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Fallacy of Capacity Performance & Innovation Improvement of Jack-In Piling in Malaysia

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ABSTRACT: Jack-in piling is gaining rapid acceptance in Malaysia piling industry with its commonly recognised advantages of proof loading every single pile installed, quick installation process and high pile capacity performance. However, this piling system is still subjected to the problem of large soil displacement inducing lateral and vertical movements of earlier installed piles, premature refusal to penetration of pile due to intermittent obstruction and also inadequate pile embedment due to shallow end bearing stratum. Hence, pre-boring technique with or without infill is used to overcome the obstruction problem and to ensure adequate pile embedment. The proof loading pile termination criteria appear to produce favourable pile performance and quality assurance. There is inherent long-term performance deterioration associated with shallow embedment of end bearing piles and incomparable short-term and long-term toe resistances, particularly in meta-sedimentary formation, which is prone to stress relief softening effect. Innovation of improving the installation process using customised recording device to guide the pile installation and pile termination has proven good quality assurance of pile installation and improved productivity. This paper presents the misconceptions of this high capacity jack-in piles, solutions and some proven innovative improvements

Keywords: Jack-in pile, Proof loading, Preboring, Meta-sedimentary formation, Stress relief, Toe softening

1. INTRODUCTION

Displacement pile is usually the initial option that most designers have in mind whenever deep foundation is required for transmitting the imposed building load to the hard stratum beyond practical reachable depth of shallow foundation. Preference of displacement pile compared to bored cast-in-situ pile foundation is mainly to due to cost effectiveness. Installation of displacement piles using jack-in method is getting more popular and has higher demand in the piling industry compared to driven pile method, particularly for new developments in centre of city. Jack-in pile allows the piles to be installed without generating excessive nuisance to the neighbours. Figure 1 shows the typical jack-in rig and Figure 2 shows the plan view layout of the jack-in rig with provision of side jacking capability. The advantages and disadvantages of the jack-in piling system are briefly summarised below:

Advantages:

During jack-in pile installation, all piles are structurally proof loaded with compressive stress throughout the pile body. There is literally no high surging tensile or compressive stresses generated as in the percussion driving stressing the pile, particularly the potentially workmanship weakness at joint welding and connection weakness at the pile main reinforcement and the end plate under tensile stress, and compressive crushing for concrete. Besides, the pile penetration corresponding to the applied jack-in pressure enables continuous profile of jacking resistance to be recorded for construction quality control and review of potential problems encountered during pile installation, which is similar as cone penetration test revealing the ground condition in continuous manner.

Disadvantages:

However, jack-in machine requires massive reaction kentledge rig and clumsy rig movement that requires stable working platform or else generating excessive lateral spreading ground movements distressing the installed piles. All installed piles need to be initially cut below working platform for rig movement during pile installation and second round cutting is necessary for pile cap connection, thus requiring two stages of pile cutting, and difficult to re-jack the piles after cutting if pile re-jacking is needed. In certain circumstances, it could cause pile eccentricity due to the wandering movement of the jack-in machine under sustained compressive jacking load for long pile in weak soil with low confinement or intermittent obstruction during pile penetration.

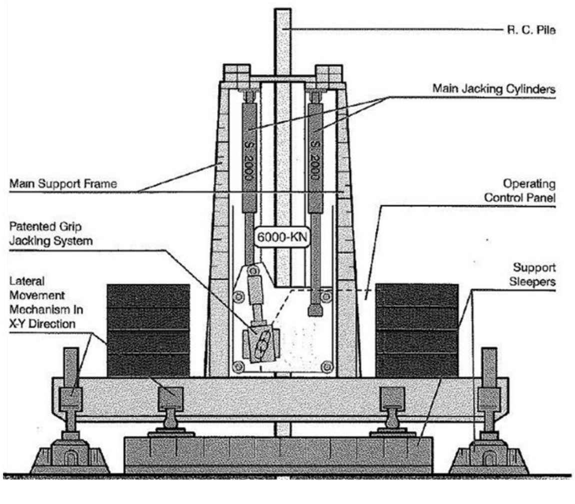


Figure 1 Typical details and basic components of jack-in rig

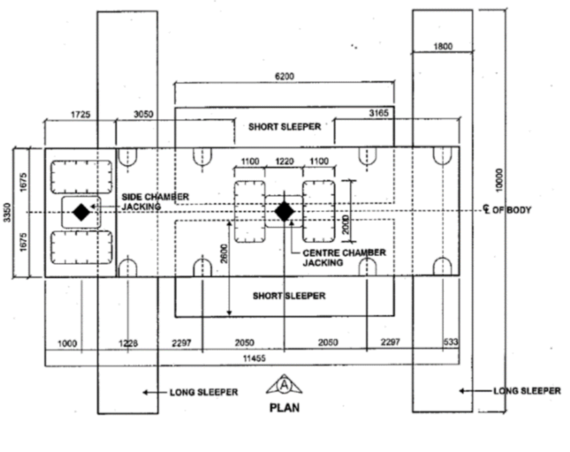


Figure 2 Typical plan view details of jack-in rig

With the increasing demand on using jack-in installation pile to support taller high-rise buildings, higher structural capacity of concrete piles are produced by pile manufacturers. Introduction of Grade 90 prestressed concrete spun pile and thicker wall spun pile reveal the innovation and evolution from the pile manufacturer to meet market need to support higher high-rise building load.

However, there are inherent fallacies of capacity performance on the jack-in piles in certain condition, particularly for short end bearing piles with insufficient embedment into the competent bearing materials where the pile end bearing capacity is developed from and also the myth of the proof loading assurance at the end of pile termination. This paper will highlight the possibilities of such fallacy on the general perception and myths in the faith on this getting popular piling system, thus proper use and due consideration can be given to ensure sustained success.

2. COMMON PRACTICES OF JACK-IN PILE INSTALLATION

The advantage of jack-in piling method is the applied jack-in force to overcome the penetrating resistance of subsoil can be recorded during the pile penetration as compared to driven pile with only indicative hammer blow counts. This can resemble a total penetration resistance profile similar to a cone penetration test, in which the ultimate pile capacity is perceived to be ascertained at pile termination.

The applied pressure against the pile penetration is conventionally recorded by manpower and the accuracy of the recorded information is solely at the discretion of the operator. Generally the applied pressure readout unit is located in the control room at upper compartment of the machine while the worker who records the pile penetration is stationed at lower compartment of the machine. Both the record of injection load and pile penetration will rely on passing of information between the operator at the upper compartment and the assisting worker at the lower compartment. This conventional method of manual recording leaves the data collected unverified and accuracy of the data can be doubtful.

2.1 Installation Method

For the purpose of exerting the jacking force onto the piles, it can be performed by either direct compressive contact at the pile head from the hydraulic jack or using the frictional grip onto the pile shaft by hydraulic grip as shown in Figure 3. The main difference between the two jacking systems is that the earlier direct pile head contact jacking system will require a frame height higher than the exposed pile length above the piling platform whereas the later frictional grip system can have lower jacking frame because the gripping can be at any mid height of the exposed pile shaft. From the view point of pile structural buckling and potential of energy release of high elastic compressive strain stored in the pile body especially for long pile carrying high capacity, the gripping system seems to have better technical advantage. Due to the requirement of high friction, the hydraulic gripping mechanism may require enlarged gripping area onto the pile shaft and also strengthening of the pile body structurally to take the high radial compressive gripping force.

Figure 3 Hydraulic gripping system with gripping blocks

When obstructions to pile penetration are expected as shown in Figures 4 and 5, pre-boring method is a very common approach to overcome the premature pile termination. Pre-boring makes way for the jack-in pile to pass through the obstructions encountered within the expected terminating pile length as shown in Figure 6. Sometimes, backfilling in a pre-bored hole is carried out for gaining the lateral pile confinement. If the pre-bored hole is not backfilled, the existence of open gap between the circular pre-bored hole and the pile shaft can have serious implication to the pile performance with the potential stress relaxation as discussed in the first case history later.

Figure 4 Typical core boulders within granitic residual soils

Figure 5 Intermittent cemented materials or less weathered meta-sedimentary formation

Figure 6 Reinforced concrete square piles in non-backfilled pre-bored holes at weathered meta-sedimentary formation

It is logical to raise a few following technical concerns with the existence of the annulus of the pre-bored hole when it is not being properly backfilled.

a. Buckling of unrestrained pile in the overly enlarged pre-bored hole.
b. Loss of pile frictional resistance as a result of lower normal contact stress or even uncontacted with gapped annulus.
c. Potential void beneath the pile toe when the pile cannot fully penetrate below the base of pre-bored hole due to obstruction at pile edges.
d. Surface runoff enters through the annulus to soak the bearing soils at the pile toe, thus resulting in toe softening.
e. Reduction of pile toe capacity resulted from existence of free surface near pile toe and the reduced pile toe confinement as shown in Figure 7. As the failure mechanism of pile toe bearing capacity is governed by the bulb-shaped plastic zones and failure wedges developed in the bearing soils under concentrated bearing pressure failing the soils. It is similar to the analogy of bearing failure of a shallow footing where the over-burden weight above the footing base level and surface surcharge on the ground surface will significantly enhance the bearing capacity. The vertical free surface generated from the pre-bored hole near the pile toe can have the similar effect of the shallow footing. When the annulus free surface of the pre-bored hole fall within the plastic zone of the failure bulb, the condition of near to zero lateral contact stress at the pile and vertical face of pre-bored hole reduces the pile toe capacity significantly. Full pile toe ultimate capacity can only be restored when such annulus free surface is sufficiently away from the plastic bulb or the annulus is filled with grout.

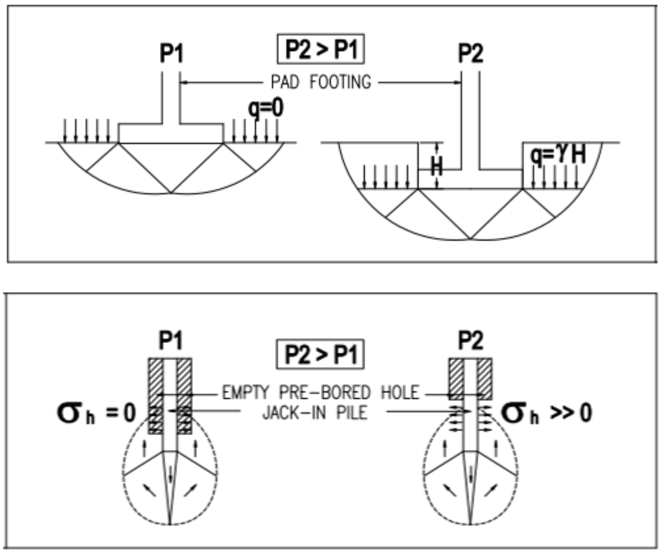


Figure 7 Effect of pile toe confinement to pile base capacity

Yang (2006) stressed that the depth of influence zone at pile toe is complicated and influenced by many factors such as angle of shearing resistance of the founding soil at proximity of pile toe, pile diameter, stiffness, in-situ effective stress at pile toe, homogeneity of the soil and etc. For piles in more compressible silty sand with fines content over 15%, the upper plastic zone is between 0.5D and 1.5D and the lower plastic zone ranges from 1.5D to 3D where D is pile size. Meanwhile, the influence zones for sand with Ø' =30^0 are 1D to 3D upwards and 3D to 5D downwards as presented by Hirayama (1988). As such, it is worthwhile to seal-off the annulus between oversized pre-bored hole and pile shaft to remove the condition of free surface and to prevent ingression of water potentially leading to softening of pile toe founding materials within the plastic zones of pile toe. This can be possibly resolved by backfilling the pre-bored hole with the excavated materials from the pre-boring before inserting the pile into the pre-bored hole, or seal up the annulus between the pile and the pre-bored wall face after jacking installation. The recommended minimum grout sealing depth shall be approximately 5 times pile size above the base of pre-bored hole. Notwithstanding to the above, it is always better to have the grout fully filled up the annulus gap in the empty pre-bored hole to avoid any potential buckling of pile if the free standing length in the pre-bored hole is significant.

2.2 Installation Problems

When inserting a displacement pile into the ground, it is not uncommon to observe large soil displacement. The same outcome of such soil displacement can cause two prominent types of soil movements as shown in Figure 8: firstly soil upheaval movement at shallower subsoil due to the free ground surface from the displacing action of pile insertion where the confining stress to resist the pile displacing action is lower; and secondly the lateral soil movement at the deeper depth by the pile insertion inducing volumetric displacing action where the overburden pressure has sufficient confinement to restrain the significant upheaval soil movement, but comparatively lower lateral restraints leading to more lateral soil movements. This is not difficult to comprehend as the lateral earth pressure at depth under geostatic condition is always a fraction of the overburden effective stress. When a volumetric expansion is induced within the soils at large depth below the ground, likes the volumetric displacing soil movement by pile insertion, the lower stress zone will be subjected to increasing disturbing stress in mobilising the soil strength to reach an overall static equilibrium. The soil displacement is very prominent in saturated soft clayey soil, where the pore water is virtually trapped in the voids between the clay particles due to inherent low permeability in clayey soil, thus the soil matrix behaviour tends to be incompressible disallowing any volumetric change when it is subjected to rapid loading or shearing. The weak strength of the incompressible soft clayey soils will have flow characteristics in the localised soil movement.

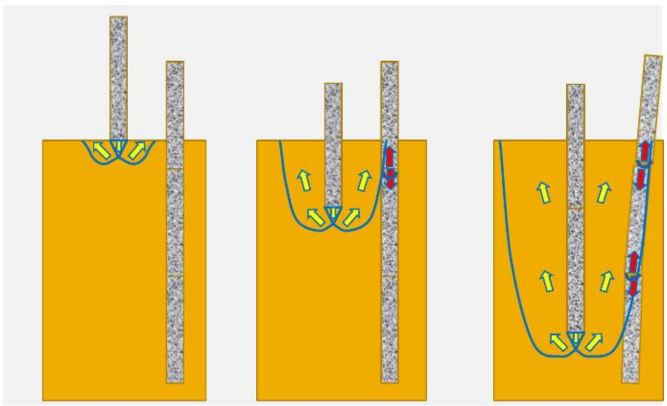


Figure 8 Soil displacing actions at penetrating pile toe to adjacent installed piles

There are consequences resulted from these two types of volumetric soil displacements to the earlier installed adjacent piles next to the subsequently installed piles. The upheaval of subsoil at shallow depth will induce tensile stress to the earlier piles, in which the pile may suffer structural tensile damage. In some cases, it may even uplift the earlier installed piles shall the pile geotechnical capacity below the zone of installation disturbance from the subsequent installed pile is not able to sustain the excessive upheaval pulling to the affected piles. On the other hand, if the pile geotechnical capacity is significantly relied on the end bearing contribution, this uplifting movement will reduce the pile toe contact pressure attained at pile termination and thus jeopardise the pile geotechnical capacity performance significantly. Although the pile geotechnical end bearing capacity can be re-developed when the pile is subsequently subjected to impose compressive load but the pile will have to experience substantial settlement before re-gaining the geotechnical capacity in end bearing condition. Whereas the lateral subsoil displacement at greater depth may potentially induce damaging flexural stress to the adjacent installed piles, which may either result in bending failure of the pile body or the pile joints. The other adverse effect of the soil displacement is the associated pile deviation with the soil displacement. When large pile group clusters are involved, the amount of soil movements not only will cause serious structural integrity problem of the installed piles, but also serious deviation to the earlier installed piles. The impact from the displacing soil is primarily controlled by the pile spacing and also the displacing volume of the installing piles.

To overcome the potentially damaging effect of the soil displacement by the displacement pile installation, two approaches are normally considered; firstly, using open end spun pile toe to allow inflow of the displacing soil at the pile toe into the inner annulus forming soil plug, thus reducing the soil displacing volume around the pile toe; and secondly, to install the vertical relief wells in between the pile clusters to empty up partially the embedding soil around the piles within the pile embedment depth in order to absorb any displacing soil volume during pile jacking. For stability of relief wells, either empty borehole with weak stabilising slurry or installing perforated or slotted steel pipe casings can be a mean for passive intrusion of the displacing soils. However, regular clearing of the intruded soil shall be carried out to ensure its effectiveness in mitigating the induced soil displacement from pile insertion.

With the open end pile, it can be a wishful perception expecting the soil plug to fill up the entire annulus, thus the displacing soils along the penetrating pile can be minimised. The formation of soil plug depends heavily on the first encountered materials at the onset of pile penetration, plug friction resulted from the strength of the entering subsoil, roughness of the inner annulus surface and the ratio of the pile external diameter and the inner hollow annulus diameter permitting entering of compressed soils at pile base. Such false expectation can be demonstrated statistically in the case history to be presented later. Nevertheless, leaving the pile with open ended condition will still mitigate the upheaval of weak soft soil and lateral pushing to adjacent piles with some degree of success. More study is needed to investigate the formation of soil plugs within the pile annulus with penetration depth and also the effect to the actual soil displacement when the soil plug is gradually formed. It shall be noted also that the first materials entering the pile hollow annulus is usually the piling platform materials made of reasonably good materials overlying the soft and weak top subsoil materials. It is worth to perform pilot pile to investigate whether longer pile plug can be formed when pre-bored hole to remove the top platform materials from the soil plug formation. Another doubt of the existence of the unfilled and possibly dry pile annulus above the soil plug may potentially provide opportunity for water ingression from higher external groundwater to soften the pile base founding material and also the stiff soil plug over time. Perhaps partially infill with cement mortal or grout above the soil plug may mitigate such concern if the softening effect is real. More field data and case histories to validate this concern in mind shall shed better design decision on this aspect.

For piles installed in karstic limestone/marble formation in Kuala Lumpur, Malaysia, that may encounter immediately weak slime zone above the limestone/marble bedrock with potential inclined surface, high percentage of damaged piles is not uncommon. With such drastic contrast of stiffness at two adjoining materials and potential inclined bedrock surface where the pile will likely reach installation refusal. The weak lateral support from the slime zone in restraining the highly stressed pile from buckling and pile deviation due to the kicking-out effect from inclined pile toe contact surface provide a very conducive environment for pile damage. It is a very well established scientific facts that catastrophic failure can occur in an uncontrolled energy release when a less stiff material is injected with significant amount of elastic strain energy by a much stiffer loading system and the stiff support until reaching a failure point. In the case of jack-in pile installation in limestone/marble karst formation, pile damage is mainly due to insufficient response time for the operator to stop jacking the failing pile. When the failure is observed, it is always too late to response. As such, an innovation was initiated by the authors to utilise automated monitoring system for faster recording, diagnostic algorithm and feedback response to guide the jack-in rig operator to cease the jacking at the onset of pile distress being diagnosed. This will be further discussed in the later sections and case history.

2.3 Pile Performance

Theoretically, jack-in pile with the following inherent technical advantages shall have assured good pile performance. This view probably still remains valid if the aforementioned adverse effects resulted from the post pile installation influence and time dependent deterioration are well addressed in design and construction.

a. The short-term pile resistances can be readily verified and prove-loaded during and at the end of pile penetration. On this aspect, the capacity performance demonstrated is nearly undisputable if the founding materials have contractive behaviour when being stressed or sheared because future increase in material strength can be expected after the dissipation of positive excess pore pressure induced during the pile installation. However, reverse trend can be expected if the founding materials have dilative behaviour when being stressed or sheared because future reduction in material strength is expected after the dissipation of negative excess pore pressure induced during the pile installation. Note shall be taken that the overall unfavourable behaviour of pile performance in the later case may be masked or softened by the beneficial factor as discussed in item (b) below.
b. The installed pile with repeated unloading and reloading cycles shall perform much stiffer load settlement performance from the pile load test, then the actual service load from the superstructure are imposed later. This can be evidenced with the existence of lock-in axial compressive stress in the pile after unloading. This is because the installed pile is initially subject to compressive load during pile jacking operation until attaining pile-soil interface slippage at the upper pile shaft when the mobilised interface strength reaches both the ultimate pile shaft and toe resistances during jacking, however upon unloading, the elastic rebounce of the initially compressed pile will subject to downdrag load where the pile-soil resistance will have to act in the reverse direction at the upper pile portion where the earlier slippage at pile-soil interface occur during pile jacking. As a results, the initially compressed pile cannot have a full recovery of the elastic straining, then the lock-in stress exists within the pile as schematically shown in Figure 9. This phenomenon of lock-in stress is equivalent to prestressing the pile before loading the pile with actual service load. The overall load-settlement behaviour of the pile with lock-in stress shall be much stiffer as shown in Figure 10 because the gradually imposing service load to the pile will take over the downdrag without further straining the pile.
c. There is no doubt that the entire pile capacity is a sum of pile toe bearing resistance and pile shaft frictional resistance. However, the uncertainty of capacity performance of toe bearing resistance can be many order higher than the pile shaft frictional resistance. This is not difficult to comprehend as the area of contact for pile toe is many time smaller comparing to the pile shaft contact surface of a pile with reasonable penetration length. The area of contact ratio of pile toe area and pile shaft area with pile length of k times of the pile diameter, D, can be derived as below:

Area Ratio = Pile shaft area / Pile toe area
$= (\pi D \times kD) / (\pi D^2/4) = 4k$

With a reasonable pile length of k = 6, this area ratio will be about 24.

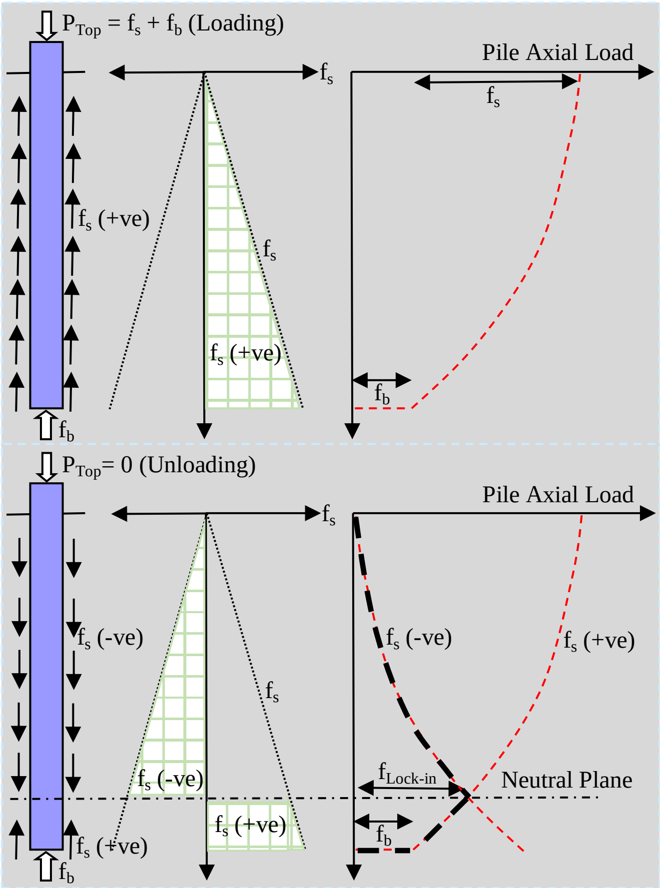


Figure 9 Effect of lock-in stress in the loading and unloading cycles in a jack-in pile

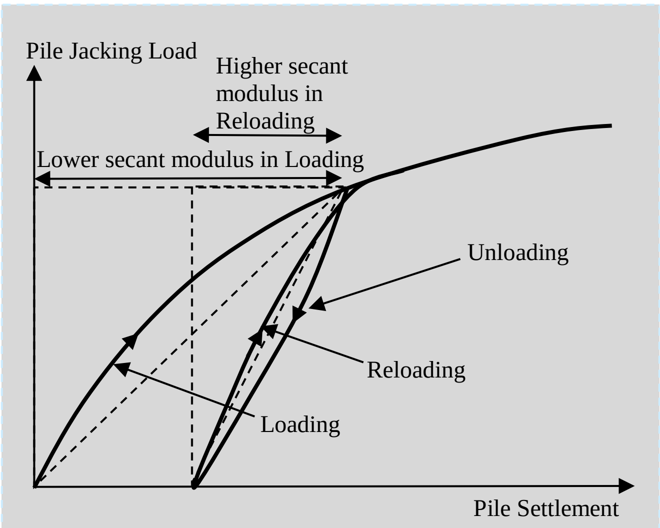


Figure 10 Pile stiffening effect in the loading and unloading cycles of a jack-in pile with lock-in stress

When reviewing the unit area capacity contribution between the pile shaft and pile toe, it is not uncommon to realize that high toe resistance during pile installation is usually the reason for leading to pile termination whether the pile is installed by statically jacking or percussion driving.

It is also common to accept that pile toe resistance can be of high variance and sudden reach extreme resistance when penetrating vertically through a stratified ground with hard stratum, whereas the change of pile shaft resistance can be of much more gradual and accumulating trend. In summary, there is high possibility of having higher variance of pile capacity for pile with relatively little embedment length resting on shallow competent stratum that offers majority of the pile capacity.

3. IMPLEMENTATION OF DATA LOGGER

Instead of relying on manpower in recording various piling information intermittently, it will be more efficient to devise an electronic recording system to perform these simple, but monotonous tasks like recording the applied jack-in pressure and the pile penetration depth simultaneously throughout the entire pile installation. As shown in Figure 11, only three simple instruments and one computer console are involved in the entire monitoring system, namely one wire spring penetration decoder for measuring movement of the hydraulic jacks and two hydraulic transducers; one with high measurement accuracy for the hydraulic pressure exerting in the jacks and another one to perform as a flagging signal indicating if the gripping jacks are engaged or not. This can optimize the data storage on the pile jacking records as the data logging system with computer programme will only record the measurements during the jacking operation and not recording when the jacking clamps are released for retracting to the next jacking operation in the repeated manner. This can provide reliable and accurate installation information of the piling records. A schematic diagram of the instrumentation installed in the jack-in pile machine is presented in Figure 12.

Figure 11 Wire spring penetration decoder, pressure transducers at the hydraulic system and LCD panel at the operator cabin

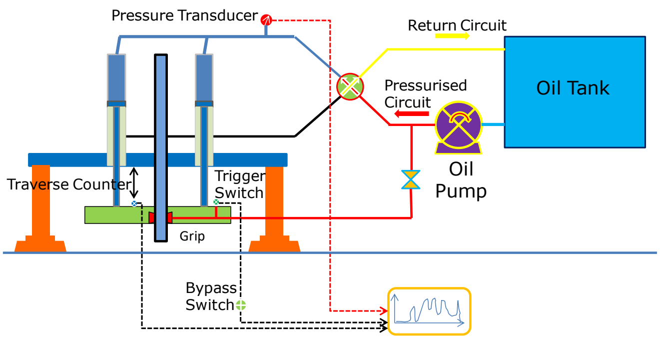


Figure 12 Schematic Diagram of Instrumentation on Jack-In Pile Machine

It is interesting to compare the manual pile installation record and recording from data logger as presented in Figure 13. Obviously the data obtained from manual recording is not reflecting the actual condition. From the aspect of human psychology in performing such monotonous tasks, the recorder will virtually pay more attention to record the expected portion of measurements that the data reviewer will be looking at most of the time. In this case, it will be the last few records before reaching pile jacking refusal, whereas the rest of the penetration profile will thus unlikely be recorded with similar efforts.

With hundreds of working piles installed, it is inhuman to expect precise and accurate manual recording. This continuous pile penetration record can be useful to see the consistency of the penetration resistance to the subsurface profile as interpreted from the borehole investigation.

To maximize the use of such automated recorded information, there are few features below worth to be explored to attain additional benefits.

a. There is clear pile capacity setup at every unloading and reloading cycle. With longer duration of resting and longer pile penetration, larger additional jacking load will be required to re-penetrate the pile into the ground, especially during the pile joint welding that usually takes about 10 to 15 minutes (as evidenced at the stoppage at 11.5m pile penetration in Figure 13). If such pile setup effect can be compiled during all the interim stoppage of jacking with varying duration and also the pile penetration before resuming jacking, it will reveal useful pile performance during construction.
b. Since pile jacking load and pile penetration are both recorded simultaneously, it is possible to compute the work done to each installed pile for the carbon footprint assessment, where green construction is a promoted practice in the construction industry.
c. Similarly, the recorded profile of jacking load and pile penetration can also be used as a controlling of pile installation to prevent structural damage of pile due to toe slippage under high axial load. This phenomenon of uncontrolled energy release from a pile stored with high elastic strain energy and resting over inclined bedrock surface has been elaborated in last paragraph in Section 2.2. With high frequency interval data acquisition for the pile load change and the computed rate of pile penetration, it is possible to stop the pile jacking process before the pile has suffered significant structural damage. The second case history will present the details of construction control of jack-in pile to minimize pile damage in the karstic limestone formation.

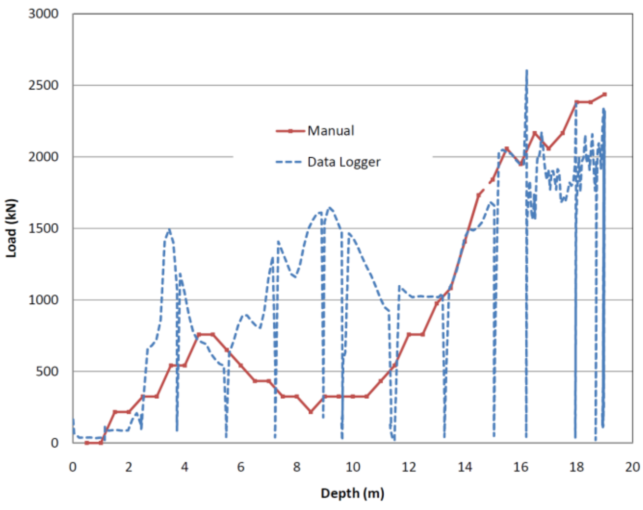


Figure 13 Comparison between manual recording and data logger recording

4. CASE HISTORIES

Two case histories are presented in this paper to demonstrate the discussed problems and solutions of the jack-in piling system over different geological formations highlighted above.

4.1 Case A – Meta-sedimentary formation

4.1.1 Background

This case history presents an investigation consisting of 400mm reinforced concrete (RC) square pile installed in meta-sedimentary formation in Kuala Lumpur with empty pre-bored hole as reported by Liew, et al (2013). The installed piles failed to achieve the required pile performance in the maintained load tests. During the investigation, subsurface investigation factual reports, pile foundation design concept, pile construction records, construction method and pile test reports were carefully studied in order to narrow down the probable causes of unfavourable performance of test pile results. Additional maintained load tests were proposed and conducted to verify the suspected probable causes from the investigation. Results of both contractually scheduled and investigative maintained load tests are presented and discussed. Some lessons learnt will also be discussed for improvement of the future jack-in pile installation with pre-boring method.

4.1.2 Subsurface conditions

The construction site is underlain by Hawthorndern formation mainly consisting of metamorphosed sedimentary rocks like phyllite and schist. As the observed rapid disintegrating rate of the exposed weathered bedrock formation and instability of many cut slopes formed in the same formation, it was expected that swelling and flaking behaviours of this formation can be prominent characteristics when subjecting to stress relaxation. Interpreting from the exploratory boreholes, the overburden weathered materials mostly consist of sandy CLAY and at fairly consistent depth of encountering competent hard stratum (SPT-N ≥50) as shown in Figure 14.

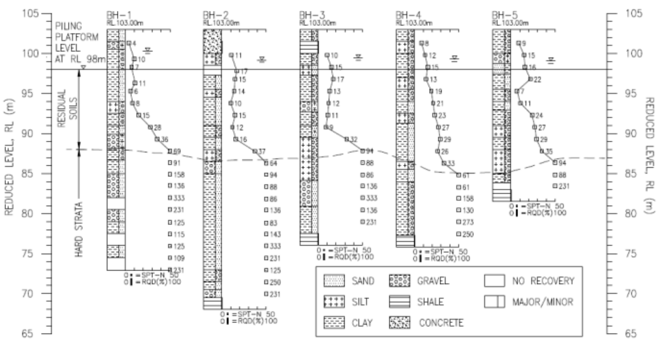


Figure 14 Subsurface conditions from exploratory boreholes

4.1.3 Construction installation of jack-in piles

Jack-in piling method was adopted to install 400mm RC square piles to achieve the specified pile termination criteria (2.2 times of specified pile working load with minimum 30 seconds maintaining period and pile settlement during the maintaining period should not exceed 5mm/cycle for two cycles). The piles were designed to take working load of 1300kN and were statically jacked until 2860kN before termination. All piles were installed in an empty pre-bored hole of 9m below piling platform at RL98m with the aim to facilitate deeper pile penetration for pile fixity and assurance of resting or within the hard weathered formation with SPT-N value more than 50. Three (3) different diameters of empty pre-bored hole had been used during the early stage of pile installation. Initially, several piles were installed using 600mm diameter pre-bored hole but it was later changed to 500mm diameter to avoid free standing condition of the installed pile in the oversized empty pre-bored hole without adequate lateral support. Finally, majority of the working piles were installed with a compromised 550mm diameter empty pre-bored hole as 500mm diameter pre-bored hole was found undersized resulting in premature termination after installing a few initial 400mm RC pilot piles.

Certain piles were terminated either at the base of empty pre-bored hole or with noticeably short penetration below base of the pre-bored hole. These piles were expected to experience capacity reduction resulting from stress relaxation due to overall low confining effective stress near the pile toe as illustrated in Figure 7.

4.1.4 Initial maintained load tests

Initial maintained load tests (MLT) were performed on five (5) selected working piles (MLT 1 to MLT 5) to verify the required proof load factor, workmanship quality and pile performance.

MLT results in Table 1 indicate majority of the initially tested piles settled more than the requirement of 12.5mm at pile working load. MLT 1, MLT 2 and MLT 4 with corresponding 0.4m, 0.3m and 0.5m penetration below the base of pre-bored hole had recorded relatively more pile top settlement compared to MLT 3 and MLT 5, which penetrate 3.5m and 4.5m respectively below base of the pre-bored hole. These piles recorded unfavourable performance with excessive pile settlement and were unable to achieve the required maximum test load except for MLT 3. Therefore, it can be reasonably expected that the potential reduction in load carrying capacity of the test pile as indicated in the test results could be strongly related to the pile penetration below the base of empty pre-bored hole. Subsequently, additional MLTs were conducted on specifically selected three (3) working piles with 0.5m, 1.5m and 2.0m penetration below base of 550mm diameter pre-bored hole respectively to verify this suspicion since MLT 1 and 2 were terminated at different maximum jacking forces and pre-bored diameters. MLT 1 was terminated at maximum jack-in force lower than other production piles due to the earlier targeted pile working load (950kN) is lower during first pile installation. MLT 2 cannot achieve maximum targeted test load due to insufficient counterweight of the kentledge blocks provided during initial stage of the pile jacking after upgrading the pile working capacity from 950kN to 1300kN.

Table 1 Performance of the contractually scheduled test piles and additional investigative test piles

MLT	Pre-bored Diameter (mm)	Pile Penetration below Piling Platform (m)	Max. Jack-in Load at Termination (kN)	Achieved Maximum Test Load (kN)	Pile Top Settlement At Working Load (mm)	Pile Top Settlement At Max. Test Load (mm)
MLT 1	600	9.40	2160	2220 (1.71x WL)	14.0	46.00
MLT 2	500	9.30	2600	2220 (1.71x WL)	23.50	42.00
MLT 3	550	12.50	2860	2600 (2.00x WL)	5.80	21.80
MLT 4	550	9.50	2860	1406 (1.50x WL)	16.50	24.50
MLT 5	550	13.50	2860	1950 (1.50x WL)	8.50	13.00
MLT 6	550	9.50	2860	1950 (1.50x WL)	15.08	42.38
MLT 7	550	10.50	2860	2400 (1.85x WL)	11.29	41.93
MLT 8	550	11.00	2860	2600 (2.00x WL)	10.30	50.35

4.1.5 Additional maintained load tests

All additional three MLT piles (MLT 6, MLT 7 and MLT 8) had been selected on piles installed with termination criterion reaching 2.2 times of working capacity but MLT 6 and 7 failed to achieve the required maximum test load, except for MLT 8. This clearly implies the high possibility of pile capacity degradation resulted from stress relaxation. MLT for piles with deeper penetration below the base of pre-bored hole have obviously shown better settlement performance at one (1) time working load in the first cycle. The load-settlement curve of all test piles with slight penetration beyond the pre-bored base as shown in Figure 15 and Figure 16 have generally gentler gradient in the first loading cycles whereas the gradient of subsequent reloading cycles becomes steeper. This is the clear evidence of phenomenal soil softening after the termination of jack-in pile with pre-bored condition resulting minimum soil displacing action. However, further reloading of the pile to higher load in the subsequent load test cycles had allowed the founding soil stratum regaining the soil compactness and thus stiffer pile base response during reloading. The test results further enhance the findings of potential stress relaxation at pile toe due to insufficient stress confinement within the effective stress bulb of the end bearing pile toe as a result of insufficient pile penetration below the base of pre-bored hole. The restoration of initial higher pile capacity in second load cycle as a result of further pile penetration into soften subsoil near to the pile toe implies that this is solely a pile settlement problem.

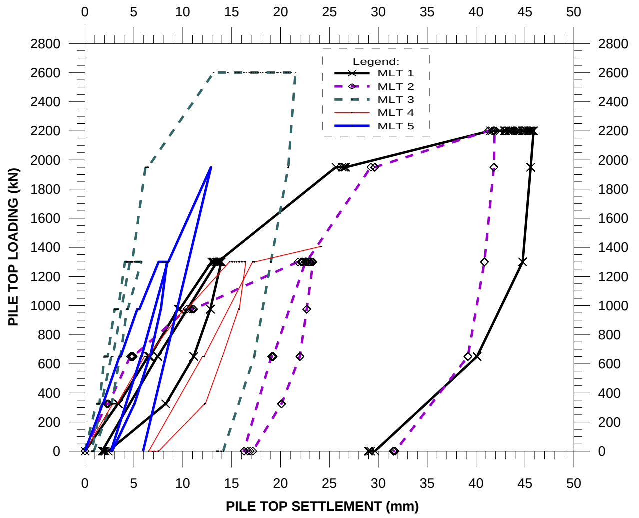


Figure 15 Pile top loading versus pile top settlement for contractually scheduled MLT results

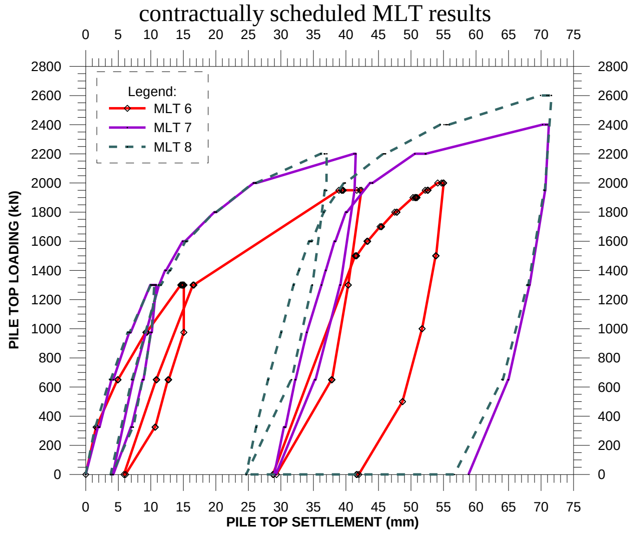


Figure 16 Pile top loading versus pile top settlement for additional investigative MLT results

4.1.6 Lessons learnt and recommendations

Piles installed into pre-bored hole without backfilling the annulus are exposed to the risk of pile toe softening, hence consequently lead to reduction in pile load carrying capacity and softer response in pile toe stiffness. The pile toe softening effect in the bearing soil stratum affecting the end bearing capacity of the pile can be logically expected when the empty annulus in the pre-bored hole is nearer to the pile base. The empty annulus with virtually zero confining stress provides pre-requisite condition for time dependent stress relaxation of soils to take place especially when the free surface is exposed to water. When the pile has sufficient penetration below the pre-bored base, the stress relaxation effect at the upper most soil (beyond influence zone of the stress relaxation above pile toe) would not affect the effective stress bulb near the pile toe, thus the pile end bearing capacity. Figure 17 shows a schematic diagram of the stress relaxation and the stress bulb of pile toe end bearing.

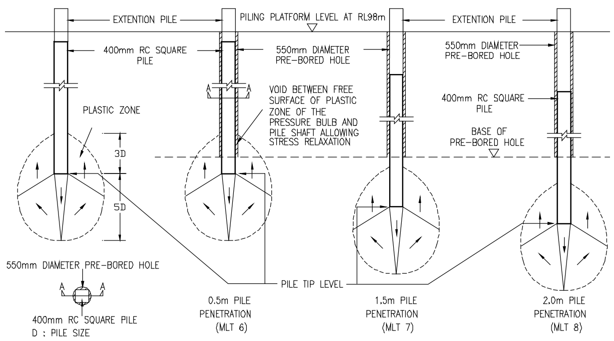


Figure 17 Schematic diagram of stress relaxation effect with relative position of pre-bored hole and pile stress bulb

4.2 Case B – Karstic limestone formation

4.2.1 Background

This case history describes various engineering challenges and innovative solutions developed during designing the alternative foundation system for a residential condominium project in Ampang, Selangor presented by Liew and Pan (2013). The project consists of two tower blocks of 18 storeys high for a total 250 units, with podium car park and ground floor retail shops. The site with land size of about 2.5 acres is located within an urban environment and is surrounded by shop lots, high-rise and low-rise residential development.

4.2.2 Site geology and subsurface conditions

The site geology consists of the notorious karstic Kuala Lumpur limestone formation. There were sixty nine (69) boreholes planned and commissioned by the original designer at almost every critical column locations. There are 156 columns in total. The limestone bedrock level varies from 16m to 40m below ground. The rockhead profile is highly variable with an inclination varying from 30° to more than 50° covering almost half of the site. The overlying overburden soil consists predominantly of clayey SILT and silty SAND with SPT 'N' varying from 5 to 20 for the overburden soil. At a localized area, silty CLAY with SPT 'N' less than 5 were found at the top 10m soil layer. In terms of groundwater level, it is relatively high at average level of 3m below ground.

As part of the verification program to the provided subsurface information, five (5) additional boreholes were carried out with full-time supervision. Two (2) of the boreholes were carried out at the test pile locations while three (3) others were carried out to investigate the extent of karstic limestone features.

4.2.3 Alternative foundation design

The main objectives of this alternative design were initiated by the design-and-build contractor looking for cost and time savings. There was an imperative intention to complete the piling works in 100 calendar days including the pile load tests. In order to achieve the tight program, the followings were proposed to attain the objectives:

1. Replace all bored piles in the conforming foundation design with spun piles – this simplifies the construction works, has less congestion on site and also reduces the mobilization cost. Jack-in piling has advantage of proof loading individually over every installed pile for better performance assurance. The other requirement is to ensure that the spun piles can achieve its maximum capacity especially those founded over limestone surface with adverse features such as inclined rockhead and limestone cavities. Cavities were treated with cement-sand grout.
2. Reduce the number of different spun pile sizes of 300, 450 and 500mm diameter provided in the conforming design to only 400mm and 500mm diameter spun piles as this will streamline the jacking operation with better efficiency and improve site management of pile storage.
3. Upgrade the utilization factor of the spun pile capacity – the conforming design utilized 75% of the allowable pile structural capacity. This is fairly common practice by some local designers for spun piles installed in limestone geology with the intention to minimize the possibility of pile damage when installing the piles on inclined rockhead surface with lower jacking load. However such a low utilization factor would result in a higher number of piles. Hence the utilization factor was increased to 85% by careful installation procedures and real-time monitoring for the jacking operation, which will be discussed later. In addition, Class B extension piles and C starter pile spun pile with thicker wall to withstand the gripping load were used to minimize risk of pile damage during installation.
4. Remove the pointed pile shoes in the conforming design. It was of the opinion that it would be inappropriate to use pointed pile shoe with structural bearing plate welded to spun piles with hollow section taking high concentrated jacking reaction load. One of the benefits with open end pile is to reduce lateral soil displacement at shallow depth during installation of the piles. In addition, in the event that underpinning is required for piles resting over limestone with large cavities beneath, micropiles can be installed through the hollow section of the spun piles which would not have been possible had the steel pile shoes been used.

The piles were then redesigned using 400 and 500mm diameter spun piles, with 85% utilization factor and to be installed without pile shoes. The piles were expected to be founded on the limestone rockhead with its capacity mainly derived from the end-bearing. Two 500mm diameter test piles were specified to verify their design capacities and to explore if the utilization factor can be further optimized. The pile lengths for PLT1 and PLT2 are about 26m and 20m respectively from the jacking platform level (which is about 2m above the original ground level). The two test locations were selected to represent location where the rockhead profile is highly inclined and where the loading is relatively small such that it will be easy to install replacement piles if the test piles were damaged during the testing.

The first test pile, PLT1, was carried out without any instrumentation but with the aim of verifying the maximum pile capacity that the pile can achieve in the jacking installation. The jacking load during installation was recorded manually at this stage as the automated monitoring device was not ready for the test pile installation. Figure 18 shows the pile jacking penetration recorded manually for both test piles. The pile was able to sustain a jacking load of up to 4,600kN before it failed. In the installation of second test pile, PLT2, instrumentation using the Global Strain Extensometer (Glostrext) system as presented by Hanifah and Lee

(2006) and as shown in Figure 19 was carried out to measure the movements of pile segments for computing global axial strains on the pile corresponding segment, load transfer behavior during jacking and also interpreted locked-in/residual stress in the pile after the unloading cycle can be then assessed. PLT2 eventually failed at a much lower jacking load of 2,200kN, which was less than twice the intended working load.

Based on the test pile results, it appears that the pile capacity achieved from the 500mm diameter pile was much less than its structural capacity in PLT2 revealing that the existing weak ground condition will not warrant the higher pile capacity. Furthermore, the soft underlying ground also discourages the use of jacking rig with excessive reaction weight for higher jacking reaction. Hence smaller 400mm diameter pile size with a lower jacking load was then preferred and the alternative pile design was then revised accordingly. A third test pile, PLT3 was then carried out on a 400mm diameter spun pile to twice its design load, which was optimized to 1,200kN. The pile was jacked and terminated at a load of 2,400kN. The final proposed alternative design is shown in Table 2.

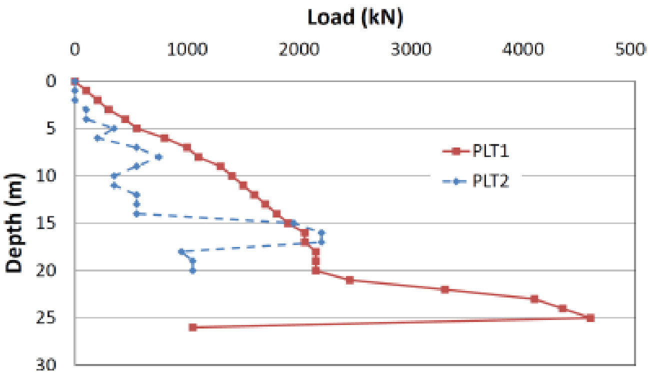


Figure 18 Jacking Installation Records of Test Piles PLT1 and PLT2

Figure 19 Installation of Glostrext Instrumentation System for the Test Pile

Table 2 Details for the proposed and revised alternative foundation design

Pile Size (mm)	Before Pile Load Test		After Pile Load Test	
	Pile Nos	Proposed Pile Capacity	Pile Nos	Proposed Revised Pile Capacity
400	43	1100kN	1148	1200kN
500	725	1700kN	-	-

In this jack-in installation, attempt was made to measure the pile plug length formed within the pile annulus after terminating the pile jacking. Figure 20 presents the statistics of actual soil plug formed after the jack-in 400mm diameter pile installation. From Figure 21, it is observed that majority of the soil plug length for pile penetration length lesser than 30m is typically in the ranges of 5 to 25 times of the pile inner diameter. Long pile exhibits larger variation of the formation of soil plug length when compared to short pile of less than 30m. However, the formation of soil plug length varies probably depending on the variability of top most materials at the piling platform first entering the pile annulus.

More detailed measurement can be performed for future projects by having the soil plug length measured at every segment of pile penetration in order to find out the minimum pile penetration, in which the stabilised soil plug can be fully formed (no further increment of soil plug length for subsequent pile penetration). It is also useful to compile the pile plug data from different pile sizes, annulus diameters, materials forming the pile plug for understanding the effect of reducing volumetric soil displacement at open-ended pile toe. One of improvements can be considered to have pre-bored hole through the stiffer fill materials at the piling platform to encourage longer pile plug from the underlying incompressible weak soft fine soils.

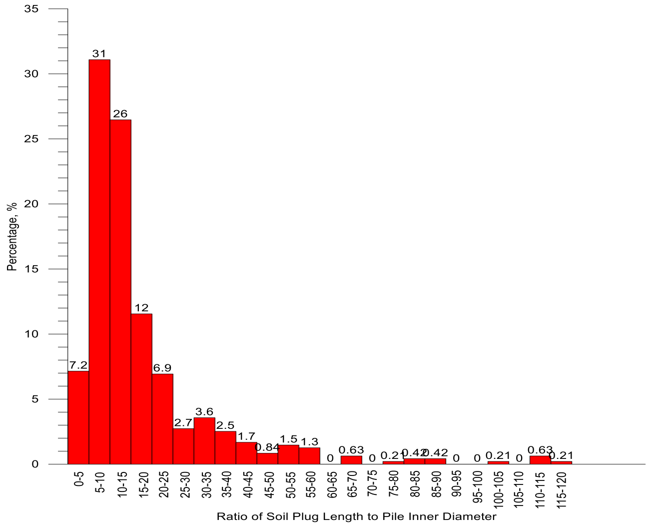


Figure 20 Statistics of pile plug formed within the 400mm diameter spun pile inner annulus with varying pile termination length

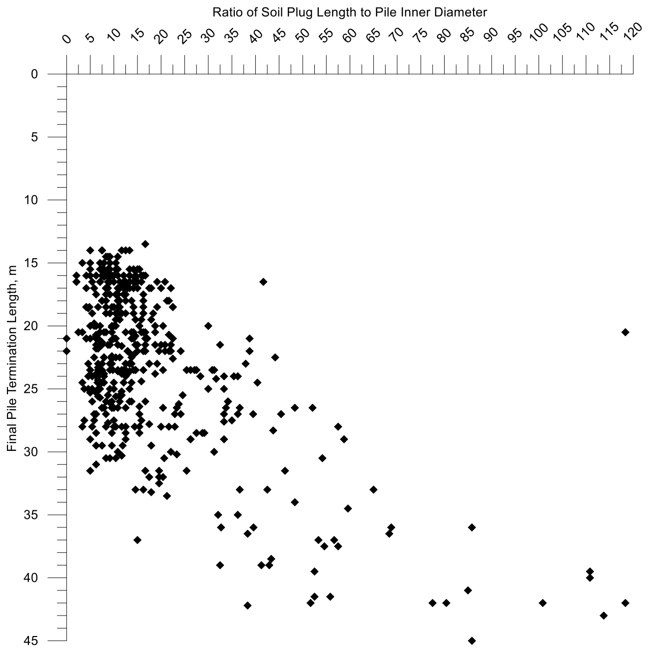
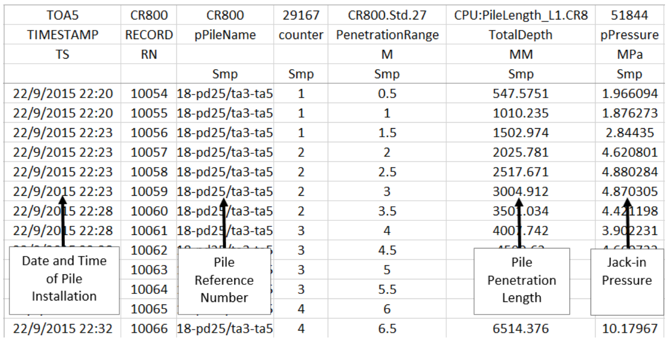


Figure 21 Final pile termination length with the respective soil plug length (400mm diameter spun pile)

4.2.4 Innovation in jack-in piling construction

One of the main challenges on this project is to optimize the pile capacity utilization factor during pile installation without overstressing or damaging the piles, especially piles which are founded on highly inclined or erratic limestone rockhead. The design team together with the piling contractor and specialist instrumentation contractor jointly devised an electronic recording system to monitor the pile jacking reaction load with pile penetration depth during pile jacking. The main feature of data logger is the visualized user interface that permits efficient judgment to the rig operator during the installation so that timely decisions can be made to control the jacking operation during the course of pile installation. Figure 22 shows example of the data recorded for the electronic data logger, in which the applied jack-in pressure at respective pile penetration length is captured precisely.



TOA5	CR800	CR800	29167	CR800.Std.27	CPU:PileLength_L1.CR8	51844
TIMESTAMP	RECORD	pPileName	counter	PenetrationRange	TotalDepth	pPressure
TS	RN			M	MM	MPa
		Smp	Smp	Smp	Smp	Smp
22/9/2015 22:20	10054	18-pd25/ta3-ta5	1	0.5	547.5751	1.966094
22/9/2015 22:20	10055	18-pd25/ta3-ta5	1	1	1010.235	1.876273
22/9/2015 22:23	10056	18-pd25/ta3-ta5	1	1.5	1502.974	2.84435
22/9/2015 22:23	10057	18-pd25/ta3-ta5	2	2	2025.781	4.620801
22/9/2015 22:23	10058	18-pd25/ta3-ta5	2	2.5	2517.671	4.880284
22/9/2015 22:23	10059	18-pd25/ta3-ta5	2	3	3004.912	4.870305
22/9/2015 22:28	10060	18-pd25/ta3-ta5	2	3.5	3501.034	4.421198
22/9/2015 22:28	10061	18-pd25/ta3-ta5	3	4	4001.742	3.902231
[illegible]	10062	[illegible]	3	4.5	[illegible]	[illegible]
[illegible]	10063	[illegible]	3	5	[illegible]	[illegible]
[illegible]	10064	[illegible]	3	5.5	[illegible]	[illegible]
[illegible]	10065	[illegible]	4	6	[illegible]	[illegible]
22/9/2015 22:32	10066	18-pd25/ta3-ta5	4	6.5	6514.376	10.17967

Figure 22 Example of data recorded in the electronic data logger

A standard pile jacking procedure was formulated to prevent overstressing the installed pile. The jacking operation will be stopped at 100%, 150% and 200% of the allowable pile capacity for thorough checking of the results. Under normal circumstances, whereby the pile is successfully jacked to twice its allowable capacity, the pile will be terminated by maintaining the jacking load for at least 20 seconds with an incremental pile head settlement of not exceeding 2mm in three consecutive cycles. During the installation, if there is an observable reduction of pile reaction load increment and/or increasing rate of pile penetration being recorded, the pile jacking operation will cease and the pile will be re-jacked to 95% of the final jacking reaction load achieved. The allowable pile capacity is then downgraded to half of the reduced jacking reaction load to achieve proof load factor of 2. This is carried out so that the pile can still confidently take a reduced working load instead of complete abandonment of a damaged pile, in which attempt was made to load the pile to the anticipated working load with proof load factor of 2. With such controlled pile installation, a total of 336 piles out of total 1224 installed piles were downgraded without observable pile structural damage, but only unable to reach the full working capacity. Minimum pilecap design modification was required due to minimum piles to compensate pile groups with downgraded piles mentioned earlier. It would be probably a disaster in the pile design with many full compensation piles and pilecap enlargement if such controlled jacking procedures with the assistance of the pile jacking monitoring system were not available at the time.

4.2.5 Summary of case history

This case history demonstrates the values created through innovations and improvements to the existing practice of jack-in piling system in the karstic limestone geology. The use of real-time monitoring for jacking operation has enabled the pile capacity utilization factor to be maximized and preventing overstressing of the piles until irrecoverable pile damage. The proposed piling system is able to achieve both cost and time savings to the project and is highly recommended to be adopted especially for site with difficult ground conditions. The existence of lock-in stress is also proven with beneficial stiffening effects in the pile performance from the fully instrumentation programme during the jacking process of the test pile.

5. CONCLUSIONS AND RECOMMENDATIONS

This paper has highlighted the following aspects of jack-in piling system and recommended the practical solutions:

a. Gaining popularity of this piling system with the perceived belief on the advantages of on-site proof loading the installed piles, overcoming the driving damage from the rival driven piling system and more receptive installation information with static jacking resistance profile for all installed piles are discussed.

b. Common installation problems of pile penetration obstruction and the corresponding solution by pre-bored holes to achieve the desire pile penetration length with minimum embedment length for lateral stability and also to achieve end bearing resistance resting on or within competent founding stratum are highlighted. The adverse impacts of the pre-bored holes, likes toe softening problems, insufficient lateral support to the pile with annulus gap with buckling potential and lack of pile toe capacity if pile terminates above the base of pre-bored hole are also presented. For achieving the full pile toe capacity, the plastic zone (upper zone of 0.5~1.5 pile size and lower zone of 1.5~3 pile size for silty sand with fine content over 15%; upper zone of 1~3 pile size and lower zone of 3~5 pile size for sand) at pile toe is recommended to be kept away from the base of the pre-bored hole.

c. The vertical and lateral soil displacements can be induced by the injection of large displacement pile where open-ended pile with hollow annulus is commonly used to reduce the soil displacement. However, from the measurements in the case history presented, there is large variability in forming the fully developed soil plug to relieve such soil displacement. Statistically most of the soil plug length is about 2 to 25 times of the pile inner annulus diameter.

d. The behaviour of good short-term pile toe capacity in dilative soils, which is prompt to stress relaxation resulting in lower long-term pile toe capacity is demonstrated. The reliability of the high pile toe capacity with shallow competent stratum, where the component of pile shaft resistance is relative lower, is highlighted and cautioned.

e. The beneficial effect of the lock-in stress from statically jacking installation enhances the pile stiffness performance when the pile is subjected to test load and also subsequent service load is explained.

f. The innovation of the data logger monitoring device has shown advantages in revealing the short-term pile setup at temporary stoppages for repositioning the hydraulic grip in the jacking cycle and also pile joint welding. Potential computation of the work done on installed piles for carbon footprint assessment and good feedback control for stopping the pile jacking to prevent structural damage to the piles in karstic limestone formation are illustrated.

g. Two case histories are presented to demonstrate the problems of this jack-in piling system and solutions devised to overcome the problems.

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