The Hong Kong Institution of Engineers Transactions, Vol 18, No 2, pp34-49
Paper T0819-201002; Received 8 March 2010; Accepted 13 July 2010

The Application of PDA/CAPWAP to Ensure Quality and Capacity in Driving Long Steel H-piles

The methodology of application of Pile Driving Analyser (PDA) test and Case Pile Wave Analysis Programme (CAPWAP) analysis for quality control and capacity assessment in driving long steel H-piles is fully illustrated in the project ´Sun Yat Sen Memorial Park and Swimming Pool Complex´. The deficiencies of using the Hiley Formula as a field control to assess pile capacity have been well known. These deficiencies are more prominent when the pile is long. In the use of the Hiley Formula for long driven piles, out of range of final set table has been a common phenomenon and this has rendered the formula not applicable. The paper discusses the use of a hybrid method making use of advanced method (PDA/CAPWAP) in conjunction with simple method (Hiley Formula) for field control and pile capacity assessment. It has been shown that this method will not only improve the reliability of the pile foundation but also contribute greatly to the overall cost-effectiveness of installation of the piles by reducing the chance of over-driving and damage to the pile. This method also enables the successful use of hydraulic hammers in taking final sets.

Keywords: *Pile Driving Analyser (PDA), Case Pile Wave Analysis Programme (CAPWAP), Hiley Formula, Steel H-piles, Final Set Table, Hydraulic Hammer*

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Introduction

For pile driving in Hong Kong, the Hiley Formula has been widely used as a field control to assess pile capacity despite its deficiencies. All dynamic formulae suffer from inherent problems including poor representation of the hammer, driving system, pile and soil. These deficiencies are more prominent when the pile is long. In the use of the Hiley Formula for long driven piles, ie pile length that is greater than about 55 m to 60 m, out of range of final set table has been a common phenomenon and this has rendered the formula not applicable. A pile-driving criterion that is applicable to long driven piles as well as user-friendly is called for. Accurate determination of the driving criterion will not only improve the reliability of the pile foundation but also contribute greatly to the overall cost-effectiveness of installation of the piles by reducing the chance of over-driving and damage to the pile.

Quality Control on Pile Driving

Fung *et al.* (2004b) investigated the reliability of CAPWAP analysis in pile capacity prediction as compared with results from static load tests in local soil conditions. It has been shown that CAPWAP analysis is a fairly accurate method for driven pile capacity prediction.

Fung *et al.* (2004b) also compared pile capacity predicted by CAPWAP analysis and by the Hiley Formula (where the coefficient of restitution (e) and drop efficiency (E_h) were obtained by back analysis from CAPWAP results of trial piles) of 327 piles from 18 sites. The results revealed that 94% of the piles have capacities predicted by the Hiley Formula deviating from their corresponding CAPWAP capacities by less than ± 10%.

So and Ng (2009) pointed out that among the pile capacity prediction methods of PDA, the Hiley Formula and the HKCA Formula, PDA gave the best estimation. It could be used in lieu of a static loading test for verifying pile capacity when the piles are long.

In the last decade, the Architectural Services Department (ArchSD) used Pile Driving Analyser (PDA) test and CAPWAP analysis for quality control and capacity assessment of driven piles installation. PDA has been used to detect the integrity of piles, hammer efficiency and give a rough indication of pile capacity. The data of force and velocity of piles obtained from the PDA tests can be further analysed by CAPWAP to give a more accurate assessment of pile capacity.

Carrying out PDA test and CAPWAP analysis for each pile for acceptance may affect the site progress of projects with restricted driving time to a certain extent. Therefore, a simple tool to assess pile capacity during the final set of the driving process is still in need. Despite various shortcomings of the Hiley Formula as mentioned above, it has been shown that the Hiley Formula can take up the role after calibration by the CAPWAP analysis to determine the efficiency of hammer (E_h) and the coefficient of restitution of hammer cushion (e) of the hydraulic hammer (Fung *et al.*, 2004a). Strictly speaking, the ArchSD's method is a hybrid method making use of advanced method (CAPWAP) in conjunction with simple method (Hiley Formula). The resulting method is therefore handy to use but avoids the drawbacks of the Hiley Formula. The details of the pile acceptance process are as follows:

Simple Ground Conditions – The efficiency of the hammer (E_h) and the coefficient of restitution of the hammer cushion (e) of the hydraulic hammer shall be determined from/verified by CAPWAP analysis of trial piles. The combination of E_h and e shall be so chosen such that when these values are substituted into the Hiley Formula, the average of the predicted bearing capacity of the trial piles is not higher than 85% of the average CAPWAP capacity. Once these parameters are determined, a final set table can be established for the contract. The factor of 85% accounts for possible deviations of CAPWAP predictions from static load test results and for the gradual decrease in driving system efficiency due to deterioration in the condition of the hammer cushion. Normally, the e value of cushion decreases with sustained use and the efficiency of the driving system becomes lower when compared with that observed during trial pile installation (Fung *et al.*, 2004b).

Difficult Ground Conditions – In cases where measured final sets are out of range of the set table with E_h and e so chosen, all the piles falling into this category shall be subject to CAPWAP analysis. To further ensure quality, at least the pile with the lowest CAPWAP capacity shall be load tested for acceptance. This provision can eliminate the risk of pile over-driving which is common for very long pile or difficult ground conditions, and as a result causing less pile damage.

The above methods have been used by the ArchSD as a field control to assess pile capacity since 2003. From experience obtained to date, they provide an accurate and workable tool to fulfil the requirement of quality control and assessment of pile capacity. It also enables the successful use of hydraulic hammers in taking final sets.

Difficult Ground Conditions – Sun Yat Sen Memorial Park and Swimming Pool Complex

Ground Conditions

The project site is a challenging and difficult site in difficult ground condition with bedrock at 56 m to 94 m below ground.

The borehole information indicates that the stratification of site comprises mainly, Fill, Marine Deposit, Alluvium and Completely Decomposed Granite.

Fill ranges in thickness from 19 m to 27 m with many boulders, cobbles and gravels.

Marine Deposit with thickness ranging from 2 m to 5 m lies beneath the Fill.

Alluvium, which is encountered beneath the Marine Deposit, has a thickness ranging from 2 m to 6 m.

Completely Decomposed Granite with thickness ranging from 21 m to 62 m lies beneath the Alluvium. Rockhead level lies between 56 m and 94 m below ground.

Difficulties of the Site

It was expected that the steel H-piles would be founded at the residual soil with SPT N-values > 200, which was in general approximately at -52 mPD. However, the geological profile of the site shows that the layer of soils with SPT N-values > 200 is deeper in the central portion of the site where longer piles were expected. The installation of driven steel H-piles with some of them longer than 70 m in a site with many boulders and cobbles would be the main challenges to the Contractor. Since the piles would be long, out of range of final set table was expected if the Hiley Formula was used. The presence of many boulders and cobbles might also affect the progress of works significantly.

Trial Pile Installation

In this project, the Contractor adopted 305 x 305 x 223 kg/m steel H-pile as foundation. 328 steel H-piles at an estimated average depth of 56 m were adopted with theoretical safe pile capacity of 3,600 kN.

Eight trial piles were selected for PDA tests with CAPWAP analysis to determine the efficiency (E_h) and the coefficient of restitution of the hammer cushion (e) of the 20-tonne hydraulic hammer for final set. These piles were spread evenly across the site including the area where the longest pile depth is expected. Before the start of pile driving, visual inspection of the type and quality of the hammer cushion was carried out. The ram and helmet of the hammer were weighed. The results of correlation are shown in Appendix A.

It was observed that all trial piles were out of range of the final set table calculated using the correlated Hiley Formula based on the E_h and e determined by CAPWAP analysis. In view of the trial pile results, the Contractor then chose to use the acceptance method for difficult ground conditions, ie carrying out PDA test and CAPWAP analysis for every pile for assessing pile capacity and load testing the pile with the lowest CAPWAP capacity for acceptance.

It was also observed that for long piles where the bearing strata are deep, the CAPWAP capacities are lower (some of them are even lower than 7,200 kN) because the Contractor was reluctant to drive the piles to a deeper level to achieve a higher capacity for fear of damaging the piles. This, however, may be acceptable due to potential set-up effect.

PDA Tests

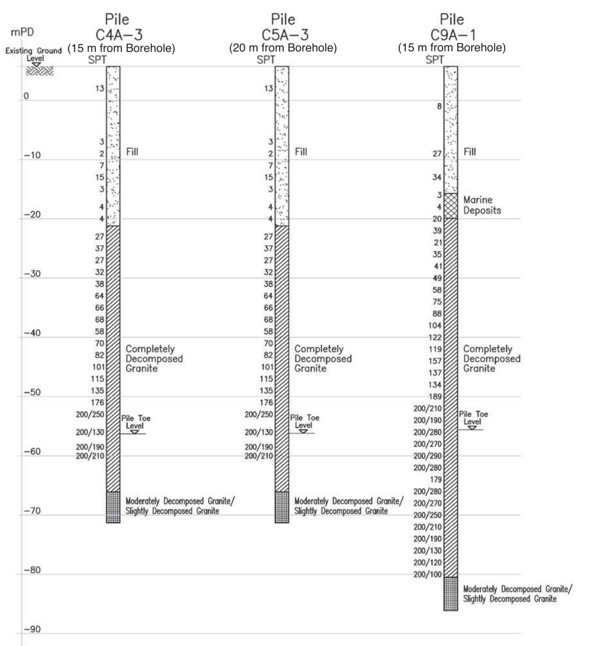
In order to start the pile cap works early by phases, the site area was originally divided into four zones and further increased to six zones later due to difficulties encountered in driving piles to achieve the design capacity in some areas of the site. PDA tests were carried out for each pile by a testing firm employed by the Contractor. Based on the results of the PDA tests, the piles with lower capacity (determined by Case method) were selected to be tested by PDA again by an independent testing firm employed by the ArchSD. CAPWAP analyses were also carried out for these piles and the pile with the lowest CAPWAP capacity of each zone was load tested for acceptance. Table 1 below summarises the number of piles selected for PDA test for each zone.

Zone No	No of Piles	No of PDA Tests Carried out by the Contractor (%)	No of PDA Tests Carried out by the ArchSD (%)
1	64	64 (100%)	9 (14%)
2	97	97 (100%)	16 (16%)
2A	6	6 (100%)	6 (100%)
3	77	77 (100%)	18 (23%)
4	132	132 (100%)	14 (11%)
4A	9	9 (100%)	1 (11%)

Table 1 – PDA Tests Carried out for Different Zones

First Loading Test

After the completion of pile installation for the first zone, loading test was carried out for the pile with the lowest CAPWAP capacity (C9A-1, 6,715 kN). The loading test was carried out on 16 December 2008, seven days after the PDA test. The maximum test load applied was 7,200 kN. The load test results failed to satisfy the contract specification. According to the contract specification, if a pile fails the static load test, additional load tests need to be carried out for other two piles for acceptance. Therefore, piles with the second and third lowest CAPWAP capacity of the zone, ie C5A-3 (6,728 kN) and C4A-3 (7,147 kN), were chosen for static load test. Both piles passed the static load tests. The borehole logs and test results of the three piles are shown in Fig 1 and



(SPT of 200 blows in 250 mm penetration is denoted as 200/250)

Figure 1 – Borehole Logs

Pile No for Loading Test	Embedded Length of Pile (m)	Set/Blow (mm)	Date of PDA Test	CAPWAP Capacity (kN)	Date of Loading Test	No of Days after PDA Test	Test Load (kN)	Allowable Total Settlement (mm)	Actual Total Settlement (mm)	Allowable Residual Settlement (mm)	Actual Residual Settlement (mm)
C9A-1	60.0	1.8	9/12/08	6,715	16/12/08	7	7,200	80.903	80.268	20.067	24.309
C5A-3	60.3	0.2	9/12/08	6,728	24/12/08	15	7,200	81.336	65.167	16.292	3.864
C4A-3	60.4	0.6	9/12/08	7,147	2/1/09	24	7,200	81.435	60.839	15.21	0.589

Table 2 – Loading Test for Zone 1

Table 2 respectively. C9A-1 was re-driven to a depth of 72.7 m (12.7 m deeper than the original depth). PDA test was carried out again and no sign of damage was observed. The CAPWAP capacity was increased from 6,715 kN to 7,741 kN.

In Table 2, it can be seen that pile no C9A-1 with CAPWAP capacity of 6,715 kN, which is less than the maximum test load of 7,200 kN, marginally passed the total settlement requirement but failed to satisfy the residual settlement requirement. However, for piles C5A-3 and C4A-3, although their CAPWAP capacities (6,728 kN and 7,147 kN respectively) are also lower than 7,200 kN, they passed the loading test. Same as other testing methods, CAPWAP can only compare with load testing method within a certain tolerance. From Table 2, it is however clearly indicated that the lower the CAPWAP value, the lower the load test result, and CAPWAP can predict the lower bound pile capacity confidently. The two piles C5A-3 and C4A-3 that passed the loading test may also be due to the following considerations:

Pile Capacity Not Fully Mobilised during the Dynamic Load Testing

It should be noted that dynamic load testing (eg PDA test) only indicates the activated or mobilised pile capacity at the time of testing. At very low set per blow, dynamic test methods tend to produce lower bound capacity estimates because the resistance is not fully activated particularly at and near the toe as the movement is less than the quake value (quake is the minimum relative movement between the pile and the soil for activation of ultimate resistance) (Goble and Likins, 1996). Using dynamic testing to determine the capacity of a pile with low set per blow is analogous to a static load test that could not be run to failure due to limited loading capacity or limited reaction system capability (Hussein *et al.*, 2002). In both cases, the loads determined are the proof loads of the piles but not the ultimate loads.

The set per blow for piles no C5A-3 and C4A-3 are very small, ie 0.2 mm and 0.6 mm respectively. Therefore, the pile capacity may not be fully mobilised during the dynamic load testing. This argument is supported by the CAPWAP analysis results which show that the shaft friction near the toe for piles C5A-3 and C4A-3 has not been fully mobilised (Figs 2 to 4).

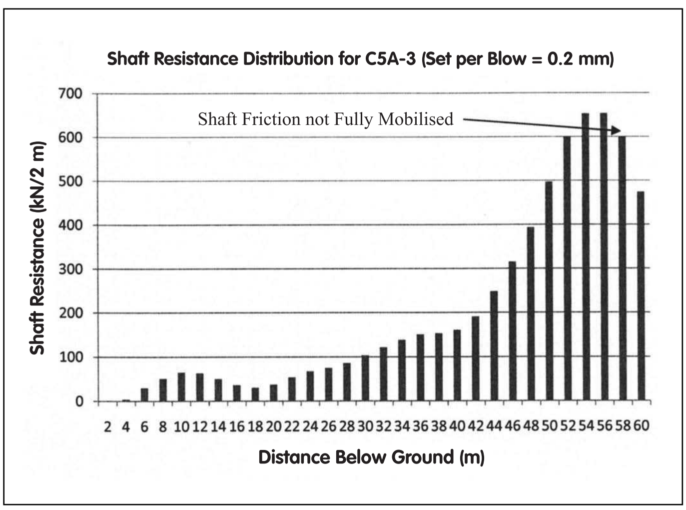


Figure 2 – Shaft Resistance Distribution for C5A-3

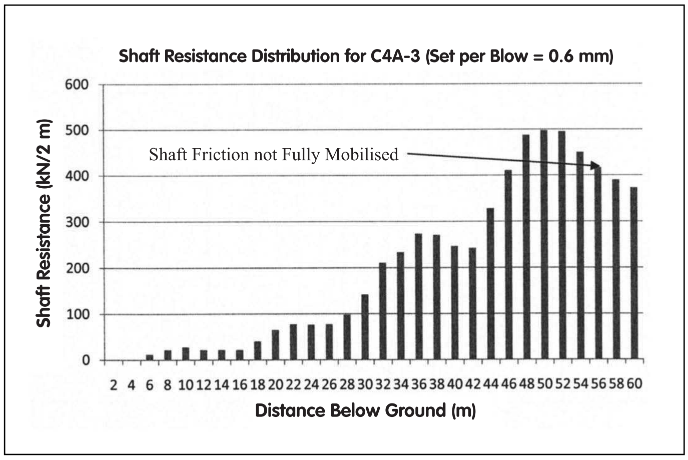


Figure 3 – Shaft Resistance Distribution for C4A-3

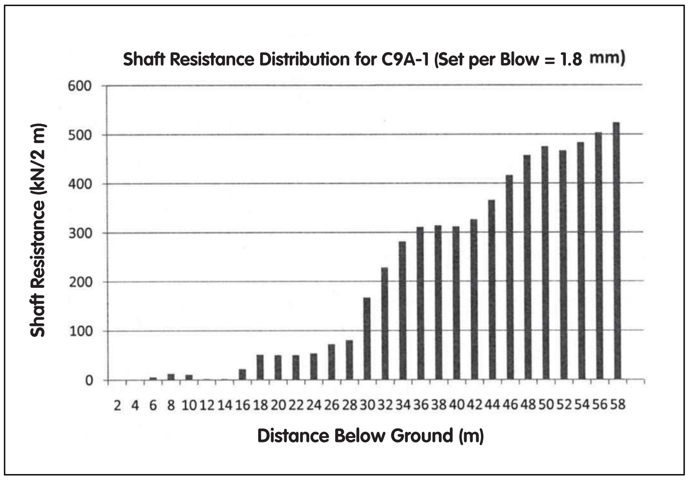


Figure 4 – Shaft Resistance Distribution for C9A-1

Set-up Effect

The time lag between the PDA test and the static load test for piles no C5A-3 and C4A-3 are 15 days and 24 days respectively. Increase in the pile capacity may occur after PDA test due to soil set-up.

The set-up effect of this site has been studied. PDA tests and CAPWAP analyses were also carried out for 51 piles at different time interval after the end of driving in order to investigate the set-up effect. Static load tests had not been carried out on these piles. The results show that pile capacity increases ranging from 1 – 52% between restrike tests carried out after time lapse in the range of 3 to 187 days. It is also noted that there was a decrease in the set per blow of the second restrike tests. If more energy had been used to drive the piles in the second restrike tests to produce a larger set per blow, the CAPWAP prediction would have been even higher (ie higher set-up effect). The plot of pile capacity variations with log time is shown in Fig 5.

Fung *et al.* (2006) pointed out that the timing of both the static load test and the dynamic restrike test relative to the end of driving (set-up

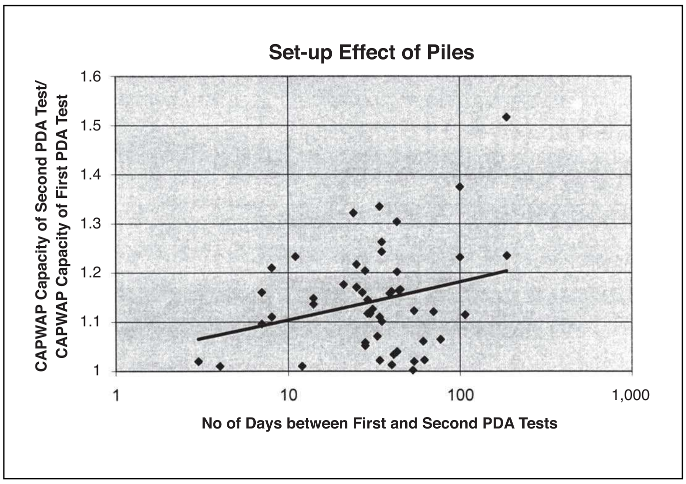


Figure 5 – Set-up Effect of Piles at Different Time Intervals

effect), and the capacity mobilisation (set per blow) have a significant effect on pile capacity prediction. The CAPWAP analysis can predict the static load test results very accurately for piles where the influence of pile capacity mobilisation and set-up effect is minimal. When the influence of the two factors is large, CAPWAP analysis underestimates the static load test results fairly significantly.

Triantafyllidis (2001) pointed out that in assessing pile capacity using the Hiley Formula, if the pile is relatively long, it is worth considering only that portion of the pile that is affected for the duration of the impact (L_c). During impact between hammer and pile, waves are generated travelling towards the pile toe. In the case of friction acting at the perimeter of the pile or toe resistance, these waves are partly reflected, generating waves travelling upwards to the pile top and causing separation between hammer and pile.

L_c is related to the duration of the impact where pile and hammer are still in contact (t_c) (ie compression stresses are applied at the pile top) and speed of wave propagation in the pile material (c) as follows:

$$2L_c = ct_c$$

From Triantafyllidis (2001), it is noted that L_c depends on weight of hammer, maximum compressive stress/impact velocity, pile impedance and skin friction on the pile shaft while driving.

The PDA results of all piles in this project site indicate that the time, t_c, where pile head is in compression, is greater than 2L/c, where L is the total pile length. This implies that the whole pile length was affected for the duration of impact. This may be due to the fact that a 20-tonne hydraulic hammer was used for driving in this project instead of a 6.2-tonne diesel hammer as quoted in the illustrative examples of Triantafyllidis (2001).

The Use of Lower Capacity Piles

Some of the trial piles were driven to 76 m deep but still could not achieve the pile capacity of 3,600 kN. The Contractor proposed to use a mixture of 305 x 305 x 180 kg/m and 305 x 305 x 223 kg/m steel H-piles instead of all 305 x 305 x 223 kg/m steel H-piles. 305 x 305 x 180 kg/m steel H-piles can be driven to final set at shallower depths. In some areas where the bearing strata are deep, the Contractor proposed to use lower capacity steel H-piles, ie 2,800 kN instead of 2,950 kN for 305 x 305 x 180 kg/m steel H-piles. The use of lower capacity steel H-piles can result in shorter piles and facilitate the final set process as well as reducing the risk of damaging long piles due to prolonged driving. Since the cost and time incurred as a result of damaged piles, removal of damaged piles and re-design of pile caps outweighed the additional cost due to increase in number of piles, therefore, there is an overall benefit in cost and time though the number of steel H-piles was increased to 385. The theoretical safe loading capacities of piles for each zone are shown in Table 3.

The loading test results of piles for Zone 2, 2A, 3, 4 and 4A as summarised in Table 3 all satisfied the contract specification.

Zone No	Theoretical Safe Loading Capacity (kN)	No of Pile
1	3,600	64
2	3,600	97
2A	3,600	6
3	3,450	77
4	2,950	132
4A	2,800	9
	Total	385

Table 3 – Six Zones of Piling Construction

Underground Obstructions

Obstructions were observed in some of the ground investigation boreholes. While driving the steel H-piles, a large amount of obstruction was encountered which caused damage of piles. The total number of damaged piles was 76. There were no obvious signs before the piles were damaged, even though the Contractor monitored the driving using PDA. During the installation of trial piles in difficult ground conditions, it may be worth using inclinometer reservation channels welded to the steel H-pile profiles in order to monitor deviations. Some damaged piles are shown in Fig 6. Figs 7 and 8 show extraction of damaged piles using vibration clamp and hydraulic jack methods respectively. Pre-boring using Odex method with temporary casing of 610 mm diameter was adopted to penetrate through obstructions. The pre-bored hole was backfilled with 10 mm aggregate and sand before driving steel H-pile. However, the Contractor reported that the piles would easily hit the boulder again at the pre-bored level. They revised their method by installing the steel H-piles at the pre-bored level before backfilling the hole with aggregate and sand. After backfilling, the steel H-piles were driven to final set and PDA tests were carried out. Five pre-boring rigs were used to pre-bore the obstructions and the total number of pre-bored piles was 193 at an average depth of 40 m.

Founding Stratum

The average embedded pile length for this site is 60.6 m with the maximum length of 79.0 m. Most of the steel H-piles were founded on

Figure 6 – Photo of Some Damaged Piles

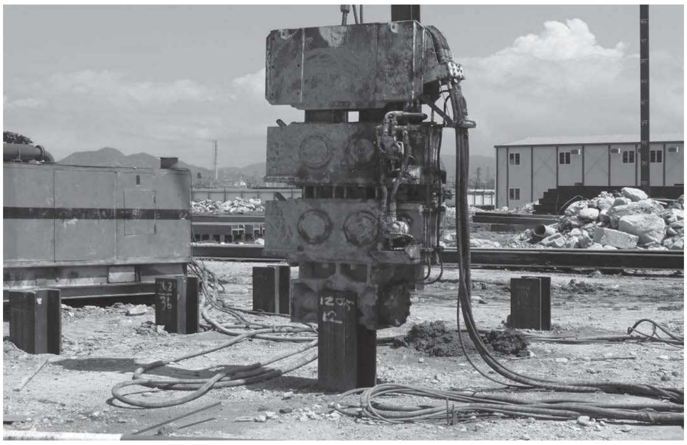


Figure 7 – Extraction of H-pile by Vibration Clamp

Figure 8 – Extraction of H-pile by Hydraulic Jack

bearing strata with N values greater than 200. The depth of penetration into N > 200 zone of some piles is as much as 15 m. This agrees with the findings of Zhang and Dasaka (2010). Therefore, the rule of thumb used by many engineers that steel H-piles could be founded on strata with three consecutive N values greater than 200 may not be applicable especially for 223 kg/m steel H-piles founded on soils with weak top layers or if pre-boring is required.

Observations

(i) PDA test and CAPWAP analysis are valuable tools used for field control and pile capacity assessment especially when the piles are long, resulting in out of range of final set table.

(ii) In the situation where the bearing strata are deep and the risk of damaging piles while driving is high, using piles with less pile capacity can result in shorter piles which will reduce the risk of damaging long piles, facilitate the final set process and shorten installation time.

(iii) Whenever using CAPWAP (same for the other dynamic formulae) to predict pile capacity, the set per blow and the timing of both dynamic restrike test and static loading test relative to the end of driving should be taken into consideration as pile capacity may not be fully mobilised if set per blow is small. Also, an increase in the pile capacity may occur after PDA test due to soil set-up. Therefore, when piles having similar CAPWAP capacities are considered for loading test, the pile with the largest set per blow should be chosen for test.

(iv) The set-up effect of piles for this site is significant. It can also be taken into consideration in pile design.

(v) Steel H-piles may not be founded on strata with three consecutive N values greater than 200 especially for 223 kg/m steel H-piles founded on soils with weak top layers or if pre-boring is required.

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Appendix A

Correlation of Parameters for the Hiley Formula

Pile No	$C_p + C_q$ (mm)	Set/ 10 Blows (mm)	Pile Length (m)	CAPWAP Capacity (kN)	Hiley Capacity for E_h = 0.93, e = 0.65 (kN)
DC9-2	54	16	52.0	8,131	6,681
C3F-1	56	3	65.0	7,529	6,517
C6H-2	59	3	66.9	7,234	6,185
C8A-1	54	24	67.0	7,205	6,262
C9G-1	56	27	82.3	7,177	5,810
CAP4-5	55	25	67.0	7,159	6,146
C10D-3	55	34	60.3	7,153	6,083
C11G-1	56	36	72.2	6,851	5,786
			Average =	7,305	6,184
				7,305 x 0.85 = 6,209 > 6,184	

C_c = 5 mm for Plastic Cushion
20t Hammer Weight = 197.2 kN
Helmet Weight = 31.22 kN
Drop Height = 1.5 m

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Ir Li graduated from the Hong Kong Polytechnic in 1976. After working in the private sector for a number of years, he joined the then Public Works Department, the HKSAR Government and worked as Structural Engineer in the Architectural Office in 1982. Since then, he has worked his way up to his present rank of Assistant Director (Structural Engineering) of the Architectural Services Department. Ir Li has taken part in various building projects including a significant number of public hospitals. His main interests include foundations and structural concrete design.

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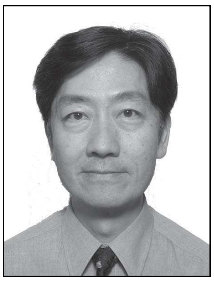

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