

PREVENTIVE MEASURES OF FOUNDATION FAILURES – CASE STUDIES

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SYNOPSIS : Many foundation failures are known to be attributed to inadequate protection to the foundation soils, lack of consideration in the engineering characteristics of soils under different stress and loading conditions, improper construction sequence and inadequacy of design against possible worst condition in long term. This paper describes several cases of foundation failures occurred in this region which include failures of slopes in close proximity to building foundations, failure of oil tank during water test, failure of sand bund during reclamation work and buckling of piles due to construction effect in soft ground. Possible cause(s) of failure and stabilization methods adopted in the temporary and permanent restoration works are addressed. Geotechnical recommendations and protective measures for preventing foundation failure are described.

INTRODUCTION

The types of foundation failure commonly encountered in the construction field can be classified into the following categories :

- i) *Type 1* - Inadequate protection resulted in weakening of foundation soil leading to instability and failure of foundations.
- ii) *Type 2* - Incompetent bearing stratum resulted in collapse or excessive deformation of foundation soil under the imposed loads.
- iii) *Type 3* - Improper sequence/method of construction resulted in foundation failure caused by induced damaging effect.
- iv) *Type 4* - Inadequate design not taking into consideration of the most critical and worst combination of stress and force which may occur during its service period.

- v) *Type 5* - Poor construction quality affecting the performance of design and lead to subsequent failure of foundation.

The following sections present several cases of foundation failures attributed to the causes as described above.

FAILURES DUE TO INADEQUATE PROTECTION

Two (2) earth slopes in the eastern part of Singapore failed during a prolonged rainfall period in 1983 and affected the safety of two (2) residential buildings situated on top of the failed slopes as shown in Figs. 1a & 2a. According to the results of study, both of these two (2) slope failures are found to be attributed to the effect of soil saturation and build-up of porewater pressure in the slope ground resulted from the critical storm weather. In order to minimize further damage to the existing buildings, temporary

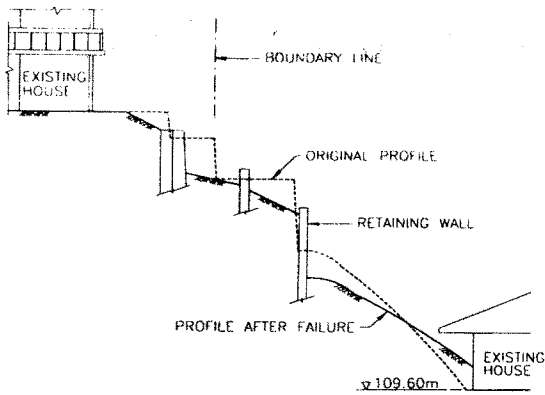


Fig. 1a Temporary Stabilisation Work at Site A

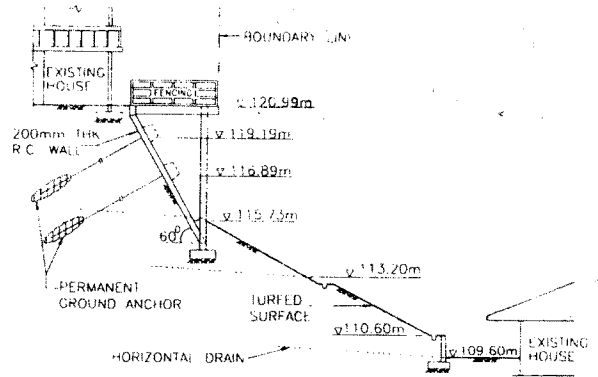


Fig. 1b Permanent Stabilisation Work at Site A

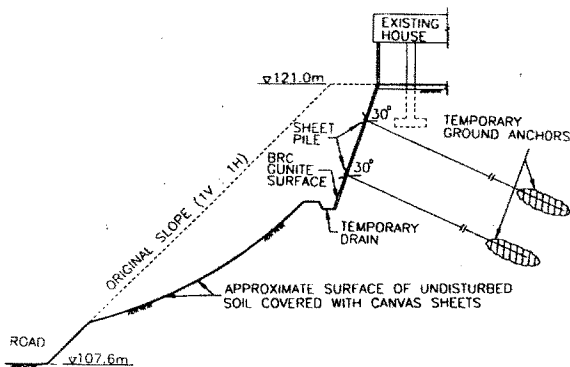


Fig. 2a Temporary Stabilisation Work at Site B

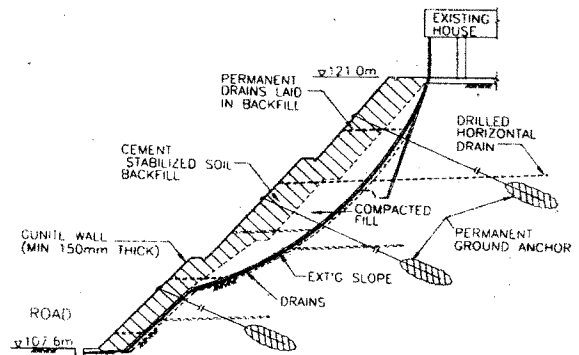


Fig. 2b Permanent Stabilisation Proposal for Site B

stabilisation method consisting of reconfiguration of slope profile and temporary anchors with surface protection was adopted for Site A & Site B as shown in Figs. 1a & 2a, respectively. For permanent stabilisation of these failed slopes, strengthening measures such as permanent ground anchors/retaining walls and protective measures such as gunite surface and horizontal drains/weep holes for preventing surface erosion and build-up of porewater pressure, respectively, have been used as shown in Figs. 1b & 2b.

FAILURE DUE TO POOR CONSTRUCTION QUALITY

A 28m high fill slope with a single flight of 1V:1.75H to 1V:2.0H constructed by using compacted fill placed on original ground with

relatively milder slope was found with localized failure near the toe of slope after a spell of heavy downpour in 1992. Subsequently, the slip failure was arrested by placement of temporary fill at the toe of slope and by restraining surface water from percolating into the slope surface as a temporary measure.

The original design of the compacted fill slope which supports a series of 2-storey housing on top of the fill has incorporated the installation of a drainage blanket at average depth of 2.5m below the design slope surface (Fig. 3a) for eliminating infiltration effect on the slope stability. According to the design drawing, the drainage blanket consisted of 500mm thick crushed rock enclosed in geotextile filter cloth. However, results of field investigation indicated that the drainage

blanket exposed in the earthslip zone where the drainage outlet was located was found full of fine material and its thickness was far below the design requirement and as a result the functioning condition of the drainage system was affected.

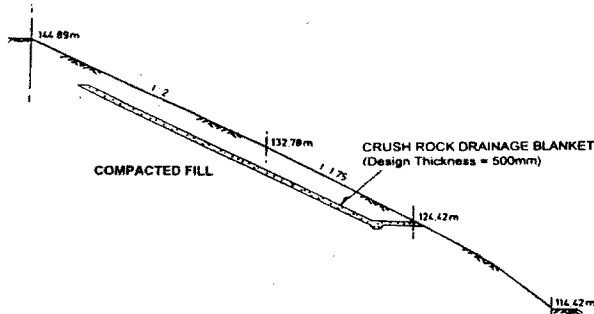


Fig. 3a Details of Drainage Blanket

The results of investigation showed that the stability of the fill slope is sensitive to the effect of wetting band which may be created on the existing drainage blanket due to its poor quality as observed in the earthslip zone. The causes of slip failure were concluded to be inadequacy of the existing drainage system (i.e. both the drainage blanket and the surface drains) and the effect of prolonged rainfall. The failed slope was finally restored by using stabilisation measures consisting of i) improvement of existing drainage system using horizontal drains and interceptor drains at mid slope, ii) ground anchor system for temporary and permanent stabilisation at critical slope section, iii) restoration of slipped ground using replacement and compaction method, iv) surface trimming to remove weak soil in the upper slope and v) close turfing to all exposed slope surfaces as shown in Figs 3b and 3c.

FAILURE DUE TO INCOMPETENT BEARING STRATUM

Two (2) cases of failures which were attributed to the existence of incompetent bearing stratum were encountered in the construction

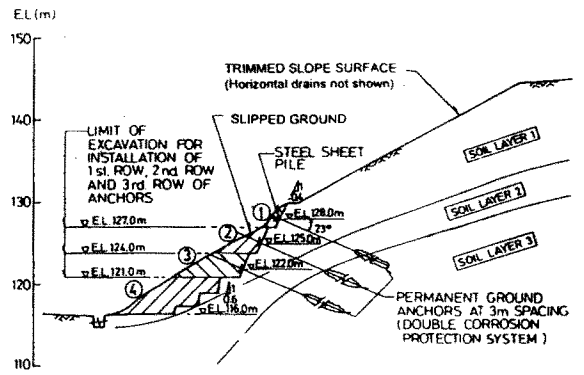


Fig. 3b Permanent Stabilisation to a 28m High Fill Slope (Step 1)

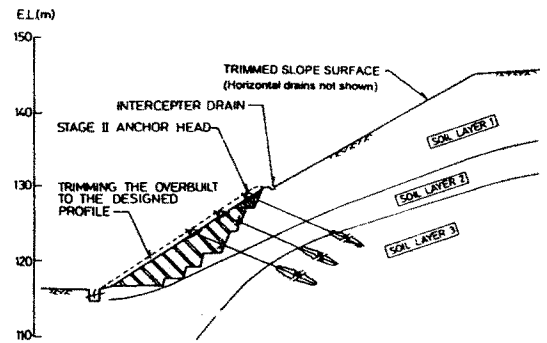


Fig. 3c Permanent Stabilisation to a 28m High Fill Slope (Step 2)

of a Land Reclamation Project and a Shrimp Framing Project both carried out in the soft clay formation in 1992 and 1996, respectively.

For the reclamation work, the construction consisted of 4 major steps : i.e. i) sand key dredging, ii) filling sand for sand key, iii) bund construction and iii) reclamation, a major slip failure occurred during pumping of the sand to level approaching the designed height (i.e. 5.5m) of the bund. The failure resulted in a volumetric difference of +173,620 cu m (i.e. increase) in the seabed profile and a volume loss of about 3,200 cu m in the sandfill of the newly reclaimed land. The results of stability analysis (Fig. 4) indicated that the design profile of sand bund did not possess adequate factor of safety against punching failure, translational and circular failure due to existence of very soft marine clay/peaty clay found below the sand bund. On the basis of the

measured volume changes and results of analysis as shown in Fig. 4, it is believed that the fill sand has already been transported (i.e. washed and/or progressively slipped) into the sea even before reaching the designed height of sand bund due to existence of the incompetent clay layers.

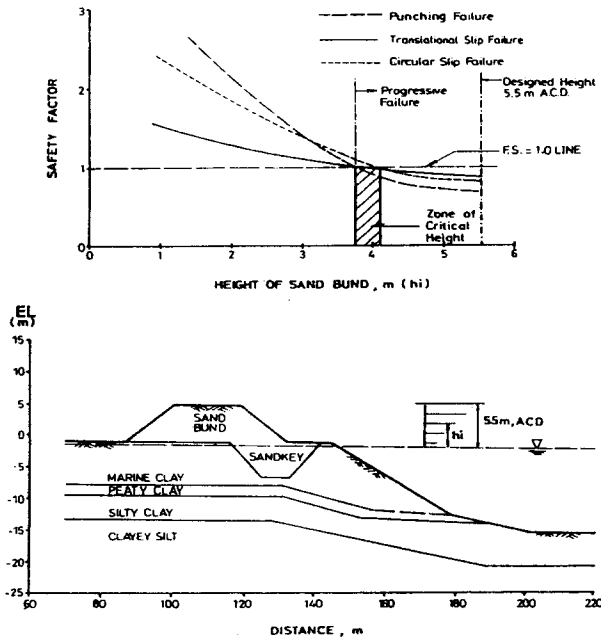


Fig. 4 Changes of Safety Factor Versus Various Height of Sand Bund

For the Shrimp Framing Project involving the construction of 126 nos. of ponds using cut and fill method, slip failures of embankment were encountered at 15 major locations where very soft unconsolidated marine clay of low shear strength and high compressibility was

found. The embankment with an overall height of 1.5-3.1m were formed by cutting the existing soft marine clay to depth of 0.3-1.9m below the original grade and filling of 0.3-2.1m above original grade using soft clay excavated from the ponds (Fig. 5). Inspection carried out after failure found that deep and wide cracks developed at top and slope surface of the failed embankment. The observed mode of failure showed that the slip failure was likely to be attributable to the development of tension cracks and deep sliding plane through the underlying soft clay due to the inadequate shear strength in supporting the embankment surcharge. This possibility was further verified by the results of stability and bearing capacity analyses which gave low factor of safety values of less than 1.0 (Table 1).

Table 1 Stability Analysis of Embankment

Case No.	Tension Crack	q kN/m ²	Most Critical Slip Surface	Computed Min. Factor of Safety
1 (Right)	Not Considered	0	1	0.86
1 (Left)	Not Considered	0	1	0.81
2 (Right)	Considered	0	2	0.73
2 (Left)	Considered	0	2	0.68
3 (Right)	Not Considered	10	3	0.74
3 (Left)	Not Considered	10	3	0.67
4 (Right)	Considered	10	4	0.64
4 (Left)	Considered	10	4	0.57

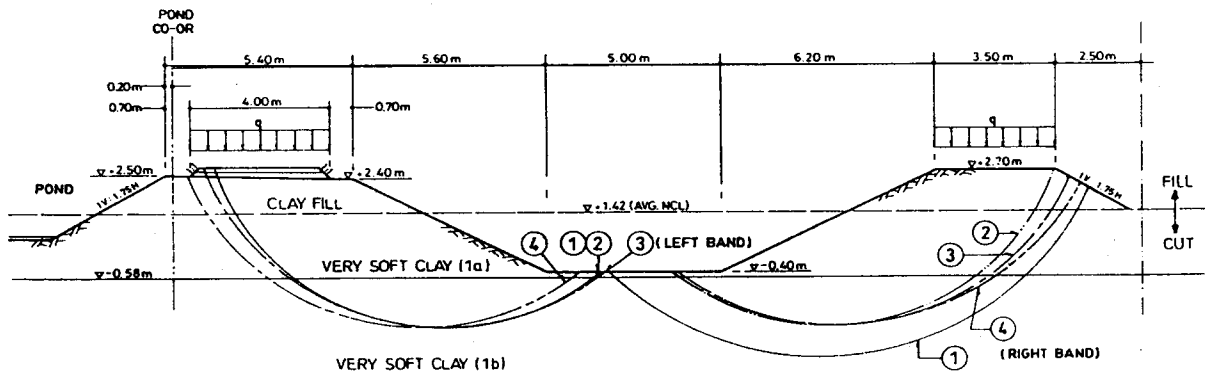


Fig. 5 Typical Profile and Stability of Embankment

FAILURES DUE TO IMPROPER SEQUENCE AND METHOD OF CONSTRUCTION

Two each of 11.3m and 6.2m diameter steel tanks constructed on shallow foundation failed the hydro-testing due to excessive settlement, tilt and suspected base plate deformation. Fig. 6 shows the time-settlement curve of one of the tank (TK-3A) prepared based on the record of hydro-test. Results of cause study on excessive settlement experienced by the four tanks found that the failures were attributed to the existence of soft organic clay/marine clay which were encountered below a 3m thick fill layer and extended to a depth of 12m below the tank base. The analysis of tank settlement (Table 2) showed that the observed settlement were within the expected final settlements of the four tanks founded on shallow foundation. Therefore, the excessive settlement of tanks could be avoided if proper assessment on tank settlement and application of soil improvement technique were considered in the tank foundation design.

Foundation failures caused by detrimental effect induced by the construction work were encountered in two (2) building construction projects. One was founded on footing foundation embedded at deep depth in sloping ground consisting of residual soil and the other one was supported by RC driven piles embedded in soft marine clay. The piles and pilecaps were constructed by the piling contractor but without the construction of ground beams. Serious offset of column stump positions were found in both projects after the completion of soil backfilling surrounding the footings and pilecaps. Pit exploration carried out at the disturbed foundation locations found that shear failure of ground beams/stumps of footings and bending failure of RC piles occurred at the offset stump locations. The results of investigation showed that the causes of failures were likely due to improper backfilling and surcharge effect of heavy construction machines which resulted in additional stress and soil movement in the vicinity of the footings and pilecaps and poor lateral resistance of RC piles against movement in soft clay. The damaged ground beams were demolished and reconstructed and the damaged RC piles were compensated by using bored piles and strengthened by using tied beams.

DESIGN CONSIDERATION AND PREVENTIVE MEASURES

As described in the previous sections, it is noted that many foundation failures could be avoided if the design does take into consideration of the most critical condition and adequate protection to ensure the integrity of design in long term.

- i) The foundation shall be designed to withstand the most critical combination of loads which the foundation may be subjected to during its service period.

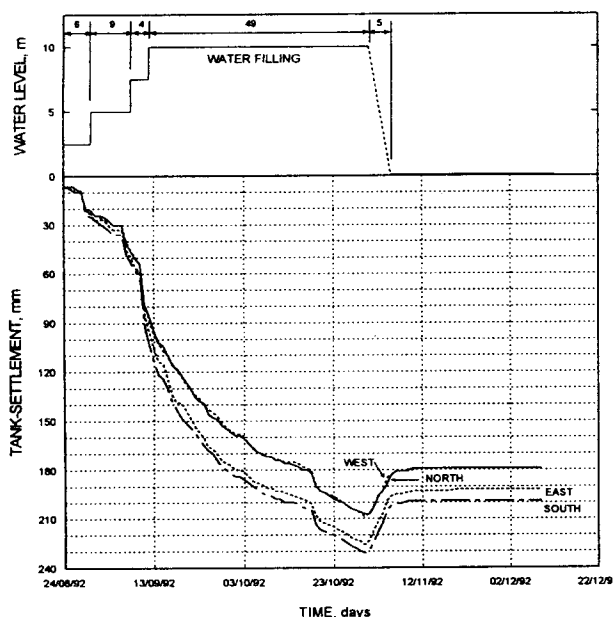


Fig. 6 Time-Settlement of Tank No. 3A under Hydro-Test

Table 2 Settlement of Tanks

Tank No.	Computed Final Settlement, mm	Average Observed Settlement, mm
2A	159	122
2B	159	132
3A	278	218
3B	296	222

Additional forces induced by downdrag effect of subsiding ground on piles, rising of groundwater table on retaining wall and effect of wetting band on slope stability shall also be considered.

- ii) The characteristics of foundation soil including its mode of failure shall be well delineated for ensuring the most representative approach of analysis and valid assumption in the design, e.g. cohesive or cohesionless soils, saturated or unsaturated conditions, drained or undrained shearing, modes of failure and rapid or slow application of loads, etc. For the type of $c-\phi$ soil commonly found in Singapore except for the soft marine deposits it is recommended that the bearing capacity of foundation soil should be analyzed and checked for both drained and undrained cases to determine its safe bearing capacity. When compressible materials exist within influence zone of foundation, settlement of the foundation should also be assessed to ensure that no excessive settlement will be induced under the design foundation pressure.
- iii) Suitable types of field and laboratory tests shall be assigned based on the soil types, existing and future stress conditions to determine the representative soil parameters. It should be noted that c_u and ϕ_u values of clay reported in some of local soil investigation works do not truly represent the undrained shear strength of clay and therefore are not recommended for foundation design.
- iv) For design of foundations located on slopes, the effect of ground profile on the bearing capacity of foundation should be properly analyzed to determine the possible reduction of soil bearing capacity. Earth slopes supporting foundations of structures should be adequately designed in terms of effective stress taken into consideration of the most critical piezometric profile which may be induced

by the critical rain storm and the worst loading condition during its service period. Adequate surface protection and drain system including surface and subsoil drains shall be provided to prevent surface erosion and soil disturbance due to infiltration of rainfall. The slope should possess adequate factor of safety against possible slip failure in long term.

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