# GEOTECHNICAL ENGINEERING CHALLENGES FOR HIGHWAY DESIGN AND CONSTRUCTION ON SOFT GROUND

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**ABSTRACT:** The success of geotechnical works on soft ground relies on important factors such as proper planning, analysis, design, construction control and supervision. However, this is usually easier said than done, and therefore there are still repeated failures of geotechnical works such as embankments, approaches to bridges and culverts. Most of the failures are quite similar in nature in that they are caused by failing to comply with one or a combination of the above factors. This paper presents case histories of geotechnical failures investigated by the Authors. The causes of failures, remedial works proposed and lessons learned are discussed. Finally, some simple guidelines to prevent failures related to geotechnical works on soft ground are provided.

*Keywords*: Soft Ground; Failure; Embankment; Culvert; Bridge; Abutment

## 1. INTRODUCTION

The success of geotechnical works on soft ground relies on important factors including proper planning, analysis, design, construction control and supervision. However, most of the geotechnical failures investigated by the Authors are usually quite similar in nature and they are caused by failing to comply with one or a combination of the factors stated above. Case histories of geotechnical failures of embankments, foundations and excavations are presented together with the causes of failures, remedial works proposed and lessons learned. Finally, some simple guidelines to prevent failures are also discussed.

## 2. TYPES OF FAILURE ON SOFT GROUND

Failures of projects on soft ground in this paper can be broadly classified into two broad categories. The first category includes those of total or partial collapse of embankments, excavations, foundations, etc. This category often needs reconstruction and/or strengthening measures. The second category of failures is those due to lateral and vertical movements resulting in severe distortion to complete or adjacent structures causing loss of serviceability. The affected structures usually need expensive repairs or strengthening works.

The following are the case histories discussed in this paper:

- **(i)** Failure of an Embankment
- **(ii)** Failure of Bridge Foundations and Approach Embankments
- **(iii)** Failure of Approaches to Bridges and Culverts

## 3. FAILURE OF EMBANKMENTS

Failures of embankments often occur during construction. A case history of an embankment initially constructed using vacuum preloading method with prefabricated vertical drains was investigated by the Authors. Figure 1 shows the cross-section of the proposed embankment. After the 1$^{st}$ failure, the remedial works involving stone columns were proposed and constructed. The embankment, with stone columns, failed when the embankment reached 3.2m of the planned 5.5m fill height. Figure 2 shows the embankment after the 2$^{nd}$ failure.

As the effectiveness of the vacuum preloading method is dependent on many factors, close monitoring of the pore water pressures in the subsoil during filling is vital to prevent failure. In view of this, instruments such as piezometers, settlement gauges and vacuum meters have been installed at site. Prior to embankment failure, measurements from piezometers indicated that the vacuum suctions were not functioning properly after finish fill

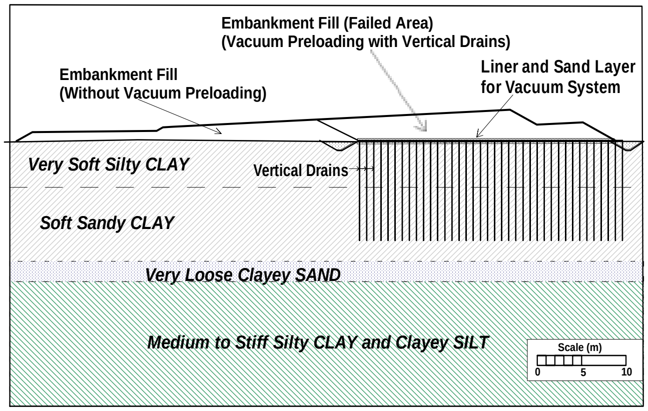


**Fig. 1. Cross-section of Embankment A (After Gue et. al., 2001)**

height of 5.5m. This in turn has caused the increase of pore water pressures in the cohesive subsoil.

The trend of increase in pore water pressures has been observed for more than one month but no action was taken to review the monitoring results and prompt for preventive action. In fact, it is clear that the 1st failure of Embankment could have been avoided if the observational method (Peck, 1969) was employed properly. Details of back-analyses and methodology of monitoring using the observational method are presented by Gue et. al. (2001) and Tan & Liew (2000) respectively.

### 3.1. Failure Due to General Shear

Our review indicates that 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. Based on the investigation, 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 should be given to probable failure due to general shear instead of over relying on a single method. For remedial measures, it is also important to determine the representative "disturbed" strength (remoulded and regaining of strength through thixotropy effects) of the subsoil to be used in the analyses. In addition, load tests should be carried out on stone columns to verify the design assumptions as there are large differences among methods of analysis. However, many embankments on very soft ground treated with stone columns have been successfully constructed with the help of the observational method.

Failures of embankments due to design are commonly caused by the following inadequacies :-

**(i)** Settlement Analysis
**(ii)** Stability of Embankment – especially general shear

## 4. FAILURE OF BRIDGE FOUNDATIONS AND APPROACH EMBANKMENT

Among the many case histories of bridge failures investigated by the Authors, failure induced by bearing capacity and stability of the embankment appear to be the major contributing factors (Gue & Tan, 2003). The layout of the proposed bridge is shown in Figure 3. Figure 4 shows an overview of the failed embankment.

The approach embankments were constructed over 25m thick of soft coastal and riverine alluvium clay underlain by dense silty Sand and very stiff silty Clay. The approach embankments were supported by reinforced concrete (RC) piles and cast with individual pilecaps. The abutments and piers were supported by spun piles driven to set in the hard layer at more than 30m depth.

A deep seated slip failure occurred at the approach embankment with a sheer drop at about 25m behind Abutment II. Figure 5 shows the sheer drop after removal of some of the fill behind the abutment. Abutment II tilted

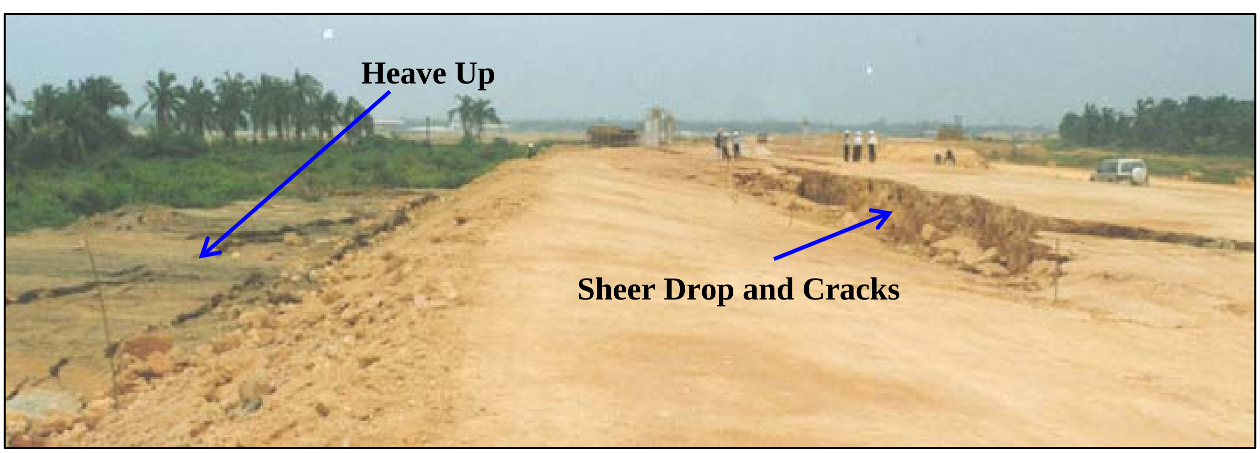


**Fig. 2. Failure of Embankment A treated with Stone Columns (After Gue & Tan, 2004)**

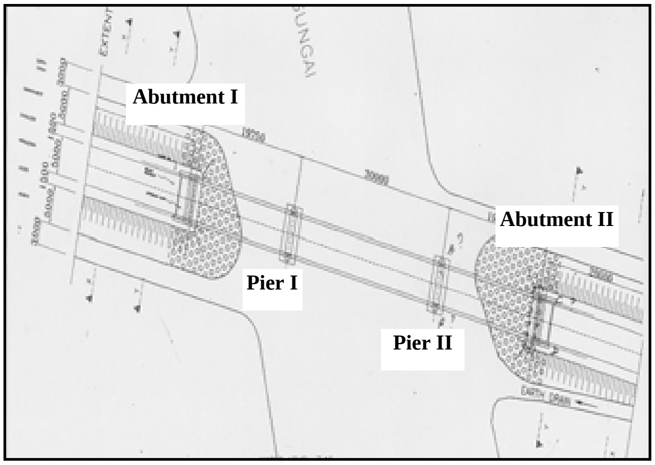


**Fig. 3. Layout of Piers and Abutments (After Gue & Tan, 2003)**

away from the river, with a magnitude of about 550mm at the top of the abutment at the time of the site inspection by the Authors, who were carrying out the geotechnical investigation of the failure. The tilt translated into excessive angular distortion affecting the integrity of the spun piles. Due to the tilt of the Abutment II away from Pier II, a gap about 300mm wide was observed between the two bridge decks at the pier. Figure 6 shows the photograph of the tilt at the Abutment II and the gap between two bridge decks at Pier II. The failure also caused Pier II to tilt slightly. Figure 7 shows the schematic diagram of the possible slip plane relative to the deformed structures.

The above failures were caused by the following factors :-

- **(i)** Inadequacy of geotechnical design for the approach embankments and abutments.
- **(ii)** Lack of understanding of the subsoil conditions and awareness of the possible problems/failures that could happen during construction.
- **(iii)** Lack of construction control and site supervision by the Consultant.

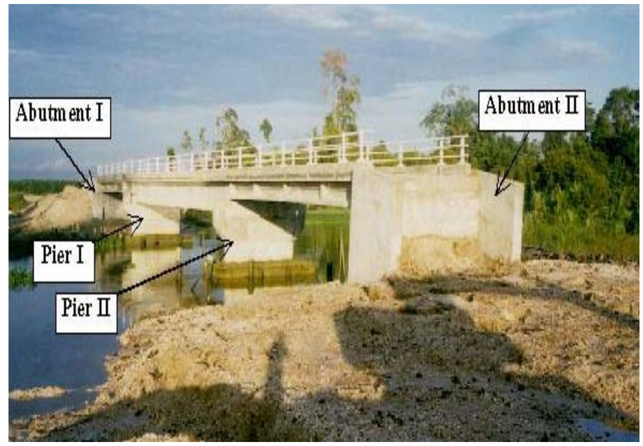


**Fig. 4. Overview failed embankment (After Gue & Tan, 2003)**

The following lessons learned are:

- **(i)** Check for lateral soil pressure imposed on piles by the embankment fill behind an abutment to prevent failure of the pile group supporting the abutment.
- **(ii)** The design consultant should review the submitted method statement to avoid slip failure due to instability of additional loads imposed by temporary fill. Full-time site supervision by site engineers having geotechnical experience and an understanding of design concepts should also be maintained to ensure compliance of both temporary and permanent works.

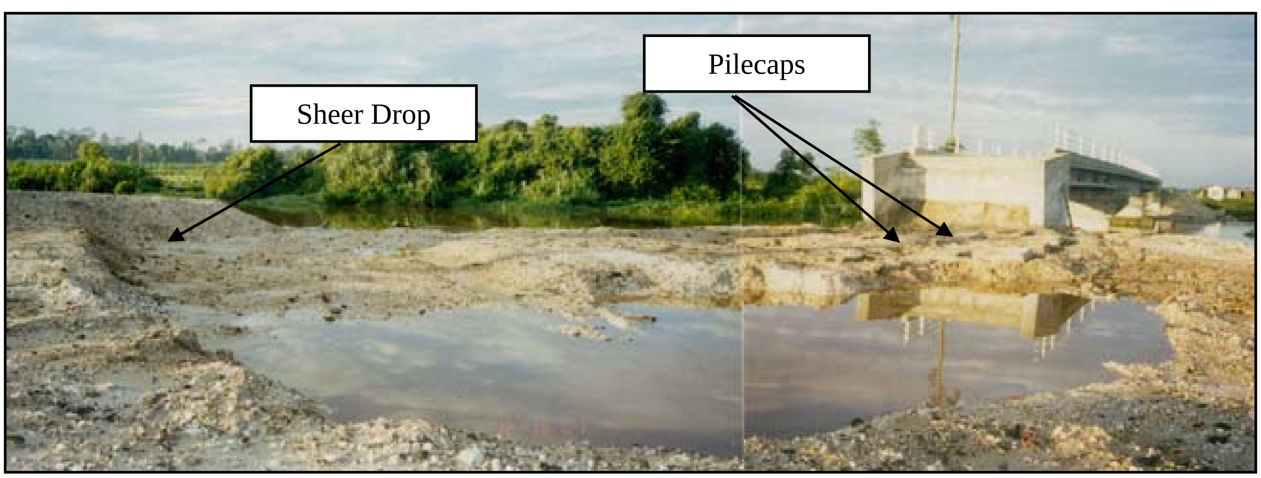


**Fig. 5. Sheer Drop at about 25m behind the Tilted Abutment (After Gue & Tan, 2003)**


**Fig. 6. Tilted Abutment and Observed Gap between Bridge Decks (After Gue & Tan, 2003)**

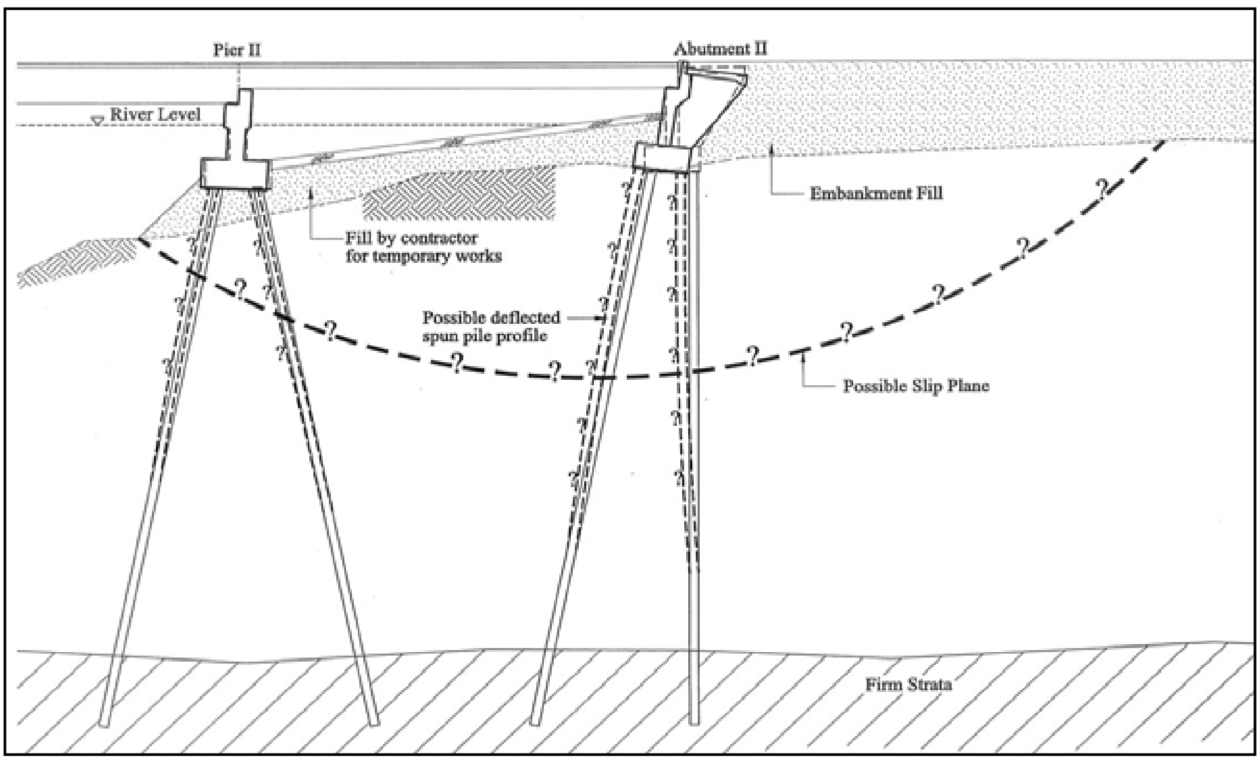


**Fig. 7. Schematic of Slip Failure (After Gue & Tan, 2003)**

**(iii)** The design consultant should ensure the removal of temporary fill after construction or to design the piles to accommodate negative skin friction, as temporary fill would cause the compressible subsoil to settle with time.

Detailed information on the above case history is documented in Gue & Tan (2003) and Gue & Tan (2004). Similar mode of failure was encountered in the case histories of bridge failures investigated by the Authors, as documented in Gue (1988).

## 5. FAILURE OF APPROACHES TO BRIDGES

The most common old practice of bridge approach design is shown in Figure 8. Bridge abutments over soft deposits are normally supported by piles. The piles for the abutments are usually driven to set at a firmer layer below. The long term settlement of the abutments is hence negligible. The embankment adjacent to the abutments would settle due to the consolidation settlement of the subsoil under the embankment load. The time and magnitude of the consolidation settlement depend on the thickness and the consolidation properties of the compressible deposits and the height of embankment. Figure 9 shows the common problem i.e. settlement of bridge approaches that need regular resurfacing or topping-up to ensure smooth riding surface.

Figures 10 and 11 show some of the innovative solutions to the problems. Figure 10 shows the use of transition piles to provide a smooth transition to a bridge abutment. The transition piles are designed to settle, and the piles close to the unpiled section would have a smaller differential settlement. This area can further be refined with an approach slab as indicated. High quality of field tests, sampling and laboratory tests are needed to obtain reliable soil parameters for analysis and prediction of the settlement (Gue, 2000).

Figure 11 shows the use of Expanded Polystyrene (EPS) to reduce the weight of the embankment particularly near the bridge abutment. However, this technique is very sensitive to the high water table and the design is usually controlled by floatation (NRRL, 1992).

The other methods are surcharging with or without vertical drains and stone columns to smoothen the transition. As the transition is a short section of the alignment, the first two methods are generally more economical, particularly the use of transitional embankment piles, as this technique does not need another set of plant and equipment, thereby saving on the extra mobilization cost.

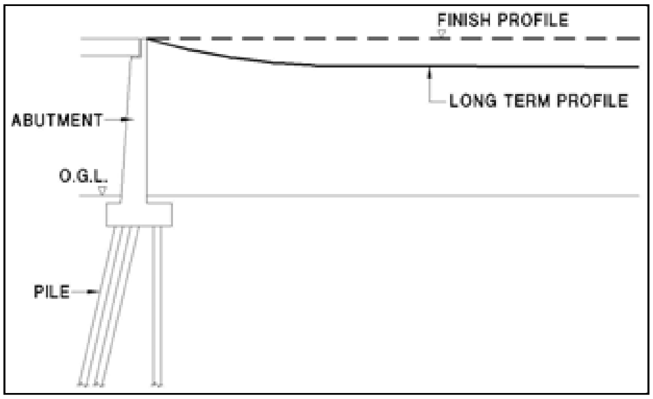


**Fig. 8. Settlement of a Bridge Approach (After Gue, 2000)**

**Fig. 9. Hump at Bridge Approach (After Gue, 2000)**

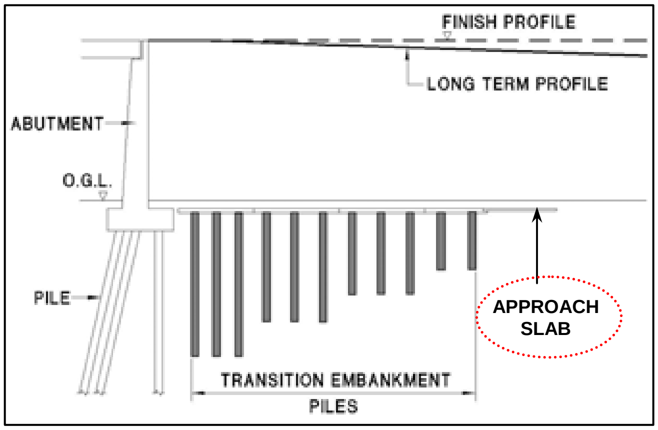


**Fig. 10. Transition Embankment Piles (After Gue, 2000)**

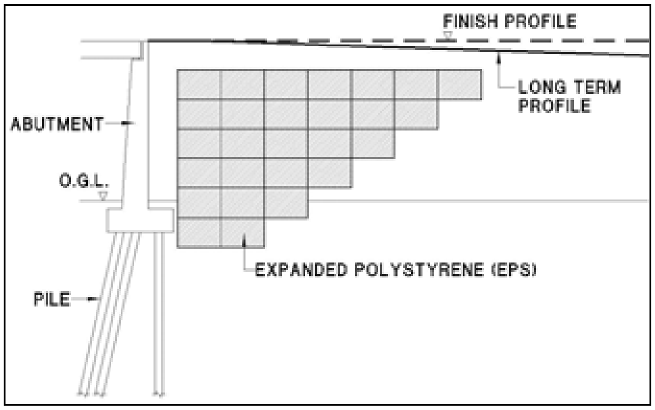


**Fig. 11. Expanded Polystyrene (EPS) (After Gue, 2000)**

## 6. FAILURE OF APPROACHES TO CULVERTS

Very often, culverts are designed and constructed as shown in Figure 12 to ensure that the area of flow of the drain through the embankment remain unchanged with time.

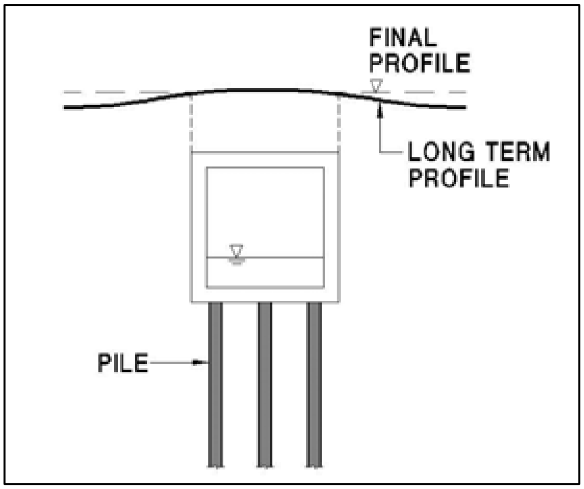


**Fig. 12. Piled Culvert (After Gue, 2000)**

This is achieved by using piles to provide a rigid platform. The consequence of having rigid platform as shown induces differential settlement between the rigid piled culvert and the unpiled embankment. The unpiled embankment over compressible soil will settle in time, as shown in Figure 13.

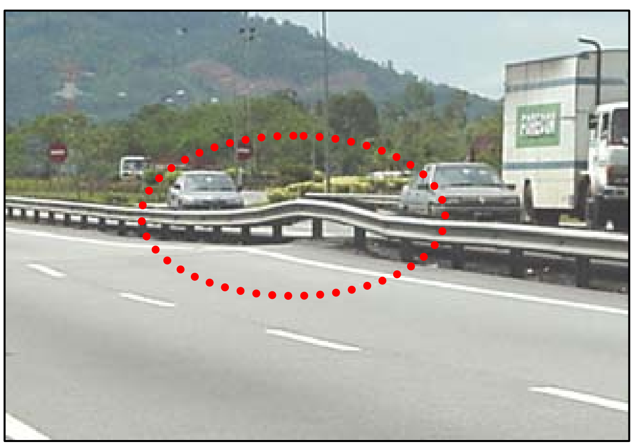

**Fig. 13. Hump over Piled Culvert (After Gue, 2000)**

The possible solutions to eliminate the differential settlement are:-

- **(i)** Provide a larger culvert to allow for long term settlement.
- **(ii)** Provide a transition piles to the approaches to a culvert.

The first option is shown in Figure 14, which allows the culvert to settle together with the embankment. Hence, the size available for flow will reduce with time as the culvert settles and a section of the culvert will be silted up as shown. The net flow area after taking into consideration of settlement and siltation should have a size not smaller than that required for the volume of flow designed just like the piled culvert as shown in Figure 12. The second option as shown in Figure 15 is similar to the transition piles described earlier (in section 5). This option is generally more costly.

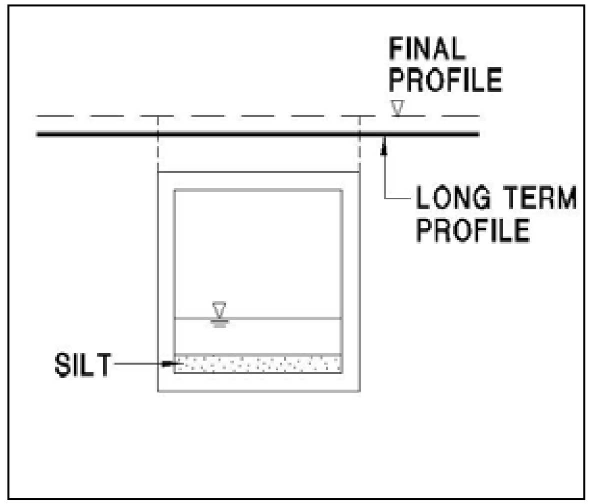


**Fig. 14. Oversized Culvert (After Gue, 2000)**

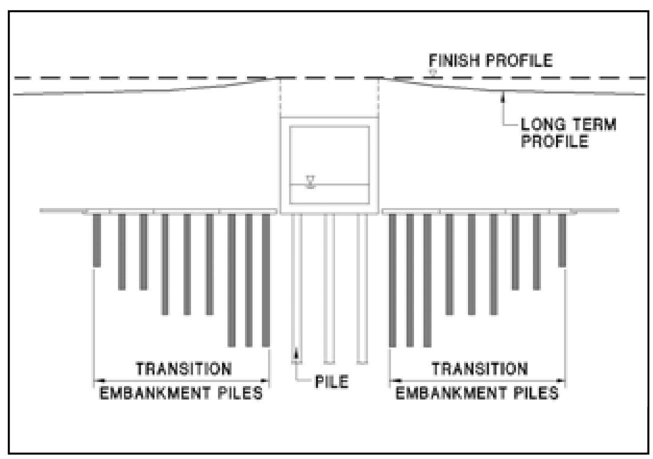


**Fig. 15. Transition Embankment Piles for Piled Culvert**

## 7. GUIDANCE NOTES ON SUBSOIL INVESTIGATION, ANALYSIS, DESIGN & CONSTRUCTION FOR HIGHWAY CONSTRUCTION

As demonstrated in the above sections, the success of highway construction on soft ground relies on proper planning, design, construction control and site supervision as summarized below:

**(i)** Awareness of the project requirements in terms of serviceability criteria (deformation tolerances, bearing capacity, etc.), site constraints, time (construction time and service period) and costs (construction and maintenance costs).

**(ii)** Knowledge of the site and subsoil conditions through proper desk study, gathering of geological information and well planned and supervised subsurface investigation and laboratory tests to acquire the necessary reliable parameters for geotechnical designs.

**(iii)** Proper geotechnical design to address both stability of the embankment and deformation, particularly differential settlements.

**(iv)** Full time proper site supervision for the construction works by qualified personnel/ engineers.

**(v)** Careful and proper monitoring on the performance of the embankment during and after construction through instrumentation scheme.

### 7.1. Guidance Notes on Subsoil Investigation

Based on numerous investigations carried out by the Authors, it has been proven that direct determination of undrained shear strength for soft marine clayey deposits is very beneficial in subsequent design and construction control. This is in addition to collecting undisturbed samples from conventional boreholes for unconfined compression and consolidation tests. The use of piezocone is useful to detect presence of sand lenses and indirect determination of undrained shear strength. The detection of sand lenses is particularly important for the assessment of ground treatment. The selection of surcharge alone or surcharge with prefabricated vertical drains to accelerate consolidation depends on this information for economical design. For example, when intermittent layers of sand lenses within the clayey marine deposits, vertical drains may not be necessary.

The spacing of vertical drains is also sensitive to the permeability or consolidation properties of, the clay. Hence, SI needs to provide reliable subsoil data for design. The details on subsoil subject can be obtained from the papers by Gue & Tan (2000), Gue (1999) and Tan (1999).

### 7.2. Localised Weak Zone

Conventional selection of subsoil design parameters are based on individual judgment and experience, in particularly on the selection of moderately conservative design line (MCL), as shown in Figure 16. However, scattered data that are weaker than the selected MCL is always unavoidable (see Figure 16). This is also to ensure practical and economical design being implemented. Therefore, special attention should then be paid to localized weak zones where more intensive ground improvement procedure should be adopted. If there are no attempts to solve those localized areas, embankment failures are likely to occur in areas with these localized weak zones.

For a linear infrastructure project carried out by the Authors, the selected MCL for a regional undrained shear

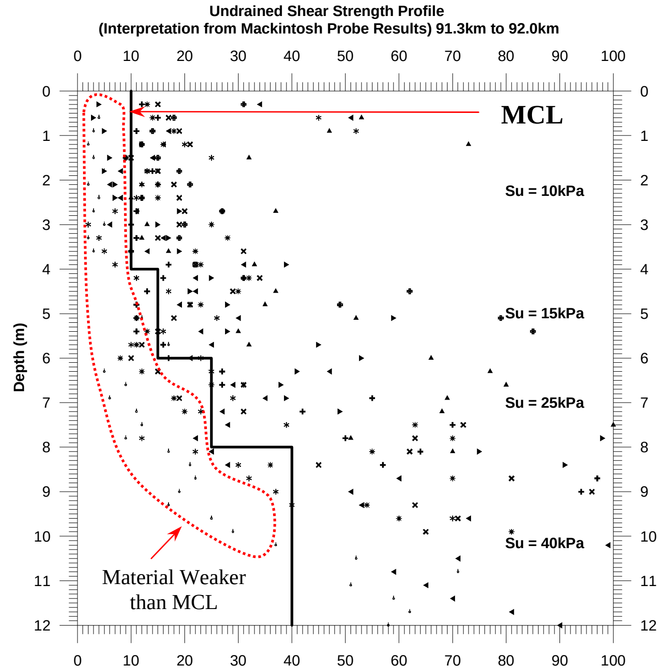


**Fig. 16. Generalised MCL across 1400m**

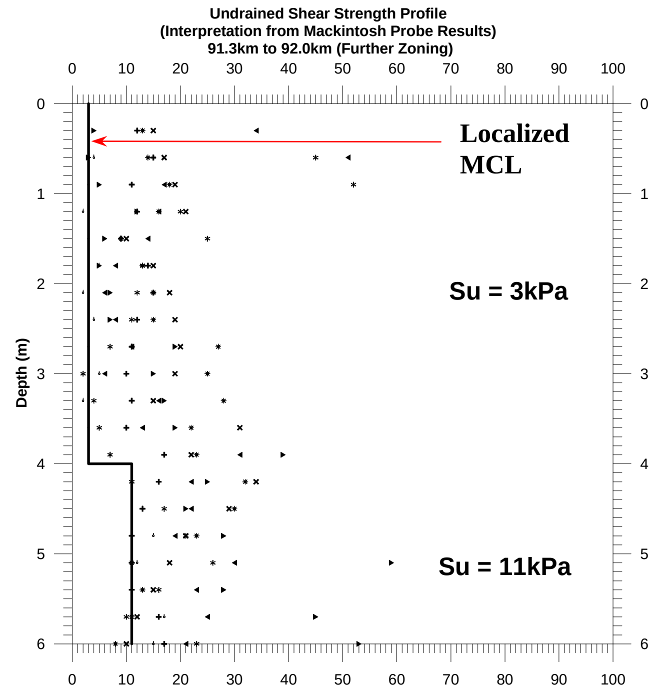


**Fig. 17. MCL for localised weak zone**

strength (Figure 16) has been further zoomed in to identify the local weak zone for special ground treatment design (see Figure 17). Such zoning has enable the usage of ground surcharge with Prefabricated Vertical Drains as a general ground treatment across the regional while adopting Piled Embankment at localized weak zones. This should be further verified by additional in-situ Vane Shear Tests, Piezocones and Mackintosh Probes carried out after site clearance for mapping out of local weak zones.

### 7.3. Guidance Notes on Embankment Design over Soft Ground

Embankment design of roads needs to satisfy two important requirements among others; the stability and settlement. The short term stability for embankment over soft clay is always more critical than long term simply because the subsoil consolidates with time under loading and the strength increases. In design, it is very important to check for the stability of the embankment with consideration for different potential failure surfaces namely circular and non-circular. Non-circular failure is often more critical.

It is also necessary to evaluate both the magnitude and rate of settlement of the subsoil supporting the embankment when designing the embankment so that the settlement in the long term will not influence the serviceability and safety of the embankment. The details of the embankment design can be obtained from Tan & Gue (2000).

A quick preliminary check on the stability of the embankment is possible using simplified bearing capacity equation below (Gue & Tan, 2004):

$$q_{allow} = \frac{s_u N_c}{FOS} \tag{1}$$

where :

$q_{allow}$ = allowable bearing pressure (kN/m$^2$)
= ($\gamma_{fill}$.H + 10)
$\gamma_{fill}$ = bulk unit weight of the compacted fill (kN/m$^3$)
H = allowable height of embankment (m)
$s_u$ = undrained shear strength of the subsoil (kPa)
$N_c$ = 5 (suggested by Authors for ease of hand calculation)
FOS = Factor of Safety (e.g. minimum of 1.2 for short term using moderately conservative $s_u$)

Note : The 10kPa allowance in the $q_{allow}$ is to cater for the minimum vehicle load.

Of course more detailed analyses are required when more refined soil layers and properties are obtained.

## 8. CONCLUSION & RECOMMENDATIONS

The geotechnical challenges presented in the paper are aimed to reduce or if possible avoid the occurrence of global and serviceability failures of highways due to design and construction errors. As such, necessary precautionary measures should be taken along the entire process of project implementation, from planning, analysis, design to construction.

Among others, the following are some of the simple and yet effective methods recommended for all design consultants:

**(i)** Always emphasise on engineering assessment of bearing capacity at least by crude check prior to detailed analyses. This can be done by exercising the following suggestions:

a) Do not abuse geotechnical design, detailed analysis is a must.

b) Do not overlook localised weak zones where special ground treatment techniques should be incorporated.

c) Do not overlook the importance of structural detailing (e.g. to replace approach slabs by transition slabs)

**(ii)** A systematic check and review process should be implemented for all designs. Reviews in particular must be done by engineers experienced in soft ground design.

**(iii)** Structured training programmes should be scheduled for practitioner of all levels to enhance the technical understanding and to provide a platform for the sharing of lessons learned.


## ACKNOWLEDGEMENTS

The Authors would like to express appreciation to Mr. Tan Tau Chuien for his contribution in preparing the necessary subsoil information for the compilation of this paper.


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