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New technique of ground improvement: driven cast-in-situ thin wall concrete pipe piles (PCC)

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The driven cast-in-situ concrete thin-wall pipe pile (PCC) is a new technique for soft ground improvement and has been recently approached in China. It has become one of the most cost-effective soft soil improvement methods that can increase the bearing capacity and reduce the settlement of the soil. This paper presents a field application in a highway foundation improvement project. The field study has demonstrated that PCC pile composite foundation is a cost-effective way of improvement. The cast-in-PCC composite foundation piles; design theory, construction technology and monitoring are described. From the test results of cast-in-reinforcement it is clearly easier to control and the PCC technology is economically advantageous. The advantages of the method are proposed to be promoted in expressway construction to have greater social and economic benefits in the world.

Keywords: PCC pile; soil improvement; expressway; bearing capacity; settlement; field test

1. Introduction

Piles and drilled shafts are structural members used to transfer loads to deep strata through skin friction and end bearing. These deep foundations are necessary when the upper soil layers are too weak to prevent excessive settlement of the structure that is subject to large lateral forces. Ship impact, wind, earth and water pressure are all sources of lateral loads on deep foundations.

Recently, one of the significant solutions for soft soil improvement is the use of thin-wall pipe pile using cast-in-situ concrete (PCC) as a new technique. As new techniques, the load transferring mechanism for single PCC piles and PCC piles composite foundations are still uncovered. The new technology for PCC piles construction involves firstly driving a double walled open-ended steel casing by a vibratory driver with the protection of the tapered expendable driving shoe into soil stratum, secondly pouring concrete into the annulus void while vibratory system is withdrawing the steel casing forming the thin wall concrete pipe pile (PCC). After the PCC pile cured in place, a certain length (0.5 m) of soil plug at the pile head is excavated and concrete is poured to form a concrete pile head. A layer of 30–50 cm of broken gravels cushion with one or two layers of geogrid embedded in between is then placed at the pile head. In such a way, most of loads of the embankment are taken by the rigid pile group, the geogrid embedded gravel cushion is acting as a rigid plate to increase the pile head load and reduce the surrounding soil settlement (Liu *et al*. 2007).

The composite foundation consisting of PCC piles and the geogrid embedded gravel cushion is getting more popular now for soft ground road foundation improvement because of its low cost, high bearing capacity, embankment stability, and less post construction settlement. The static load test and field excavation can be concluded that the new type of pile is convenient to construct with high bearing capacity and reliable quality, which has great potential in practice (Liu *et al*. 2006, Xu *et al*. 2006). This study presents a successful application of PCC piles in a highway project and the preliminary study on its improvement mechanisms and effects.

Liu *et al*. (2003) have developed a cost-effective piling technique for soft ground improvement using a large diameter cast-in-place pipe pile, named to PCC pile. The PCC pile equipment has been designed and the technology is applied at highways embankment projects improving the bearing capacity of the soft soil foundation and reducing settlements (Liu *et al*. 2004a, 2004b, Fei *et al*. 2004a, 2004b, 2004c). Also, application of PCC piles technique in sea embankment on soft soil foundation (Liu *et al*. 2005), for expressways Zhou *et al*. (2005). The effectiveness of using the PCC pile for reinforcing soft soil has been demonstrated by Ma *et al*. (2006). The advantages of the technique are: (1) Shortening the construction duration time and (2) good workability.

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2. Construction equipment

In view of the inadequacy of existing program-based approach and relying on their own strengths in geotechnical engineering research, a large-diameter cast-in-place concrete thin-wall pipe with vibration mode technology of vibrosinking machinery has been patented and developed by Hohai University – Nanjing, China in 2001 as shown in Figure 1. Equipment components include:

(1) General gantry tower: to drive pile, a vibration mode is applied to annular double-wall of the hollow case.
(2) Lifting equipment: the lifting power is more than ordinary power to raise large scale pile.
(3) Vibration hammer: the vibration hammer has a weight enough to drive the pile according to the piling rate requirements through additional pressure, so driving can be achieved quickly.
(4) New double-walled steel casing: a combination of concentric annular pipe of 8-mm steel thickness with different diameters. The inner and outer diameters of pipes are 0.76 and 1.016 m, respectively. The inner pipe is open-ended whereas the annulus is fitted with a temporary conical-shaped driving shoe.

2.1 *Principle and mechanism of piling*

Construction machinery of in-situ thin-wall concrete pile is run by using vibratory hammering to the top of the pile casing forming a hollow annulus up to the designed depth, then pouring concrete into the hollow annulus; and finally vibratory is withdrawing the steel casing. In this way, an annulus concrete pipe pile can be formed with soil column inside the PCC pile. The vibratory hammer for pile driving consists of contra-rotating eccentric masses attached to the pile head. This vibratory system has quite heavy weight and very high

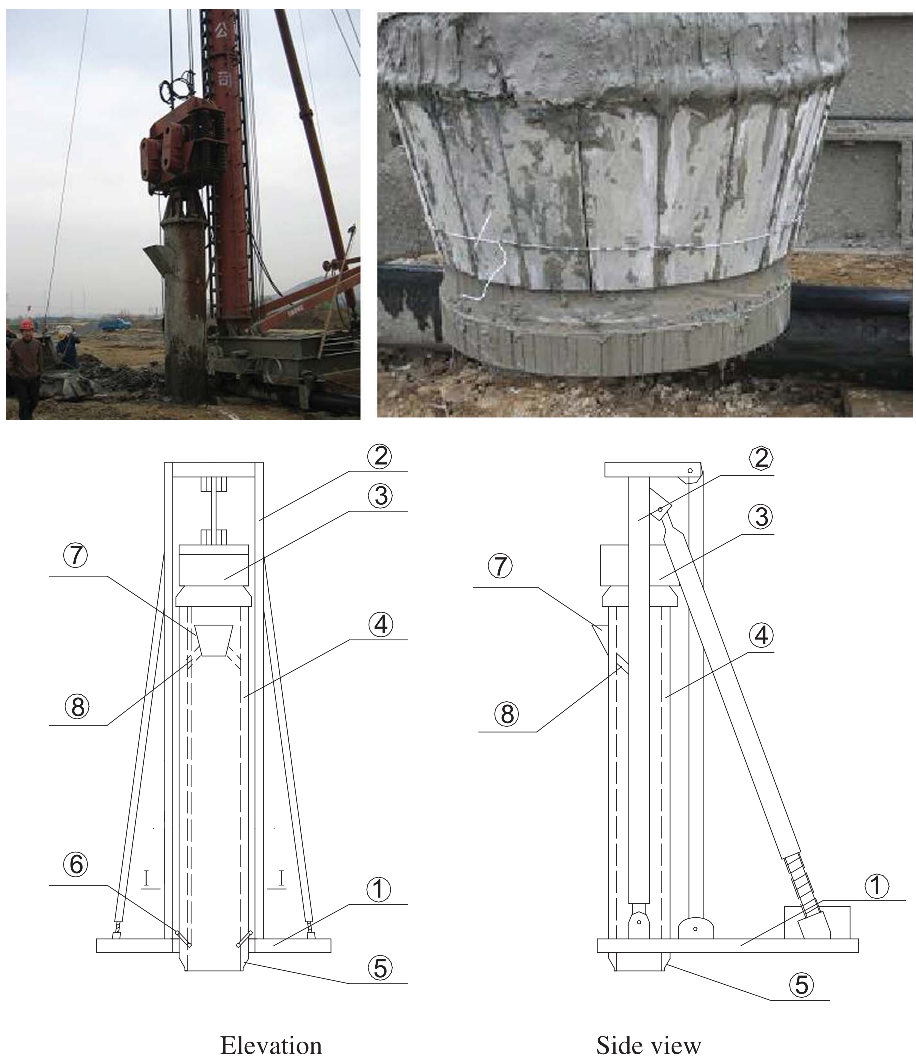


Figure 1. Schematic drawing of construction equipment. (1) Base of the equipment (including windlass); (2) Gantry crane; (3) vibration head; (4) double-walled steel casing; (5) valve pile boots; (6) into mud mode; (7) feed inlet; and (8) concrete shunt.

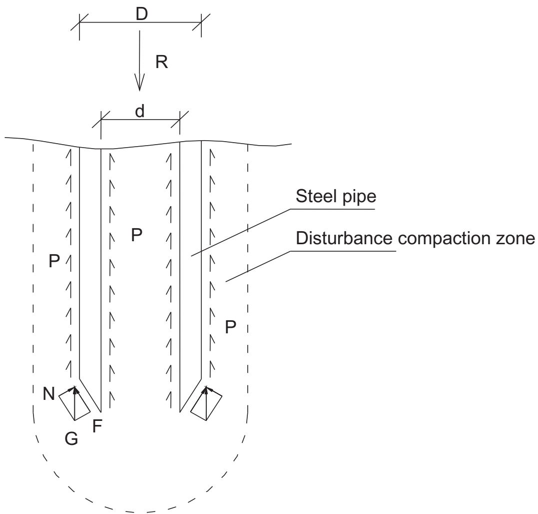


Figure 2. Immersed vibration stress scale diagram.

frequency resulting high impact force to drive the steel casing into the soil quickly with a rate of 0.8–1.2 m/min. The pile penetration rate is related to the working efficiency of the vibratory hammer, the weight of the vibratory system, density, cohesion, and the particle size of the soil layers. The pile penetration resistance during pile driving depends on the dynamic penetration resistance developed both along the pile shaft and on its base. While the driving force R is greater than the resistance force during penetration which consisting of the vertical components of the resistance forces developed at the tapered pile shoe as force N and force F and the resultant friction force along the pile shaft P, hence the pile is able to penetrate. At equilibrium the driving of pile will stop, the relevant forces at penetration are shown in Figure 2. During the driving process, the soil is displaced into the inner pipe and outside the outer pipe. This creates a 120-mm thick annulus between the outer and inner pipes for in situ concreting. As thin-wall vibration system makes the soil squeezing under the action of vibration force, hence the skin friction is declined sharply involving yield resistance and consequently high sinking rate.

2.1.1 *Casing, Vibration, and compaction*

The construction process of PCC piles include: locating equipment in place, driving steel casing, pouring the annulus with concrete, pulling the steel casing, and moving to another spot. The construction flow chart is depicted in Figure 3. During in situ concreting the annulus, the casing is withdrawn at a steady rate of 0.8–1.2 m/min. The pile casing should vibrate for 10 s before withdrawal. The withdrawing should be stopped temporarily with continuing vibration of the pile casing at every 1 m withdrawal for 5–10 s until the casing is completely withdrawn. This precaution may cause compaction of the PCC pile concrete and the squeezing pressure generated towards the inside and outside the thin wall casing is supporting the poured concrete forming thin wall thickness of the PCC pile. An appropriate concrete head varying from 0.3 to 0.5 m is always maintained within the annulus to provide stability, whereas the casing is withdrawn.

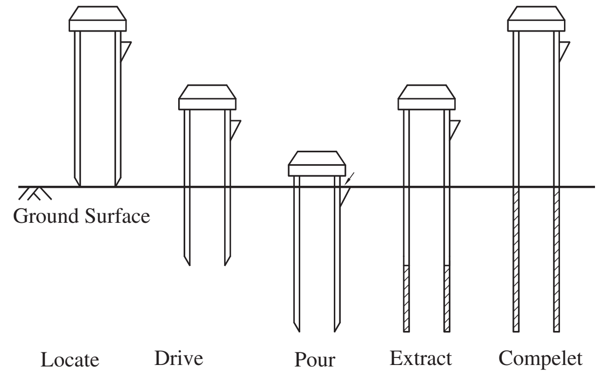


Figure 3. Construction flow.

During penetration of the steel casing, large soil displacement occurs due to driving, vibrating and squeezing effects which make the surrounding soil being compacted to a certain degree. The disturbance extent depends on the wall thickness of the steel casing and the soil properties.

3. Static load testing of single piles

3.1 *Excavation pile appearance description*

The piles with dimensions of 100 cm in diameter, 10–12 cm in thickness, and 15.0 m in length are installed with spacing of 3.30 m can be seen forming excellent, smooth inside wall and complete construction of the thin-wall pipe. Construction quality is good, smooth, well integrity, no pile necking (see Figure 4a), but the excavation of 1.0 m around the top of some piles show unequal thickness of the pile, and not smooth inside wall (see Figure 4b), this was mainly due to shifting the pile by redundant pushing dawn and pulling up the annular casing to promote a smooth concrete surface (see Figure 4). The soil profile at the project of the proposed expressway in Nantong, Jiangsu Province – China is described as 1.8 m thick layer of fill materials, overlaying 9.20 m thick of soft silty clay underlain by stiff clay. The soft clay layer can be classified as low to

(a) (b)

Figure 4. Piles excavation plans.

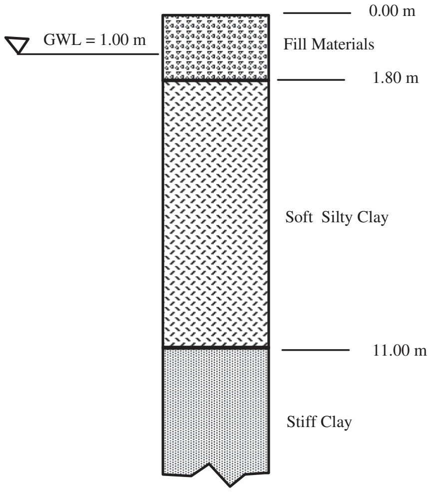


Figure 5. Profile of soil foundation at the site of Nantong Expressway project.

medium plasticity clay. The ground water level was recorded at 1.00 m depth (see Figure 5). The physical and mechanical properties of the soil deposits are presented in Table 1 and Figure 6.

3.2 *Static load test*

To promote the new approach method of PCC piles hence the purpose of static loading test (Figure 7) is to determine the ultimate bearing capacity. The static load test has been done by

Figure 7. Photo of field static load test.

Table 1. The physical and mechanical properties of soil

Pile	Depth	W_c %	γ_d g/cm^3	S_r %	Void ratio e	W_l %	I_p %	I_l	a_vMPa^{-1}	c'kPa	ϕ'^{o}	C_c	E_sMPa
K31+534	1.2	26.5	1.52	92	0.779	28.8	8	0.72	0.23			0.077	7.72
	4.3	40.8	1.27	98	1.124	34.3	8	1.83	0.92			0.307	2.3
	5.4	39.7	1.30	99	1.084	33.1	8	1.83	0.68			0.227	3.05
	8.5	34.0	1.40	98	0.935	35.2	8	0.84	0.22			0.073	8.77
	11.5	41.4	1.29	100	1.098	31.5	8	2.21	0.73			0.243	2.87
	15.7	31.4	1.40	90	0.95	51.4	22	0.09	0.51			0.168	3.85
	18.6	29.4	1.47	95	0.825	27.7	5	1.38	0.07			0.024	25.64
	21.3	29.1	1.49	97	0.809	30.0	5	0.80	0.11			0.038	15.87
	24.8	31.2	1.45	98	0.858	31.5	6	1.02	0.19			0.062	10.00
K30+793	0.9	24.6	1.48	81	0.818	28.0	8	0.59	0.33	30.5	26.9	0.174	5.34
	3.3	29.8	1.48	97	0.825	29.4	8	1.05	0.25	28.2	28.7	0.164	7.05
	5.3	39.3	1.28	96	1.105	32.1	6	2.20	0.94	22.3	34.7	0.292	2.06
	10.3	38.5	1.35	100	1.000	31.8	11	1.63	0.37	16.1	31.2	0.204	5.41
	12.3	42.0	1.26	99	1.142	31.9	10	2.02	0.87	39.4	23.2	0.420	2.38
	15.3	28.9	1.44	98	0.885	46.3	20	0.13	0.27	28.8	25.3	0.257	6.77
	18.8	26.9	1.46	86	0.845	27.6	7	0.89	0.23	0.0	42.6	0.120	8.83

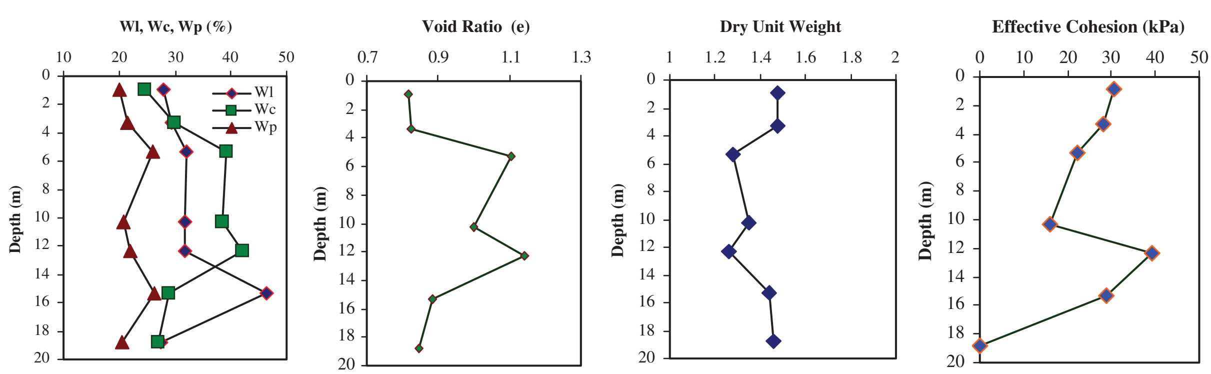


Figure 6. Physical and mechanical properties of soft clayey soil at the site of Nantong Expressway project.

Table 2. Static load test results in the table

Section	Pile No.	Max. load (kN)	Broken(√) or not (×)	Max. load of single pile (kN)	Settlement (mm)	Rebound (mm)
K30+778−K30+808	A5-18	1650	×	1650	13.63	7.53
	A6-20	1800	√	1650	42.74	21.64
K30+868−K30+898	A8-16	1500	√	1350	50.98	12.81
K31+509−K31+559	A14-10	1000	×	1000	13.18	6.22
	A15-8	900	×	900	11.70	5.33

applying the load at the top of the pile with a hydraulic jack, dial gauges are fixed to measure the settlement of the pile top.

3.3 *Test results and analysis*

A total of five single piles have been chosen for static loading test at the north side of the river area taking into account the dynamic effects of vehicles. The static load test results are obtained in Table 2.

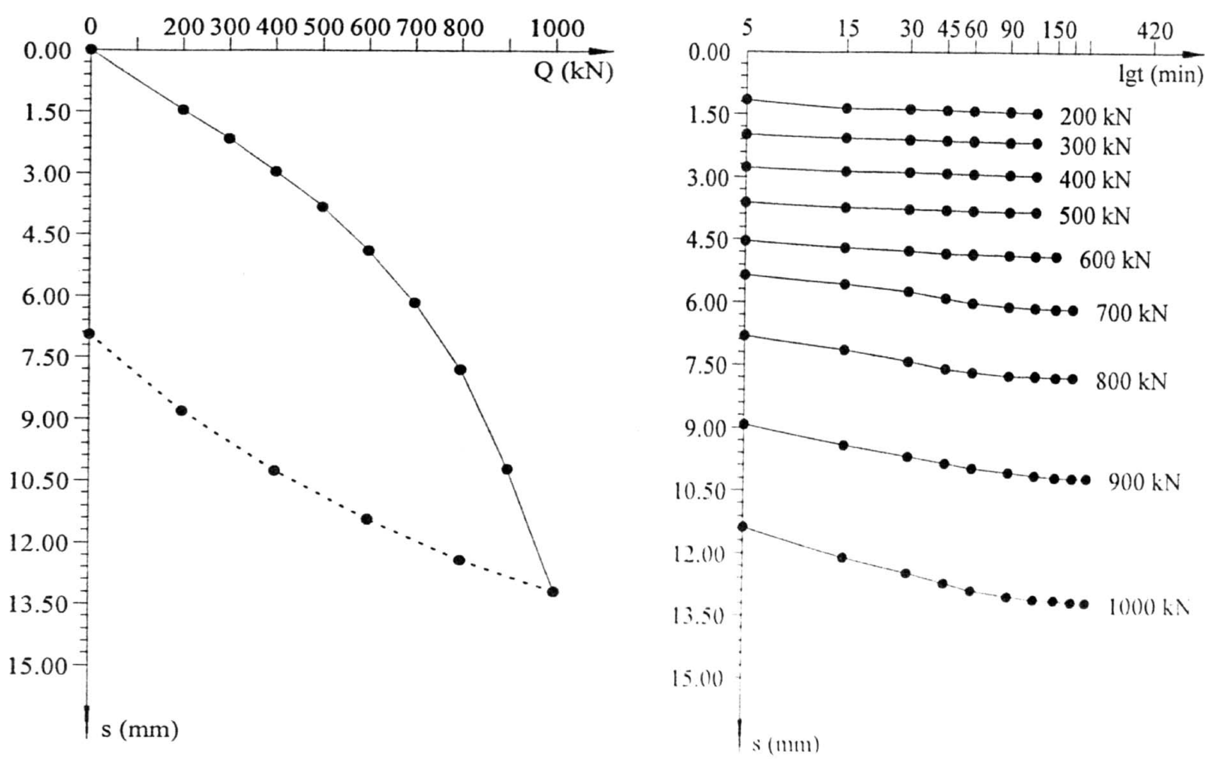
Some of the applied load for static pile load tests did not meet the failure. Thin wall concrete pile of 100 cm in diameter, 15.5 m in length, and 10 cm in thickness could support load up to 1350 kN with increase of 11% of the designed load of 1215 kN. The increase over the value of the design pile of 15 m in length, 124 cm in diameter, and 12 cm in thickness is 9%, the pile could support load up to 1650 kN. Typical curves are shown in Figure 8 depicting that for the pile number A14-10 the ultimate bearing capacity is 1000 kN inducing a settlement of 11.4 mm and rebound of 6.9 mm and large creep can be noted under higher loading. The static load test results also showed excellent construction quality of this pile.

4. Monitoring and improvement

The research program include bearing capacity and vertical displacement monitoring as load-settlement of the embankment, multilevel displacement, pore water pressure, and lateral displacement. To monitor these observations surface settlement plates, inclinometer, layered settlement system, pore water pressure transducer, etc. are needed. The monitoring instrumentation is depicted in Figure 9.

4.1 *Settlement*

Surface settlement values with the increase of height of the embankment are recorded. The monitoring of settlement has



(a) Q-S curve of single Pile

(b) Single Pile S − lgt curve

Figure 8. Static loading test curve of pile No. A14-10.

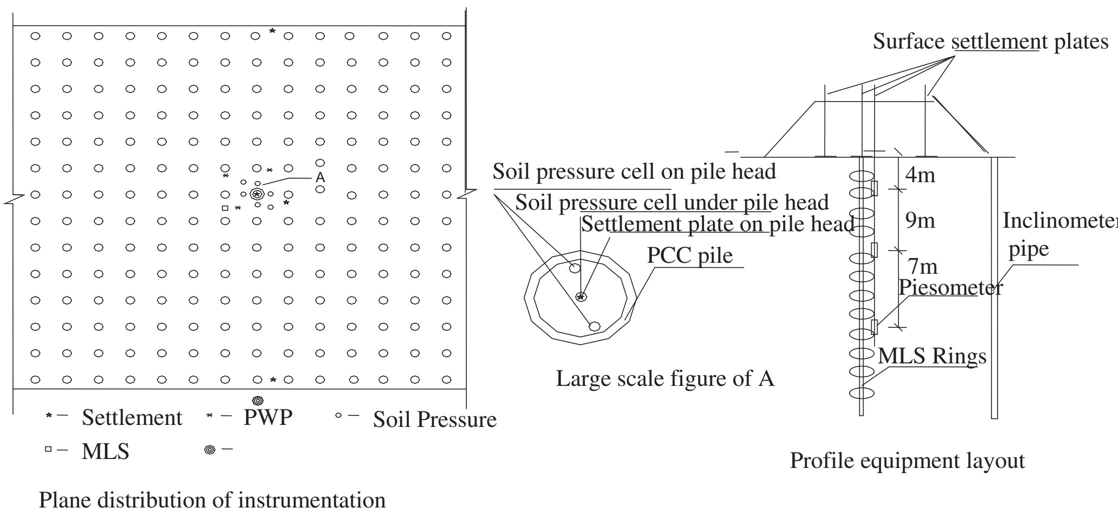


Figure 9. Monitoring plans.

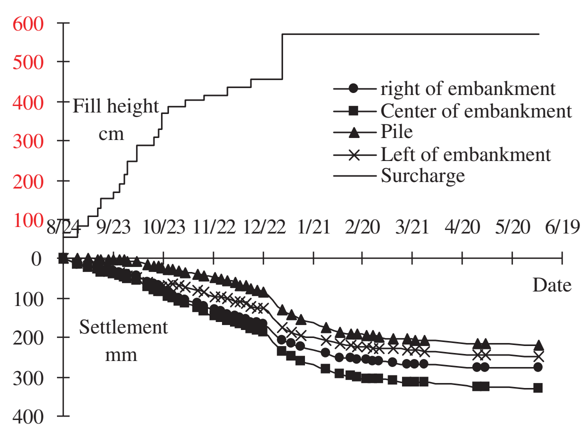


Figure 10. Surface settlement at section K30+884.5.

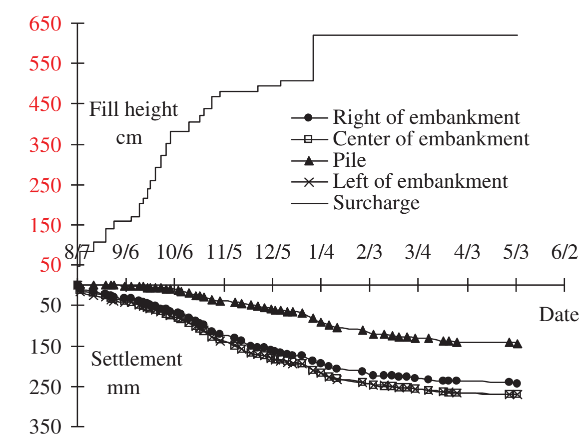


Figure 11. Surface settlement at section K31+578.8.

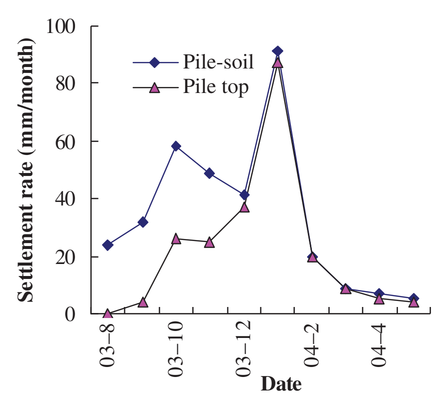


Figure 12. Settlement rate at section K30+884.5.

been divided into two aspects: The settlement observation at the pile head and the other settlement observation of soil between piles as load-settlement-time curves are shown in Figures 10 and 11, and settlement rate-time curves are depicting in Figures 12 and 13, all can be analyzed according to the following rules:

(1) In all sections, with the increase of height of embankment fill the surface settlement increased correspondingly. Early settlement of the pile top is smaller than that occurred at the ground between piles and with a slower rate. At embankment height of 2.5–3.0 m, almost no settlement of the pile head is recorded because the surround soft soil is loaded and hence give settlement response earlier than the pile head, and the pile - soil filling rate has increased, inducing settlement with rate of pile-soil is greater than the top of the pile.

(2) The rate of subsidence of soft soil foundation is fast converged. Rapid embankment construction is causing faster settlement rate. The initial settlement rate of embankment – soil foundation between the top of the piles are greater than the settlement rate of piles themselves, the two rates are converged when filling height reached around 4.0–5.0 m. Throughout the embankment

Table 3. Final differential settlement between piles and soil

Section	K30+756.5	K30+794.5	K30+822	K30+853	K30+884.5	K31+535.4	K31+578.8
Pile-soil differential settlement (mm)	100	108	114	125	108	113	127

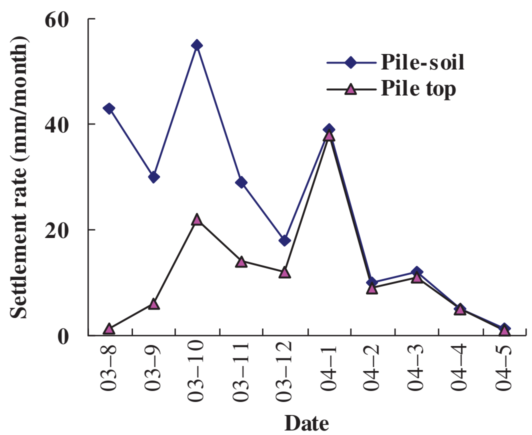


Figure 13. Settlement rate at section K31+535.4.

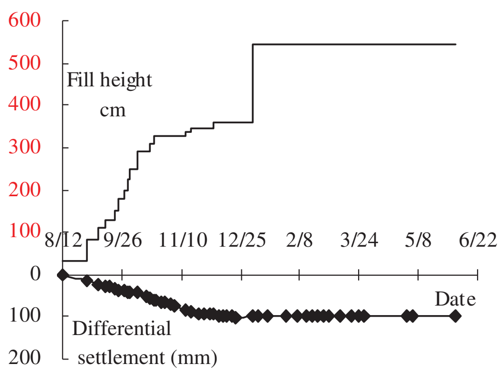


Figure 14. Pile-soil differential settlement at K30+756.5.

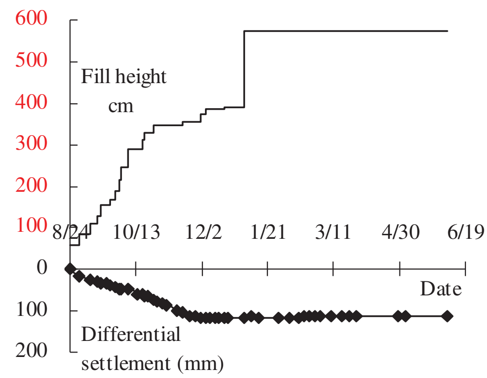


Figure 15. Pile-soil differential settlement at K30+822.

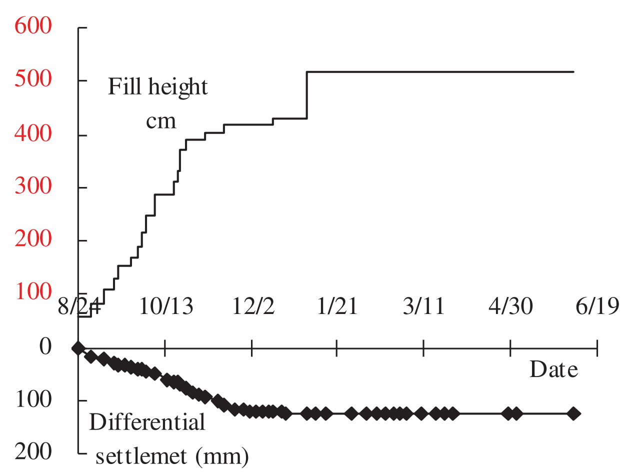


Figure 16. Pile-soil differential settlement at K30+853.

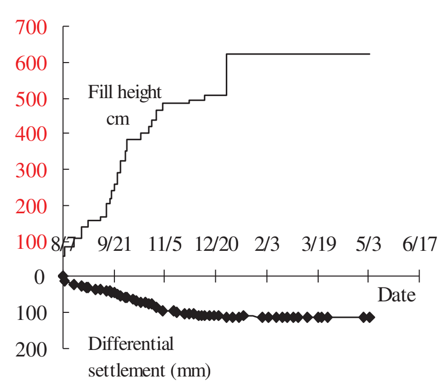


Figure 17. Pile-soil differential settlement at K31+535.4.

construction stages, the maximum pile-soil settlement rate is 93.3 mm/month at (Section K30+884.5). The average settlement rates of soil between piles are 40.0, 41.3, 44.4, 45.8, 40.8, 34.2, and 41.4 mm/month, and on the top of piles the average settlement rates are 16.6, 19.2, 19.0, 18.6, 18.4, 18.4, and 14.0 mm/month.

The differential settlement at different sections between the soil and pile head are shown in Table 3. The differential settlement – time relationships have been depicted in Figures 14, 15, 16, and 17, emphasizing that at sections K30+756.5 and K30+822 the maximum differential settlements have reached 100 and 114 mm, respectively, after fill surcharge of almost 3.5 m (see Figures 14 and 15). However at sections K30+853 and K31+535.4 the maximum differential settlements have reached 125 and 113 mm with fill surcharge of 4 and 5 m, respectively (see Figures 16 and 17).

4.2 *Multilevel settlement*

Observation hole is accurately derived vertically with diameter 110 mm to depth of 25 m, the verticality of hole with accuracy

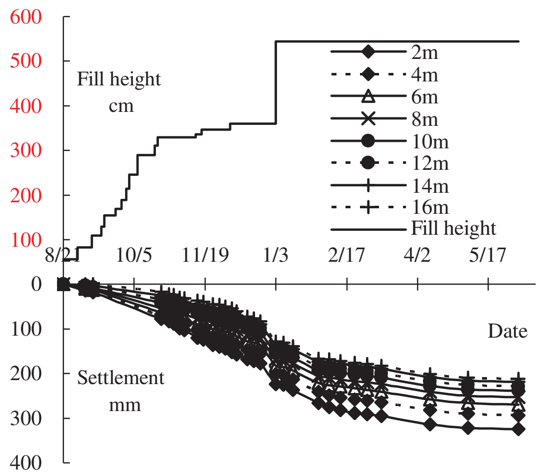


Figure 18. Multi level settlement at section K30+794.5.

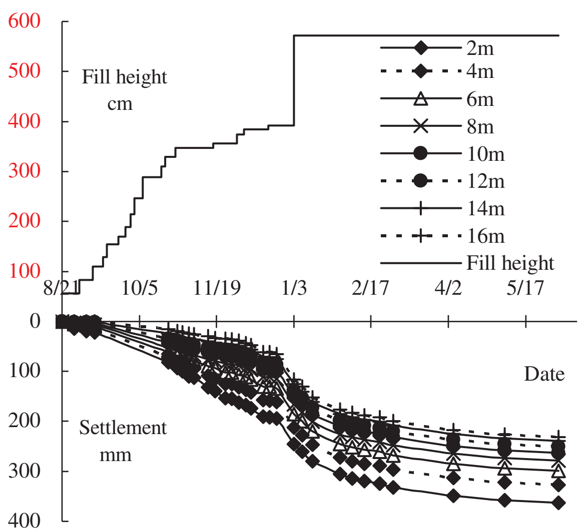


Figure 19. Multi level settlement at section K30+822.

of $\leq 2°$. Settlement magnetic rings are installed every 2 m by sequence to monitor the sub layers settlement of the soil.

The relationships of the monitored multilevel settlement are shown in Figures 18, 19, 20, and 21. The settlement at 16 m depth was more than 200 mm, means equal to 66% of the surface settlement (325 mm) at the center of the embankment or equal to the pile head settlement (almost 200 mm), this is emphasized that the pile is well compressed.

4.3 *Horizontal (lateral) displacement*

A pipe with diameter 110 mm is driven in the soil to reach a depth of 25 m, the hole with accuracy of $\leq 2°$. Inclinometer is using for monitoring the horizontal displacements at different depths.

Poor physical and mechanical properties of soft ground and low ability to resist deformation of embankment foundation

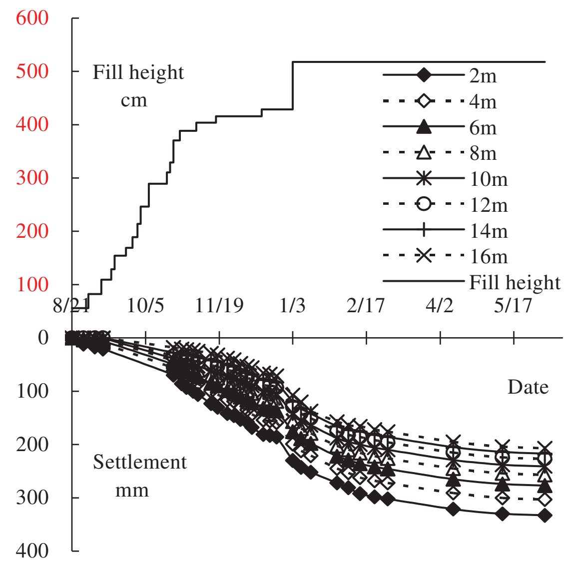


Figure 20. Multi level settlement at section K30+853.

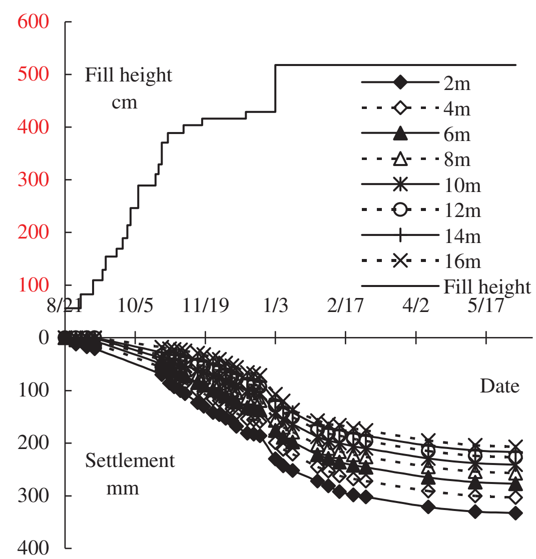


Figure 21. Multi level settlement at section K30+884.5.

under which the shear stress along the weak band often have a greater level of displacement, typical lateral displacement recur as are shown in Figures 22 and 23.

The largest lateral displacement was 20 mm and the results for the embankment of 6.0 m height are very concerned indeed. Moreover, there is no mutation of the lateral displacement, despite the faster embankment construction.

The main purpose is to understand the process of readjusting the distribution of surcharge pressure of embankment load between soil and pile composite. The stresses between soil and the top of the pile changes due to the process of surcharging fill as shown in Figures 24, 25, 26, and 27. Changes are similar with the increased load of embankment, the stresses on soil and the pile corresponding is increased, but there is a

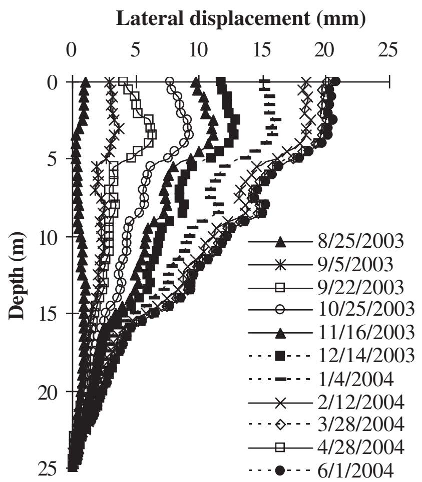


Figure 22. Lateral displacement at section K30+794.5.

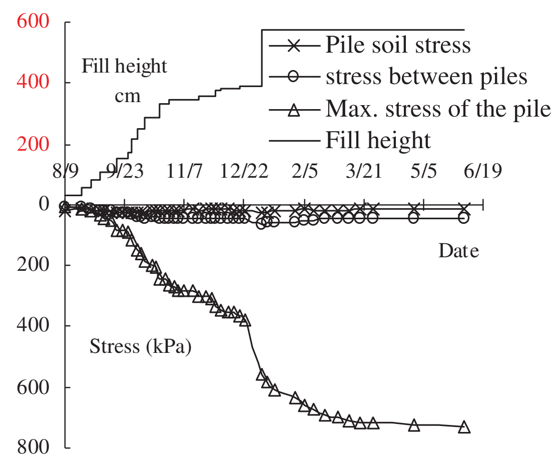


Figure 25. Pressures changes diagram at section K30+822.

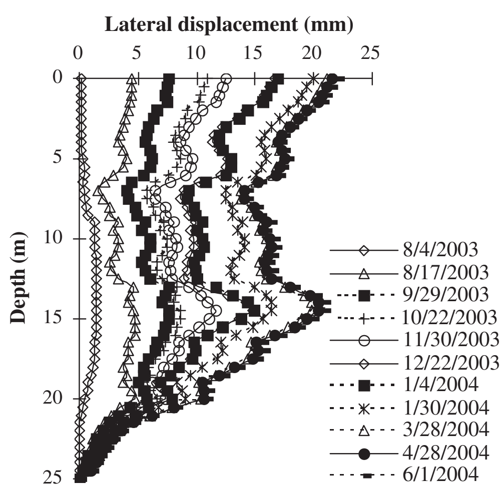


Figure 23. Lateral displacement at section K31+535.4.

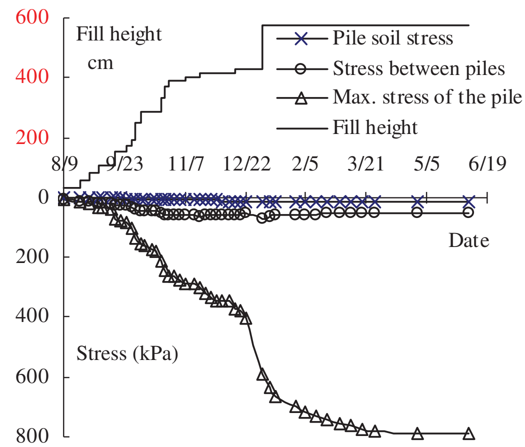


Figure 26. Pressures changes diagram at section K30+853.

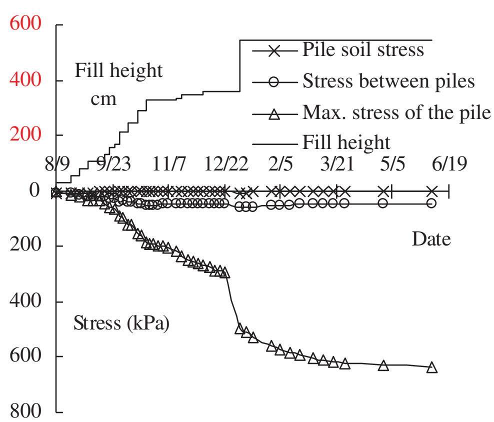


Figure 24. Pressures changes diagram at section K30+794.5.

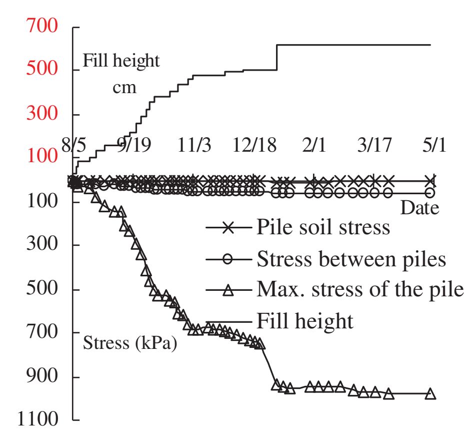


Figure 27. Pressures changes diagram at section K31+535.4.

greater difference as in section K30+822. When filling height is from 0.5 to 3.5 m means the induced stress on the pile-soil is increased from 10.49 to 46.50 kPa, respectively, and stress on the pile head is increased from 11.54 to 268 kPa. After completing the embankment height of 5.7 m the average stress on soil between piles and on the pile head is reached 58.29 and 606.19 kPa, respectively. Preloading is a certain level of stress between piles during the surcharging as in section K30+822 the stress on soil between piles at the completion of filling is 63.59 kPa then dropped to 45.27 kPa. Stress has been passed the value of 606.19 kPa on the head of the pile to 724.53 kPa, currently stress on the pile head and the stress on soil between piles have been basically stabilized and interacted whenever pile-soil settlement being stable at fill height of 3.5 m.

4.4 *Pore water pressure*

The pore water pressure read out unit was mounted for monitoring. The sensors are assembled and then immersed in water to ensure no bubbles and then sensors are driven to design depths of embedment. The pore water pressure changes at different depths with the height of the embankment filling process as shown in Figures 28 and 29.With increased of the surcharge load the pore water pressure is gradually increasing and then subsequent dissipation is observed. The rate of the increase is small because a large part of the load supported by the cushion of embankment on the piles and filling is in the hierarchical manner to pile-soil, hence load sharing between the adjustment process caused by the fill and pore water pressure that early dissipated. At different depths the pore water pressures are increased significantly indicating compression of the soil and monitoring results show pore pressure is dissipated quickly where at 25 days the pore pressure dissipation is 60% and after 2 months became 80%, indicating good foundation drainage conditions causing rapid consolidation.

4.5 *Piles-soil load distribution*

Pile-soil stress for the assessments means changes in the pile-soil stress ratio, as can be seen in Figure 30 pile-soil stress ratio increased gradually with the increase of the load of embankment. When the filling is still increase at section K30+822 from 0.5 to 3.5 m height, pile-soil stress ratio increases from 1.1 to 5.77, when preloading is finished the pile-soil stress ratio will be 9.53 until stabilization to be of 16.0. The pile-soil stress is increasing with the embankment height, but after construction the pile-soil stress ratio is slowly increase and eventually become stable. Finally the monitoring at the end of construction showed, the pile-soil stress ratio are 14.4, 16.0, 15.6, 14.0, and 17.0 (see Figure 30). The curves of different sections also showed some differences, largely to the performance of two laws:

(1) The pile-soil stress ratio in section K31+535.4 is increased earlier and faster than in section K30+884.5
(2) The pile-soil stress ratio in section K30+822 and section K30+853 are increased in the latter part; may due to dense cushion laying – geotextile that induce tension was tighter than the early increase of pile-soil stress otherwise, large pile-soil stress ratio performance for the latter part of the curve.

Pile-soil embankment stress changes show cast-in-pile-soil stress ratio is a variable and the value is not only due the height of the filling but other characteristics of the cushion.

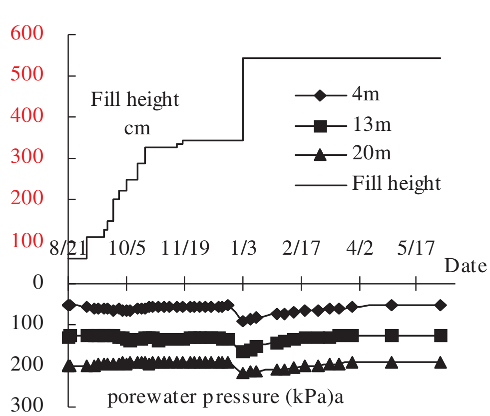


Figure 28. Pore water pressure curve at K30+794.5.

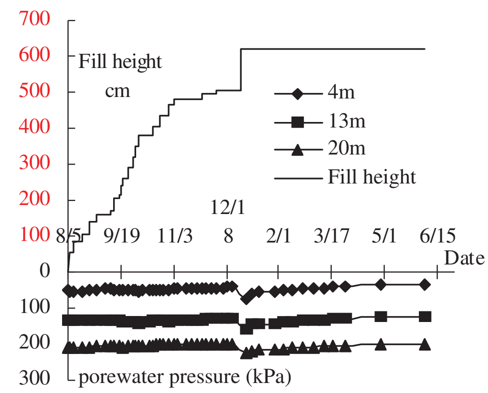


Figure 29. Pore water pressure curve at section K31+535.4.

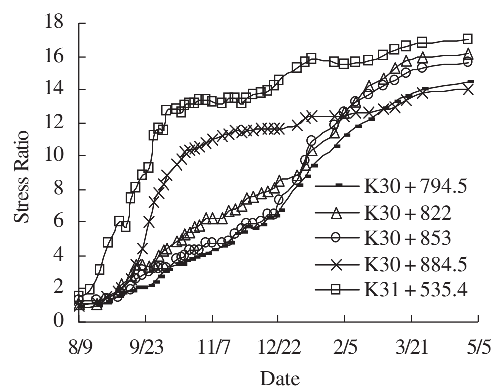


Figure 30. Stress ratio vs. time.

5. Conclusions

The PCC piles technique for ground improvement of expressway is recently applied in China and proposed to be promoted applied in the world. From the test results the quality of construction is easier to control and the method is economically advantageous. The following conclusions can be drawn:

(a) The program of soft ground improvement can significantly reduce the settlement of the foundation, and reinforce soft ground for highways and other buildings
(b) The scene of deep horizontal displacement data shows that the cast-in-reinforced foundation has a good ability to resist horizontal displacement where the maximum horizontal displacement has a value of 20 mm at the ground surface, hence embankment sub-layer stability has been greatly enhanced and there is no instability problems may be occur.
(c) PCC Pile settlement after treatment induces fast convergence, hence small settlement after, thus avoiding maintenance costs later of the expressway.
(d) With different pile diameter, thickness and spacing, the reinforced observation results show that 10–12 cm thickness provides sufficient capacity to meet the bearing capacity of soft foundation and pile construction process requirements economically. Large diameter PCC pile can significantly increase the bearing capacity of the soft soil.
(e) Pile-soil deformation can be reduced by increasing pile cushion stiffness using geotextile material and reducing pile spacing.

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