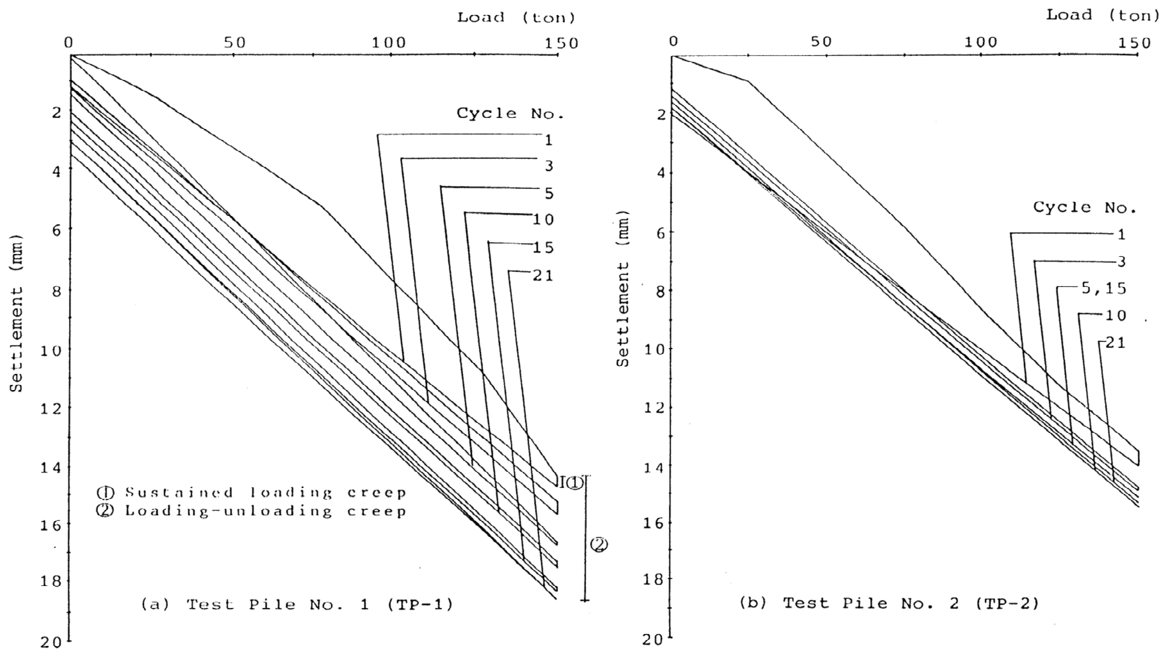


Fig. 5 Load Settlement Curve of the Proof Test Piles (1 ton ≈ 10 kN)

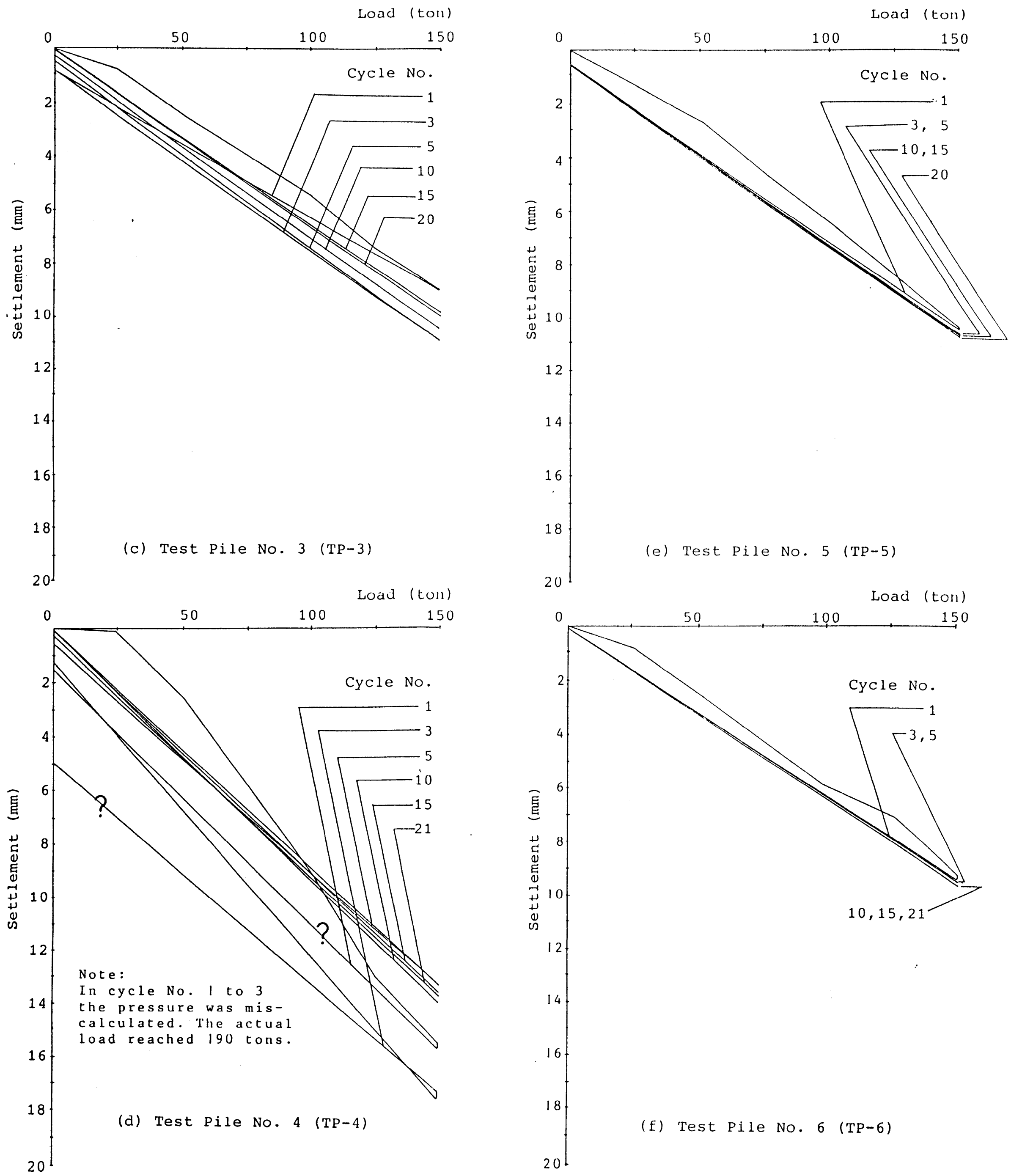


Fig. 5 (con't) Load Settlement Curve of the Proof Test Piles (1 ton ≈ 10 kN)

a. The test load was applied by a hydraulic jack reacting against a kentledge platform of 6 m x 6 m. The load was applied in six equal increments, i.e. 250 kN increment at each stage. Each increment was kept constant for 30 minutes and the full test load was maintained for two hours. Then the pile was unloaded for 30 minutes. At each step, the load applied was measured and the settlements of the pile were recorded through four dial gauges with 0.01 mm accuracy. The dial gauges were placed at each corner of the pile and measured against two parallel reference beam. With this arrangement tilting of the pile cap, which indicated buckling, could be observed. Readings were taken every three minutes.

b. Next, the pile was subjected to 20 cycles of loading-unloading between full (1500kN) and zero test load. The full test load was maintained for two hours and the zero load was maintained for 30 minutes. Upon completion of the final cycle, the load was reduced to zero and the residual settlement was measured. Settlement readings were taken every 15 minutes.

The loading-unloading cycles were intended to see whether the pile kept on or stopped creeping with increasing number of loading-unloading cycles. This was an important criteria in determining whether the RGMs could really serve as a gantry crane foundation where loading and unloading was an imminent characteristic of the structure. In addition to the six proof tests, one unused pile constructed outside the working area was tested until failure occurred at 2000 kN load. Figures 5 and 6 show the results of the proof tests (TP-1 to TP-6) and the destructive test (TP-F), respectively. Tilting of the pile caps at their maximum test load is shown in Table 1.

Table 1 Tilting of Pile Cap at Maximum Test Load (Size of Pile Cap 60 cm x 60 cm)

Test Pile No.	Max. Reading (mm)	Min. Reading (mm)	Tilting of Pile Cap (mm)	(%)
TP-1	22.29	15.28	7.01	1.2
TP-2	18.86	11.61	7.25	1.2
TP-3	10.90	9.64	1.26	0.2
TP-4	20.85	16.82	4.03	0.7
TP-5	10.99	9.79	1.20	0.2
TP-6	9.97	8.74	1.18	0.2
TP-F	25.55	25.00	0.55	0.1

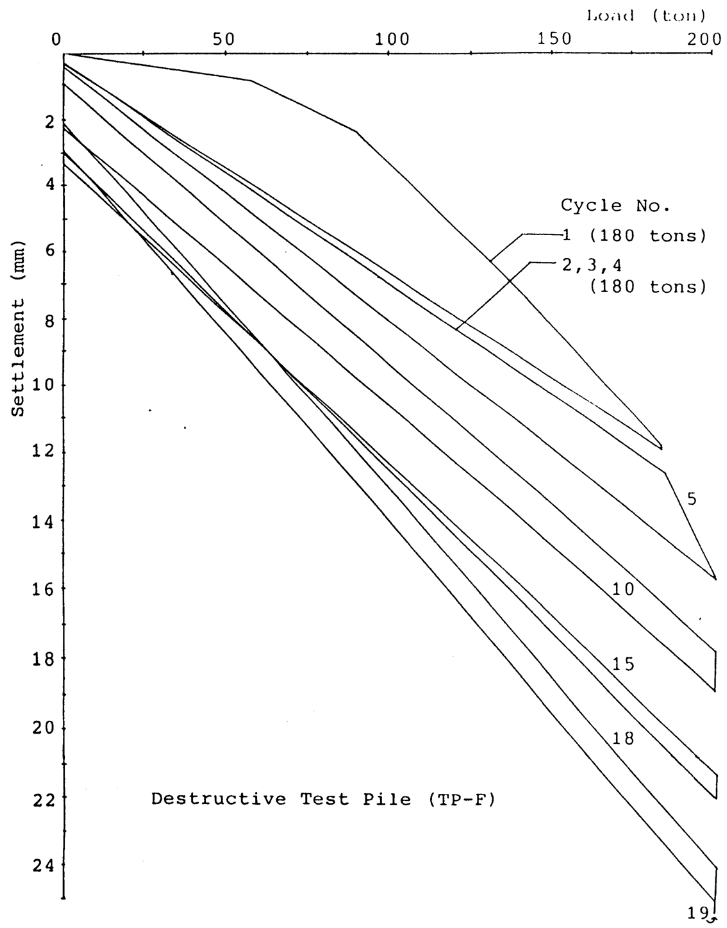


Fig. 6 Load Settlement Curve of Destructive Pile Test (1 ton ≈ 10 kN)

ANALYSIS and DISCUSSION

The RGM was considered to have failed when the loading test gave either one of the following results :

- Total settlement > 20 mm
- Residual settlement > 10 mm
- Creep induced by sustained loading > 5 mm
- Creep induced by loading-unloading > 10 mm
- Tilting of the pile cap > 2.5 %

Figure 5 and Table 1 shows that none of the criteria was exceeded by the proof test piles. On the contrary, Figure 6 shows that the destructive test pile exceeded the total settlement and the creep criteria. The load settlement curves in Figs. 5 and 6 shows that three settlement phenomena took place, i.e. elastic settlement, residual settlement and creep. The magnitudes of the settlements are tabulated in Table 2 below,

Table 2 Settlement of the Test Piles

Test Pile No.	Total Settl. (mm)	Elastic Settl. (mm)	Residual Settl. (mm)	Creep	
				a* (mm)	b*(mm)
TP-1	18.75	15.50	3.25	0.40	4.30
TP-2	15.60	13.50	2.10	0.55	2.10
TP-3	11.10	10.25	0.85	0.05	2.00
TP-4#	17.64?	16.36?	1.28	0.40?	2.20?
TP-5	10.66	10.24	0.42	0.30	0.40
TP-6	9.66	9.54	0.12	0.10	0.40
TP-F	25.24	22.70	2.54	1.30	13.00

a* sustained loading creep (maximum record)
b* loading-unloading creep (maximum record)
\# doubtful results (see Fig. 5d). The hydraulic pressure required to reach the test load was miscalculated. The first three cycles had actually reached 1900 kN before the engineer came and made correction.

Table 2 shows a clear trend that the settlements were getting smaller from test pile No.1 to No. 6 (TP-1 to TP-6). Data from test pile No.4 was not representative due to operator's error in calculating the hydraulic pressure of the load applying jack. One note needs to be mentioned is, while the quantity of the cement-grout injected was approximately the same for all piles, the piles constructed later performed better. This was due to the fact the workers skill and familiarity with the local soil condition were getting better, and hence the borehole quality and the grouting techniques were greatly improved. Slower grouting rate was found more beneficial.

Total Settlement

Figures 5 and 6 shows that the total settlements of all the test piles during the first cycle of loading were smaller than 20 mm. At 1500 kN (150 metric ton) load, even after the last cycle, the worst total settlement was less than 10% of the drilling diameter of the pile i.e. less than 20 mm. The largest settlement, 18.75 mm, happened at the first test pile (TP-1) which was also the first constructed pile. The 20 mm criteria was exceeded only after more than ten cycles of loading-unloading with 2000 kN test load were applied during the destructive test. The result of the destructive test pile showed that the failure was due to creep.

Elastic settlement

Among the three type settlements, it is clearly seen that the elastic settlement was the major component. The fact that none of the test piles was instrumented made it difficult to interpret the distribution of the elastic settlement between the free length and the bond length of the piles. However, since the free length had relatively no resistance, it was logical that this part of the pile contributed most to the elastic settlement. Assuming the 12.05m free length as a column, for a 1500 kN load Hooke's law gave an elastic settlement of 15.9 mm which was greater than the data shown in Table 2. The actual elastic settlement varied from 9.54 mm to 15.50 mm with an average of 12.05 mm. Analyzing these numbers, it may be appropriate to assume that 2/3 of the elastic settlement comes from the free length. Based on this, the following equation is proposed,

$$\Delta l_e = \frac{P(0.50L_f + 0.25L_b)}{E_p\, A_p} \qquad \text{(4)}$$

Where Δl_e is the elastic settlement of the pile, P is the load, E_p is the Young's Modulus of pile, A_p is the area of pile, L_f is the free length and L_b is the bond length. The above equation gives an error of ± 25 %.

It should be noted here that relaxation (rebound at zero load) also occurred but the influence was very small. Data shows that the relaxation observed in the last 30 minutes interval was in the order of 0.01 to 0.26 mm. These were small numbers and only in the order of 0.6% elastic settlement.

Residual Settlement

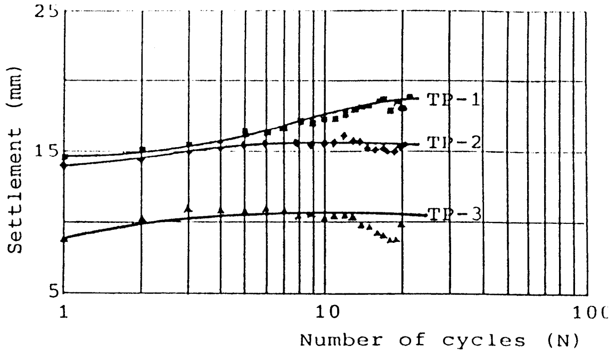
The loading tests revealed that the residual settlements of the RGMs were very small compared to the total settlement. The maximum residual settlement was 3.25 mm which was much smaller than the given failure criteria of 10mm. Even under 2000 kN load (TP-F) the residual settlement was only 2.54 mm. It is clear that the residual settlement was not a determining factor.

Creep

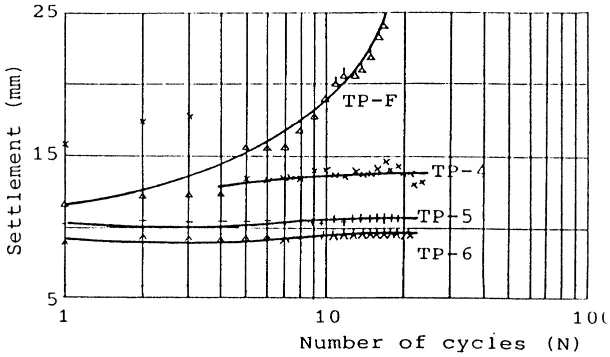
Two creep phenomena occurred in the loading tests, i.e. creep due to sustained loading and creep due to loading and unloading cycles. Table 2 showed that none of the used piles failed by either sustained loading creep or loading-unloading creep. On the other hand, the destructive test pile (TP-F) failed due to loading-unloading creep at 2000 kN load.

The sustained loading creep of the proof test piles generally reached the maximum value during the initial loading cycles and then decreased toward zero with increasing number of loading cycles. This phenomenon can be observed in Fig. 5. On the contrary, as can be seen in Fig. 6, the maximum creep of the destructive test pile was reached in the final cycle.

The failure caused by loading-unloading creep can be clearly seen by plotting the settlement of the pile vs. the logarithm of the number of cycles as presented in Fig. 7. The graphs of the proof test piles, TP-1 to TP-6, were asymptotic to the horizontal axes (log cycles). This meant the settlement was more or less remained constant after a certain number of cycles. The graph of destructive test piles, TP-F, on the other hand, was asymptotic to the vertical axes (settlement axes). This meant after a certain number of cycles the rate of settlement was very large, in other word, the pile failed.



(a) TP-1, TP-2 and TP-3



(b) TP-4, TP-5, TP-6 and TP-F

Fig. 7 Settlement vs. No. of Cycles

Buckling

From the tilting of the pile cap, presented in Table 1, it was concluded that none of the test piles (including the destructive test) was buckled. Calculation by Davisson method (POULOS & DAVIS, 1980) gives the critical buckling load of about 4000 kN.

All the above analysis shows that the RGMs have an ultimate capacity of about 2000 kN. Therefore, it can bear the 1000 kN working load safely. It was finally proven by the fact that no significant settlement was observed when the construction was put into service.

CONCLUDING REMARKS

The loading tests results show that the settlement characteristics of the repetitively grouted micropiles (RGM) differ from the conventional bored pile. In the RGM, the elastic settlement was much higher than the residual settlement. The slenderness and the free length of the pile are responsible for the large elastic settlement. The small residual settlement comes from the fact that the soil characteristics are improved by the injected cement-grout. It is also concluded that buckling is not a critical factor and the RGM can act as a foundation subjected to repeated loading. However, a full scale research on instrumented piles is necessary to develop a better design method.

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