Estimation of driven pile capacity in soft soil based on limited geotechnical data: A case study

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Abstract. The lack of qualified geotechnical data for the foundation design process is a present and growing concern among the consultancy practitioners. The clients have been increasingly reluctant to commit to extensive but necessary site investigative program that has no immediate economic benefits or perceived to impede work progress. Consequently, the risk of foundation failure and over-design are a potential reality. Even so, the construction pace is unrelenting. The research attempted to address such a quandary via a project case study with not too dissimilar limitation with respect to the availability of geotechnical data. Selecting the best available data for the type of foundation selected, further analyses were carried out in order to obtain collaborative strength information with the appropriate application of the factor of safety. Next, selection of the governing value from among the derived strength parameter had been based on the currency and accuracy of the testing process. Finally, provisional qualifiers had been attached to the design proposals in case alternate scenarios were encountered during the implementation stage. In the context of the case study, a reasonable driven pile capacity of 15.55 tonnes as determined from the PDA test has been recommended contingent upon the prescribed project scale and ground conditions.

1. Introduction
Geotechnical failure is not an exception to the construction industry but almost akin to a rule somewhat. In fact, the Public Work Department (PWD) had carried out many geotechnical forensic investigation involving varying contributing factors from vibration, soil erosion, foundation failures, retaining structure failures, slope stability, ground settlement, water infiltration to collapsible soil with close to 252 cases for 2010-2015 period [1]. Furthermore, the probability of failures cannot simply be waived with detailed ground investigations, sophisticated calculations, strict site supervision and monitoring, as soils and rocks are complex engineering materials with properties and parameters that are commonly non-linear in behaviour, unique or constant [2]. In many cases, failures do not occur because of one deficiency but rather a misfortunate combination of factors, which adds to the difficulties in its prognosis [3].

Nevertheless, failure to plan is tantamount to planning to fail. If geotechnical failures were to occur, the extent of the calamity could be exacerbated without fore-knowledge of the ground condition made available through site investigation works. One of the greatest causes of foundation failure is due to insufficient knowledge of ground conditions leading to significant cost overruns and time delays for both client and contractor. As such, the role of site investigation is precisely that to reduce the
uncertainty of ground conditions by various combinations of field and laboratory testing [4]. When data is sufficient, safe and optimal foundation design could be produced; lacking such, potentially risky and/or expensive design undertaking could be meted out.

However, economic as well as progress concerns usually force the issue. The foundation designers are forced to work with limited, irrelevant or sometimes sub-standard geotechnical data. It is the premise of this research to explore the possibility and naturally limitations of producing practical foundation design using limited but prequalified geotechnical data. For this purpose, a project case study had been engaged and all available information as they may had been were obtained through site reconnaissance, documentary review, main contractor’s site reports and specialist contractors’ test reports.

2. The Case Study
The proceeding discussions detail out information on the case study which is available at the time of research. It would reveal the extent of informational scarcity so as to hinder conventional approach in geotechnical assessment and process for the foundation design.

2.1. Project site
The project involved a two-storey school at site in Penang, Malaysia which commenced on March 2019 with the foundation works. According to the Geological Society of Malaysia, the geological bedrock formation of the site consists of sedimentary rock of shale, mudstone, siltstone, phyllite, slate or hornfels with marine and continental deposits of clay, silt, sand, peat or minor gravels near the surface. Through site reconnaissance, it was discovered that the site was a paddy field which was later backfilled and converted into flat palm oil plantation ground for years before the current development. As a result, the upper ground strata were formed of a mixture of fibrous organic and fine/granular soils matters, and the lower strata of primarily fine soil material. It further suggested of high potency for ground compressibility and material decay under the imposed loads. A suitable extensive soil exploration program involving sampling, and field and laboratory testing were also prescribed as a result. However, due to cost constraint it was settled upon a singular JKR probe testing program. This compromise although unsatisfactory would however allow for some measure of foundation selection between shallow and deep types which would evidently need to be further collaborated with more rigorous test later on. Figure 1 shows the project site layout and the six test locations referenced as BH1 to BH6.

2.2. JKR probe test
The test was based on principle laid out by Huarslev (1948) for drive rods for sounding and sampling, and recommendation by the European Group Subcommittee (1968) for static and dynamic soundings. In the project context, it consisted of screwing a cone penetrometer to the lower end of approximately 120 cm long steel rod and a hammer to its upper end. Several intervals of 300 mm were measured and marked on the rod from the tip of the cone up to as close as possible to its top. Then, the assembly was carried to the testing spot and set up vertically, and the hammer was pulled to its maximum reach and allowed to fall freely to drive the cone and rod into the ground. The number of blows required for each 300 mm penetration was recorded. The test should be ceased upon reaching 400 blows per 300 mm penetration or 15 m deep penetration whichever comes first. Of the two manually operated probe variants commonly employed in Malaysian practice, the JKR variant which was used in the test consisted of a 25 mm diameter cone penetrometer with a 60° pointing angle and a 5 kg hammer freely dropped from a height of 280 mm resulting in 27,972 Nm/m² of impact energy.

Six numbers of JKR probe tests were carried out at site. Five locations were on or close to the periphery of the building representing foundations supporting columns of relatively low load intensity while a location at the centre of the building representing foundations supporting columns of relatively high load intensity. Figure 2 shows the results of the prescribed JKR probe tests. High blow counts of 70 and above were recorded as deep as 0.6 m from the surface but beyond this and up to a depth of
roughly 6 m the blow counts swerved to 30-70 range. However beyond this point, the blow count indicates satisfactory and consistent blow counts at well over 70 and approaching the threshold of 400 at various depths from 8.1 m to 9.3 m. No other soil information beyond the stated strength parameter was recorded during the test with preclude among others determination including subsurface soil classification. Interpretation of the results may require a bit of judgement and contingent upon the realization of the following mitigating factors: human errors, low impact energy and resistance to penetration from seemingly localized obstacles. In this context, some explanations shall be attempted here by classifying the depth into shallow, intermediate and deep regions based on the aforementioned blow count characterization. At shallow region where relatively high blow counts were recorded, this could be due to prior activities on the surface resulting in limited and intentional or unintentional compaction effort and effect which did not theoretically translate deep into the ground. On the other hand, at the intermediate region where erratic blow counts were observed, the ground could be disturbed in situ material close to the surface or even inadequately compacted backfilled material. Lastly, at the deep region where a more consistent blow counts were anticipated, this was due to natural in situ ground or the presence of persistent overburden mass bearing upon undisturbed layer over the millennia. Naturally, this layer could potentially accord the best founding support but at such a depth only deep foundation type would be practical. The shallow layer would only provide superficial founding strength as it masked the more determining mass of the intermediate layer with its inconsistent load-carrying capability.

![Figure 1. The project site and the JKR probe test location](image1.png)

![Figure 2. The results of JKR probe tests](image2.png)

2.3. Selection and installation of foundation
The preceding discussion alludes to the selection of deep foundation type as opposed to the shallow ones being the more practical all-round solution for the project in question. Notwithstanding the
difficulty of making a practical piling recommendation in absence of a more comprehensive soil investigation data, a prefabricated reinforced concrete square pile was selected for installation based solely on technical judgement having the parameter prescribed in table 1. A 12 m long square pile of 200×200 mm dimension which is limited to a tentative working load of 30 tonnes had been used for the project and the resulting pile group formation of 1 to 4 piles had been capped with reinforced concrete pile cap to provide adequate seating for the building stumps.

In preparation for driving, the pile segment was hoisted to an upright position using crane and placed into the lead of the pile driver in order to deliver a concentric blow to the pile upon each impact. A pile cushion consisting of wood, metal or composite material was placed between the pile and the hammer to reduce pile stresses during driving. For driving the pile into the ground, a single acting hydraulic impact hammer with the energy rating of 67.8 kNm was engaged consisting hydraulic actuator and pump to retract the heavy ram weight to the top of the stroke, and released to freefall under the gravity, striking the anvil and thereafter imparted the energy to the pile head causing pile penetration. Several intervals of 300 mm were measured and marked on the pile beforehand from the base of the pile up to as close as possible to its top. Pile driving record was kept of the number hammer blows required to cause 300 mm penetration. Pile jointing to subsequent pile segment was established through perimeter butt welding and the prescribed processes were repeated until the required embedment depth was attained. The whole processes were completed within half an hour.

| Items                                 | Values  |
|---------------------------------------|---------|
| Nominal pile size (in mm)             | 200×200 |
| Manufactured length (in m)            | 6       |
| Embedment length (in m)               | 12      |
| Manufactured working load (in tonnes) | 45      |
| Allowable working load (in tonnes)    | 30      |

As arbitrary as it might have seemed with the respect to the pile selection process thus far, the pile driving record may provide an avenue for estimating the pile axial capacity for the first time albeit through simple dynamic formula that empirically relates energy delivered by pile hammer to the energy absorbed during pile penetration. The Hiley (1930) equation was selected in this context for the determination of the dynamic pile capacity as prescribed by Bowles [5].

Given the weight of hammer and pile as 3 tonnes and 1.2 tonnes respectively, pile penetration into pile cap of 0.1 m, hammer drop height of 90 cm and the blow per 300 mm penetration at set of around 10, through iterative process the ultimate axial pile capacity was estimated at 51.93 tonnes. Applying a Factor of Safety (FOS) of 2.0, the allowable axial pile capacity was 25.97 tonnes. Further discussion on the selected FOS shall be made on subsequent section.

2.4. Pile Driving Analyzer (PDA) test
The PDA test was first developed in a Case Project at Case Institute of Technology (now Case Western Reserve University). In the project context, it consisted of attaching two sets of sensors namely a strain gauge and an accelerometer on diametrically opposite sides of an installed pile above the surface (Figure 3), applying successive hammer impacts on the pile head (at 3 tonne hammer weight and 0.15 m hammer stroke) (Figure 4), and recording strain and acceleration signals (Figure 5). The force-time signals were then analysed using the Case Method to arrive at an estimated ultimate static capacity. A more rigorous CAse Pile Wave Analysis Program (CAPWAP) analysis involving iterative curve-fitting technique in which force-time signal response from a wave equation model is matched against the measured signal response from a single hammer blow was also carried. It allowed
for the determination of ultimate static and dynamic resistances through an idealized elastoplastic soil model (in which the quake parameter defines displacement for elastic-to-plastic transformation), and a viscous damping model (which is the function of a damping parameter and the velocity) respectively [6].

The PDA were performed on two critical installed pile locations deemed as such on the account of the projected higher load concentration as obtained from the results of prior super-structural modelling and analysis. These locations are indicated on the structural layout of Figure 6 and referenced as D1/1 and D3/4. To allow for convenient sensor attachment, a 3 m extension segment was added to the original 12 m long installed pile; thus somewhat extending the embedment length and may not quite represent the surrounding pile condition. Since the pile had been driven days ahead of the actual test, it can be construed that the test was performed as a re-drive or restrike. Figure 7 shows the simulated load vs. settlement curves for both test locations. The ultimate capacities obtained from Case Method and CAPWAP analyses are summarized in table 2. The pile integrity was found to be satisfactory due to no indicative early velocity wave reflection from damaged parts apart from that of toe and/or joint detections. In addition, both tests had also shown that at the pile set values of 7.6 mm/blow and 2.5 mm/blow of the hammer energy were sufficient to completely mobilize the soil resistance. This is collaborated by prior researches in which blow counts in excess of 10 blows per 2.5 cm may not cause enough displacement to fully mobilize the soil resistance [7], [8]. The range of shaft quake, toe quake, shaft damping factor and toe damping factor values from the CAPWAP analysis was 2.744-2.937 mm, 4.282-4.352 mm, 0.544-0.585 s/m and 1.073-1.097 s/m respectively, which were acceptable. Test ref. D1/1 and D3/4 experienced maximum hammer energy of 0.32 tonne-m and 0.30 tonne-m respectively resulting in 11.8 MPa and 12.4 MPa of maximum compressive driving stress each. The total mobilised ultimate capacities for test ref. D1/1 were 32 tonnes and 31.1 tonnes (of which significant contributor was the shaft friction at 24.9 tonnes) for PDA and CAPWAP analyses respectively, and for test ref. D3/4 were 45 tonnes and 43.2 tonnes (once again the significant contributor was the shaft friction at 33.9 tonnes) for PDA and CAPWAP analyses respectively.

Figure 3. Installation of strain gauge and an accelerometer on diametrically opposite sides

Figure 4. Successive application of hammer impacts

Figure 5. PDA recording strain and acceleration signals
Figure 6. The structural layout of the two-storey school and the two location of the PDA marked as D1/1 and D3/4

Figure 7. Simulated load vs. settlement curve for D1/1 and D3/4

Table 2. Summary of PDA and CAPWAP analysis results

| Test Ref. | Pile Set (mm/blow) | Max. Comp. Stress (MPa) | Max. Energy (ton-m) | Case Method Ultimate Capacity (ton) | CAPWAP Ultimate Capacity (ton) |
|-----------|--------------------|-------------------------|--------------------|-----------------------------------|-------------------------------|
| D1/1      | 7.6                | 11.8                    | 0.32               | 32                                | 24.9                          |
| D3/4      | 2.5                | 12.4                    | 0.30               | 45                                | 33.9                          |

2.5. Comparison between Hiley’s formula, PDA analysis and CAPWAP analysis results

At this juncture, perhaps it would be useful to bring all the findings together for the purpose of comparison, re-evaluation and arriving at a more generic outcome. For this purpose, the lower strength value from the PDA and CAPWAP analyses shall be used for practical reason. In addition, it would be prudent to apply a FOS to the ultimate values to account for parametric uncertainties and design assumptions. In practice, FOS values of 2 and 3 have been used for skin friction and end-bearing components respectively where sufficient geotechnical information are available. In addition, it is further recommended that an overall FOS of 2 to 3 be applied for small scale construction covering relatively limited land area which is not anticipated to differ significantly from the known surrounding ground condition. Moreover, it is also applied to account for the transitory nature of the soft soil over time under imposed loads. For simplicity, a blanket FOS value of 2.0 had been selected for all cases but further consideration might be required when circumstances are more complex than those prescribed.

Table 3 indicates the comparative ultimate as well as allowable pile capacity values as obtained from Hiley’s formula, Case Method analysis and CAPWAP analysis. The Hiley’s formula indicated the highest values at 51.93 tonnes and 25.97 tonnes for ultimate and allowable capacity respectively, followed by the Case Method analysis at 32 tonnes and 16 tonnes respectively, and lastly the CAPWAP analysis at 31.1 tonnes and 15.55 tonnes respectively. Although the difference between that of PDA and CAPWAP analyses for all intents and purposes immaterial, we would be hard pressed to
gloss over their differences to that of Hiley’s as determined from the piling record, as opposed to wave propagation record on pre-installed pile for the former. To account for these differences, the context in which the test was performed must be stipulated namely the actual construction stage in the project lifecycle. The piling record was obtained during the piling installation process in which penetration was made to the ground in its natural state. If the fine-grained constituents (clays, silts and even fine sands) were dominant, full scale resistance to the pile penetration could be mobilized through soil remoulding and excess pore water pressure build-up; thus giving the impression of elevated but transient soil strength. On the other hand, the PDA test was carried out days following the pile installation process where soil relaxation and excess pore water pressure dissipation might have occurred. To effectively mobilize the effective soil stresses on the pile, some amount of ground deformation under prolonged and sustained load must have occurred first which was not the case at the time of the latter test. So, it can be construed that the PDA test was carried out close to the service condition and as such more realistically representative of the pile behaviour. As such, taking the smaller CAPWAP value of 15.55 would be more prudent considering the construction scale and ground condition at the time of test.

Table 3. Pile capacity comparison between Hiley’s formula, Case Method analysis and CAPWAP analysis

| Hiley’s Formula | Case Method Analysis | CAPWAP Analysis |
|-----------------|----------------------|-----------------|
| Ultimate Values (ton) | Allowable Values (ton) | Ultimate Values (ton) | Allowable Values (ton) | Ultimate Values (ton) | Allowable Values (ton) |
| 51.93 | 2.0 | 25.97 | 32.00 | 2.0 | 16.00 | 31.10 | 2.0 | 15.55 |

3. Lesson Learned

The conventional approach to soil investigation can be readapted to some extent to address crucial informational gaps in foundation design process even with financial constraint. This includes the following as illustrated in Figure 8:

- Desk study – It essentially involves collection of extant preliminary information including formal documents, technical drawings, reports, etc. in order to obtain projected geotechnical and geological information on the site and adjoining areas which are suggestive of the condition of the site. This step is easy to accomplish with limited monetary overhead.

- Site reconnaissance – It essentially involves visual inspection of the site and surrounding areas from the surface and close to the surface in order to collaborate some of the documentary evidences in the previous step, obtain undocumented site information and plan for the soil investigation program in the next step. This is also performable.

- Soil investigation program – It essentially involves soil exploration, sampling, field and laboratory testing, etc. in order to obtain real geotechnical and geological information on the site and in the event of data scarcity serve as a framework to interpolate and extrapolate other information relating to the site. With constraint, JKR probe test or equivalent which measures instantaneous soil strength parameter is recommended for foundation selection and preliminary strength parameter in the case of shallow types.

- Analysis of foundation installation record – It essentially involves using empirical formula in order to determine preliminary strength parameter of the foundation when such options are intrinsically available such as for piles. This is also performable with little additional cost but must be explicitly spelled out in the contractual terms of reference.

- Performance testing – It essentially involves plate load test or equivalent for shallow foundation, and static load test PDA test or equivalent for deep foundation in order to
collaborate or reassess the preliminary strength parameter. With constraint, one of the appropriate test would do over pre-selected locations that denote singular criticality such as load concentration.

![Diagram](image-url)

Figure 8. The process of foundation analysis and design with limited geotechnical data

4. Conclusions
Limited but relevant geotechnical data may assist in determining the foundation type and capacity. However, such design proposals must be accompanied with provisional qualifiers. In the context of the case study, a reasonable driven pile capacity of 15.55 tonnes as determined from the PDA test had been recommended contingent upon the prescribed project scale and ground conditions.

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