Investigation and Analysis of a Well Instrumented Small Displacement Pile Founded on Rock

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The construction of driven piles on rock has been practiced for years in Hong Kong. However, detailed field studies to investigate the performance of this type of foundation are still limited. A well instrumented driven H-pile was recently installed and founded on granite. The load transfer mechanism and the pile performance are studied based on comprehensive pile tests. Analyses show that the pile design is controlled by the structural capacity of the pile section but not the engineering properties of the bedrock. The pile toe resistance back-calculated from strain gauges is found to be considerably higher than the allowable values for bored pile design. The Hiley formula commonly used in Hong Kong is found to underestimate the ultimate pile capacity of the pile founded on rock. A more rational wave equation analysis is performed and shows consistent results with PDA tests and the CAPWAP analysis.

Keywords: Instrumented Pile, Load Transfer Mechanism, Rock, Wave Equation Analysis, Dynamic Formulae

Introduction

The use of driven piles as foundation systems is one of the common engineering practices in building developments and civil engineering works in Hong Kong. Among different types of driven pile, small displacement H-pile attains popularity because it is easier to handle and has better driving characteristics. The pile length is dependent on the ground conditions. Whenever the rockhead is shallow or the karstic surface exists, it is common to drive the pile to refusal on rock (Li et al. 1994). However, it lacks detailed field study and the behavior of this type of piles is still not well understood. Recently, a well instrumented driven H-pile (denoted as P1) was installed and founded on slightly decomposed granite. Comprehensive pile testing, including high strain dynamic tests (PDA Test) and slow maintained static loading test has been performed. These tests provide an unusual opportunity to study the load transfer mechanism and to evaluate the performance of driven piles founded on rock.

Site Location and Ground Condition

The site is located on the Kowloon Peninsula, to the east of Hong Hom Bypass. The ground condition and the uncorrected N values from the standard penetration test (SPT) measured in each soil stratum are shown in Figure 1. The site is on marine reclaimed land and the ground level is at approximately 4 m above principal datum (PD). The groundwater level is at about 3 m below the ground surface. The soil profile comprises approximately 12 m of fill material, 3 m of marine deposits and 9 m of weathered granitic saprolites overlying granite bedrock.

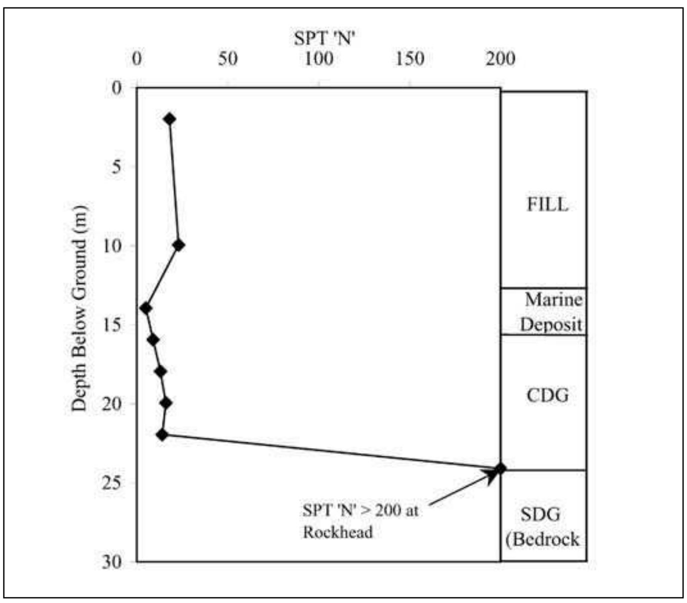

Figure 1 – Ground Condition

Pile Installation and Load Test Arrangement

A total of 14 vibrating wire type strain gauges (Geokon model VSM-4000) spaced at 7 levels was installed on P1, as shown in Figure 2. Two strain gauges were installed at each level. A protection box, consisting of a pair of 150 x 75 x 17 kg/channel, was used to cover each strain gauge mounted. Tapered steel shoes were welded to the end of the protective channels for the strain gauge assembly. Considering the added sections, the cross-sectional area of the pile becomes 27450 mm².

The pile was driven by a 10-ton hydraulic hammer with an energy rating of 180 kNm. High strain dynamic tests were performed at the end of pile installation with a 10-ton drop hammer according to ASTM (1995). A pair of strain transducers and a pair of accelerometers were bolted

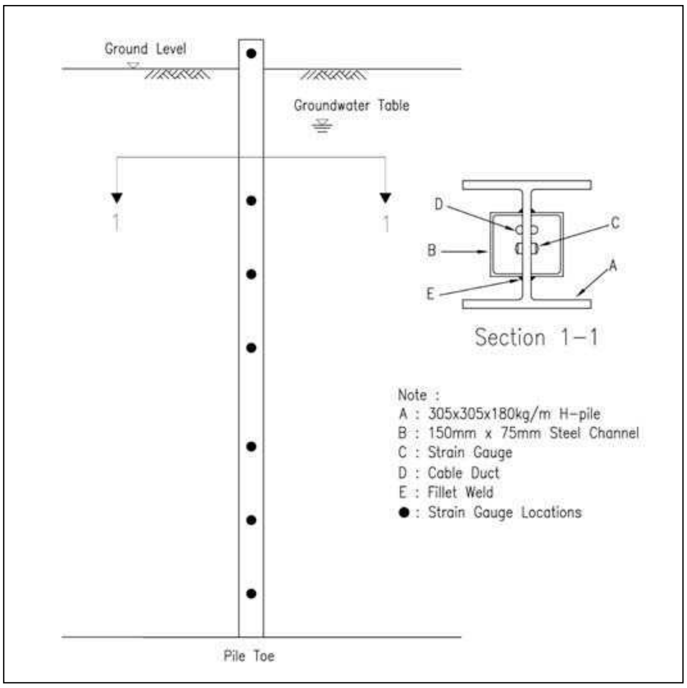

Figure 2 – Pile Instrumentation Details

1.5 m above the ground level below the pile head to measure the stress wave signals. The recorded signals were stored in the microcomputer of the Pile Driving Analyser (PDA) on site. The embedded pile length at final set was 24 m. A total of 50 blows were recorded. At the last 10 hammer blows, the measured transferred energy and Case's capacity showed more or less the same. A representative blow is chosen for CAPWAP analysis. The final set was 8 mm per 10 blows with the sum of temporary compressions of ground and the pile being 35 mm. The results of the dynamic tests are presented in Table 1. A static load test on P1 was carried out 3 days after the PDA test. A kentledge reaction system was adopted with testing arrangement generally followed BS8004 (BSI 1986). The load was applied in 3 cycles to 33% of the testing load (TL), 66%TL and 100%TL (9735 kN).

Test/ Analysis	Pile Capacity (kN)	Maximum Driving Stress (MPa)	Energy Transfer Ratio	Smith Shaft Damping (s/m)	Smith Toe Damping (s/m)	Shaft Quake (mm)	Toe Quake (mm)
PDA Test	9420 - 9470	360	0.7	N/A	N/A	N/A	N/A
CAPWAP Analysis	9450	348	N/A	0.03	0.85	4.4	3.7

Table 1 – Results for High Strain Dynamic Load Test and Relevant CAPWAP Analysis

Load Movement Relationship and Load Transfer From Static Loading Test

The load movement curve of the static loading test for P1 is plotted in Figure 3. Several common failure criteria of graphical offset type, ie BD (2002), HKHA (1998), ASD (1993) and BS8004 (BSI 1986), are applied to interpret the pile capacity. It is found that only the ASD failure criterion is reached giving a pile capacity of 6500 kN. Pile failure as defined by the other three failure criteria has not reached.

The load movement curve is still highly elastic with a very small residual settlement of 1 mm upon unloading from 9735 kN. Therefore, there still should be some reserve of the pile capacity above 9735 kN. According to the trend of the load movement curve, it is estimated that the pile capacity determined by the BD and HKHA failure criteria should be greater than 9735 kN with an upper bound of about 11900 kN, which is the structural capacity of the pile.

For comparison purpose, the load movement relationship of a driven H-pile (denoted as P2) from another site and one additional case (denoted as P3) reported by Li et al. (2001) are presented in Figures 4 and 5, respectively. Similar highly elastic load movement relationships with small

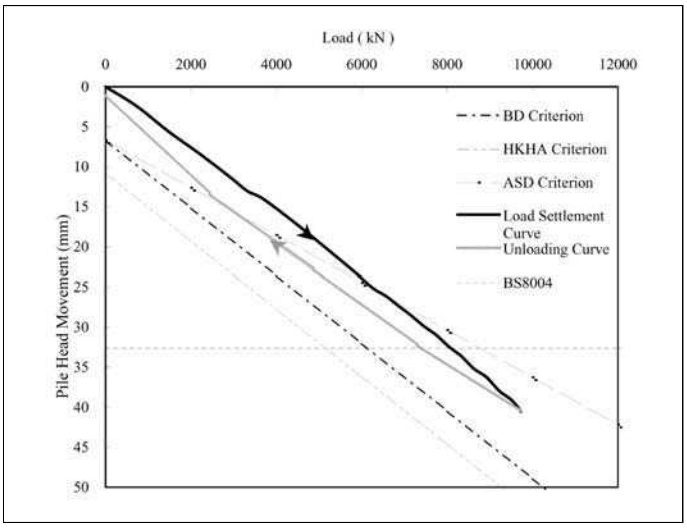

Figure 3 – Load Movement Relationship for P1

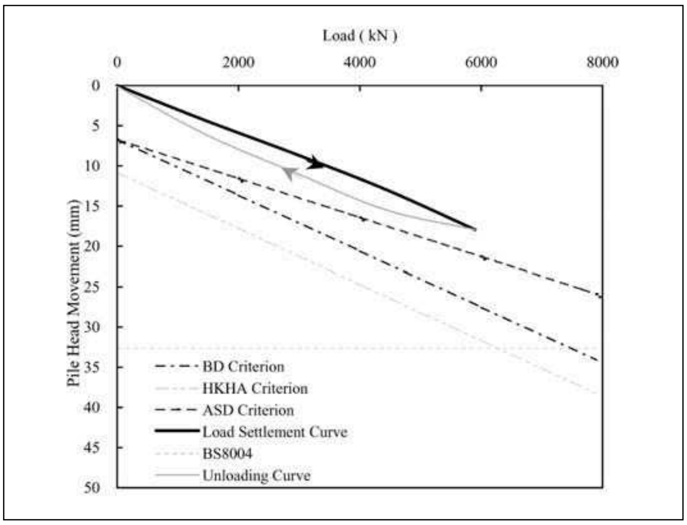

Figure 4 – Load Movement Relationship for P2

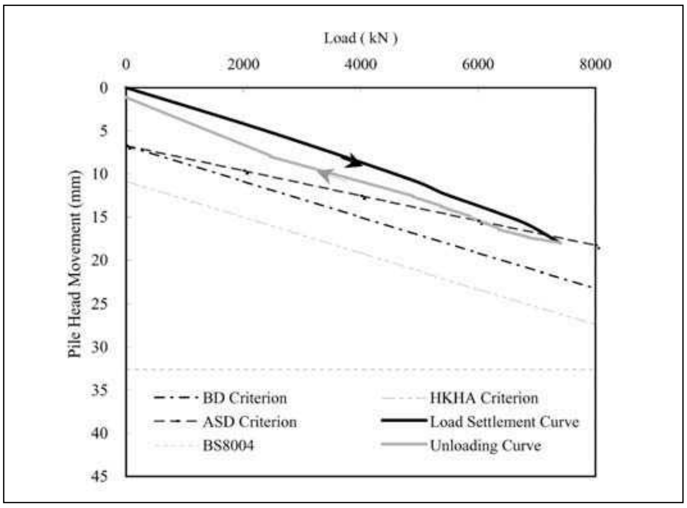

Figure 5 – Load Movement Relationship for P3

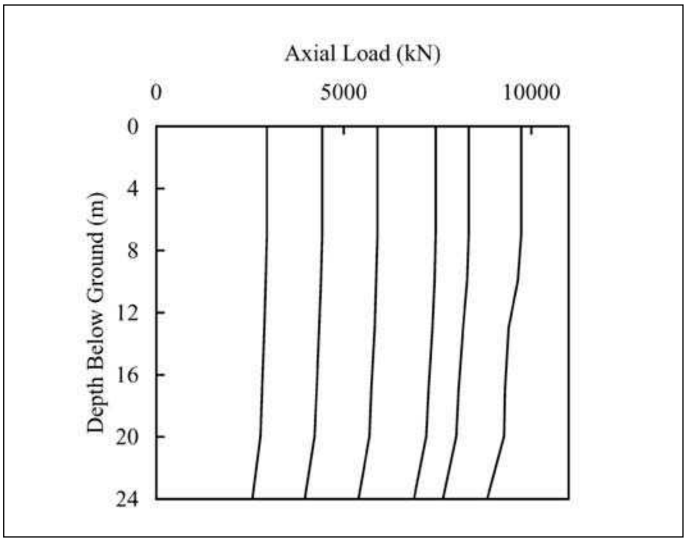

Figure 6 – Pile Load Distribution Curves

residual settlements are identified. The pile information and corresponding interpretations of these static loading tests are summarised in Table 2.

The load distributions along the pile P1 at different stages of loading are shown in Figure 6. It is found that the shaft resistance is generally over 80% of the total pile resistance after the applied load is greater than 1500 kN. At the 100% TL, the shaft resistance is about 905 kN.

Reference	Pile No	Pile Length (m)	Bearing Stratum	Maximum Testing Load (kN)	Residual Settlement (mm)	Pile Capacity			
						ASD[a] (1993)	HKHA[b] (1998)	BD[c] (2002)	10% Diameter[d] (BS8004 1986)
This Study	P1	24	Grade II Granite	9735	1.0	6500	N/A[f]	N/A[f]	8200
This Study	P2	17	Grade III Granite	5900	0.1	N/A[f]	N/A[f]	N/A[f]	N/A[f]
Li et.al.(2001)	P3	10	Rock[e]	7400	1.0	7200	N/A[f]	N/A[f]	N/A[f]
[a] 0.7PL/EA + d/120 + 4 mm [b] PL/EA + d/120 + 4 mm [c] PL/EA + d/30 [d] Diameter is taken as the least lateral dimension of the pile [e] Rock type and rock grade are not specified [f] Defined pile failure not reached [g] All piles are driven H-piles (305 x 305 x 180kg/m)									

Table 2 – Pile Information and Results of Static Loading Tests

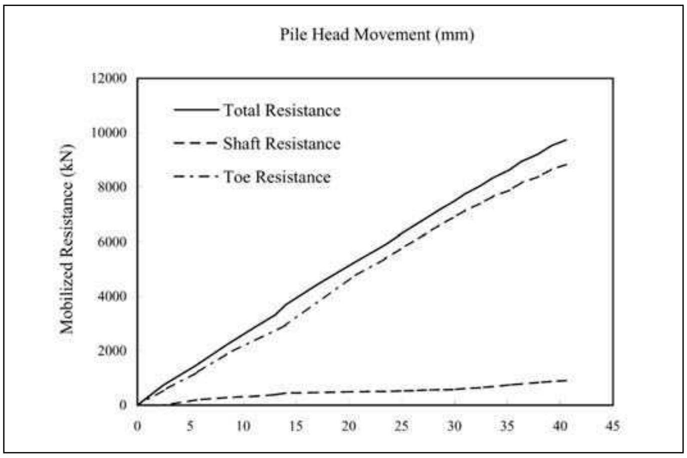

Figure 7 – Pile Resistance Contribution with Pile Head Movement

In turn, the toe resistance contributed by bedrock is 8830 kN, exceeding 90% of the total resistance. The corresponding pile toe movement at maximum test load is about 5.5 mm The mobilised pile resistance with pile head movement is shown in Figure 7. It clearly shows that the developed pile resistance with response to the applied load is mainly contributed by the toe resistance from the bedrock.

Static Analysis for Bearing Capacity of Driven Piles Founded on Rock

The results of the back-calculated maximum static compressive stresses at the pile toe on rock for P1, P2 and P3 at the maximum testing load are summarised in Table 3. It is found that the stress is the highest for P1, which is 322 MPa. This situation is not uncommon for driven piles on rock. From soil mechanics principles, the shaft resistance is proportional to the effective overburden stress. Longer piles will have a larger shaft resistance contribution. As P2 and P3 are relatively short and based on the experience of P1, it is assumed that 20% of the pile resistance is contributed by the shaft resistance. The equivalent compressive stresses at the pile toe are estimated to be 206 MPa and 258 MPa for P2 and P3, respectively.

Pile No.	Maximum Testing Load (kN)	Pile Toe Load (kN)	Maximum Compressive Stress on Rock (MPa)
P1	9735	8830	322
P2	5900	4720[a]	206[a]
P3	7400	5920[a]	258[a]
[a] Assume 20% pile capacity contributed by shaft resistance			

Table 3 – Maximum Compressive Stresses at Pile Toe

Rehnman and Brom (1971) performed an experimental study to investigate the toe resistance of piles in different types of rock including granite, limestone and sandstone. It was concluded that the ultimate unit toe resistance of rock is high and can exceed considerably the unconfined compressive strength (UCS) of the rock. This finding was consistent with the test results on gnesis performed by Bergfelt (1958). The following empirical correlations between bearing capacity (qu) and unconfined compressive strength (UCS) were suggested:

qu = 4~6 UCS (Rehenman & Brom 1972) (1a)

qu= 5 UCS (Bergfelt 1958) (1b)

The engineering properties of Hong Kong granite have been studied by several researchers. GEO (1993) reported a typical range of UCS of Grade II rock in Hong Kong as 100 ~ 200 MPa with granite inclusive. Hope et al.(2000) reported a range of UCS for Grade II granite as 75~200 MPa.

Based on equations (1a) and (1b), the bearing capacity of the slightly decomposed granite in P1 can range from 300 MPa to 1200 MPa. This range of values is much higher than the allowable bearing pressure of 5 MPa to 7.5 MPa (BD 1995) commonly used in bored pile design in Hong Kong. Moreover, this range of bearing capacity is comparative to the yield strength (fy) of steel H-piles, which is 430 MPa in case of Grade 55 305 x 305 x 180 kg/m steel H-pile. It implies a geotechnical pile capacity from 8300 kN to 33200 kN. Compared with the limiting structural capacity 11900 of P1, the bedrock should have adequate bearing capacity without being overstressed when high loading is transferred from the pile.

It is noted that the allowable pile capacity by adopting 0.3 fy is 3570 kN. The evaluated factor of safety based on the structural approach is at least 2.7 for pile P1. It implies that the pile design is controlled by structural capacity of the pile section rather than the engineering properties of the bedrock and it is feasible to utilise a higher allowable strength of the H-pile sections.

Performance of Dynamic Formulae

Hiley's formula is widely used in Hong Kong for determining the pile capacity. The formula estimates the capacity of driven piles taking into account the efficiency of driving hammer, the pile set and temporary compressions of the ground, the pile and the helmet:

$$R = \frac{k_1 k_2 W H}{(s + c/2)} \qquad (2)$$

Where R = ultimate resistance of the soil to penetration by the pile
k1 = mechanical efficiency of hammer
k2 = efficiency of hammer blow
W = weight of hammer
H = drop distance of hammer
S = set
c = sum of temporary elastic compressions of the helmet, the ground and the pile

Despite it is widely used in Hong Kong, Hiley's formula suffered from inherent problems (GEO 1996). These problems include its poor representation of the driving system and system energy losses and neglecting pile axial stiffness. Fraser et al. (1990) and Yiu et al. (1990) reported case histories in which the pile capacities are underestimated by the Hiley formula while HKCA (1995), on the other hand, reported case histories of overestimations. With a hammer efficiency ranging from 75% to 90%, the Hiley's formula gives pile capacity values between 5450 kN and 6150 kN.

Based on energy measurements from a PDA test, Broms and Lim (1988) proposed a modified dynamic formula as follows:

$$R = \frac{k_3 \, W \, H}{(s + c/2)} \tag{3}$$

where k3 = overall energy transferred ratio that can be determined from measured stress wave signals during a PDA test

This modified formula, or called measured energy method by Paikowsky et al. (1994), eliminates the uncertainty in assessing the transferred energy to a certain extent. Given a measured k3 value of 0.7 for P1, the pile capacity predicted by equation (3) is 7820 kN. Hence, the modified formula, even after calibration with stress wave measurements, underestimates the pile capacity for the driven pile founded on rock.

Allowable Stress of Steel H-pile

It is observed from the test results that a small displacement steel H-pile founded on rock can develop a very high capacity. According to the results of PDA tests on P1 carried out 3 days before the static loading test, the maximum compressive stress developed along P1 during hard driving with a set of 8 mm/10 blows is 360 MPa. This value accounts for 85% of the yield strength of grade 55 C steel. According to the author's experience, the maximum driving stress up to 90% fy is common.

From the static loading test results, it is shown that the static stress of P1 in this study is 322 MPa at the maximum test load. At this load level, the load displacement curve is still elastic. It demonstrates that a steel H-pile that has experienced a very high driving stress below its material yield strength still performs very well. It agrees with the findings of Farid (1979). Farid carried out an experimental study on the effect of high impact stress on the structure of 254 x 254 x 63 kg/m piles during hard driving. The metallurgical examination on the part of the pile most affected by the impact was performed. It is found that the expected localised plastic deformation was only confined to the top layer of several millimeters thick. The microstructure below the top layer appeared to be normal. Therefore, the structural strength of the pile section may not be depreciated by high driving stress.

Wave Equation Analysis

Wave equation analyses utilise the one-dimensional wave equation to estimate pile stresses and the pile capacity. Smith (1960) developed the first computer solution of the wave equation. In the solution, the driving equipment, pile and soil resistance are modeled as a discrete set of masses, springs and viscous dashpots. It is capable of evaluating the effect of change in pile properties or pile driving system. The use of wave equation analyses to study the driven pile behavior in Hong Kong has been reported by Makredes et al. (1982) and others. The approach combining the use of PDA tests and wave equation analyses has been successfully used in installing steel pipe piles in a public housing project.

A wave equation analysis using the computer program GRLWEAP 2003 (GRL. 2003) is performed to analyse P1. The widely used Smith's model is adopted. The shaft resistance distribution from the strain gauge measurements is used. The toe resistance is evaluated using the static analysis based on soil mechanics principles. The quakes and damping constants are according to soil types as recommended by GRL. (2003).

From the results of analysis, the estimated pile capacity is approximately 8600 kN and the maximum driving stress is 400 MPa, which are consistent with the results of PDA tests and CAPWAP analysis. It is noticed that the CAPWAP capacity is the pile capacity being mobilised under a certain hammer blow. A larger hammer or a more efficient hammer blow may activate a higher pile capacity. Both the pile capacity predictions made by GRLWEAP and CAPWAP are underestimations when making reference to the results of static loading test as discussed before. As indicated by Choi et al. (1982), Ng et al. (2000) and Zhang et al. (2003), the correct interpretation of the results requires knowledge on wave equation mechanics and pile dynamics and is subject to engineering judgement. The prediction accuracy may improve by adjusting more reasonable wave equation parameters.

The predicted pile penetration resistance from the driveability analysis is shown in Figure 8. As the hammer used in the PDA test differed from the hammer adopted in pile driving, there is a certain degree of deviation between the observed and predicted pile penetration resistance. Such deviation may be also due partly to the assumed wave equation parameters.

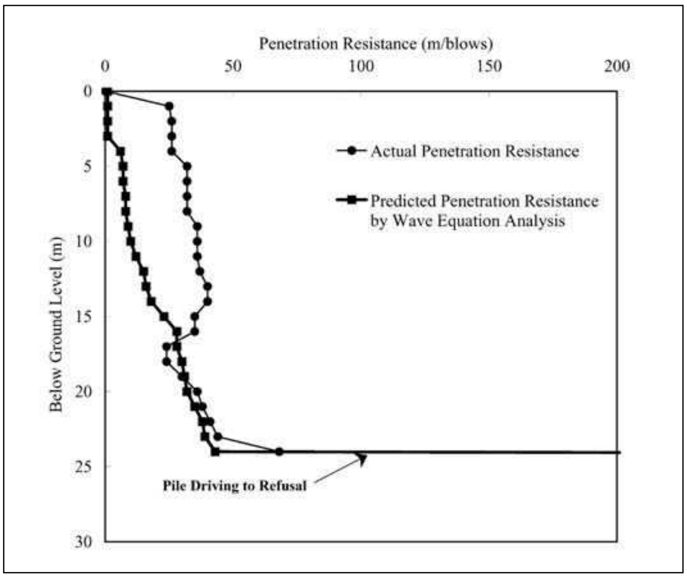

Figure 8 – Comparison between Observed and Predicted Pile Penetration Resistance

Conclusions

A case history of a well instrumented driven H-pile founded on rock is investigated. The site geology and comprehensive pile tests provide an opportunity to study the behavior of the small displacement pile founded on rock. Several conclusions can be drawn based on the study:

1. From the field test results, 90% capacity of the pile is contributed by the toe resistance. A large portion of the toe resistance can be mobilised within a toe displacement of several millimeters and the load movement relationship is generally linear.
2. The load tests together with reported case histories demonstrate that design of steel H-piles penetrating into Hong Kong granite, particularly when the bedrock is shallow, is controlled by structural capacity of the pile section rather than the engineering properties of the bedrock. The toe resistance obtained from the tests is greater than those for bored pile design. It may be possible to utilise a larger portion of the strength of the H-pile sections.

3. It is shown that the Hiley formula, even after calibration against stress wave measurement, underestimates the pile capacity considerably in this case history.
4. A more rational wave equation analysis by the application of the programme GRLWEAP is used to analyse the pile performance. Three aspects of results are evaluated. The predicted pile capacity and maximum driving stress are generally consistent with those from high strain dynamic tests and CAPWAP analyses.

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