Common Blind Spots in Ground Investigation, Design, Construction, Performance Monitoring and Feedbacks in Geotechnical Engineering

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ABSTRACT: In geotechnical engineering dealing with risks and uncertainties, the processes involved start from the investigation with the fundamental intention to attain better understanding of the subsurface conditions and acquisition of the engineering parameters for the subsequent engineering analyses, designs, detailing, tender documentation and calling, followed by design validation tests at field and construction problem solving. With the forensic investigation experiences by the author in the past, some interesting findings and surprises are compiled in this paper to illustrate these common blind spots at the aforementioned engineering processes. The importance of desk study and sound geological knowledge in planning of investigation programme have not received sufficient emphasis in the higher education system, thus resulting in significant wastage by the trained graduate in using the investigating tools and generating excessive amount of redundant information. Some of the mistakes are fundamental errors in perceiving the engineering behaviours when using the software with intuitive and illusive perception rather than based on sound engineering understanding. There is also strain compatibility issue in mobilising material strength of composite materials with drastic stiffness contrast when approaching failure state of a soil structure interaction problems. Design validation tests are crucial to ensure design methods adopted able to reasonably behave as intended. However, the tests usually do not reveal the overall behaviours of the design in actual scale and time factors, but rather a behaviours of a special case or prototype. Geotechnical instrumentation on a larger scale with time might be a more representative of practical performance with totality. This will be more useful for review and back-analysed of a big picture performance of the geotechnical structures.

KEYWORDS: Mechanism, Forensic investigation, Case study, Soil structure interaction

1. INTRODUCTION

This paper aims to share the work experience, engineering practices, thoughts and innovations arising from the author's 25 years of professional career in geotechnical engineering. Liew (2009) presented a brief summary on the role of geotechnical engineer in Malaysia.

2. COMMON BLIND SPOTS IN GEOTECHNICAL ENGINEERING

2.1 Planning, Execution and Interpretation of Site Investigation (SI)

As most practicing geotechnical engineers, subsurface investigation planning is an inception stage of geotechnical engineering, in which each is supposed to go through. Some went through with remarkable training on the use of investigating tools and some just have exposure with interpreting factual reports. It is not uncommon to notice that there is an obvious lacking of knowledge in general geology and geography when a geotechnical engineer plans site investigation programme. The level of desktop study performed up front of the exploratory works are usually far from satisfactory if a logical scientific sense is humbly expected. The understanding of geological and geographical conditions of the project site is often treated as an unrelated section to the following planning of exploratory works, which emphasise much on sample recovery for laboratory testing, in-situ testing at discrete locations and even geophysical survey for interpolation of the subsurface profiles in between the discrete exploratory probing points. Very little attention is given to the adjoining subsurface conditions beyond the project site boundary. Perhaps there is shortcoming in traditional geotechnical engineering courses in training of possession of macroscopic view of the site, but rather to capitalise all investigative efforts in exploring the localised site within the project boundary. There are many possibilities that the external conditions beyond the project site have serious impacts to the geotechnical designs. For instance, the hydrogeological conditions of a hill site project, where groundwater can fluctuate significantly resulting in serious instability to high cut in the design. Another example is the coastal area projects where the saline intrusion may pose durability design consideration of the embedded substructural elements and the tidal effects can cause significant uplift pressure in the design of a submerged basement.

Monitoring of certain site parameters to reveal the long-term variation to a seasonal changes or a single event trigger expected in the design is also an important investigation objective in the planning of SI works.

Because of over-emphasis of acquiring engineering parameters of a specific site for design input, there is little efforts invested to understand the genesis of ground formation, its formation sequence, potential alteration of the changes of stress field and hydrogeological condition after the disturbance of construction. One shall bear in mind that most investigation results may only be valid in its pre-disturbance conditions. Of course some practice of empiricism may still reasonably calibrate the post construction condition to the pre-construction without causing remarkable error and often this can be even noticed by the geotechnical engineer, who adopts the empirical approach. However, such empiricism might only remain reasonably valid with certain consistent practice strictly abided. Simply changing the practice beyond its range may cause serious problem.

If SI planning is performed with reasonable effort on desktop study with the following available information reviewed, the investigation planning can then be started off with reasonable and logical expectation of the ground conditions in advance. The investigation shall aim to validate the postulated geological model resulted from the desktop study rather than exploring the ground in total blindness without any clues. This approach will result in more precise and efficient investigative objectives and keep the generation of repeat SI information to a minimal level within same geological units.

a. Topographical and terrain maps
b. Geological and hydrogeological maps
c. Pre and post site disturbance terrain survey
d. Aerial photographs with historical land use development information
e. Available existing and adjacent SI information

With a sensible geological model established, then allocation of limited investigative resources can be prioritised to validate and acquire necessary engineering parameters for characterisation of the subsurface conditions for design purposes.

There is also a practice that, once the SI planning is established, the geotechnical engineer execute the planned investigation based on scheduled sampling, in-situ field testing and laboratory testing without reviewing the deviation of site conditions from the postulated geological model. Often, such practice renders many inappropriate sampling and testing. When deviation from planning is observed, necessary adjustment in the investigation strategy shall be timely made to attain the investigative objective. One shall understand that planning is just a reasonable aim by the investigator at best effort, the actual fieldworks will reveal the actual site conditions. This is why SI works requiring supervision by experienced personnel, who is fully aware of the investigative objectives and can make timely decision to adjust the execution based on actual site condition. Leaving SI works to the SI contractor alone without guidance from the planning engineer may have the risk of not yielding meaningful information for subsequent interpretation, which will defeat the intent of the investigation.

Geophysical survey methods are not well understood by many geotechnical engineers, who may be in the position in specifying such investigative tools. As such, the planning and execution of these geophysical survey remains as the duty of specialised contractors, who are non-engineering graduates. However, the gap in communicating the investigation objectives and expectation between the two parties is often huge. As a results, many unsuccessful results arise causing low confidence in these survey techniques. The powerful visual impacts of the interpretative presentation from these survey techniques has attracted the eyeballs of project clients and also the geotechnical engineers. Without in-depth understanding of the project geotechnical engineer in specifying the appropriate survey methodology, communicating with the survey specialists and collectively interpreting the survey outcome with his/her exploratory borehole probing at discrete locations, it will be an unrealistic demand to these method to be useful. It is the author's personal opinion that the project geotechnical engineer shall not overly rely on the specialist contractors to produce expected survey outcome unless he/she has reasonable knowledge of such survey technique. This is especially valid if the operator possess inadequate engineering knowledge in planning the survey configuration and understand the limitation of the method in attaining the investigative objectives other than producing computer-processed and interpreted outcome by the equipment vendor's software.

2.2 Blind Spots identified from Case Studies of Forensic Investigation

The following case histories from the author's forensic investigation experience presents the lessons learnt that are commonly overlooked by engineers in the application of geotechnical engineering knowledge and principles.

2.2.1 Erroneous external stability of piled retaining wall analysis

This case study involves a distressed piled retaining structure of 7.5m high over soft soils as shown in Figure 1. In view of the underlying weak alluvial deposits at the predevelopment natural stream as shown in Figure 2, the retaining structure was supported by five rows of vertical 200mm driven precast concrete square piles with carrying capacity of 450kN. During backfilling of the constructed retaining wall, excessive lateral movement was observed. As evidenced in Figure 1, rising groundwater level was evidenced from the water staining at weephole drains probably resulted from building up of seepage at the previous stream.

Investigation was conducted to assess all possible modes of wall failure and reveal the probable causes of the wall distress. From Table 1, it was obvious that the overall stability assessment using limit equilibrium method by slide method had over-estimated the safety margin of the piled wall and same for the lateral resistance of the foundation piles. The vertical effective stresses at the underlying soil beneath the wall for computing the sliding resistance of the wall were over-estimated without considering the reduction of vertical effective stress in the foundation soil due to the vertical support from the piles. An unrealistically optimistic safety factor was computed for accepting the wall foundation design. The lateral structural resistance of the vertical piles was also not adequate to provide the required equilibrium for lateral stability of the wall under the increasing lateral earth pressure after rising of groundwater level within the wall backfill due to extreme raining event.

Figure 1 Overall Site Condition and Evidence of Rising Groundwater Level in the Wall Backfill

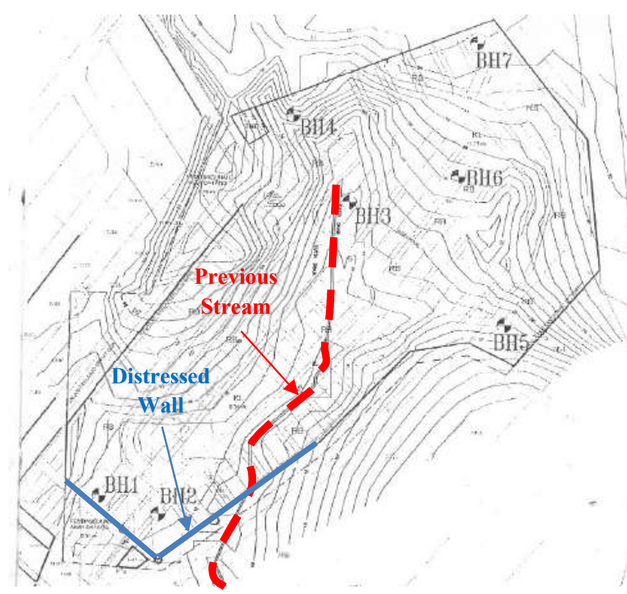

Figure 2 Predevelopment Stream and Distressed Wall

Table 1 Factors of Safety in Geotechnical Assessment

Ground Water Level	External Wall Stability			
	Overturning (>2.0)	**Lateral Stability (>1.5)**	**Bearing Capacity (>2.0)**	
RL40.4m	3.8	1.50	2.5	
RL42.5m	3.7	1.34*	2.5	
RL45.0m	2.9	0.97*	2.5	
	Global Stability			
Ground Water Level	**Without Self Weight**		**With Self Weight**	
	Short Term (>1.2)	**Long Term (>1.4)**	**Short Term (>1.2)**	**Long Term (>1.4)**
RL40.4m	1.19	1.70	1.25	2.39
RL42.5m	1.16	1.25*	1.24	1.92
RL45.0m	1.13	0.80*	1.17*	1.64

* Note : The underlined FOS means design inadequate. Ultimate limit condition prevails if FOS < 1.0.

The remedial solution is to excavate down the foundation formation further to cut off the piles to a lower level after demolishing the distressed wall. Then a compacted free draining crusher run material was placed over the cut piles to rebuild the same retaining wall with internal drainage and a low stabilising berm in front of the new wall. The details of this case study presented by Liew (2007) shows a treacherous situation on adding the vertical piles to support the wall, but in fact jeopardising the wall stability instead.

2.2.2 Embankment distress due to strain incompatibility of basal geotextile reinforcement

The role of basal reinforcement to provide a stable temporary working platform for embankment construction over soft ground has been a traditional approach in many constructions over soft ground. Sometimes, such temporary stability condition is also needed for the overall stability of the embankment construction with staged construction until the gain in undrained strength at every staged filling in the ground treatment design reaches a sufficient strength level to sustain the next temporary staged filling, final permanent embankment fill after removal of the surplus of the surcharge fill for a complete ground treatment. However, the adopted design tensile strength the embankment construction based on the misleading design strength at 5% typically provided in the product catalogue can over-estimate what has actually mobilised at site, thus resulting in non-representative safety margin in limit equilibrium stability assessment. To simultaneously mobilise the shear strengths of the embankment fill and also the underlying supporting soft subsoils, the tensile strain in the basal reinforcement shall be compatible with the strains in the aforementioned embankment fill and the subsoil. Otherwise, distresses like embankment cracking and even instability can develop if ignoring such strain compatibility or slippage at the interface basal reinforcement is allowed. This case history by Liew, et al (2016) present an instrumented embankment construction with extendible basal reinforcement to illustrate the strain compatibility issue in relation to the distress occurrence.

This case study involves an embankment with the use of extendible basal reinforcement of characteristic strength 600kN/m at 10% strain and prefabricated vertical drains as ground treatment option. The embankment distresses were observed with primarily longitudinal crack lines on the embankment surface along the main alignment direction during construction stage of the embankment. Layout plan of the embankment distress is as shown in Figure 3.

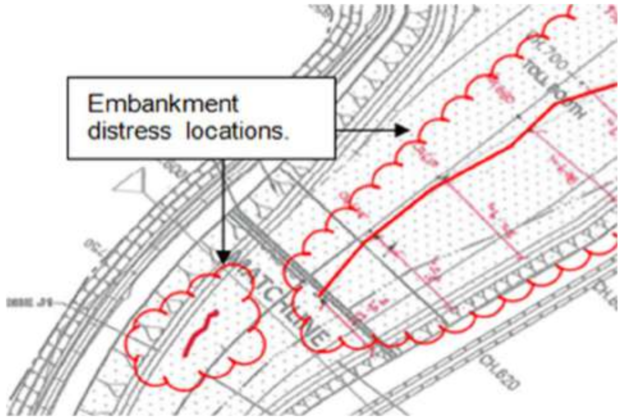

Figure 3 Layout Plan of embankment distress

Figure 4 shows a close up view of longitudinal cracks found from the embankment. The crack pattern was not of random nature, but rather a near straight line running along the longitudinal direction of the embankment alignment. From the trenching excavation across the crack line, the depth of the cracks was found to be about 300mm to 500mm from the formation level after partial fill removal of 1m from the staged constructed embankment to reduce embankment loading at the time of investigating the cracking. The crack is generally of "V" shape (i.e. wider gap at the top and diminishing as going downward). Water was poured into the cracks with seeping out observed at the bottom of the cracks confirming the depth of the cracking.

Figure 4 Longitudinal Crack found on the Embankment

As informed by the site team, the cracks appeared after a prolonged drought season, hence there was a suspicion of development of shrinkage cracks due to loss of moisture at top desiccate formation of the fill after exposing to very hot direct sunlight. However, the cracks did not have the feature of typical radial shrinkage resulting with the random honey comb crack pattern. In addition, the embankment fill was found very well compacted as evidenced by the observed resistance to the hydraulic excavator in performing the trial pit trenching during the site visit and inspection. It was not unreasonable to expect that the compacted embankment fill can be brittle and easy to crack when subjecting to any differential straining.

Generally, the development is within a relatively flat original ground and underlain by soft alluvial soils. There were three (3) stages subsurface investigation (SI) works carried out in year 2012, 2013 and 2014 respectively. All SI works mainly consist of vane shear tests to obtain and verify the performance of gain-in undrained shear strength after consolidation at each rest period. Figure 5 shows the profiles of vane shear test results before embankment construction in 2012 and after embankment construction in 2013 and 2014.

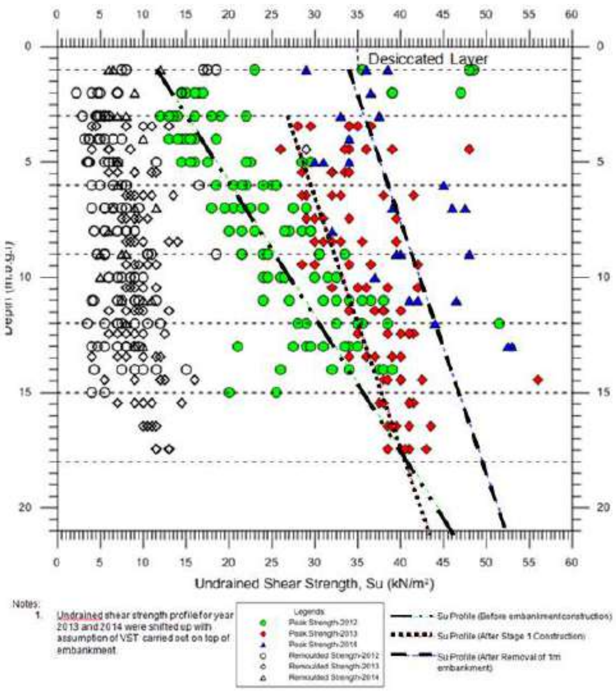

Figure 5 Interpreted Undrained Shear Strength at Different Stages of Embankment Construction

Bulk density of the proposed site prior to embankment construction works were summarised in Figure 6. The average bulk density of the alluvial subsoil samples obtained from the SI is 14kN/m3 for top 9m soil and gradually increases to 16.5kN/m3 for the subsequent depth from 9m to 18m of the alluvial clay layer.

It was worth to mention that bulk density at the surficial desiccated soil layer of 1m thick is taken to be about 17kN/m3 as top soil layer has subjected to compaction during the construction of the drainage blanket and subsequently improving its density.

Meanwhile, from the construction records of fill compaction, bulk density of the compacted embankment fill was found ranging from some 19kN/m3 to mostly 20kN/m3.

Based on the interpreted subsoil parameters, the proposed embankment and subsoil profile is summarised in Figure 7. There are obvious strength gain in the peak undrained shear strength in different time durations, whereas the remoulded undrained shear strength remains fairly consistent showing good quality of the testing. The gain-in peak undrained shear strength profiles after each stage of resting period are more prominent at the upper soil and diminishing as going done to a depth of about 18m.

Vertical settlement and lateral subsoil movement profiles of embankment were monitored by settlement gauges and inclinometers. Several settlement gauges and inclinometers were installed on the proposed embankment to monitor the performance of the embankment as shown in Figure 8. The instrumentation consists of settlement gauges for fill thickness control at every filling stage and settlement monitoring, inclinometers for horizontal subsoil displacement profile monitoring, standpipes for groundwater monitoring and piezometers for excess pore pressure monitoring.

The embankment filling started with 0.8m thick drainage blanket and prefabricated vertical drain (PVD) was installed from top of drainage blanket. After completion of PVD installation and installation of extendible basal reinforcement on top of drainage blanket, the embankment was filled up to each designed staged construction thickness and rest for consolidation.

Filling sequence and performance of the embankment at the distressed area was monitored by the settlement gauge (SG580) as shown in Figure 9. The filling sequence of embankment was divided into four stages which consist of Stage 1 filling (S1), Stage 1 rest period (R1), Stage 2 filling (S2) and Stage 2 rest period (R2).

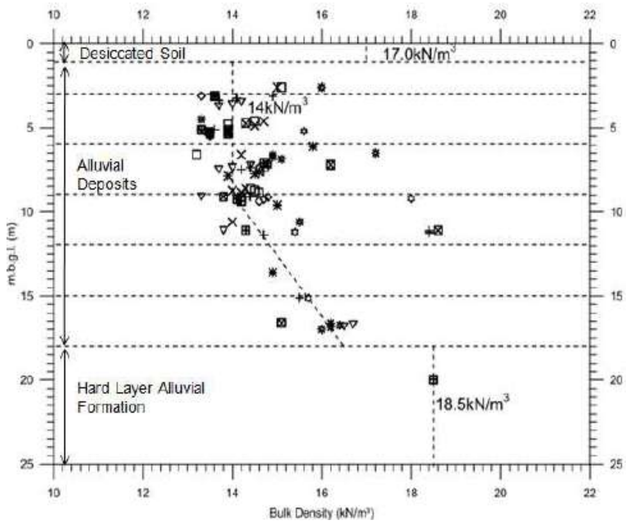

Figure 6 Interpreted Subsoil Bulk Density

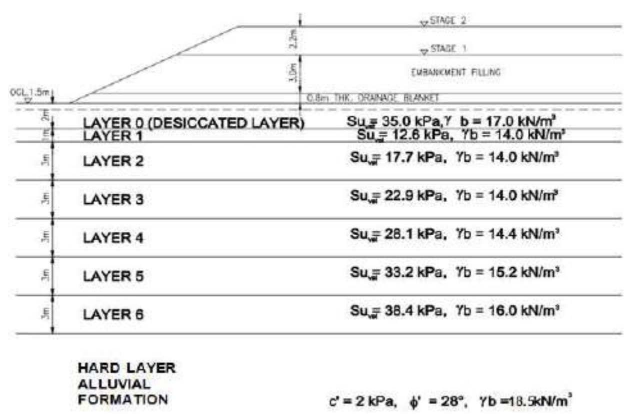

Figure 7 Typical Cross Section of Embankment in Stages

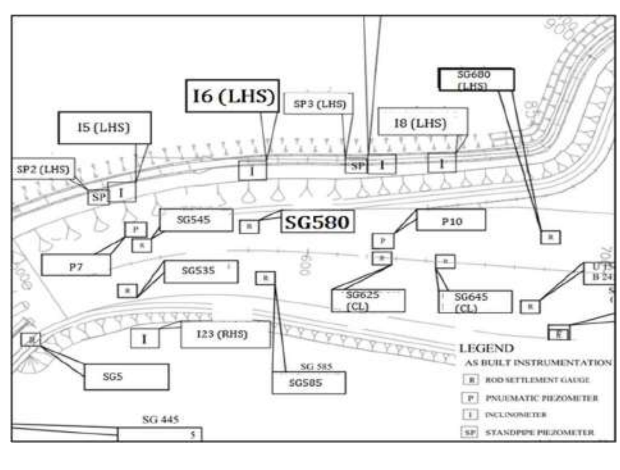

Figure 8 Installed Instrumentations on Embankment

Figure 10 presents the inclinometer monitoring results from inclinometer I6 installed at a location of about 4m beyond the embankment toe. It shall be noted that the inclinometer monitoring was only started 2 month after Stage S1. As such, it was expected that portion of lateral displacement will not be recorded in the inclinometer monitoring results. The recorded maximum lateral displacement is about 100mm. The top 11m indicated more lateral soil displacement implying larger plastic straining in the subsoils, which can develop into a slip surface leading to embankment distresses.

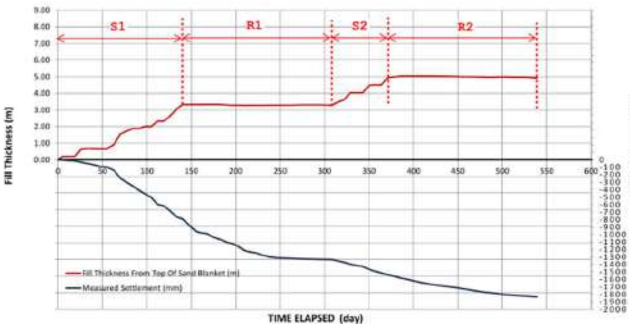

Figure 9 Filling Sequence and Settlement of Embankment Monitored by SG580

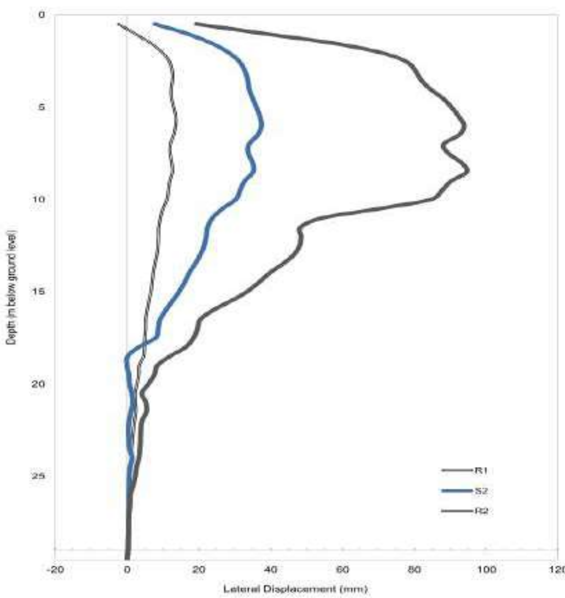

Figure 10 Inclinometer I6 Monitoring Results

Back analysis for embankment performance was carried out with Finite Element Analysis (FEA) method using engineering software "PLAXIS" to simulate the filling sequences in order to back analyse the performance of the extendible basal reinforcement. The FEA modelling is shown in Figure 11.

Back analysis with matching the computed settlement and lateral deflection profiles from analysis to the actual recorded profiles have been carried out to reveal on the performance of the extendible basal reinforcement during the construction stage.

Backfilling stages and construction sequence were modelled in accordance with the actual conditions (i.e. filling thickness and rest period). Back-analysis with reasonable range of subsoil parameters and coupled consolidation model have been performed to compare with the actual measurements. Findings from PLAXIS were tabulated in Figure 12. The back analysed settlement trend with time is compared with measured settlement profile of settlement gauge (SG580).

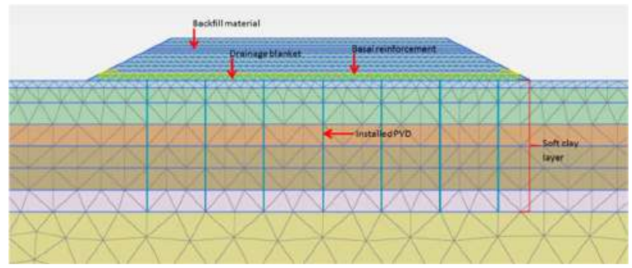

Figure 11 FEA Modelling For Back Analysis

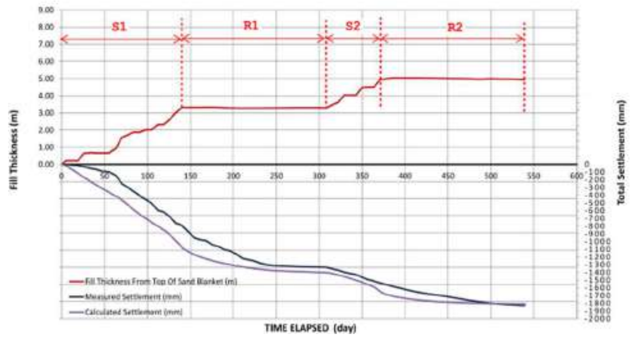

Figure 12 Comparison of Back Analysed Settlement Trend with Actual Measurement

In order to best simulate the actual condition of the constructed embankment, both settlement trend and lateral deformations profiles from back analysis are required to reasonably match with the actual performance of embankment. The lateral deformations from back analysis was plotted and compared with the actual conditions in Figure 13.

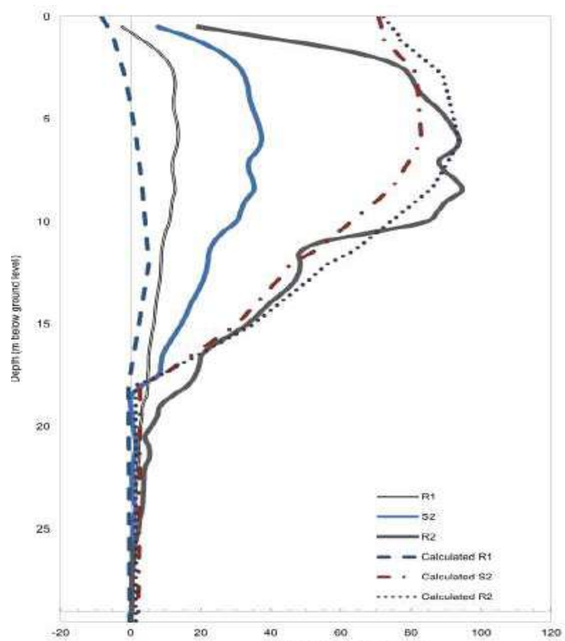

Figure 13 Comparison of Lateral Displacement Profile

As portion of the lateral displacement is not recorded in early stage of the embankment filling due to delayed installation of the inclinometer, thus, the back analysis was performed to estimate the possible lateral displacement before installation of inclinometer while the incremental lateral movement profile at subsequent construction stage still match well with the measured profiles. It was found that the subsoils have undergone lateral displacement of 160mm prior to the monitoring and it is expected that total lateral displacement of 260mm was experienced in the subsoil beneath the embankment before the cracks were observed. If taking this lateral subsoil movement of 260mm at 4m beyond the embankment toe and where the crack was discovered, the average mobilised tensile straining of the subsoil from the crack location to the embankment toe plus the distance of inclinometer I6 of 18m would be estimated to be 1.44%. This strain level is no way close to the characteristics strength of the basal reinforcement.

As both the lateral deformation and settlement profiles from back analysis matched reasonably with the measured profiles. It is fairly convinced that the back-analysis results have reflected the performance of the constructed embankment. Thus, the mobilised tensile stress and strain within the basal reinforcement from back analysis at each monitoring stage are summarised in Table 1. This analytical maximum tensile stress from the FEA refers to the localised strain at the interface between the anchorage length of basal reinforcement embedded below the embankment (where the anchoring resistance is developed at the basal reinforcement) and the active zone of the instable embankment from the side slope and the underlying supporting soil (where the destabilising force pulling the basal reinforcement). It is expected that the shear surface shall pass through this interface to create the maximum tensile force along the basal reinforcement.

The axial force of the basal reinforcement extracted from back analysis indicates that mobilised tensile strength of the reinforcement is about 67.4kN/m at end of monitoring stage (R2), which is only about 11.2% of the characteristic reinforcement strength of 600kN/m.

From Table 2, maximum strain of the basal reinforcement at end of monitoring stage (R2) is 1.12%. Maximum lateral deflection of subsoil of 425mm at the edge of the embankment has also been calculated at end of monitoring stage (S2).

Conventionally, design of the embankment with extendible basal reinforcement assumes mobilised strength of basal reinforcement with a tensile strain limit of 5 to 6%. However, it is worth to note that the optimistically assessed maximum average mobilised tensile strain of subsoils from the case study is at most 1.44% or lesser. Strain incompatibility between the basal reinforcement and embankment fill could cause embankment cracking and even instability can develop if ignoring such strain compatibility.

Table 2 Summary of Basal Reinforcement Performance from FEM

Stage	Mobilised Tensile Load / Strain	Maximum Lateral Deflection at Edge of Embankment (mm)
S1	40.6kN/m / 0.68%	267
R1	41.8kN/m / 0.70%	295
S2	64.6kN/m / 1.08%	400
R2	67.4kN/m / 1.12%	425

Based on the back analysis results, it was observed that average tensile straining of the subsoils is more than the maximum tensile strain in basal reinforcement. As such, it is deduced that the observed crack was possibly due to localised edge instability and possibly combined with lateral spreading of the supporting subsoils with inadequate strength.

From the back analysed basal reinforcement performance, the mobilised tensile strength and strain are far lesser than the conventionally adopted values for stability assessment using limit equilibrium stability analysis. In view of this, it is worth to limit the ability of the basal reinforcement to mobilise its structural strength in line with the strain limit of the compacted embankment fill if no tensile cracking of the brittle embankment is expected. However, higher strain in the underlying subsoil at maximum embankment loading maybe allowed if sufficient safety margin at the subsoils is allowed in the design to prevent catastrophic failure.

Since the crack pattern is more towards a near straight line running parallel to the longitudinal direction of the embankment, the formation of the observed cracks are likely related to some inherent mechanisms in the transverse section of the embankment and also the underlying supporting subsoils.

As the cracks are shallow and "V"-shaped by nature, it is likely a flexural crack with the tension zone at the top of the embankment. Furthermore, brittle behaviour of well compacted embankment fill is prone to cracking, when subjected to differential straining or localised straining near to the embankment slopes can be.

From the observation of the cracks at site and instrumentation results and possibly lower mobilisation of basal reinforcement, factor of safety could be lower than expected during the design stage. With such marginal stability condition, some localised plastic straining or even lateral spreading of the supporting subsoil at the embankment edge can be reasonably expected. The relative good strength in the compacted embankment fill before excessive distressing may contribute the slight extra safety margin in the overall stability which causes only shallow depth of longitudinal crack found on site.

2.2.3 Piled embankment failure distressing bridge abutments

Liew, et al (2010) presented a case study involving a construction of 3-span concrete bridge (Pier 1 & 2 at Ch 3286 & Ch 3307 and Abutment A & B at Ch 3266 & Ch 3328 respectively) over alluvial formation with ground level at about RL8.5m and river invert at about RL7.2m. Fill of 1.5 to 2m thick over the 10m thick weak alluvial deposit was required for the piling platform at RL10.50m. Generally, the fill thickness at the site is about 5.4m above the original ground level. Subsurface exploration confirmed that the underlying weathered residual soil of 7m thick is found above the weathered meta-sedimentary formation of primary sandstone derivatives. The subsoil parameters are summarised in Table 3. Due to higher embankment fill (5.4m) and relatively weaker ground condition at the Abutment B side, the approach embankment was supported on 200mm×200mm RC pile foundation at 1.8m grids for a stretch of 30m long and the lower embankment was on the treated ground using Prefabricated Vertical Drains (PVD) with surcharge. Trial Electrically-conducting Vertical Drains (EVD) was applied at the small area of 20m×20m replacing the originally designated PVD treatment. Both abutments were supported with one front row of raked piles and rear row of combined vertical and raked piles for both the lateral and vertical resistances. ϕ400mm Prestressed spun concrete piles were used for the abutment piles. Figure 14 shows the layout of distressed piled embankment and the embankment on two ground treatment techniques near Ch 3375.

Table 3 Strength Parameters of Subsoils

Subsoil	Unit Weight	Undrained Shear Strength, Su	
		Abutment A	Abutment B
Filled Materials	20 kN/m^3	40 kPa	40 kPa
Original ground	16 kN/m^3	12 kPa (about RL 6.5m – RL 9m) 20 kPa (about RL 1m – RL 6.5m)	15 kPa (about RL 8m – RL 9m) 12 kPa (about RL 6m – RL 8m) 10 kPa (about RL 3m – RL 6m) 15 kPa (about RL 1m – RL 3m)
Hard materials (SPT 'N' > 50)	18 kN/m^3	>250 kPa	>250 kPa

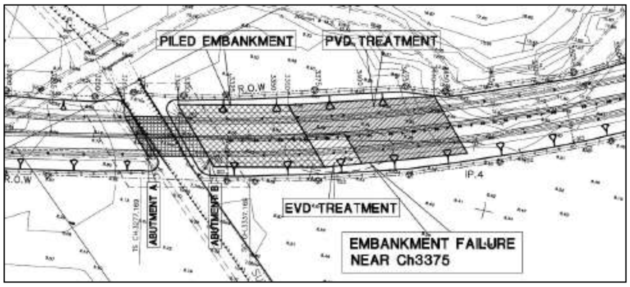

Figure 14 Layout of distressed bridge abutments and embankment

The following observations during the construction and after the distress were summarised:

a. First failure occurred at the PVD treated area immediately next to the piled approached embankment after the embankment fill was completed 6 days later.
b. Subsequently, development of tension cracks leading to sudden collapse of embankment fill was reported at the EVD treated area and the adjoining piled embankment after reaching 5m fill height.
c. Thereafter, spalling of concrete and gap opening at the bridge deck near Abutment B were observed.
d. After the failure of EVD embankment and piled embankment, most electrometric rubber bearings at Abutments A and B suffered observable shearing deformation as shown in Figure 15.
e. From the bridge bearing distortion at abutments and piers, it was confirmed that there was clockwise rotation on plan and global lateral movement of the bridge deck in the direction from Abutment B towards Abutment A. Bridge movement monitoring layout in Figure 16 revealed maximum bridge movement of 40mm in longitudinal direction as show in Figure 17. However, movement of the bridge before commissioning the monitoring works was not registered.
f. As shown in Figure 18, the filled piling platform has settled about 400 to 1000mm in magnitude beneath the piled RC slab and flexural cracks were observed at numbers of free standing piles at the piled embankments.
g. Slab of piled embankment was damaged and the slab movement led to 100mm gap at joints of the slab.

Figure 15 Shearing distortion of rubber bearing

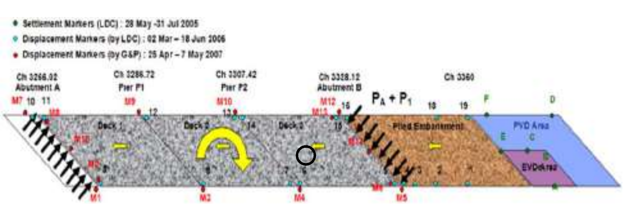

Figure 16 Monitoring layout

Back analyses of the collapsed embankment at the ground improvement areas revealed a slight increase of about 2kPa in the mobilised undrained shear strength comparing to the initial undrained strength of 10kPa. With the plastic deformation of the underlying weak subsoil under the embankment loading at the PVD and EVD treated area and marginally low safety factor (FOS), the piles would gradually approach flexural yielding condition and exhibit excessive pile movements and rotation at the plastic hinge formed. The lateral thrust of the relatively unstable embankment on treated ground with potential failure mechanism behind the piled embankment could have also imposed excessive flexural stress to the RC piles. The net horizontal thrust after deducting the lateral resistance of the group piles beneath the embankment then pushed Abutment B, bridge decks and Abutment A. The bearing distortion shape agrees well of the load path traversing from the Abutment B through bridge decks and the piers and finally reaching Abutment A. A schematic diagram of such scenario is shown in Figure 19.

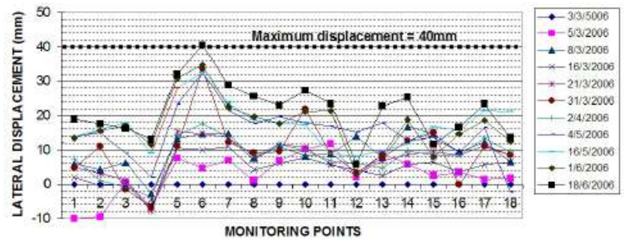

Figure 17 Movements in the longitudinal direction

Figure 18 Settlement of piling platform and flexural damage of embankment foundation piles

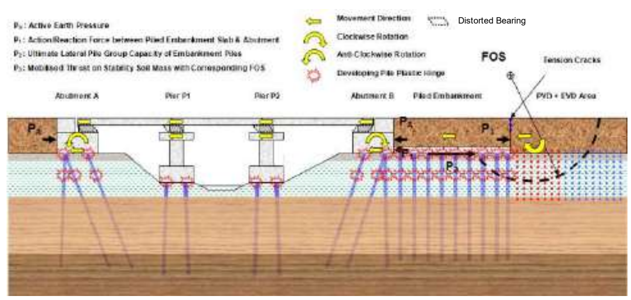

Figure 19 Schematic bridge movements

In assessing the embankment stability and giving the benefit of doubt, the lateral resistance from the piles to the bridge supports was maximised by assuming attaining ultimate limit state condition considering the excessive lateral movements of the piled embankment, abutments, bridge decks and bearing distortion as observed. Table 4 summarises of the computed ultimate resistance of all the bridge structure elements related to the lateral movements.

Assuming both abutment piles and piled embankment foundation piles had reached the ultimate pile group capacity simultaneously, the safety factor of the embankment stability in longitudinal direction is at best 1.14 to 1.26 depending on the pile eccentricity due to lateral displacement. It is also possible that some piles had compromised its lateral resistance ahead of others leading to even lower safety factor than the aforementioned marginal value.

Table 4 Summary of Ultimate Lateral Resistances

Elements	Ultimate Lateral Resistance	Remarks
Rubber Bearing (under vertical working load of 170kN/bearing)	44kN/bearing	From manufacturer's technical catalogue
Embankment Pile (391 nos. of 200mm×200mm RC pile)	39kN/pile (fixed head) Total Lateral Resistance = 371kN (pile group efficiency of 1.0)	Brinch-Hansen method using the modified limiting resistance with depth
Abutment Pile (18 nos. of ϕ400mm Class A spun pile)	102kN/pile (fixed head)	

Creeping foundation movements due to occurrence of plastic deformation of underlying weak subsoil will cause unacceptable serviceability conditions of piles and gradually undermine the structural integrity of the bridge structure. In fact, the entire system including the bridge, pile embankment and the embankment was at the marginally stable condition with very low safety factor. The summary of the safety factor of the embankment stability at the PVD and EVD treated embankment behind the piled embankment with consideration of the pile eccentricity effect is presented in Figure 20.

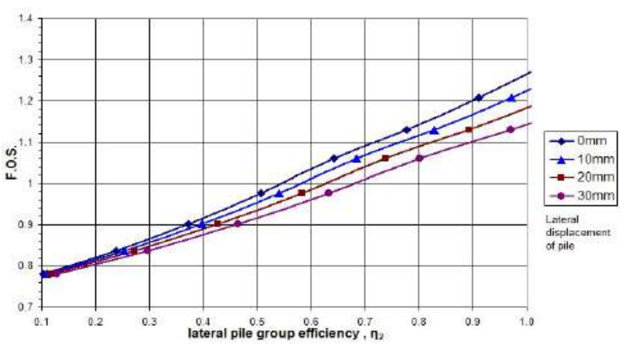

Figure 20 Safety factor of embankment stability with lateral pile group efficiency and pile eccentricity effect

As discussed by Marche & Lacroix (1972), lateral movements of weak foundation soils are likely to become significant when the embankment loading is greater than three times of the undrained shear strengths of the subsoil underlying the embankment. Stewart, et al (1992) also observed a bilinear response on the maximum recorded bending moment of abutment piles against the embankment loading, indicating plastic deformation around the piles when the embankment loading reaches 3 to 3.5 times the undrained shear strength. The embankment stability under these conditions corresponds to safety factor of about 1.5. As regard to the safety factor of embankment, Hunter, et al (2003) have reviewed thirteen trial embankments and concluded that the embankment will reach 60 to 70% and 75 to 85% of its failure height at the safety factor of 1.5 and 1.25 respectively. In this case, it is likely that the plastic deformation at the PVD/EVD treated area might have developed even before reaching the finished formation level (600mm below finished road level), in which an embankment failure could have occurred shortly after reaching the formation level.

For the remedial design, it was crucial to simultaneously remove the active lateral earth pressure behind both abutments to avoid unbalanced lateral loading and replace with geotextile reinforced backfill preventing lateral load from imposing to the abutments. The embankment removal and reconstruction were closely monitored using the movement markers. As for the distressed pile embankment structure, demolition of the distressed reinforced concrete slab and reinstalling new piles and reconstruction of slab for the embankment height exceeding 2.5m.

2.2.4 Unreliable facing capacity in soil nailed slope design with fully covered shotcrete/gunite facing

In soil nailed slope design, full facing covering the slope surface attached to the anchoring soil nails are usually considered with dual purposes, namely full surface protection preventing erosion from surface runoff and, more importantly, to attain high facing load at the nail head to mobilise higher nail structural strength when the geotechnical pull-out capacity is available with higher overall factor of safety for slope stability. However, this wishful design expectation might not be materialised when the supply of moisture between the slope material and the atmosphere is substantially cut-off by the full shotcrete or gunite facing. Without maintaining the equilibrium of moisture in the soil slope, there will be volumetric shrinkage of the soil slope resulted from depletion of moisture. The shrinkage can result in detachment of contact between the slope surface and hard rigid shotcrete/gunite facing surface. Figure 21 shows the evidence of shrinkage of soil slope forming a gap ranging from 50mm to 275mm beneath the shotcrete surface of the failed nailed slope. From the inspection of the soil material beneath the shotcrete surface, the moisture is remarkable lower than the exposed ground surface.

Figure 21 Gap Formation between Soil Nailed Slope and Shotcrete Facing

Figure 22 shows a typical nail force diagram of a soil nail embedded in the slope where the available nail resistance along the entire soil nail shall be the lowest envelope among the envelopes of passive pull-out resistance ($f_{s,p}$), active push-out resistance ($f_{s,a}$), nail structural tensile capacity (T_N) and nail head structural capacity (T_H).

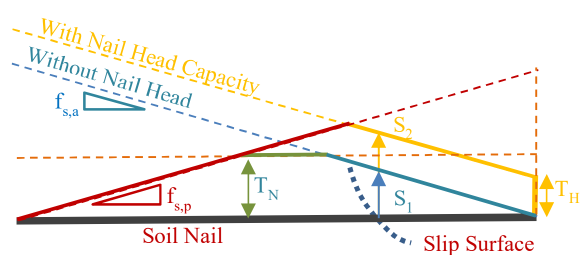

Figure 22 Nail Force Diagram with and without Nail Head Capacity

Consequentially the expected face loading (T_H) at the nail head will diminish resulting in reduction in mobilised nail resistance at any critical slip surface cutting through the nail from S_2 to S_1 if there exists a gap and thus reducing the overall factor of safety of the slope stability. As such, when considering the shrinkage factor resulting from depleting moisture content of fully covered slope surface, the porous facing design with adequately slope face exposure for maintaining the moisture equilibrium may be more appropriate to reduce variation of the resulting safety factor of slope stability. Such soil shrinkage is usually not considered by geotechnical designer for slope works as it is hard to believe its reality until being observed.

2.2.5 Illusive high end bearing pile capacity

Liew & Ho (2013 & 2016) presented an investigation consisting of 400mm reinforced concrete (RC) square pile installed onto a competent meta-sedimentary Hawthorndern formation in Kuala Lumpur with empty pre-bored hole to ensure minimum pile penetration length. 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 probable causes as identified in the investigation. Results of both contractually scheduled and investigative maintained load tests are presented and discussed.

As observing the rapid rate of disintegration of the exposed weathered bedrock formation and instability of many cut slopes formed in the same formation, it is believed that swelling and flaking behaviours of these formations can be prominent when subject 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 23.

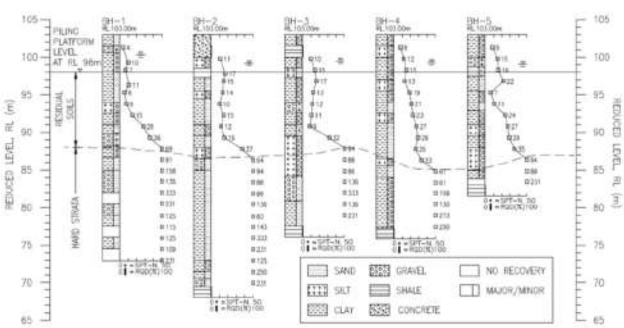

Figure 23 Borehole logs of subsoil profiles

For this case, jack-in installation 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. 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 pile in the oversized pre-bored hole without adequate lateral support. Finally, majority of the working piles were installed with a compromised 550mm diameter pre-bored hole as 500mm diameter pre-bored hole was found undersized resulting in premature termination for 400mm RC square pile.

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 tip as illustrated in Figure 24.

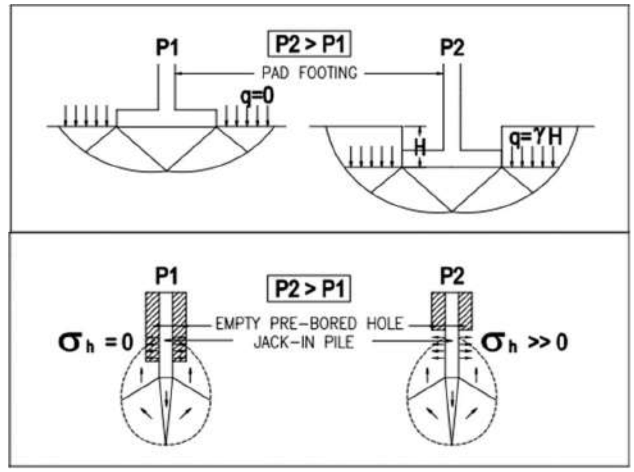

Figure 24 Pressure bulb and plastic zone for shallow foundation and pile foundation

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

MLT results in Table 5 indicate majority of the initially tested piles settled more than the requirement of 12.5mm at pile working load. MLT 1, 2 and 4 piles 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 5 piles, 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 as explained earlier.

All additional MLT piles (MLT 6 to MLT 8) had been previously installed with termination criterion reaching 2.2 times of working capacity but MLT 6 and 7 piles 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 three test piles in Figure 25(b) has 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. 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 rendering stiffer pile base response. The test results further enhance the findings of potential stress relaxation at pile tip due to insufficient stress confinement within the effective stress bulb of the end bearing pile tip 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 tip implies that this is solely a pile settlement problem.

Table 5 Performance summary of the contractually scheduled test piles and additional test piles.

MLT	Pre-bored Diameter (mm)	Pile Penetration below Piling Platform (m)	Max. Jack-in Load at End of Jacking (kN)	Max. Test Load (kN)	Pile Top Settlement	
					1×Working Load (mm)	Max. Test Load (mm)
MLT 1	600	9.40	2160	2220 1.71xWL	14.0	46.0
MLT 2	500	9.30	2600	2220 1.71xWL	23.0	42.0
MLT 3	550	12.50	2860	2600 2.00xWL	5.8	21.8
MLT 4	550	9.50	2860	1406 1.50xWL	16.5	24.5
MLT 5	550	13.50	2860	1950 1.50xWL	8.5	13.0
MLT 6	550	9.50	2860	1950 1.50xWL	15.1	42.4
MLT 7	550	10.50	2860	2400 1.85xWL	11.3	41.9
MLT 8	550	11.00	2860	2600 2.00xWL	10.3	50.4

MLT 1 was terminated at maximum jack-in force lower than other production piles due to the earlier targeted pile working load (WL) is lower (950kN) during 1st 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.

Piles installed into pre-bored hole without backfilling the annulus are exposed to the risk of pile tip softening and consequently leads to reduction in pile load carrying capacity and softer response in pile tip stiffness. The base 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 tip) would not affect the effective stress bulb near the pile tip, thus the pile end bearing capacity. Figure 26 shows a schematic diagram of the stress relaxation and the stress bulb of pile tip end bearing.

The depth of influence zone at pile tip is complicated and influenced by many factors such as angle of shearing resistance of the founding soil at proximity of pile tip, pile diameter, stiffness, in-situ effective stress at pile tip, 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 (J. Yang, 2006). Meanwhile, the influence zones for sand with $\phi' = 30^{o}$ are 1D to 3D upwards and 3D to 5D downwards (Hideki 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 tip founding materials within the plastic zones of pile tip.

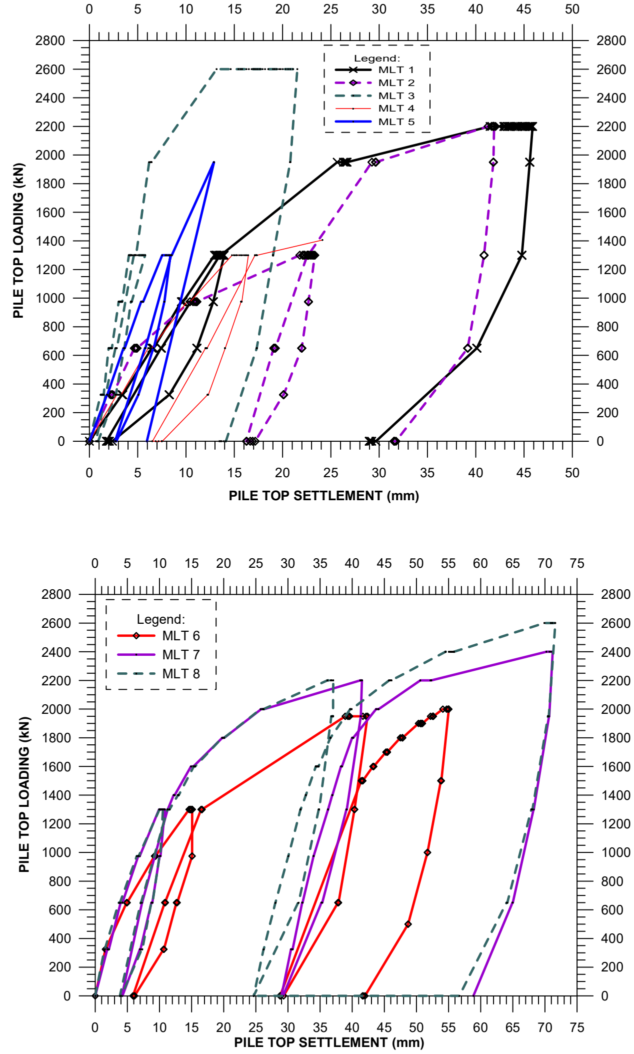

Figure 25 Pile top loading (kN) versus pile top settlement for (a) contractually scheduled MLT results and (b) additional MLT results

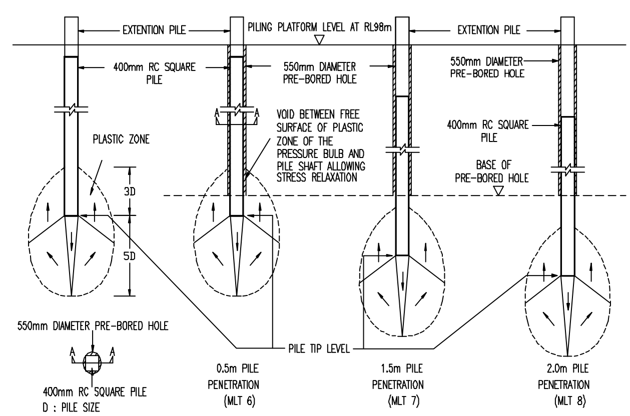

Figure 26 Schematic diagram of stress relaxation effect with relative position of pre-bored hole and pile stress bulb

This paper presents a case investigation of jack-in pile installation method with empty pre-bored hole to achieve deeper pile penetration within the competent meta-sedimentary formation to overcome any premature pile penetration length but unfortunately suffering time dependent pile capacity reduction problem. All the jack-in piles initially achieving the pile termination criteria during installation were primarily due to the high pile capacity developed from temporary high short-terms undrained strength. Subsequently the performance of MLT at selected working piles shows incomparably unfavourable performance with the performance at pile termination. Stress relaxation within the plastic zones of pile tip end bearing due to free annulus surface in empty pre-bored hole and possibly exaggerated with ingress of water at the pile tip softening the founding subsoil are suspected. This localized stress relaxation condition can significantly reduce soil strength, thus directly affecting the carrying capacity and settlement performance of mostly end bearing jack-in pile. The amount of pile capacity reduction is dependent on the subsoil material at pile tip founding level and pile penetration (embedment) below the base of pre-bored hole. However, the consequence of such pile tip softening is in fact a pile settlement problem rather than pile capacity issue. Further pile penetration under sustained imposed pile loading will allow regaining of the pile capacity to balance the pile working load imposed onto the pile.

The performance of three (3) additional MLT on working piles in this investigation provides clear evidence of the varying degree of pile capacity reduction with respect to the corresponding pile penetration below the base of oversized empty pre-bored hole without the annulus backfilled. To overcome the shortcomings, it is worthwhile to consider sealing off the annulus between oversized pre-bored hole and pile shaft to prevent ingression of water and remove the condition of free annulus surface that leads to softening of pile tip material within the plastic zones. This can be easily achieved by placing appropriate amount of cementitious grout into the pre-bored hole before lowering the pile for jacking operation. The depth of cementitious grout sealing shall sufficiently cover the upper plastic zone of the stress bulb after volumetric displacement of grout at pile termination. The recommended minimum grout sealing depth shall be approximately 5 times pile size above the base of pre-bored hole. It is always better to have the grout fully fill up the annulus gap in the empty pre-bored hole to avoid buckling condition of pile if the free standing length in the pre-bored hole is significant.

2.2.6 Non-linearity in Elasto-Plastic behaviour and hysteresis phenomenon of pile-soil interacting performance

There have been many studies on load transfer behaviour of pile foundation circled around the fundamental topic concerned by many geotechnical engineers. From observing load settlement behaviour, many researchers postulate the behavioural model of bi-linear, tri-linear or even non-linear hyperbolic function for simulating the recoverable elastic and non-recoverable plastic behaviour of overall pile response under loading at macroscopic level. Some even put remarkable effort in examining the localised load transfer of series of discretised pile segments with interfaces to soils at microscopic scale. Generally non-linear elastic behaviour is seldom observed in geotechnical materials. When the stress-strain relation start to exhibit non-linearity, unrecoverable plastic deformation is associated in the non-linearity.

Basing on the elastic behaviours of pile and also the embedding soil, all deformations within the system are expected to be fully recoverable with no residual deformation when fully unloaded. In short, the pile and soil can restore back to its initial state of deformation before loading was imposed. As for the unrecoverable plastic deformation, the deformation mostly comes from either slippage at the pile-soil interface or localised yielding or particle dislocation of the embedding soil with local shear stresses beyond the soil strength or both. Once the plastic deformation occurs, creeping behaviour under sustained loading of sufficient magnitude causing localised yielding when the redistribution of the stress field to reach new equilibrium and hysteresis phenomenon can be observed in statically cyclic loading process. The elasticity of the loaded materials and load path in dispersing the load imposition from the supporting pile to foundation soils, there will be different degree of stress mobilisation in the load dispersing process, particularly with a relatively large stress field system. Some are well under stressed, some at state of yielding, and some are stressed beyond the strength. The non-linearity of stress-strain behaviour is a gross summation of the different degree of stress mobilisation with unrecoverable plastic deformation. In both the forwarding and reversal of stressing process, localised yielding and slippage at pile-soil interface resulting to partial plastic deformation with energy loss during the loading or unloading process, thus increasing the non-linearity. It is such non-linearity causing the separation of the stress-strain paths in energy injection and energy recovery of the system.

Figure 27 shows a typical load settlement curve of static maintained load test results. The portion from Point 1 to 2 denotes linear elastic behaviour when there is no part of the pile-soil system attaining either interface slippage and dislocation of soil grains. Full recovery of elastic strain of pile structure and foundation soil is possible with this range of loading. However, when the pile loading enters beyond Point 2, either soil yielding or interface slippage at the upper portion of the pile-soil system occur. The lower pile segments may remain elastic behaviour. When the loading is stressed beyond Point 3, more soil yielding and interface slippage occur and extend to lower pile segments resulting more irrecoverable straining. Upon reaching the first maximum test load at Point 4 following with unloading process to Point 5 and subsequently to Point 6, partial restoration of the stored elastic strain energy in the pile-soil system takes place.

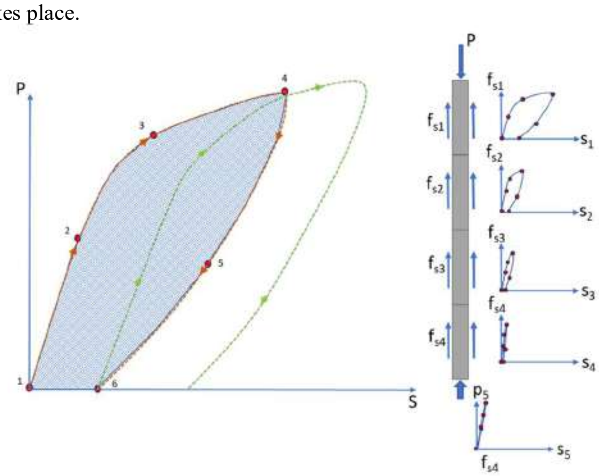

Figure 27 Schematic diagram of Pile Load Test Results

When the restoration of elastic strain between the pile and the soil becomes inconsistent due to either soil grain dislocation or interface slippage, the reaction at the upper pile segments can be in a reverse direction, hence preventing full release of the elastic strain in the piles becoming the lock-in load in the pile. As illustrated in Figure 28, the static equilibrium of the pile-soil system at this state is attained with downward drag force at the upper pile segment and upward resistances from the lower pile segment and pile toe. Maximum compressive load is located at the neutral plane where the downward and downward resistances meet. When the test pile is reloaded again, normally the initial stiffer response at the beginning of reloading can usually be observed when comparing to the earlier loading cycle. This is primarily due to much lower elastic shortening (δ_{top} - δ_{NP}) with relatively high pile stiffness when reloading of the pile by taking over the downward drag load in the soil above the neutral plane to reach static equilibrium. It can be logically expected that the pile deformation, δ_{NP}, at the neutral plane when first attained in the loading cycle shall remain unchanged in the unloading and reloading cycle as the upward resistance is the same for these three loading cycles.

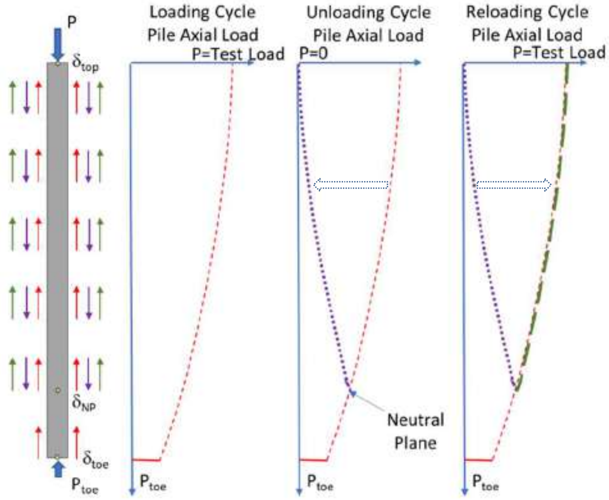

Figure 28 Schematic diagram of Pile Load Test Results

It will be interesting to examine the possible pile stiffening response when such look-in load exists in the pile due to installation process and, also preloading before pile testing. In jack-in pile system, such effect is more prominent than driven pile as static jacking can preserve better lock-in load in pile comparing to dynamic percussion piling method. For cast-in-situ bored pile, such lock-in load may only momentarily exist during the volumetric expansion due to thermal hydration. After cooling down, even tensile load can exist in the pile if not slight compressive load.

Due to the potential high creep potential when the stress-strain behaviour of a pile subject to loading with remarkable plastic deformation in embedding soil and slippage at the pile-soil interface, it is suggested to observe the stainable stabilised pile loading as the test load where the initial high rate of creeping settlement attenuates to attain the static equilibrium. For instance, when the pile is loaded reaching the aforementioned state, the recorded loading onto the pile will reduce from the last incremental test load to a slightly lower, but stable load reaching the static equilibrium. For practicality, the conventional maintained load test procedure to maintain the test load will not be useful to determine the maximum test load, but rather spending the unnecessary time in pursuing the intended test load with additional pile penetration and strain hardening.

3. CONCLUSION

From the past experience of the author's professional career, the following messages can be summarised:

a. In planning geotechnical investigation, desktop study will help to optimise the resources required to yield meaningful outcome for subsequent engineering design and construction purpose. Exploratory boreholes and testing shall not be abused to obtain repeated and redundant data.
b. Analysis assumptions used in handy commercial software
c. packages if not carefully understood can be detrimental in its optimistic output leading to catastrophic design decision. The danger of unrealistic soil resistance in computing the safety factor of global stability for a piled retaining wall with no account taken to reduce the effective vertical stress from the pile support wall self-weight has clearly demonstrated.
d. Inappropriate design parameter from technical data sheet of basal reinforcement used in permanent embankment design leads to problem of incompatible strain mobilisation with respect to the weak supporting subsoil. There is an over-expectation of basal reinforcement performance in fulfilling serviceability limit state of a permanent embankment design.
e. The PVD ground improvement treatment to support an earth embankment abutting to a piled embankment of larger thickness was unfortunately incompatible to a stiff bridge abutment and weak lateral pile support of piled embankment. The relatively higher lateral support has attracted remarkable lateral load to structurally fail the vulnerable abutment piles and embankment piles. The settlement of temporary working platform shall not be overlooked in soft ground condition that potentially results in large free standing pile length, which reduces further the pile lateral resistance.
f. Soil shrinkage of fully covered shotcrete surface in a soil nailed slope due to depletion of moisture content can reduce the nail head capacity substantially, which subsequently reduces safety factor of slope stability.
g. Stress relaxation and softening can significantly reduce pile toe capacity in mostly end-bearing jack-in pile in weathered meta-sedimentary formation. The relaxation can be due to insufficient confining stress near to the pile toe resulting from empty pre-boring hole for ensuring minimum pile penetration.
h. The non-linearity and hysteresis in pile behaviour are mostly due to interface slippage and soil yielding with soil grain dislocation. However, interface slippage and soil yielding cause lock-in load in the pile and further stiffening the pile-settlement performance.
i. For practical determination of maximum test load in a pile test, it is suggested to have the pile loaded reaching the plastic state and record the final stable pile loading in the static equilibrium with specified limit of creep settlement rate.

From the few case studies presented above, it is not difficult to observe blind spots in many applications of geotechnical engineering if invalidated perception is intuitively taken for expecting design performance. Without in depth observation and understanding of the underlying operational principles of the design and its variation of performance with time, instant failure or gradual distressing between construction and operation are not uncommon. Many valuable site observations will help the designer in making appropriate design assumptions, which would not be invalidated in the service condition later.

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