Failure Investigation of Piled Reinforcement Soil Wall & Excessive Movements of Piled Embankment at Soft Ground, Malaysia

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ABSTRACT: This paper presents two case histories covering a collapsed piled supported reinforced soil wall at weak deposits under extreme weather condition and another case of excessive movement of piled road embankment approaches affecting the bridge abutments. The first case history demonstrates an ultimate limit state condition under the most unfavorable meteorological events of which the failure was expected and justified from the design review, whereas the second case shows the problem of the consolidation settlement of working platform resulting free standing condition of the embankment piles with high lateral loading from the non-pile supported approach earth embankment of lower height. Both cases involve the reaching of ultimate lateral pile resistance of the supported structure. Sequence of events established from investigation evidences leading to the failure are illustrated in the paper and remedial solutions are also presented.

INTRODUCTION

This paper presents two case studies sharing the interesting forensic findings on piled supported structures in taking lateral loading.

1. CASE STUDY - A

1.1 Introduction

The project site is located within a new residential and commercial mixed development at the southern region of Peninsular Malaysia. The collapsed structure is a combined 2m high reinforced concrete (RC) wall sitting atop a 8m high reinforced soil (RS) wall supported on 400mm thick RC slab and six rows of ϕ300mm driven prestressed spun concrete piles at weak thick alluvium deposits overlying a more competent very hard residual soils. The 255m long combined wall was constructed as the building platform at RL29m for a commercial complex in which there is a 3.5m deep and 6.5m wide monsoon drain in front of the wall. The soffit of 400mm thick RC slab is at about RL18.6m, whereas the invert of monsoon drain varies from RL16.5m to RL 17.5m towards upstream. Figures 1 and 2 show the layout plan of the project site and cross section of the combined wall respectively.

Generally, fill consisting of medium stiff to stiff clayey SILT and clayey SAND (N=6 to 10) was found at the top 4 to 5m below the initial platform at RL20m. Subsequently, very soft to soft CLAY material (Su=40kPa) of about 4m thick was detected overlying a stiff to very stiff sandy SILT (N=10 to 30) of about 10m thick. The groundwater level fluctuated between RL14.0m and RL18.0m. The subsoil profile and soil conditions of the project site are summarised in Figure 3.

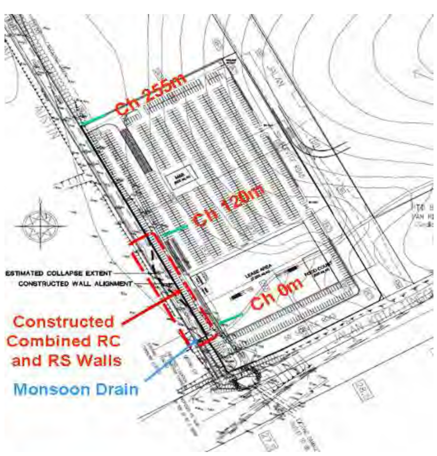

Figure 1 Layout plan of the RS wall

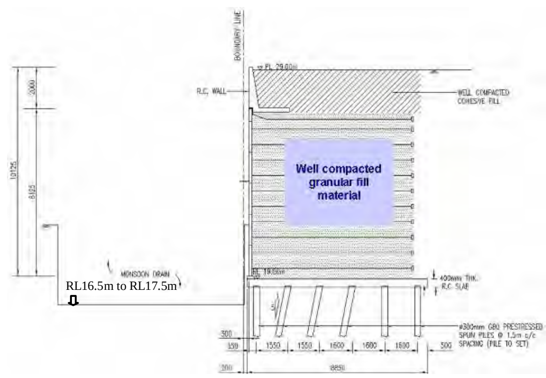

Figure 2 Cross section of the original design of combined walls

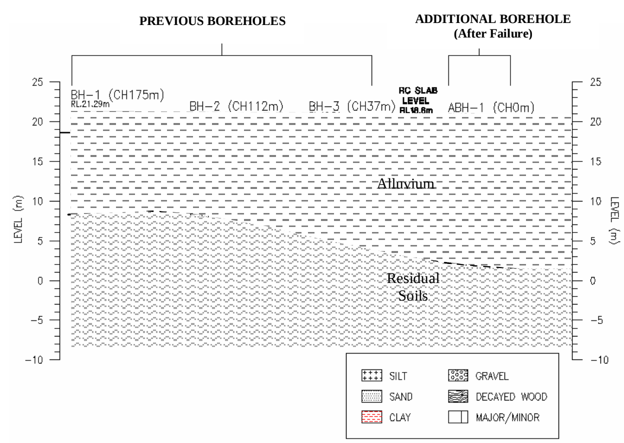

Figure 3 Subsoil conditions of the site

1.2 Chronology & Observations of Combined Wall Collapse

In early Jan 2007, a portion (Ch 0m to Ch120m) of the nearly completed combined walls collapsed towards the monsoon drain after prolonged intensive heavy rainfall. Before the collapse, the affected wall was only left with the balance of backfilling work of the completed upper RC wall and some with the erection of the upper precast RC wall stems at the beginning chainage.

The following observations at the failure scene are summarised:

a. Collapsed retained ground with water stagnation was observed at the tension cracks on the granular backfill and the upper cohesive backfill.

b. Wet wall panels and traces of watermarks were observed implying evidence of seeping water through the panel joints. The highest level of observed seeping watermarks was at the top of the RS wall indicating high water level behind the wall panels as shown in Figures 4 and 5.
c. The base slab was still well attached to the laterally translated RS wall. As shown in Figure 6, the upper part of the exposed piles had suffered flexural hinge damage at both the pile head and 1.8m below the slab soffit probably under excessive lateral loading resulting in lateral translation and rotation of piles towards the monsoon drains.

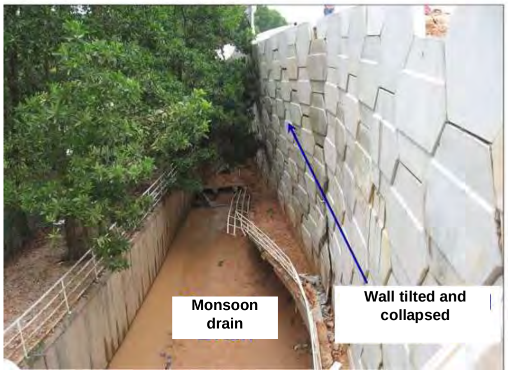

Figure 4 Side view of the collapsed combined wall

Figure 5 Observed watermarks on wet wall panels

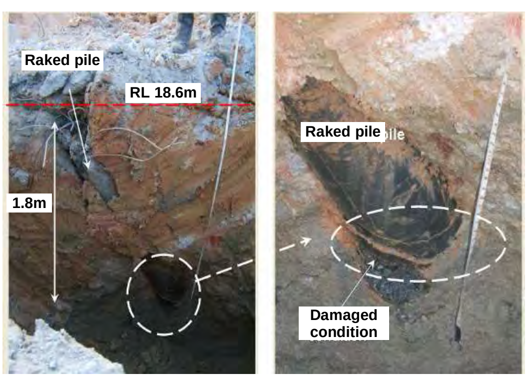

Figure 6 Damaged foundation piles

1.3 Investigation & Geotechnical Assessment

Following the wall collapse, additional subsurface investigation was carried out to confirm the ground conditions for investigating the causation of failure and proposing remedial design. Rainfall data for the corresponding period was also gathered for the investigation.

The following aspects as tabulated in Table 1 were examined in the investigation of probable failure condition of the combined walls and also the review of remedial design.

Table 1 Probable failure conditions

Type of Analysis	**Assessed Elements**		
Total Imposed Loadings on Wall & Foundation	Assessment of Lateral Earth Pressure with Water & Vertical Loads		
Pile-Soil Interaction of Foundation	Axial, Shear & Moment Loading Distribution on Piles		
Structural & Geotechnical Capacities of Piles	Axial (Struct/Geo)	Shear (Struct/Geo)	Moment (Struct)
External Wall Stability	Sliding	Overturning	Bearing Capacity
Overall Limit Equilibrium Stability	Factor of Safety against Instability		

1.4 Investigation Findings

Based on the assessment, the following findings can be deduced:

a. The foundation pile design, at service groundwater condition, had achieved optimum design requirements with adequate safety factors.
b. Rise of water table to RL25m after prolonged antecedent rainfall and follow by a triggering rainfall induced excessive lateral force on the foundation piles supporting the combined walls, in which the ultimate limit condition was reached.
c. Excessive imposed lateral stress exceeded the ultimate lateral resistance of the foundation piles resulting in the collapse of central portion (CH40-CH70) of the combined wall and pulling the adjoining wall from both ends.

1.5 Proposed Remedial Works

The following remedial works were proposed at site:

a. Dismantle the collapsed combined wall and demolish the original RC slab,
b. Replace the disturbed weak foundation soils with crusher run,
c. Install 6 rows of new offset raked and vertical foundation piles,
d. Construct new RC slab with same reinforcement details and reinstate damaged monsoon drain,
e. Re-erection of RS wall with additional weepholes, horizontal drainage pipes and drainage layer behind the wall panels. Additional weepholes and horizontal drainage pipes were also introduced to the intact wall survived in the failure incident,
f. Re-erect the new RS wall with free draining backfill materials and top compacted cohesive fill for the upper RC wall,
g. More proper construction controls and precautionary measures such as temporary cut-off drains of larger capacity, sequence and compaction of backfill granular material and close monitoring works on both intact and re-erected RS wall were carried out. Figure 7 shows the cross sectional view of the remediated wall.

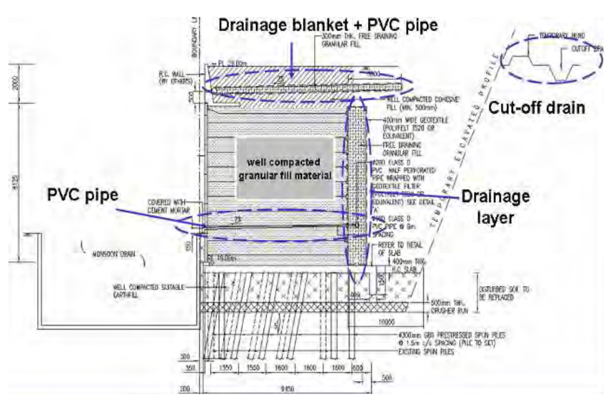

Figure 7 Cross section view of remediated combined wall

2. CASE STUDY – B

2.1 Introduction

The project located at the State of Selangor, Malaysia, involves a construction of 3-span concrete bridge (Pier 1 & 2 at Ch 3286 & Ch 3307, Abutment A & B at Ch 3266 & Ch 3328) 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 2. 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 "ICP PHC" Class Prestressed spun concrete piles were used for the abutment piles.

Table 2 Strength parameters of subsoils

Subsoil	Unit Weight	Undrained Shear Strength, Su	
		Abutment A	Abutment B
Filled Materials	20 kN/m³	40 kPa	40 kPa
Original ground	16 kN/m³	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/m³	>250 kPa	>250 kPa

Figure 8 shows the layout of distressed piled embankment and the embankment on two ground treatment techniques near Ch 3375.

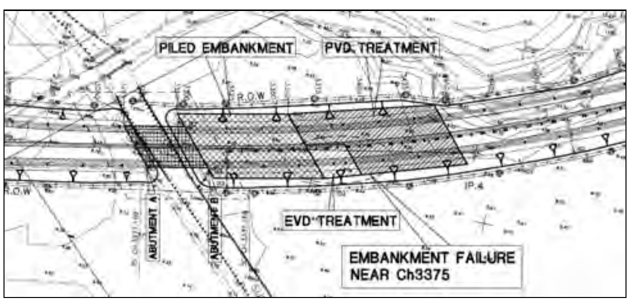

Figure 8 Layout of distressed bridge abutments and embankment

2.2 Site Observations

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 9.
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 10 revealed maximum bridge movement of 40mm in longitudinal direction as show in Figure 11. However, movement of the bridge before commissioning the monitoring works was not registered.
f. As shown in Figure 12, 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 9 Shearing distortion of rubber bearing

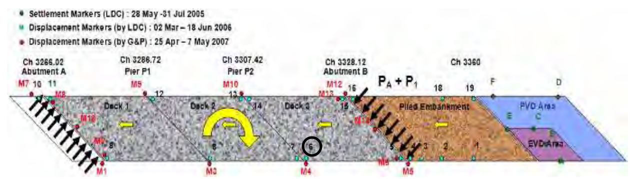

Figure 10 Monitoring layout

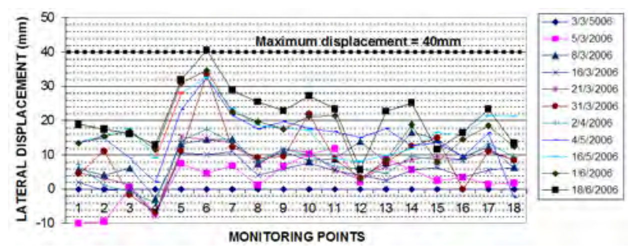

Figure 11 Movements in the longitudinal direction

Figure 12 Settlement of piling platform and flexural damage of embankment foundation piles

2.3 Geotechnical Assessment & Forensic Findings

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 13.

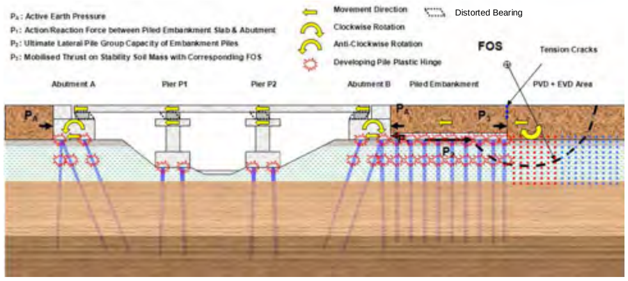

Figure 13 Schematic bridge movements

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 3 summarises of the computed ultimate resistance of all the bridge structure elements related to the lateral movements.

Table 3 Summary of ultimate resistance

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)	

Assuming both abutment piles and piled embankment foundation piles had reached the ultimate pile group capacity, 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. 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 as shown in Figure 13 is presented in Figure 14.

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.

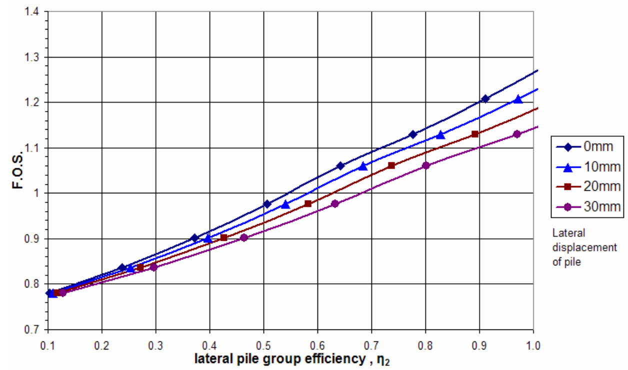

Figure 14 Safety factor of embankment stability with lateral pile group efficiency and pile eccentricity effect

2.4 Proposed Remedial Works

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, it was suggested to demolish and reinstall with new piles and RC slab for the embankment height exceeding 2.5m.

3.0 CONCLUSIONS

From these two case studies, it is obvious that the weak lateral resistance of the foundation piles in supporting heavy vertical loading can often be overlooked in the conventional retaining structure design despite optimum pile design had been well achieved in the first case with the serviceable groundwater condition, but it just failed suddenly with worst possible condition of water table. Huge contrast between the vertical resistance and lateral resistance of concrete piles, and the inherent brittle failure in lateral pile resistance can easily lead to poor robustness of foundation design. As such, it is suggested to consider ground improvement technique to strengthen the weak underlying foundation soils for supporting heavy gravity retaining structure as a first priority instead of brittle concrete pile foundation. If concrete piles were to be used for a specific reason, it is important to have raked piles with lateral self-balanced pile arrangement to improve its design robustness in taking lateral loadings with uncertainties in addition to the vertical loading.

4.0 REFERENCES

[1] Hunter, G. & Fell, R. (2003), "Prediction of Impending Failure of Embankments on Soft Ground", Canadian Geotechnical Journal, pp209-220.

[2] Marche, R. & Lacroix, Y (1972), "Stabilite des clues de points etablies sur des pieux traversant une couche molle". Canadian Geotechncial Journal 9(1), pp2-24.

[3] Stewart, D.P., Jewell, R.J.& Randolph, M.F. (1992), 6th Australia–New Zealand Geomechanics Conference, Christchurch, New Zealand, pp199-204,1992.