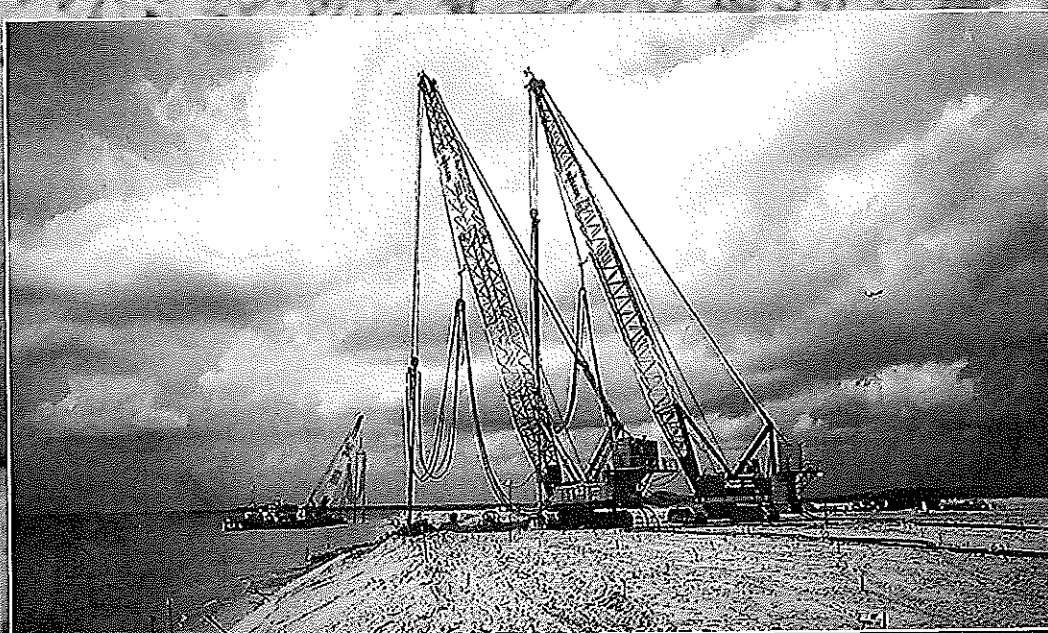




ELSEVIER GEO-ENGINEERING BOOK SERIES
SERIES EDITOR: JOHN A. HUDSON
Volume 3

GROUND IMPROVEMENT

Case Histories



EDITED BY
BUDDHIMA INDRARATNA
and **JIAN CHU**

Chapter 23

Failures of Ground Improvement Works in Soft Ground

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ABSTRACT

Various ground improvements have been used to support embankments on soft ground. The success of the ground improvement works depends on many factors from planning, investigation, analysis, design, specification of works, construction and closed supervision by design consultants. Flaws in any of the above stages would compromise the effectiveness of a ground improvement. The failure of ground improvements can either be short-term ultimate limit state failure (e.g. slip failure and tension cracks) or long-term serviceability limit state problems (e.g. excessive differential settlement). This paper presents three case histories of failures related to ground improvement works in soft ground, namely vacuum preloading with vertical drains, stone columns and piled supported embankments. The causes of failures, remedial works proposed and lessons learned are discussed.

1. INTRODUCTION

Various ground improvements have been used to support embankments on soft ground. The success of the ground improvement works depends on many factors from planning, investigation, analysis, design, specification of works, construction and closed supervision by design consultants. Flaws in any of the above stages would compromise the effectiveness of a ground improvement and even cause failure. This paper presents three case histories of failures related to ground improvement works in soft ground, namely vacuum preloading with vertical drains, stone columns and piled supported embankments. The causes of failures, remedial works proposed and lessons learned are discussed.

2. EMBANKMENTS TREATED WITH THE VACUUM PRELOADING METHOD

Two embankments at a site in Peninsular Malaysia (State of Selangor), namely Embankment A and Embankment B, were reviewed by the authors. Other than measuring

the settlements of the embankments with time, piezometers were also installed in the subsoil beneath the embankment at three different depths of 3, 6 and 8 m respectively, in the very soft cohesive soil. The measurements of these piezometers were taken during construction of the embankment and during the resting period. Embankment A failed not long after reaching the final height but Embankment B, which is not far away from Embankment A and employs the same ground treatment, did not fail. Figure 1 shows the cross-sections of the embankment and subsoil profile.

The embankments were constructed on a very soft silty clay 4.5 m thick and underlain by a layer of soft sandy clay to a depth of about 12 m. Beneath these very soft to soft-cohesive soils is a layer of loose clayey Sand followed by layers of medium to stiff silty clay and sandy clay. Figure 2 shows the undrained shear strength (s_u) profile of the subsoil obtained from the field vane together with the adopted design values. The sensitivity of the soft clay ranges from about 2 to about 10 and can be categorized as sensitive to extra-sensitive clays according to the definition of sensitivity by Skempton and Northey (1953).

2.1. Construction and Monitoring

Instruments like piezometers, settlement gauges and vacuum meters have been installed with the intention of monitoring the performance of the embankments treated using vacuum preloading. The original vacuum design suction is 40 kPa. For this case history investigated by the authors, only the results of the piezometers showing the response of pore water pressures will be discussed as the results of the settlement monitoring and vacuum meters did not show any trend to indicate signs of failure.

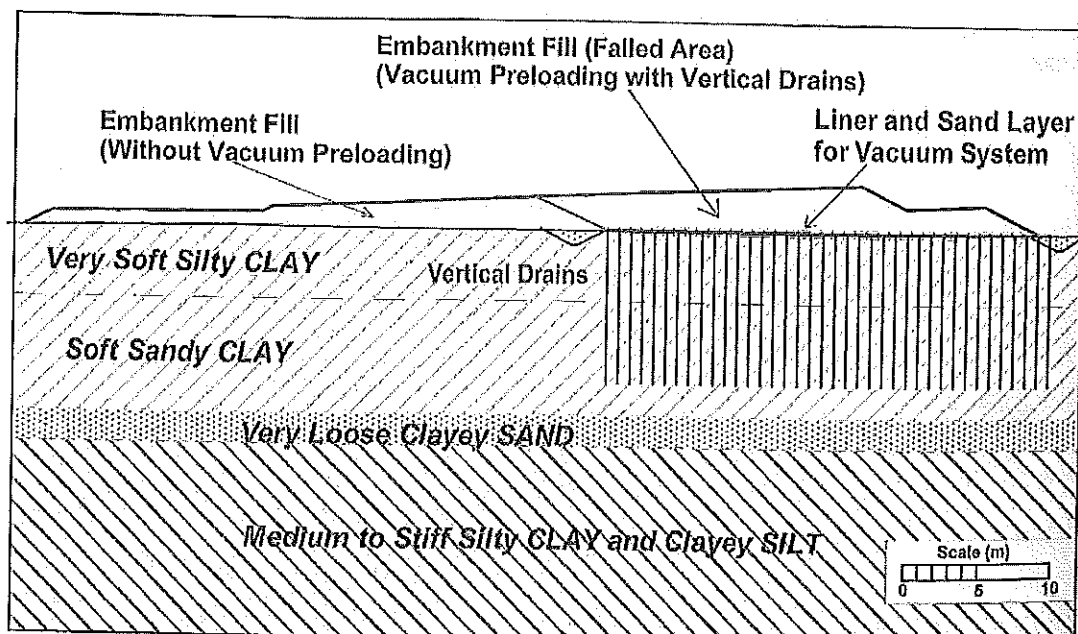


Figure 1. Cross-section of Embankment A.

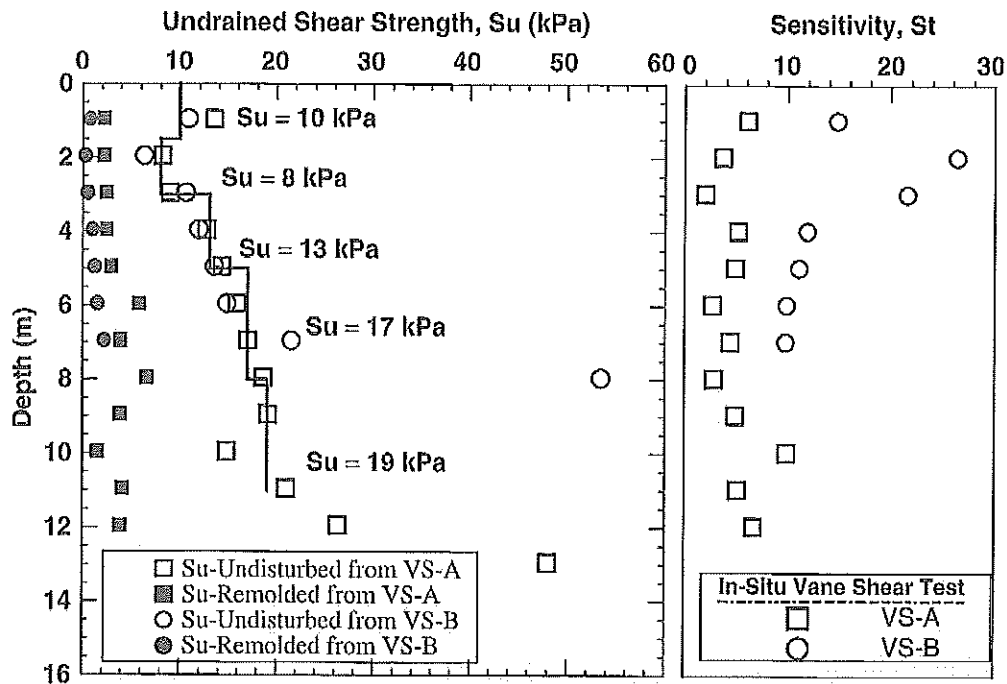


Figure 2. Undrained shear strength profile.

The construction sequence of Embankment A and changes in pore water pressures of the piezometers in the subsoil at depths of 3, 6 and 8 m throughout the construction are shown in Figure 3. Embankment A failed not long after reaching the final height.

As shown in Figure 3, from Stage C filling onwards, the pore water pressures measured from piezometers PZ-A2 and PZ-A3 at depths of 6 m and 8 m respectively, increased until failure at day 162 after reaching the final fill height. Piezometer PZ-A1 at depth of 3 m did not show an increase in pore water pressure until it was out of order after day 130. In brief, the measurement from piezometers PZ-A2 and PZ-A3 at Embankment A indicated that the vacuum suction at these depths was not functioning properly to prevent increase in pore water pressures in the cohesive subsoil with respect to the filling.

The trend of increase in pore water pressures has been observed for more than 1 month but no contingency action was taken by the contractor, who was also responsible for the design, to investigate the causes and to stop filling until the pore water in the subsoil returned to the allowable design values.

The independent analyses carried out by the authors employing both Undrained Strength Analysis (Ladd and Foott, 1974; Ladd, 1991) and the Effective Stress Method also indicate that the design of the vacuum preloading was acceptable if the vacuum system performed as designed. Therefore, if the contractor had taken the initiative to review the monitoring results of the piezometers installed in the subsoil as part of the required procedures for vacuum preloading, the failure could have been prevented because the trend of increase in the pore water pressures in the subsoil was very clear and easily identified.

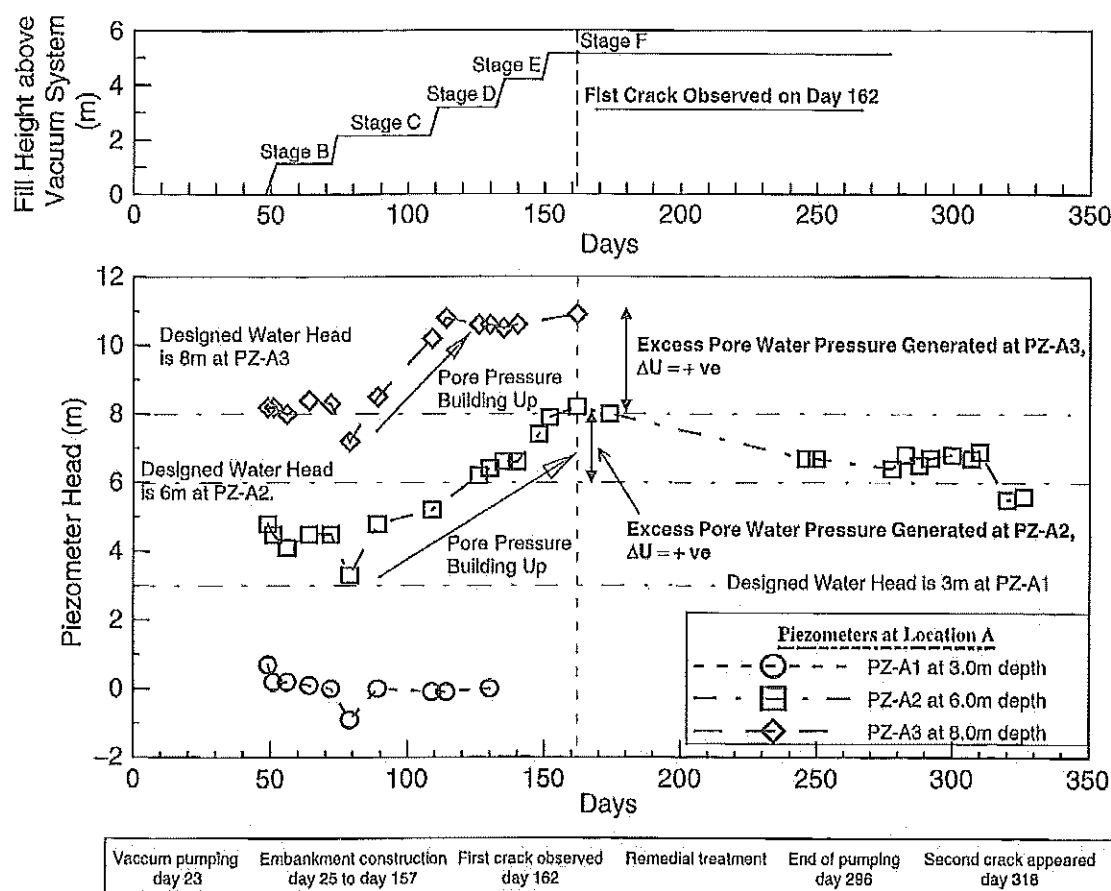


Figure 3. Construction sequence and monitored pore water pressure changes of failed embankment A.

Embankment B, which was not far away from Embankment A and employed the same vacuum preloading ground treatment, was successfully constructed. Figure 4 shows the changes of pore pressures in the piezometers at different depths throughout the construction of Embankment B. The filling sequence is also presented in the same figure for easy reference. The pore pressures in all the piezometers installed were within the designed range indicating the vacuum suction performed as per design. This finding further confirms that the failure of Embankment A was not due to design but rather due to lack of proper construction control.

The observations from two embankments clearly show the importance of the observational approach when employing the vacuum preloading method for embankment construction. It also shows the effectiveness of the observational approach in identifying problems well before failure, provided that the site engineer supervise the work and the design engineer constantly review the monitoring results obtained from the site. In brief, the failure of Embankment A would have been prevented if engineers had observed the changes of pore water pressure in PZ-A2 and PZ-A3 and took necessary precautionary action.

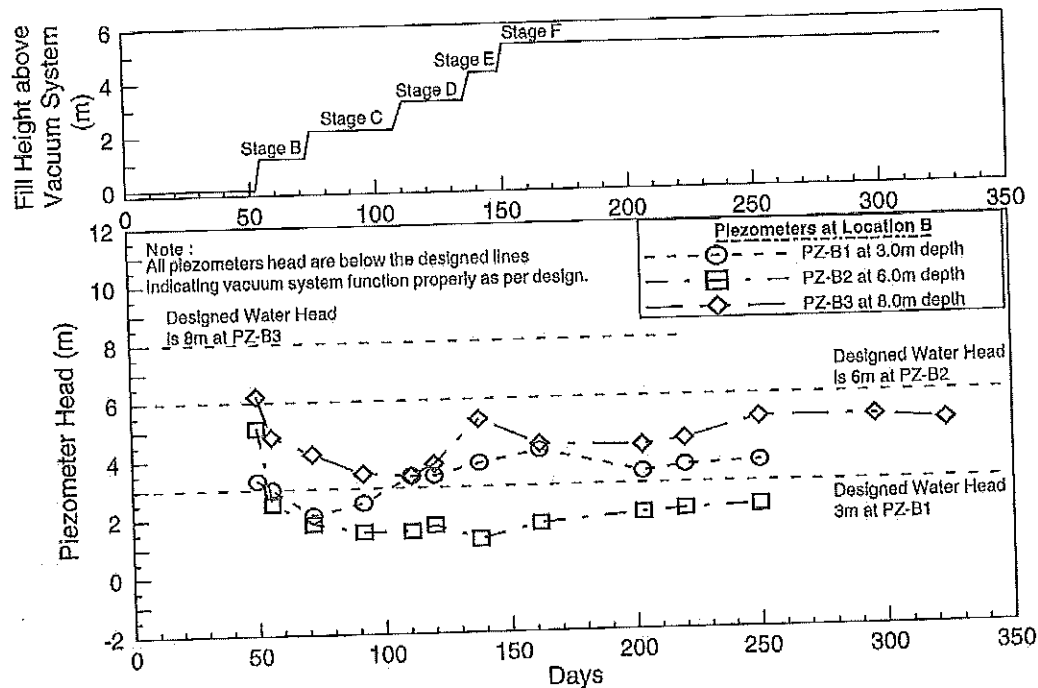


Figure 4. Construction sequence and monitored pore water pressure changes of successfully Constructed Embankment B.

2.2. Recommended construction control for vacuum preloading with prefabricated vertical drains

It is very important for the engineer responsible for the design of the vacuum preloading system to be involved in supervision and monitoring of the performance of the system during construction and also post construction. The monitoring results should be immediately reviewed once available and compared with the allowable design limit to check for any abnormalities that could cause failure or influence the expected performance of the treatment. Some details of the construction control of embankments are described by Tan and Gue (2000).

Following are some of the general procedures to be implemented:

(1) Instruments to be used are:

- Piezometers (preferably vibrating wire type) at different depths of the subsoil to be treated.
- Settlement gauges on the original ground level before filling to measure the settlement of subsoil.
- Extensometers (e.g. Sondex probe extensometer, spider magnet type, etc.) to measure the settlement at different depths of the subsoil.
- Settlement markers on top of the embankment after reaching the final filling level.
- Displacement markers or inclinometers at the toe of the embankment to measure lateral displacement.

- (2) During filling, the frequency of monitoring should be daily or every other day depending on the factor of safety (FOS) against slip failure for each stage of filling. Usually, the FOS is higher for the early stages of filling but as the height of the embankment increases, the FOS reduces. In brief, if the FOS is critical, monitoring should be carried out daily.
- (3) After completion of each stage of filling and during the rest period, the frequency of monitoring can be reduced depending on the allowed rest period, and some general guidelines are as follows:
 - Once a week for 1 month,
 - Once every 2 weeks for 2 months,
 - Once a month for the rest of the rest period.
- (4) The design engineer should closely coordinate with the supervising engineer on site. If possible, the design engineer should be on site monitoring the performance of the ground treatment during filling.
- (5) The monitoring results should be interpreted, checked and reviewed immediately after reading on site. These results should be compared with allowable design values and ultimate design values to evaluate the relative FOS on site. If the monitoring results are doubtful, redo the monitoring. If necessary, carry out back analyses with the monitored results.
- (6) If the monitored values exceed the allowable design values, contingency measures should be taken such as :
 - Increase the frequency of monitoring
 - Stop filling or slow down the filling rate
 - Find out the causes of the abnormal results
 - Carry out analyses incorporating the findings from the monitoring results to validate the design parameters used (e.g. soil strength, unit weight, etc.).

3. FAILURES OF EMBANKMENTS SUPPORTED BY STONE COLUMNS

After the first failure of Embankment A treated with vacuum preloading using prefabricated vertical drains as discussed in Section 2, remedial works using stone columns were proposed and constructed. The embankment with stone columns also failed when the embankment reached 3.2 m of the planned 5.5 m fill height. Figure 5 shows the embankment after the second failure.

The stone columns proposed and constructed by the contractor using the vibro-replacement process (wet method) were to act as remedial measures to support the reconstruction of the new embankment. The stone columns are of 1 m diameter with grid spacing of 2.5 m center-to-center up to a depth of 20 m. Crushed stones of size 15 mm to 100 mm were used as a backfill medium for the stone columns. During reconstruction of the embankment on top

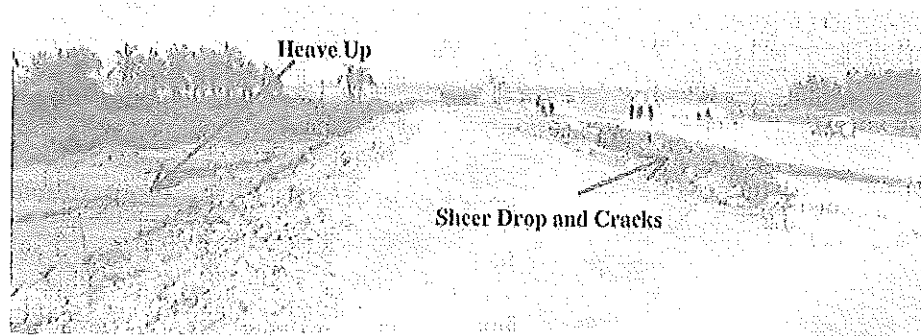


Figure 5. Failure of Embankment A treated with stone columns.

of the stone columns, the embankment failed with large cracks (as shown in Figure 5) when the fill height reached 3.2 m, which is 2.3 m lower than the required fill height of 5.5 m.

The authors' review indicates the design by the specialist contractor only used Priebe's methods (1995) to check on the stability and settlement of the subsoils treated with stone columns. There was no evidence of separate calculations using other methods to check on the bulging and general shear failures of the stone columns when determining the ultimate bearing capacity; these failure modes are not sufficiently covered in Priebe's method. Table 1 lists some of the methodologies available for bulging and general shear failure check.

From the investigation by the authors using disturbed strength of the subsoil on the methods listed in Table 1, the results show that generally bulging failure is not a concern but 'general shear failure' is grossly inadequate.

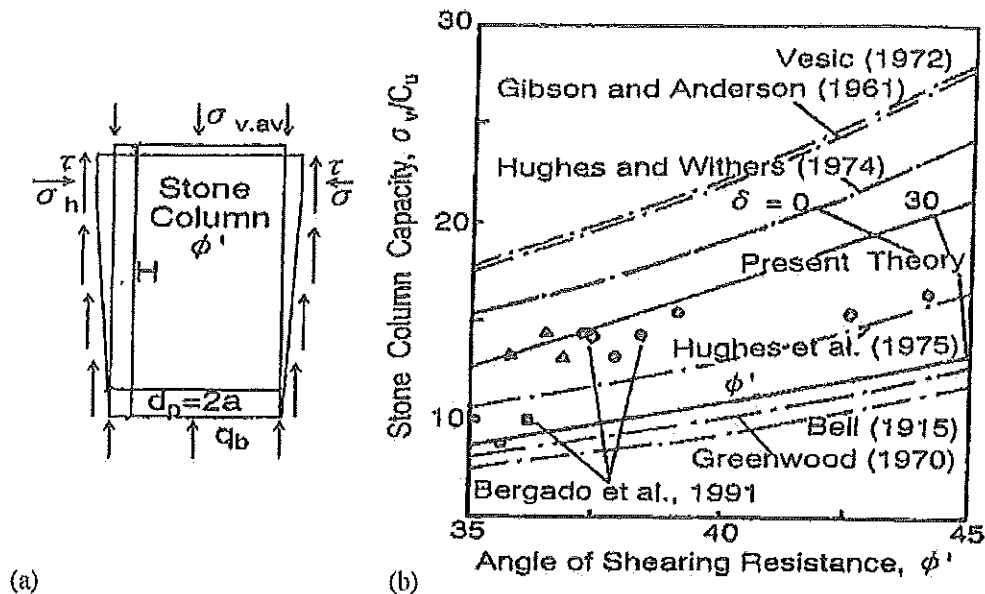
Most of the methods listed in Table 1 are reproduced in a graph by Madhav and Miura (1994) together with their proposed method as shown in Figure 6. It is observed that there is a large range of possible ultimate bearing capacities when using different methods and this tends to cause confusion to design engineers. Therefore, it is recommended that when using stone columns in very soft ground (e.g. $s_u < 15$ kPa) or as remedial measures for reconstruction of failed embankments, attention shall be given to probable failure due to general shear. In addition, load tests and close monitoring of the instrumentation via the observational method should be carried out to verify the design.

3.1. Failures of embankment C

Embankment C is located about 2 km away from Embankment A. It was initially treated with prefabricated vertical drains. Cracks were observed at the embankment after reaching the surcharge level with a fill height of 3.9 m and immediate action was taken to lower down the embankment height to finish road level (FRL) which is 1.5 m lower. The embankment was observed for 2 months and since no further cracks developed, the consultant agreed to refill the embankment to surcharge level. Slip failure occurred during the filling of the surcharge. After the first Failure, the contractor decided to use stone columns as remedial measures to strengthen the subsoil so that the embankment can be

Table 1. Methods for estimation of ultimate bearing capacity of stone columns.

Mode of failures	References
Bulging	Greenwood (1970), Vesic (1972), Datye and Nagaraju (1975), Hughes and Withers (1974), Madhav et al. (1979)
General Shear	Madhav and Vitkar (1978), Wong (1975), Barksdale and Bachus (1983)

**Figure 6.** (a) Stresses on stone column (b) Comparison of different methods (after Madhav and Miura, 1994).

reconstructed. However, the embankment supported by stone columns failed again after reaching the fill height of 3.9 m.

The subsoil at the Embankment C area generally consists of organic soil with a thickness of about 4 m. Underlying the organic soil is a layer of very soft to soft silty clay with a thickness of about 10 m followed by stiff to very stiff clayey silt. Similar to the second failure of Embankment A, the stone columns for Embankment B were also being designed using Priebe's method (1995) only without other separate checks on the bulging and general shear failures as listed in Table 1. The investigation carried out by the authors indicates that the stone columns bearing capacity against general shear failure is grossly inadequate, resulting in the failure of the embankment.

3.2. Lessons learned from the two embankment failures

The failures of Embankment A and Embankment B treated with stone columns were mainly due to inadequate design. The authors are of the opinion that when designing stone columns to treat very soft ground (e.g. $s_u < 15$ kPa) or as remedial measures for an embankment, attention should be given to probable general shear failure instead of over relying on a single method. For remedial measures, it is also important to determine the

representative "disturbed" (remoulded and regaining of strength through thixotropy effects) strength of the subsoil to be used in the analyses. In addition, load tests shall be carried out on stone columns to verify the design assumptions as there are large differences among methods of analysis. In brief, further works are necessary before a reliable unified and comprehensive design method is available for stone columns supporting embankments on very soft ground.

When stone columns are used to treat very soft ground, it is recommended that the observational method (Peck, 1969) be used with proper instrumentation and closer monitoring to prevent failure if there is a slight doubt about the design methodology. Many embankments on very soft ground treated with stone columns have been successfully constructed with the help of the observational method.

4. LONG-TERM "MUSHROOM" PROBLEM FOR A PILED EMBANKMENT WITH INDIVIDUAL PILECAPS

A piled embankment with individual pilecaps was constructed in the 1980s as part of the expressway in the northern state of Malaysia. The original design principle of this solution was intended to rely solely on the arching of the embankment materials to transfer the load to the pilecaps as the soft compressible subsoil between the pilecaps settled under consolidation. However, a few years after the expressway was opened to traffic, the embankment continued to experience large differential settlement in the form of localized depressions that required regular maintenance and repaving. The protruding parts of the embankment with pilecaps as if "punching through" the embankment look like "mushrooms", and therefore this term is used to describe the problem. Figure 7 shows features of the "mushroom" problem. Meanwhile, Figure 8 shows the differential settlement ("mushroom") observed between the area with and without pilecaps after excavation at a depth of about 300 mm. The authors were involved in the investigation of the causes of problems and design of long-term remedial measures.

The area where the 'mushroom' problems are prominent is predominantly in areas underlain by Quaternary age deposits and comprises marine deposits such as clay, silt and sand with sea shells. The alluvium deposits mainly consist of very soft to soft silty clay and clayey/sandy silt with the presence of intermittent sand layers with some sea shells and wood remnant. The deposition environment of the Quaternary deposit is believed to be from a marine environment.

The subsoil conditions of the site were obtained from a series of subsurface investigations carried out in September 2001 and October 2002 with boreholes, piezocones, Mackintosh probes, hand augers and vane shear tests. The shear strength and consolidation parameters and modified critical state parameters (Vermeer and Neher, 2000) for design and analysis are summarized in Tables 2 and 3.

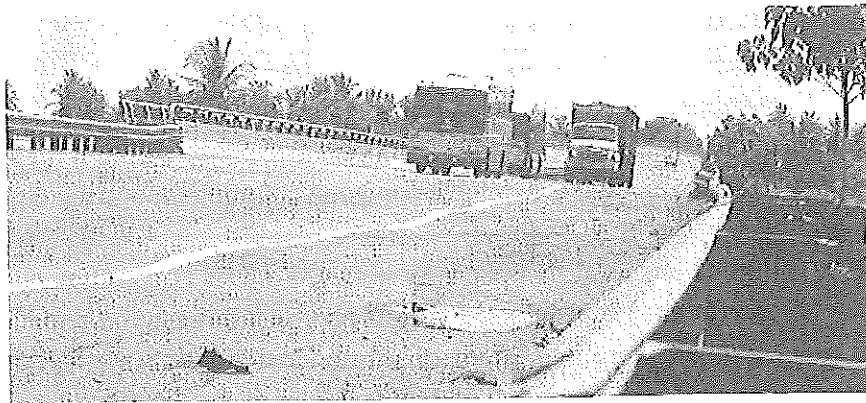


Figure 7. 'Mushrooms' and undulating surface on expressway.

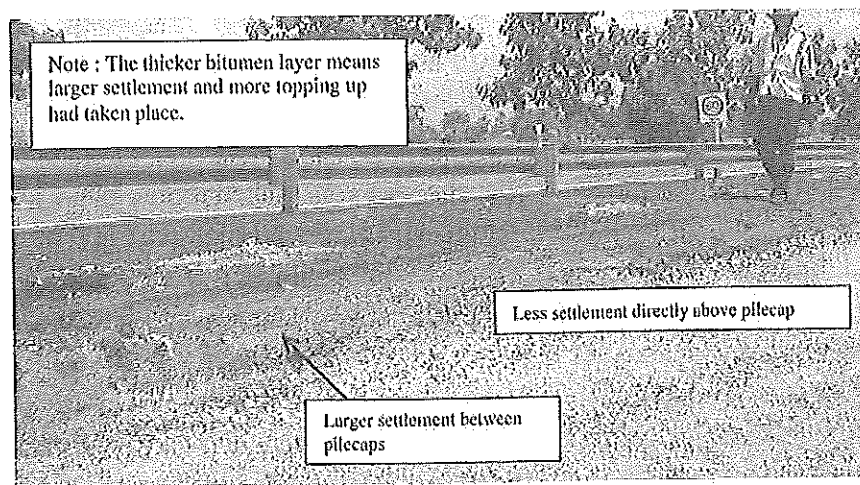


Figure 8. Differential settlement observed after excavation to a depth of about 300 mm for construction of remedial works.

Undisturbed block sampling was also carried out at the embankment fill areas to determine the properties of the fill materials. Sieve analyses carried out have indicated that the fill materials consist of sandy or gravelly / with relatively high percentage of fines (up to a maximum of 99%). Shearbox tests were also carried out and the results indicated effective stress parameters for the embankment fill materials of $c' = 2 \text{ kPa}$ and $\phi' = 25^\circ$.

4.1 Modelling of "Mushroom" Problems

The modelling of 'mushroom' problems was carried out using two-dimensional (2D) and three-dimensional (3D) PLAXIS software programmes (Brinkgreve, 2001, 2002), which are general finite element method (FEM) programs for geotechnical analysis to determine the possible causes of the problems.

A typical FEM model adopted for the investigation is shown in Figure 9. In the model, the circular spun piles beneath the embankment are modelled as linear elastic/drained

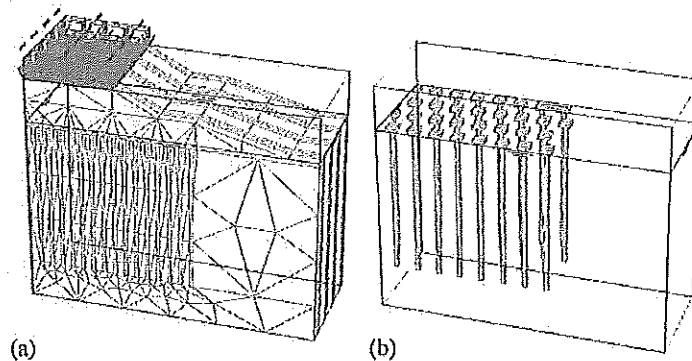
Table 2. Shear strength parameters of marine deposits

Depth from OGL	γ_{bulk} (kN/m ³)	γ_{dry} (kN/m ³)	ϕ' (deg)	c' (kPa)	S_u' (kPa)
0–7.5	15.8	12.0	22	3	16.0
7.5–12.5					22.5
12.5–17.5					35.0
17.5–22.5					47.0

Note: The undrained shear strength obtained from vane shear tests are corrected in accordance with recommendations by Bjerrum (1973) for stability analysis of embankments to cater for differences in rate of shearing of the subsoil in vane shear tests and in the field and strength anisotropy.

Table 3. Consolidation parameters of marine deposits and modified critical state parameters of marine deposits

Depth from OGL	OCR	λ^*	κ^*	μ^*	c_v (m ² /yr)	c_h (m ² /yr)
0–2.5	7	0.18	0.05	0.007		
2.5–7.5	1.6	0.18	0.065	0.007	3.0	12.0
7.5–22.5	1.2	0.22	0.075	0.0085		

**Figure 9.** (a) Typical 3D FEM Model of Embankment (b) Piles and Pilecaps Modelled.

material with properties defined by Young's Modulus and Poisson's ratio. The piles are assumed to penetrate into the stiff stratum. In both the 2D and 3D analyses, the soft clay subsoils are modelled using Soft Soil Creep model which resembles the Modified Cam–Clay model with isotropic hardening. The fill material is modelled using the Hardening Soil model (Schanz et al., 2000).

Results from the FEM analyses have shown that the differential settlement of the embankment ranges from 64 mm to 156 mm with angular distortion as high as 4% (1/25). This is in excess of the recommended values of 1% (1/100) by BS8006 (1995). Typical results from the FEM analyses showing the 'mushroom' problem are shown in Figure 10.

The settlement profile along a typical section of the embankment showing the computed differential settlement and angular distortion is plotted in Figure 11. In summary,

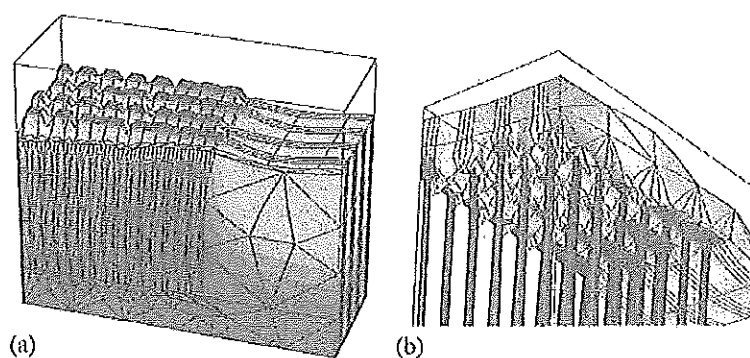


Figure 10. Results of 3D FEM analyses showing 'mushroom' problems: (a) top view (b) bottom view.

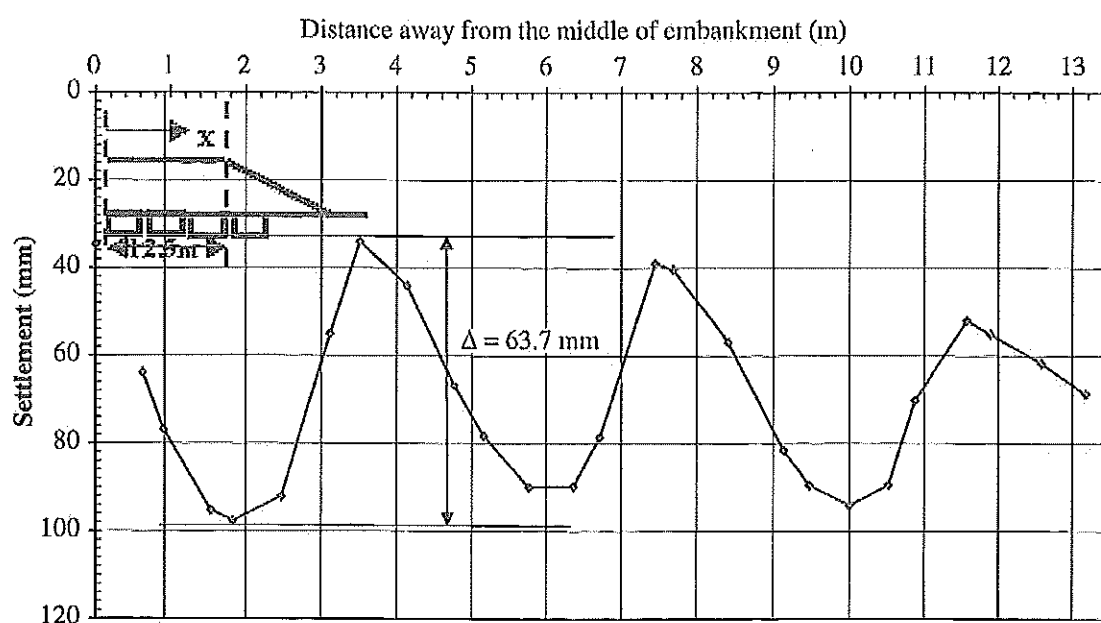


Figure 11. Settlement profile before remedial treatment across the embankment.

results of FEM analyses have shown that the 'mushroom' problems arise due to the ineffective arching mechanism influenced by the following factors:

- (a) unsuitable fill materials,
- (b) large pile spacing to the height of fill.

The FEM results are also consistent with findings from Hewlett and Randolph (1988) and Koutsabeloulis and Griffiths (1989) who showed that pile spacing (or size of voids) and properties of fill materials are important to ensure an effective arching mechanism.

4.2. Remedial design for "mushroom" problems

The occurrence of the 'mushroom' problems has necessitated regular repaving works to ensure the riding comfort and safety of the expressway. However, repaving works are only a short-term solution as the embankment will continue to settle due to additional

loads from the pavement. Therefore, an effective remedial design for the 'mushroom' problems shall satisfy the following criteria:

- (a) Minimum disturbance to operation of the expressway
- (b) Simple and fast to construct
- (c) Cost effective
- (d) Minimum long-term maintenance.

After reviewing all the feasible options such as a high-strength geogrid with granular infill (on top of pilecaps or at shallow depths) and a reinforced concrete raft (on top of pilecaps or at shallow depths), it was found that a reinforced concrete (RC) raft at shallow depth offers the best solution to the 'mushroom' problems satisfying the above criteria.

The design was checked using FEM analyses to ensure the required long-term angular distortions of 1% (1/100) as recommended by BS8006 (1995). Results of FEM analyses have indicated that the angular distortion of the embankment is below 1% upon construction of the RC raft. Figure 12 shows typical results from the FEM analyses with the computed angular distortions and maximum differential settlement. In summary, the RC raft at shallow depth solution adopted consists of

- (a) Raft thickness = 250–300 mm depending on embankment characteristics (fill height, pile spacing, etc.).
- (b) Reinforcement required = T16–150 mm c/c (Top and Bottom).
- (c) Characteristic concrete strength = 35 N/mm².

It is to be noted that the reinforcements are deliberately arranged uniformly throughout the slab due to difficulties in accurately determining the position of as-built pile position. The proposed solution is subjected to a comprehensive programme of monitoring to validate its effectiveness and possible optimization of the design for future implementation. The monitoring programme consists of taking readings from the following instruments:

- (a) Sister bar embedment strain gauges installed in the steel reinforcement to measure stresses in the reinforcement.
- (b) Ground and raft settlement markers to monitor the settlement behaviour of the raft and ground.
- (c) Inclinometers to monitor the overall performance of the embankment.

4.3. Construction of remedial works

The RC raft solution at shallow depth essentially involves the following simple construction sequence:

- (a) Excavating of minimal depth for the concrete raft and wearing course. This is typically <500 mm as the thickness of the concrete raft is approximately 300 mm and

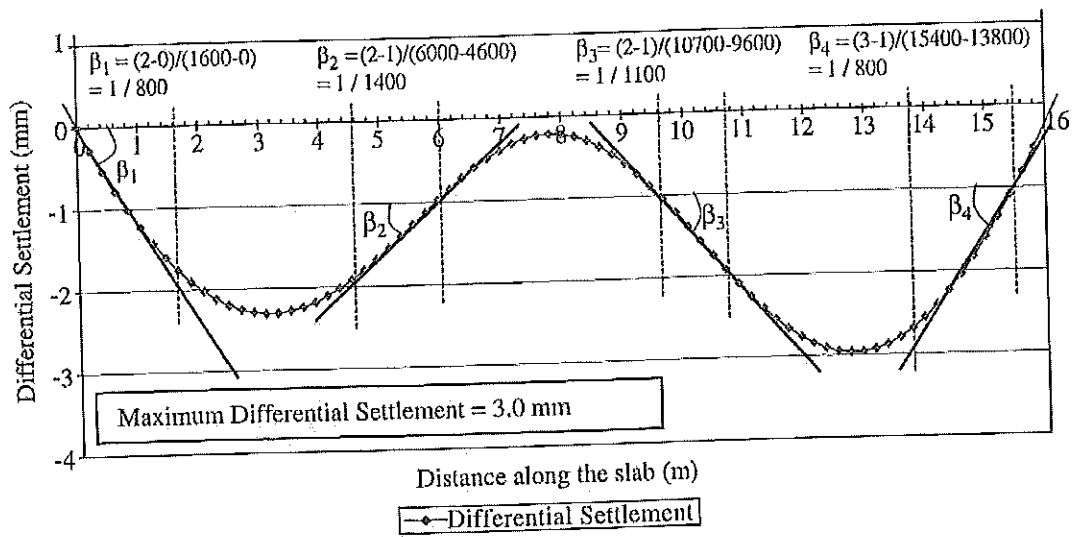


Figure 12. Settlement profile after remedial treatment across embankment using RC raft at shallow depth.

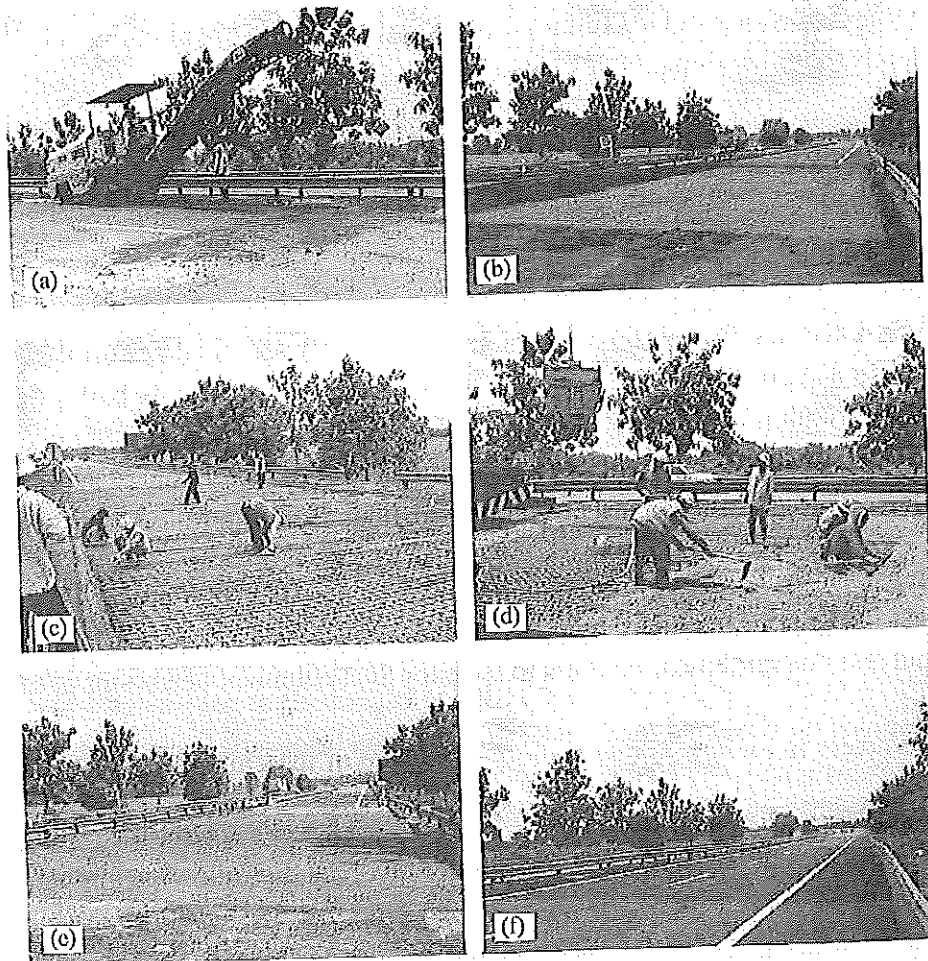


Figure 13. (a) Milling works up to 300 mm deep in progress (b) after milling (c) laying of steel reinforcement (d) concreting (e) completed RC Raft (f) pavement completed, traffic re-opened.

thickness of the wearing course is 50 mm (total ~3550 mm) and can be easily and speedily carried out using a milling machine.

- (b) Laying of steel reinforcement and casting of concrete.
- (c) Laying of the wearing course.

The simple construction sequence is very important for this site due to its location within a busy expressway. Therefore, the simple construction sequence will minimize lane closure for the construction works. In addition, the remedial solution is easy to construct and does not require a specialist contractor. The typical construction sequence of the works for a recently completed pilot stretch is shown in Figure 13.

5. CONCLUSIONS

From the case histories presented, it is important to be aware that the success of the ground improvement works depends on many factors from planning, investigation, analysis, design, specification of works, construction and closed supervision by design consultants. Flaws in any of the above stages will compromise the effectiveness of a ground improvement causing failures of either short-term ultimate limit state failure (e.g. slip failure and tension cracks) or long-term serviceability limit state problems (e.g. excessive differential settlement).

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