Role of Extendible Basal Reinforcement for Embankment Construction over Soft Soils

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ABSTRACT: 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 in the design can be over-estimated than what has been actually mobilised at various stages, thus resulting in non-representative safety margin. 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 paper will present a case study of an instrumented embankment construction with extendible basal reinforcement to illustrate the strain compatibility issue in relation to the distress occurrence.

KEYWORDS: Extendible basal reinforcement, Embankment, Soft ground, Strain compatibility

1. INTRODUCTION

Embankment is a raising up fill platform with side slopes to support structure and infrastructure developments. Often, at the areas where strength of the existing subsoils is not adequate to handle the weight of embankment, ground improvement techniques shall be considered. The design consideration of the ground improvement works shall ensure adequate factor of safety (FOS) to avoid failure during construction and to meet the necessary settlement performance at service condition with scheduled maintenance requirements.

In some cases, embankment constructed over soft ground requires additional reinforcements to achieve staged design levels and to ensure side slope stability during the temporary construction stages. Basal reinforcement can be used to minimise the spreading failure of the compacted embankment fill over weak supporting subsoils. This also includes the edge stability of the embankment side slopes where the unbalanced load imposition of embankment fill provoking the local instability. Such temporary stability condition is needed for the overall stability of the embankment construction with staged construction until the gain in undrained strength at every stage filling in the ground treatment design reaches a sufficient strength level to sustain the corresponding next staged filling and the final permanent embankment fill plus the preloading surcharge fill.

2. BASAL REINFORCEMENT

2.1 General

Basal reinforcement is defined as a created in-situ composite reinforced soil system by inserting inclusions in predetermined directions at base of embankment to enhance the stability of temporary platform. The imposition of gravitational weight of the fill embankment on the supporting soft soil will induce lateral plastic squeezing movement to both side slopes of the embankment fill when the disturbing stress reaches the ultimate soil strength. Sometimes, when the depth of the soft soils are sufficiently thick, the failure mode of the embankment distress can be in the form of sliding between two intact soil mass with distinct rupture surface or a rotational circular failure.

2.2 Design of Basal Reinforcement

Basal reinforcement, when required, can be designed in accordance with BS 8006. The partial factors adopted shall reflect the design requirements of the basal reinforcement over the service period.

The design of basal reinforcement shall take into consideration of the strain compatibility between the embankment fill and basal reinforcement system. This is to ensure that the desired force in the basal reinforcement is mobilised to keep the embankment fill intact during the construction period of the embankment. Despite of the possibility of potential lateral plastic squeezing of the thin underlying soft clay, in which the gross squeezing plastic flow movement can be more than the permissible tensile straining of the basal reinforcement, the design of the basal reinforcement shall not be strained to a level that can cause cracking of the brittle compacted fill above the basal reinforcement.

However, the adopted design tensile strength to cater for the maximum straining thorough out the embankment construction can be overestimated than what has been actually mobilised. Thus, resulting in non-representative safety margin. To simultaneously mobilise the shear strengths of the embankment fill and also the underlying supporting soft subsoil, 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 as mentioned earlier can develop if ignoring such strain compatibility or slippage at the interfaces of the basal reinforcement is allowed. If the embankment is strained to a level with excessive tensile cracking across the embankment body, one shall ask if the compacted fill with tensile cracking still has sufficient strength to contribute in overall embankment stability or not.

In view of this, a case study of an instrumented embankment construction with extendible basal reinforcement have been carried out to illustrate the strain compatibility issue in relation to the distress occurrence. This may call for a review of the permissible strain of extendible basal reinforcement under the working condition for supporting a brittle compacted fill over soft compressible deposits.

3. CASE STUDY OF EMBANKMENT DISTRESS

3.1 Background

Case study discussed in this paper involves of a newly proposed embankment with extendible basal reinforcement of characteristic strength 600kN/m and prefabricated vertical drains as ground treatment option. An embankment distresses consisting primarily of shallow longitudinal crack lines on the embankment formation along the main alignment direction was found during construction stage of the embankment. Layout plan of the embankment distress is as shown in Figure 1.

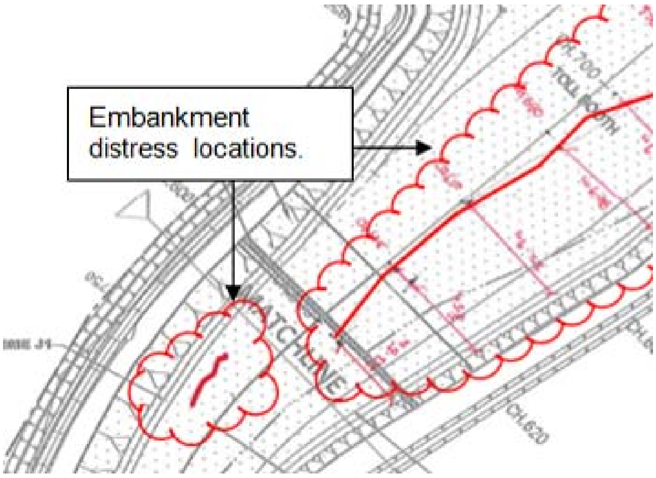

Figure 1 Layout Plan of embankment distress

Close up view of longitudinal cracks found from the embankment was as shown in Figure 2. The crack pattern was not of random nature, but rather a near straight line and running along the longitudinal direction of the embankment alignment. From the trenching across the crack line, the depth of the cracks was found to be about 300mm to 500mm from the formation level after 1m partial fill removal 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 and seeping out was observed at the bottom of the cracks to confirm the depth of the cracking.

Figure 2 Longitudinal Crack Found at the Embankment

According to 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.

3.2 Subsoil Conditions

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 for the embankment construction works which were 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 3 shows the profiles of vane shear test results before embankment construction in 2012 and after embankment construction in 2013 and 2014.

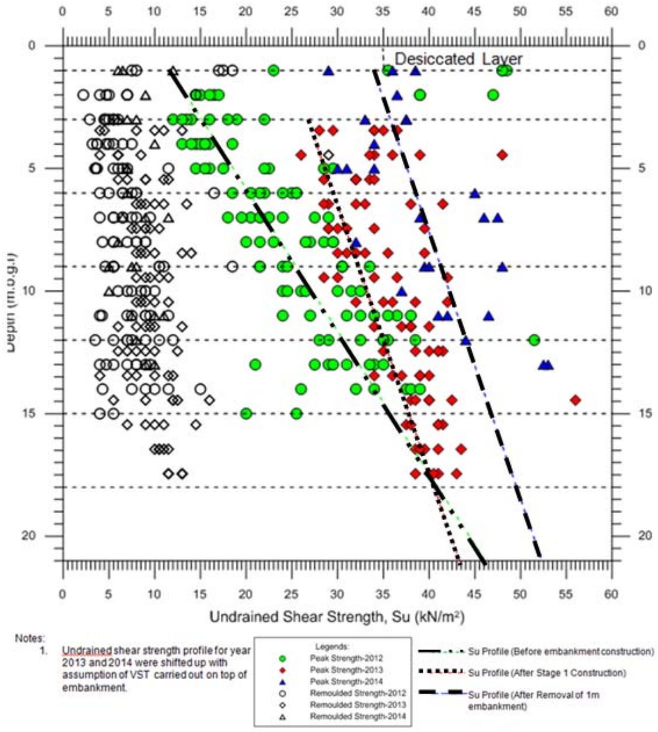

Figure 3 Interpreted Undrained Shear Strength at Different Stages of Embankment Construction

Bulk density of the proposed site prior to embankment construction works were summarised in Figure 4. The average bulk density of the alluvial subsoil samples obtained from the SI is 14kN/m^3 for top 9m soil and gradually increases to 16.5kN/m^3 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/m^3 as top soil layer has subjected to compaction process caused during the construction of the drainage blanket and subsequently improve its density.

Meanwhile, from the construction records of fill compaction, bulk density of the compacted embankment fill was found to be ranging from some 19kN/m^3 to mostly 20kN/m^3.

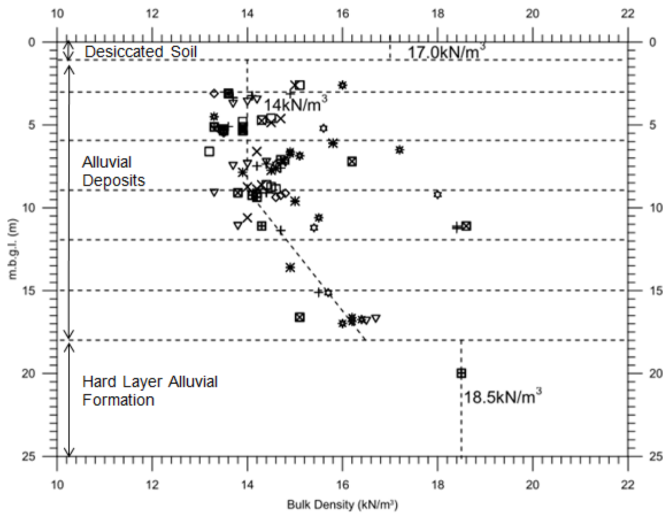

Figure 4 Interpreted Subsoil Bulk Density

Based on the interpreted subsoil parameters, the proposed embankment and subsoil profile is summarised in Figure 5. 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 rest period are more prominent at the upper soil and diminishing as going done to a depth of about 18m.

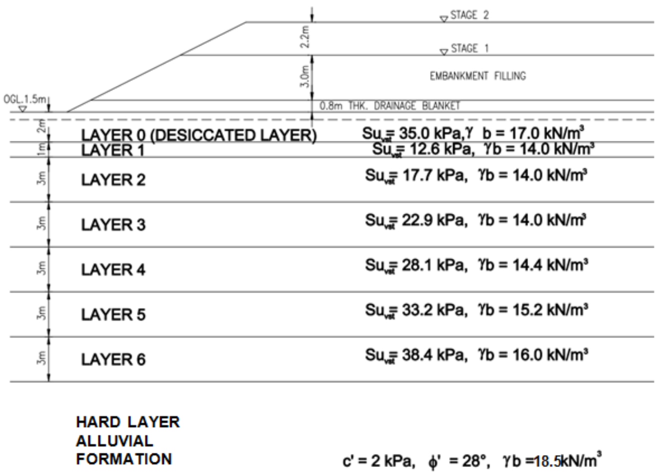

Figure 5 Typical Cross Section of Embankment For Initial Stage

3.3 Construction Monitoring

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 6.

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.

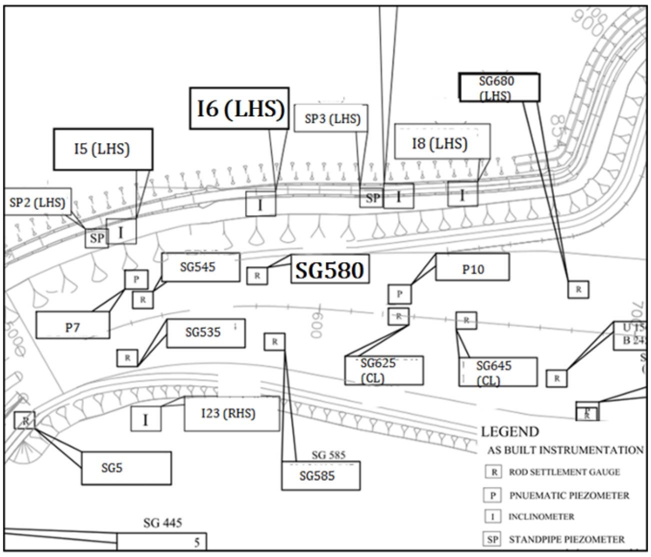

Figure 6 Installed Instrumentations on Embankment

3.3.1 Settlement 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 7. 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).

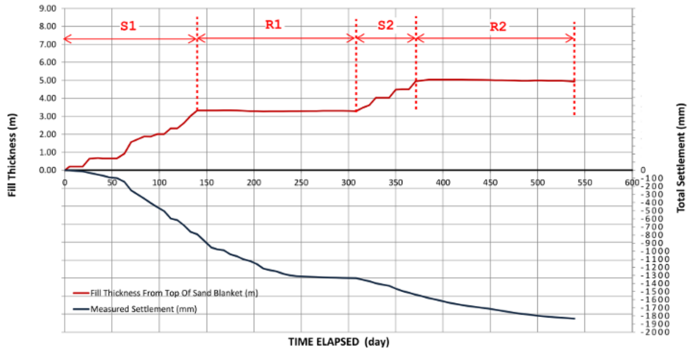

Figure 7 Filling Sequence and Settlement of Embankment Monitored by SG580

3.3.2 Lateral Displacement Monitoring

Figure 8 presents the inclinometer monitoring results from inclinometer I6, which was installed at a location of about 4m beyond the embankment toe. It shall take note that the inclinometer monitoring was only started 2 month after Stage S1. As such, it was expected that portion of lateral displacement is not recorded in the inclinometer monitoring. 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.

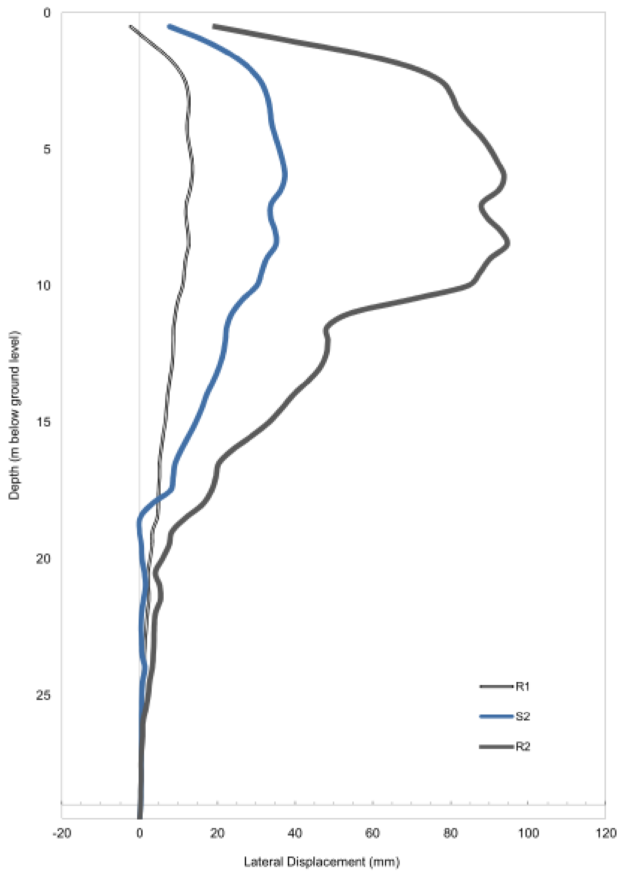

Figure 8 Inclinometer I6 Monitoring Results

4. BACK ANALYSIS

4.1 General

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 9.

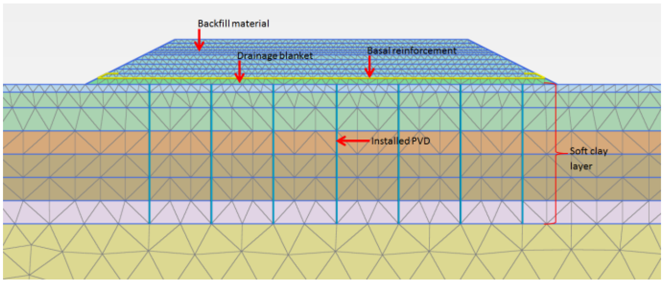

Figure 9 FEA Modelling For Back Analysis

5. RESULTS AND DISCUSSIONS

5.1 Back Analysis Results

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.

5.1.1 Comparison of Embankment Settlement Trend

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 10. The back analysed settlement trend with time is compared with measured settlement profile of settlement gauge (SG580).

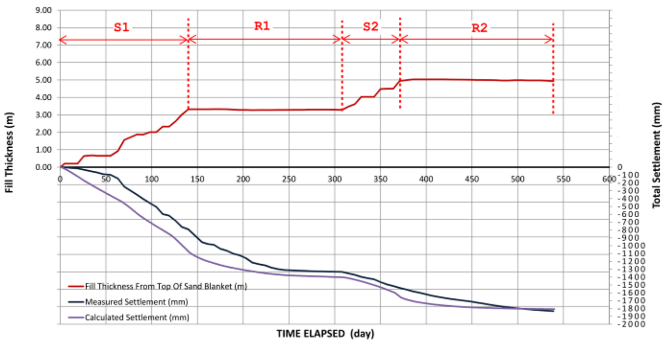

Figure 10 Comparison of Back Analysed Settlement Trend with Actual Measurement

5.1.2 Lateral Deformations Comparison

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 11.

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 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 ultimate design strength of the basal reinforcement.

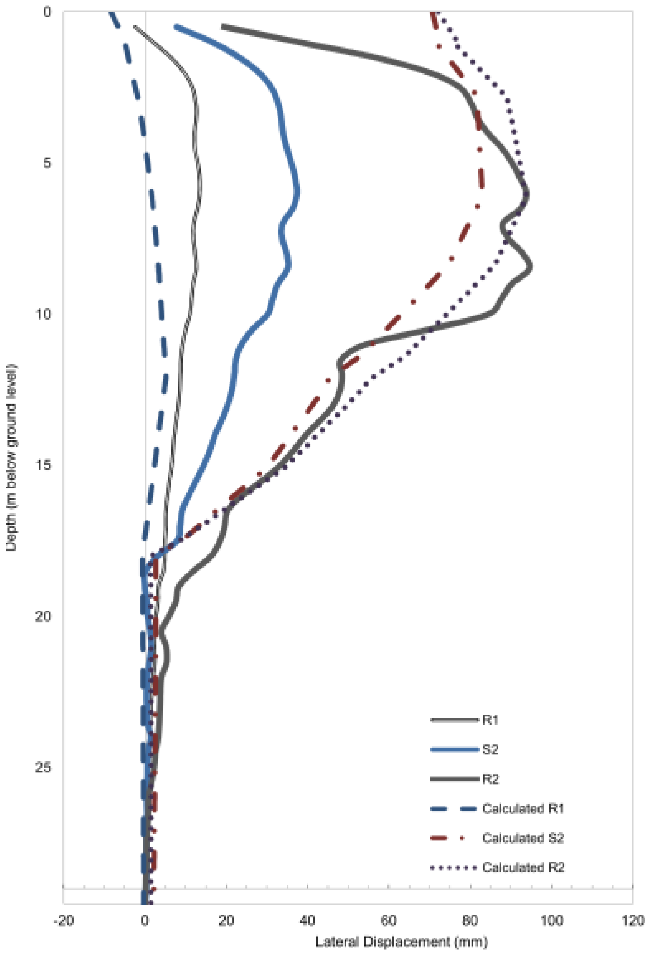

Figure 11 Comparison of Lateral Displacement Profile

5.2 Performance of Basal Reinforcement

As both of 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 for 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 ultimate reinforcement strength of 600kN/m.

From Table 1, 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).

Table 1 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

6. DISCUSSION

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.

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 assumed 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 mechanical behaviour prompted to cracking, when subjected to differential straining or localised straining near to the embankment slopes can be expected in a well compacted embankment fill.

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.

7. CONCLUSIONS

From interpreting and reviewing the monitoring data and basic analysis of the distressed embankment, the followings summary of the findings and recommendations can be deduced from this forensic investigation:

The longitudinal cracks observed on the fill embankment are probably the outcome of the plastic straining of the weak underlying treated supporting subsoil under the embankment loading. This is confirmed by the plastic straining as evidenced in the inclinometer results where large deformation observed from the monitoring results. The tension cracking with the V-shape crack opening suggest a flexural embankment deformation supporting the localised plastic straining with localised edge slope instability.

Tensile strain in the extendible basal reinforcement shall be compatible with the strains in the embankment fill and the subsoil. Otherwise, distresses like embankment cracking and even instability can develop if ignoring such strain compatibility. The wishful high tensile strain assumed in the basal reinforcement resulting high reinforcing strength to improve overall embankment stability can lead to misrepresentation of the safety margin. Thus there will be a need to review the current design practices adopting unrealistically high mobilisation of extendible basal reinforcement with high tensile straining, which possibly lead to cracking of the brittle compacted embankment. It would be useful to instrument the basal reinforcement to reveal the distribution profile of the mobilised tensile strain and compare with the distribution of mobilised tensile strain within compacted embankment fill and the underlying supporting subsoils for future embankment projects at soft ground. This will also reveal the potential interface slippage between the extendible basal reinforcement and compacted embankment and also the underlying supporting subsoils.

In view of strain incompatibility between basal reinforcement with the subsoil might cause serviceability issue for the constructed embankment, counterweight berm on sides of the embankment was proposed to minimise the deviatoric shearing stress at the subsoil beneath the embankment. The counterweight berm will also reduce the reliance of the basal reinforcement performance for assurance of embankment stability, in which the strength mobilisation in the basal reinforcement can be very sensitive in the embankment design. The configuration of the counter-weight berm was assessed based on the improvement of the factor of safety with the counterweight berms in place. The design of the counterweight berm also considers the possible post construction changes on the counterweight berm to continue serving its intended function until a stage when its presence is of no more relevance to the stability of the main embankment.

8. REFERENCES

British Standard Institute (BSI). (1995) BS8006: Code of practice for Strengthened / reinforced soils and other fills.