Pile damage due to soil heave

Dommage aux pieux à cause de soulèvement de sol

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SYNOPSIS This paper outlines the behaviour at two sites of displacement piles driven into a two-layered soil profile consisting of a top layer of soft marine clay overlying stiff residual soils. At the first site, because the piles were structurally sound and were embedded in the stiffer residual soils, pile heave was small. Pile bearing capacities were not affected by heave. At the second site, due to structurally inadequate joints, soil heave separated each pile into two lengths with a vertical gap of more than 75mm between them. This bifurcation could be identified from the load-settlement plot of the load test results. Redriving of the damaged piles with the object of closing the gap caused by soil heave was shown to be unreliable and not completely effective.

When a displacement pile is driven into a saturated cohesive soil, the soil displaced during the driving process is partly heaved up at the ground surface and partly displaced laterally from around the pile shaft. At a certain depth below the ground surface due to the overburden pressure, vertical heave ceases to prevail and only lateral displacements take place. It has been reported (BROMS, 1981) that measurements have indicated that heave within a pile group corresponds to 30 to 60% of the total volume of the piles. About half of the total volume appears as heave outside the pile group. Consequently the driving of a pile would cause adjacent piles that have already been installed to rise.

MACALLUM STREET GHAUT DEVELOPMENT

This site was identified for high rise development which included a number of blocks of 24 floors. In this site the residual soils consisting of stiff fine to coarse sandy clay formed by the insitu weathering of granite were overlain by a layer of marine clay which varied from 10 to 13 metres in thickness. For a number of years municipal refuse had been dumped on this site and covered over periodically with thin layers of imported soil. Since the thick layer of soft alluvium would demand the use of large displacement piles in order to prevent buckling, the effects of soil and pile heave were included in the engineering feasibility study.

Three test piles of various lengths were driven at each of three test pits. In each test pit the three reinforced concrete piles of cross-section 406mm by 406mm were driven with their centres forming the vertices of an equilateral triangle of side 1.37 metres. The spacing of the piles was therefore less than the pile perimeter (1.624 metres) which was the minimum spacing for friction piles as required under CP 2004:1975. The purpose of driving these test piles at a closer spacing than normal was to generate a more severe condition for heave.

Test Pit No. 1

In the case of Test Pit No. 1, Pile A1 with a penetration of 24.4m was the first pile driven. Pile B1 and then Pile C1 were driven to a penetration of 25.0m and 24.4m respectively.

On the completion of the driving of Piles B1 and C1, Pile A1 was found to rise 3mm. After all the three piles were load tested, they registered significant settlements and

TABLE I

Test Pit No. 1

REDUCED LEVEL OF PILE TOP (METRES)			DATE	TIME (HRS)	REMARKS
A1	B1	C1			
4.371			17.4.76	14.45	On completion of driving Pile A1
4.374	4.361	4.361	23.4.76	14.15	On completion of driving Piles B1&C1
4.358	4.331	4.346	7.5.76	11.50	
4.361	4.325	4.343	18.6.76	08.15	

Test Pit No. 2

REDUCED LEVEL OF PILE TOP (METRES)			DATE	TIME (HRS)	REMARKS
A2	B2	C2			
		4.885	6.4.76	15.05	On completion of driving of Pile C2
4.846		4.885	7.4.76	21.35	On completion of driving of Pile A2
4.849		4.888	7.4.76	16.00	On completion of driving of Pile B2
	4.885		2.5.76	12.25	
4.843	4.882	4.885	24.5.76	15.45	On completion of load testing of Piles A2, B2 & C2
4.846	4.879	4.885	19.6.76	20.20	

Test Pit No. 3

REDUCED LEVEL OF PILE TOP (METRES)			DATE	TIME (HRS)	REMARKS
A3	B3	C3			
		4.173	31.3.76	14.30	On completion of driving of Pile C3
		4.179	1.4.76	17.55	On completion of driving of Piles B3 and A3
4.118		4.182	2.4.76	16.00	
4.112		4.176	5.4.76	16.00	
4.115	4.179	4.179	7.4.76	16.10	
4.109	4.060	4.170	7.5.76	11.55	On completion of load testing of Piles A3, B3 & C3
4.106	4.054	4.166	18.6.76	08.15	

Piles B1 and C1 continued to settle even after the load testing was completed and the kentledge removed. (TABLE I)

Test Pit No. 2

Pile C2 was the first pile driven in this test pit, followed by Pile A2 and then Pile B2. Their respective depths of penetration were 35.9m, 24.1m and 30.6m. Immediately after the other two piles were driven, Pile C2 which had the longest length of embedment of which about 22 metres were in the stiff residual soil, was found to have heaved 3mm. After the load testing of all the 3 piles in this group, this pile was found to have settled 3mm.

Test Pit No. 3

In this test pit, the driving of Pile C3 to an embedded length of 35.6m was completed on 31st March 1976 followed by Pile A3 and Pile B3 on 1st April to depths of penetration of 24.1m and 38.7m respectively. Immediately after the last two piles were driven, Pile C3 was found to have heaved 6mm and by a further 3mm after a lapse of about 22 hours. Three days later it settled 6mm, then rose 3mm after two days and settled again 9mm after the completion of the load testing of all the 3 piles in the group. It continued to settle after that and registered a reduced level of 4.166m on 18th June 1976 which was 16mm below the observed peak level.

The observations on the nine test piles would indicate that the driving of displacement piles had caused adjacent piles already installed to heave. The magnitudes of heave were, however, small mainly because of the long embedment and anchorage of the piles in the stiffer residual sandy clays which were underneath the marine clay. The high strains resulting from the large radial displacements led to considerable re-orientation of the clay particles and produced high excess pore pressures. The resultant high hydrostatic gradients triggered off a process of consolidation at a significantly high rate resulting (a) in the rapid recovery of the loss in pile capacities of 212.3, 221.7 and 206.2 tonnes respectively as computed from the stability plots (CHIN, 1978). The adhesion which was observed to occur when a short pause in driving resulted in a considerable increase in driving resistance. This increase, however was observed to be transient because the driving resistance rapidly decreased to approximately the original values soon after pile driving was resumed, and (b) in the pile settlements which were observed to occur immediately after the peak heave had taken place.

Despite the heave and subsequent settlements, the piles were observed to have high values of ultimate shaft friction and end bearing capacities when they were load tested about a month after they were installed. For example, in the case of Pile C3 which registered the greatest heave, the three consecutive cycles of Constant Rate of Penetration tests yielded estimated ultimate shaft frictional estimated ultimate end bearing capacities viz 216.9. 254.5 and 362.0 tonnes respectively, showed significant increases with each subsequent CRP test (Fig. 1 and 2).

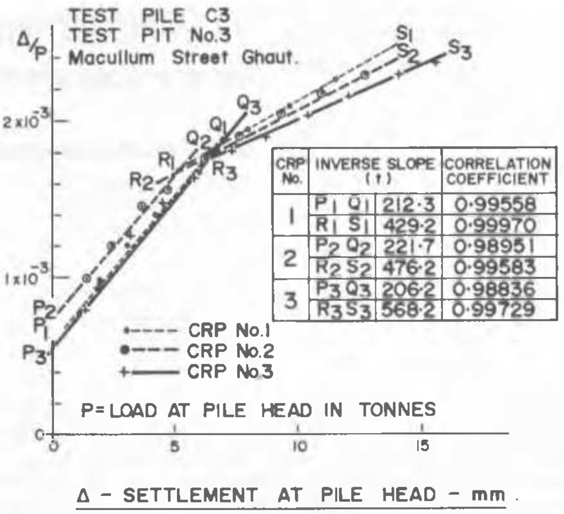

FIGURE 2: Test Pile C3: Stability plots for 3 CRP tests.

BAYAN BAHRU DEVELOPMENT

Reinforced concrete piles of similar size as those used in the Macallum Street Ghaut Development were used at Bayan Bahru. At this site, the soil formations also consisted of a thick layer of marine clay (12 to 15.5m thick) overlying stiff residual sandy clays formed by the insitu

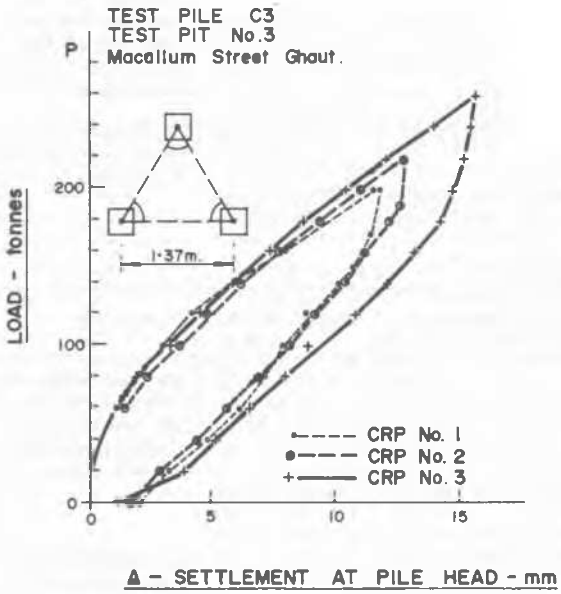

FIGURE 1: Test Pile C3: Load-settlement relationship for 3 CRP tests.

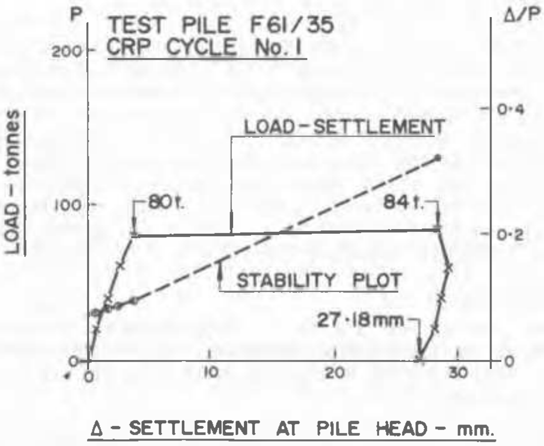

FIGURE 3: Test Pile F61/35: Load-settlement relationship for CRP test No. 1

weathering of granite. On the top of the marine clay was a layer of loose sandy clay of about a metre thick. When the engineers of this project load tested some of the piles, unusually large settlements and permanent sets were registered. Some of the test piles with embedded lengths of more than 30m had plunging failures at test loads of as low as 80 tonnes. The behaviour of test pile F61/35 was a typical example.

Test Pile F61/35

This pile had been driven to an embedment of 31.36m. When the first length of 12.18m was pitched and after penetrating the surface layer of about a metre in depth on two blows with a K32 hammer, this length of pile penetrated a further 4m into the marine clay on its own weight.

At the first CRP test, the settlement registered at the pile head increased rapidly after a test load of 80 tonnes and the pile continued to settle without any appreciable increase in load. A total settlement of 28.91mm was registered when the test load reached 84 tonnes. On releasing the entire test load, there was a permanent set of 27.18mm. The five points representing the observed test results (Fig. 3) defined a reasonably straight line with a product moment correlation factor of 0.99743 for the stability plot of Δ/p against an abscissa of Δ, where Δ was the settlement corresponding to a load of p at the pile head. The inverse slope gave a value of 89.3 tonnes

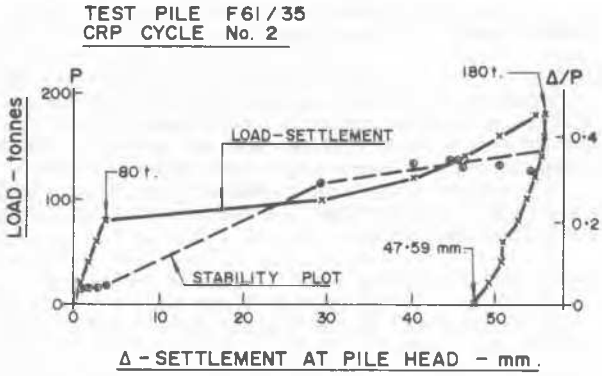

FIGURE 4: Test Pile F61/35: Load-settlement relationship for CRP test No. 2

as the ultimate shaft friction. There was no indication of end bearing capacity. This was too low an ultimate shaft friction for a pile with a penetration of 31.36 metres into the soil formation concerned. In the case of test Pile C3 in the Macallum Site with an embedment of 35.06m the ultimate shaft friction was more than 200 tonnes.

At the second CRP test, the jacking was continued after the "plunging failure". After a total settlement of about 30mm significant increase in load was observed. The testing was continued until the test load reached 180 tonnes (Fig. 4). On release of the load to zero, there was a permanent set of 47.59mm.

At the third cycle of CRP test, the plunging settlement with little apparent increase in load observed in the first and second CRP tests did not occur (Fig. 5). At a test load of 180 tonnes, the settlement at the pile head was 12.20mm as compared to 54.99mm at this load in the second CRP test. On release of the entire test load, the permanent set was also reduced considerably to 3.416mm.

At the fourth cycle of CRP test, the plunging settlement was also absent (Fig. 6). When the test load reached the value of 180 tonnes, the settlement at the pile head was 11.93mm and the permanent set on the release of the entire test load was 2.45mm.

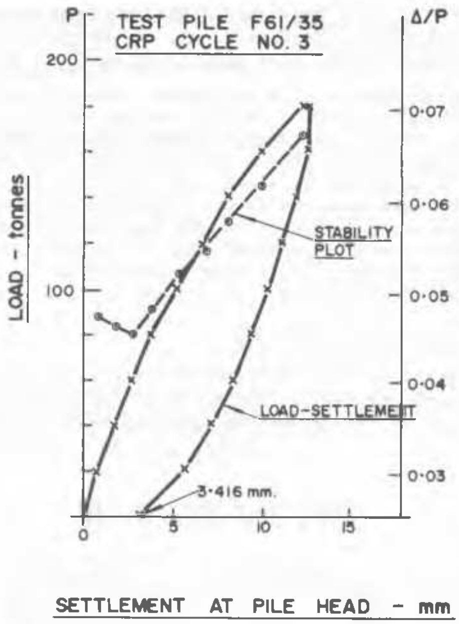

FIGURE 5: Test Pile F61/35: Load-settlement relationship for CRP test No. 3

From the above observations, it was clear that before this pile was load tested, it had consisted of two separate lengths with a vertical gap between them. As a result of a structurally inadequate joint, the heaving of the soil had lifted up the top length of this pile from its bottom length by about 27.18 + 47.59 = 74.77mm. It was only after the continued jacking in the CRP tests No. 1 and 2 that this vertical gap was substantially reduced. This pile had two joints, one at 7.0m and the other at 19.18m below the ground surface. As a 7.0m embedment in the marine clay and surface layer would not provide an ultimate shaft frictional capacity of 89.3 tonnes, it was the lower joint that had failed.

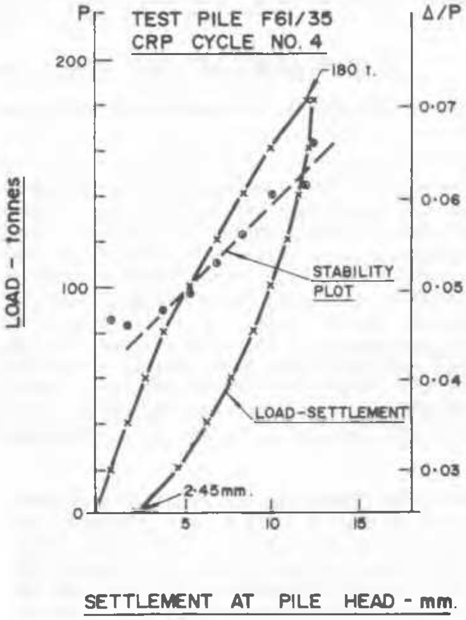

FIGURE 6: Test Pile F61/35: Load-settlement relationship for CRP test No. 4

Six other piles were load tested and as all the results revealed a similar bifurcation, all the seven hundred piles that had already been driven were suspect.

As a remedial measure, the contractor redrove some of the piles with the purpose of closing the gap which separated each pile. Using a K25 diesel hammer, the redriving of each pile was stopped when a set less than 12.5mm for the last 10 blows was attained. Of the first lot of 154 piles which were thus redriven, 31 penetrated less than 50mm, 32 penetrated between 50 and 75mm and 91 had a penetration of more than 75mm. One of these redriven piles, F62/168, which registered a penetration of 4.54mm at the last 10 blows, was load tested.

Test On Redriven Pile F62/168

Figure 7 gives the load-settlement plots of the first two cycles of the CRP tests. There was an improvement in that a load of about 160 tonnes was attained before plunging

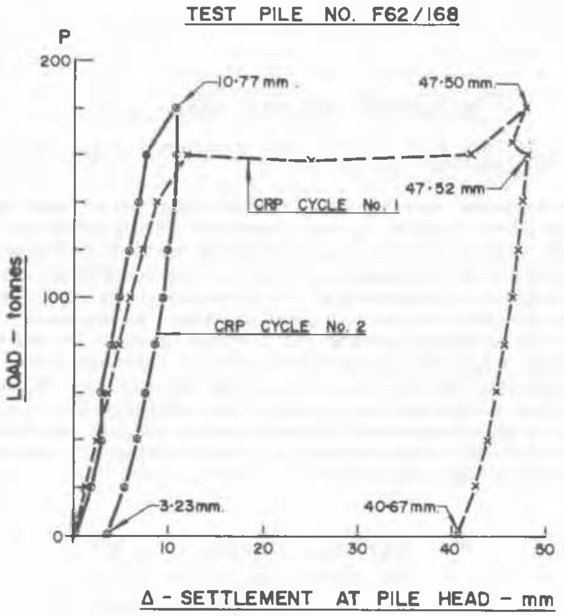

FIGURE 7: Load-settlement relationship of redriven pile.

failure commenced. The plunging failure continued over a settlement of about 30mm before there was further increase of load to 180 tonnes. The residual settlement on releasing the entire load was 40.67mm which was rather large and unacceptable. As in the tests of the piles before redriving, there was no plunging failure at the second cycle of the CRP test. At a test load of 180 tonnes, the settlement at the pile head was 10.77mm and the residual settlement was also considerably reduced to 3.23mm. On the third cycle of the CRP test, the settlement at the pile head was further reduced to 10.05mm at a test load of 180 tonnes and the residual settlement was 2.57mm.

The redriving was therefore not completely effective as a load capacity of only 160 tonnes was generated and this was 20 tonnes below the specified test load. Plunging failure still prevailed and there was no means by which any relative lateral displacements between the two ends of the bifurcation could be determined. It was also not possible to assess the extent of the structural damage sustained at these two ends of the bifurcation.

A comparison of the behaviour of the piles in the two sites would point to the importance of providing not only pile shafts but also pile joints which are structurally adequate to cater for the tensile forces generated by soil heave in the case of piles driven into stiff residual soils underlying marine clay.

Splicing of piles by welding together the two abutting steel plates using a fillet weld was shown to be inadequate in the Bayan Bahru site. Reflection waves and heave forces impose tensile stresses on the fillet weld. It is bad design practice to subject fillet welds to tensile stresses. Fillet welds should only be used in shear (CHIN, 1982). The hard driving which develops when the pile begins to penetrate into the stiffer residual soils tends to deform the steel plate at the pile head. Consequently it will not bear over its entire surface area against the plane surface of the abutting plate on the top length of pile that is added on.

Conclusions

The results of this study would indicate that:

(a) when displacement piles are driven into stiff residual soils which are overlain by a thick layer of saturated marine clay, the piles will heave,

(b) the upward displacement will be small if the pile is structurally adequate. The small upward displacement is followed rapidly by a small settlement. These pile displacements have little effect on ultimate shaft frictional capacity,

(c) Soil heave will cause a structurally inadequate pile to separate into two or more lengths. Such damage will seriously reduce pile bearing capacity. Redriving to close the vertical gap which is formed when the upper length of pile is lifted upwards by soil heave from the lower length of the damage pile is unreliable and cannot re-establish original pile bearing capacity and

(d) the design of piles to be founded in such a two-layered soil formation should provide for the tensile forces generated by the driving process and by soil heave.

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