Design and Construction of Road Embankment on Soft Ground

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ABSTRACT: The design and construction of embankment over very soft compressible alluvial deposits has always been a challenging task for Engineers. This paper presents a set of guidelines for the design and selection of construction methods for embankment taking into considerations of safety, direct and indirect costs, duration of completion and other cost benefits. Various commonly used ground treatment techniques such as excavate & replace (E&R), prefabricated vertical drain (PVD), temporary surcharge, geotextile basal reinforcement, stone columns and piled embankment were discussed. This paper also highlights the usage of piled embankment with different pile lengths as transition area to provide smooth profile between bridge abutment (rigid structure supported by piles installed to hard stratum) and embankment which is relatively more flexible. Finally a new construction control chart is proposed to be used during construction to allow high embankments to be built without compromising on the stability during construction and to meet the tight construction schedule and technical requirements.

1 INTRODUCTION

The construction of embankment over very soft compressible alluvial deposits (e.g. Clay, silty Clay, clayey Silt etc.) is unavoidable for infrastructures works especially roads and railway. The choice of construction method in this formation is not only governed by direct costs, but also the long term maintenance costs, duration of completion and cost benefits.

This paper presents a set of guidelines for the design and selection of construction methods for embankment taking into considerations of safety, direct and indirect costs, duration of completion and other cost benefits. Various commonly used ground treatment techniques such as excavate & replace (E&R), prefabricated vertical drain (PVD), temporary surcharge, geotextile basal reinforcement, stone columns and piled embankment were discussed. This paper also highlights the usage of piled embankment with different pile lengths as transition area to provide smooth profile between bridge abutment (rigid structure supported by piles installed to hard stratum) and embankment which is relatively more flexible. Finally a new construction control chart is proposed to be used during construction to allow high embankments to be built without compromising on the stability during construction and to meet the tight construction schedule and technical requirements

2 GEOLOGY OF SOFT ALLUVIAL CLAY

The behaviour of soft alluvial soils is influenced by the source of the parent material, depositional processes, erosion, redeposition, consolidation and fluctuations in groundwater levels.

Generally, alluvial deposits (materials transported and deposited by water action) consist of finest clays to very coarse gravels and boulders. Alluvial soils usually show pronounced stratification and sometimes organic matter, seashell and decayed wood are present in the alluvial deposits.

3 SUBSOIL INVESTIGATION

The subsoil conditions of the proposed embankment need to be established in varying degrees of detail during the planning and design. The basic information required for planning and preliminary design of the embankment includes :

- Site Topography;
- Geology and Landuse;
- Soil Stratigraphy;
- Soil Strength;
- Soil Compressibility;
- Groundwater Levels.

Additional soil properties may be needed depending on the construction methods to be adopted. The planning and interpretation of the site investigation and interpretation will not be covered in this paper. Details of the subject can be obtained from papers by Gue & Tan (2000), Gue (1999), Neoh (1999) and Tan (1999).

4 EMBANKMENT DESIGN

Before carrying out an embankment design and selection of the most appropriate construction methods, the following issues should be considered:

- Boundary of the embankment;
- Influence of the embankment on adjacent structures, services, slopes and drainage;
- Earliest construction start date and completion date;
- Tolerance on settlements and differential settlements of the proposed developments or structures.
- Rate at which embankment fill material can be placed;
- Availability of fill from other parts of the site;
- Availability of alternative materials;
- Cost analysis and implication of the ground treatment proposed.
- Future maintenance (frequency and cost)

4.1 Embankment Loading

The embankment loading can either be in single stage or multi-stage.

Single Stage Loading

It will cause an immediate increase in total stress and if the filling is so rapid such that dissipation of pore pressure cannot take place or insignificant, the stability of the embankment will rely on the in-situ undrained shear strength (s_u) of the subsoil.

Multi-Stage Loading

The advantage of multi-stage is that the subsoil is allowed to increase in strength as consolidation took place under the embankment load. However, the rate of increase in loading needs to be limited so that the ratio of the load to the available strength of the subsoil is within the acceptable factor of safety. This method also requires longer time of construction.

4.2 Stability Analysis of Embankment

The stability of the embankment is commonly assessed using a limit equilibrium analysis. It is important in stability analysis of the embankment to consider different potential failure surfaces, circular and non-circular, as shown in Figure 1. This is because circular failure surfaces may not yield the lowest factor of safety (FOS), particularly for embankments on thin clay layers or where discrete weaker layers occur, where translational failure generally dominates. The FOS against failure is usually defined as :

$$FOS = \frac{\bar{s}}{\bar{\tau}}$$

Where

$\bar{s}$ = Average shear strength available along the failure surface.

$\bar{\tau}$ = Average shear stress applied along the failure surface.

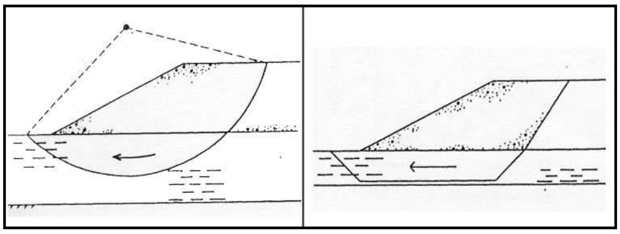


Fig.1 Circular & Non-Circular Failure Surfaces

Computer programs that offer different methods of limit equilibrium stability analysis are commonly available. Table 1 below summarises the different methods of stability analysis together with the comments.

In general, there are three types of methods in modelling the soil in the stability analysis and they are :

(a) Total Stress Analysis
(b) Effective Stress Analysis
(c) Undrained Strength Analysis

4.2.1 Total Stress Analysis

The stability of the embankment is analysed based only on the available undrained shear strength (s_u) of the subsoil prior to start of construction, taking no account of any increase in strength after consolidation. The s_u can be based on the results of

METHOD	FAILURE SURFACE	COMMENTS
Bishop (1955)	Circular	- Consider force and moment equilibrium for each slice. Rigorous method assumes values for the vertical forces on the sides of each slice until all equations are satisfied. Simplified method assumes the resultant of the vertical forces is zero on each slice. - Simplified method compares well with finite element deformation methods.
Janbu (1972)	Non-Circular	- Generalised procedure considers force and moment equilibrium on each slice. Assumptions on line of action of interslice forces must be made. Vertical interslice forces not included in routine procedure and calculated F then corrected to allow for vertical forces.
Morgenstern & Price (1965)	Non-Circular	- Consider forces and moments on each slice, similar to Janbu Generalised procedure. - Consider more accurate than Janbu. No simplified method.
Sarma (1979)	Non-Circular	- A modification of Morgenstern & Price which reduces the iterations. - Considerable reduction in computing time without loss of accuracy.

Table 1 : Methods of Stability Analysis (adapted from Geotechnical Control Office, 1984)

unconsolidated undrained triaxial compression tests (UU), isotropically consolidated undrained triaxial compression tests (CIU), vane shear tests or Piezocone (CPTU).

4.2.2 Effective Stress Analysis

The stability of the embankment can be only be analysed using an effective stress approach, provided that both the total stresses and pore water pressures can be estimated. The available shear strength, *s*, along the shear plane can be obtained as:

$$s = c' + \sigma_n' \tan\phi'$$

where c' and ϕ'define the Mohr-Coulomb effective stress failure envelope and

$$\sigma_n' = \sigma_n - u_r$$

where σ_n is the total normal stress and u_r is the pore pressure at failure.

It should be noted that effective stress analysis will lead to a more favourable (optimistic, higher FOS) assessment of the stability than the use of undrained analysis (Ladd, 1991).

4.2.3 Undrained Strength Analysis

Undrained strength method was developed by Ladd & Foott (1974) and is further refined by Ladd (1991). This method can also take account of the gain in undrained shear strength ($+\Delta s_u$) as a result of consolidation. The Undrained strength analysis (USA) extends the total stress analysis by using the current vertical normalised strength ratio of s_u/σ_v', where σ_v' is the current vertical effective stress.
There is a few ways to estimate the ratio of s_u/σ_v':

a) $s_u/\sigma_v' = 0.11 + 0.0037$ PI

For normally consolidated clay, the ratio tends to increase with plasticity indeed (PI) (Skempton, 1957).

b) $s_{u(mob)}/\sigma_p' = 0.22$;

where $s_{u(mob)}$ is the undrained shear strength mobilised on the failure surface in the field, and σ_p' is the preconsolidation pressure (yield stress) (Mesri, 1988).

Unlike effective stress method, the pore water pressures set up during shearing to failure need not be estimated, thus eliminating an unknown in the design procedure. This method is most commonly used in the analysis of short term stability and design of staged construction.

4.2.4 Factor of Safety

The factor of safety to be used in the stability analysis will depend on the following factors :

- Method of analysis
- Reliability of the design method
- Reliability of the design soil parameters
- Consequences of failure in terms of human life and economic loss.

O'Riordan & Seaman (1993) reports that BS6031:1981 gives no specific values or method for soil strength determination for use in embankment design. It only refers to a range of factor of safety between 1.3 and 1.4 for cut slopes.

Generally in practice, the factor of safety on shear strength (FOS) from total stress or undrained strength analyses used in temporary stage is usually taken as between 1.2 to 1.3. FOS of 1.4 and 1.5 are normally adopted in effective stress analyses of embankment for permanent stage. It should be noted that designing with low FOS increases the possibility of large vertical and lateral ground deformations and also risk of failure.

4.3 Settlement Calculation

Construction of embankments will cause settlement to take place in the subsoil during and after filling. This phenomenon is more obvious for embankment constructed over soft clay due to consolidation process. Therefore, it is necessary to evaluate the magnitude and rate of settlement of the subsoil supporting the embankments so that the settlements in the long term are within the specified limits and shall not affect the serviceability of the railway tracks on top of the embankments.

It is important to estimate the magnitude of settlements that occur during construction and waiting period so that the total actual thickness of the fill at site can be designed to ensure stability. An iterative process is required in the estimation of the settlement because the extra fills (more load) are required to compensate for settlement that will lead to further settlement of the subsoil and also to ensure the post construction settlements are less than the fixed allowable settlement limits. Figure 2 illustrates the important of iterative exercise to achieve the designed platform level.

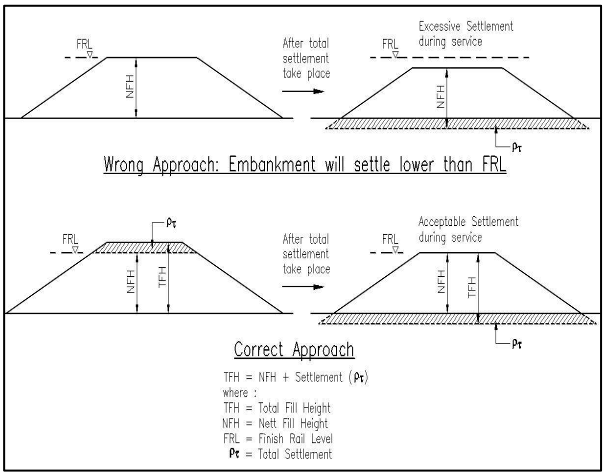


Fig. 2 Concept on Settlement Analyses

Nonetheless, traffic load shall be excluded in the settlement analyses as it is not a permanent load. With consideration of traffic load in settlement analyses will lead to overestimate of settlement and thus end up with unnecessary and costly ground treatment.

Usually the assumptions of one-dimensional consolidation are generally valid for embankment which have widths greater than the thickness of the compressible soil layer; Davis and Poulos (1972). This paper only covers the one-dimensional problem

For clay layer of larger thickness, horizontal flow of pore-water may be more significant and the one-dimensional theory tends to underestimate the rate of consolidation. The two-dimensional consolidation can be solved numerically using solutions proposed by Terzaghi (1923) and Rendulic (1936), as described by Murray (1971 and 1974)

4.3.1 Magnitude of Settlement

When a load of finite dimensions is rapidly applied to a saturated clay, the resulting settlement can be conveniently divided into three stages :

(A) Initial Settlement (also called immediate or undrained or shear settlement), ρ_i

(B) Primary Consolidation Settlement, ρ_c

(C) Secondary Compression, ρ_s

(A) Initial Settlement, ρ_i

During application of the load, excess pore pressures will set up in the clay, but relatively little drainage of water will occur since the clay has a low permeability.

Estimation of initial settlement can be carried out using elastic displacement theory as :

$$\rho_i = \sum \frac{1}{E_U}(I \cdot q)dh$$

Where

q = Applied Stress / Pressure on the subsoil (kPa).

dh = thickness of each layer (m).

E_u = Undrained Young's Modulus of the subsoil (kPa)

I = Influence factor

A useful chart is given by Osterberg (1957) and shown in Figure 3. The chart allows estimation of the initial settlement of the embankment.

(B) Consolidation Settlement, ρ_c

With time, the excess pore water pressures dissipate as drainage occurs and the clay undergoes further settlement due to volume changes as stress is transferred from pore pressure u to effective stress. The rate of volume change and corresponding settlement is governed by how fast the water can drain out of the clay under the induced hydraulic gradients.

One dimensional primary consolidation settlement can be estimated using the expression :

$$\rho_c = \sum_{i=1}^{n}[\frac{C_r}{1+e_o}\log\frac{\sigma'_p}{\sigma'_{vo}} + \frac{C_C}{1+e_o}\log\frac{\sigma'_{vf}}{\sigma'_{vc}}]H_i$$

where

ρ_c = Consolidation Settlement Magnitude (m)

σ'_{vo} = Initial vertical effective stress

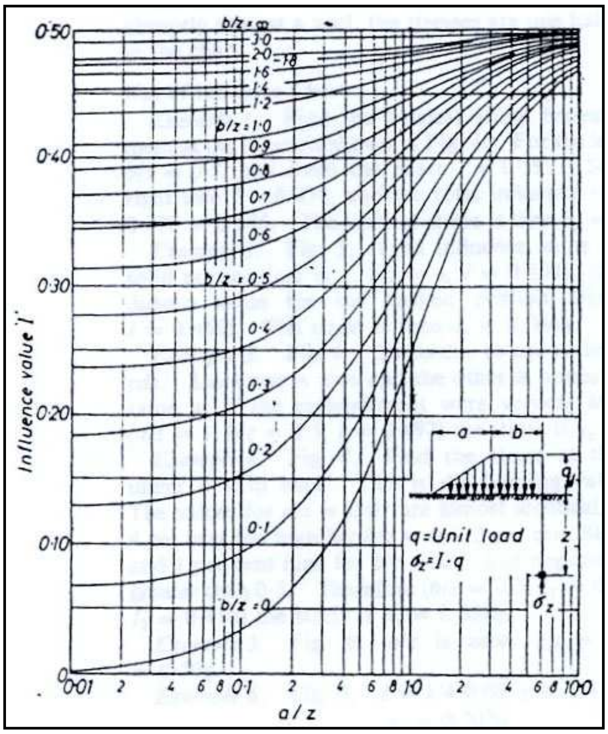


Fig. 3 Influence Chart for Vertical Stress Embankment Loading – Infinite Extent (from Osterberg, 1957)

σ'_{vf} = Final vertical effective stress

= $\sigma'_{vo} + \Delta\sigma'_{v} \geq \sigma'_{vc}$

σ'_{vc}= Preconsolidation Pressure / Yield Stress

H_i = Initial thickness of incremental soil layer, i of n.

e_o = Initial voids ratio

C_C = Compression Index

C_r = Recompression Index

Values of $\Delta\sigma'_{v}$ at the centre of each soil layer due to embankment loading can be estimated using elastic theory, Poulos and Davis (1974). The parameters σ'_{p}, e_o, C_C and C_R can be obtained from oedometer consolidation tests. The notation and terminology used are shown in Figure 4.

(C) Secondary Compression, ρ_s

Even after complete dissipation of the excess pore pressures and the effective stresses are about constant, there will generally be further volume changes and increased settlement which is termed as Secondary Compression.

$$\rho_s = \sum_{i=1}^{n}[\frac{C_\alpha}{1+e_p}\log(t)]H_i$$

where

ρ_s = Secondary Compression Magnitude (m)

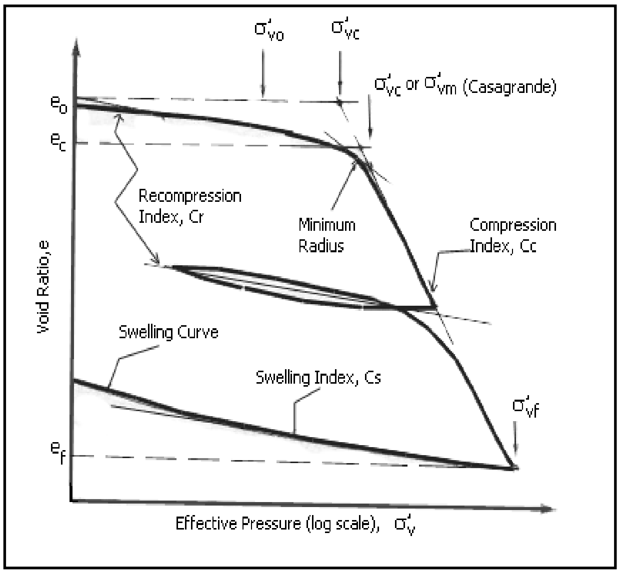


Fig. 4 Notation and Terminology used for Oedometer Compression Curves (from Balasubramaniam & Brenner, 1981)

H_i = Initial thickness of incremental soil layer, i of n.

e_p = Voids ratio at the end of primary consolidation

C_α = Secondary Compression Index.

t = Time for calculation.

Other than oedometer tests, the secondary compression ratio or Modified Secondary Compression Index, $(C_\alpha/(1+e_p))$ can be estimated from the relationship proposed by Mesri (1973) as shown in Figure 5.

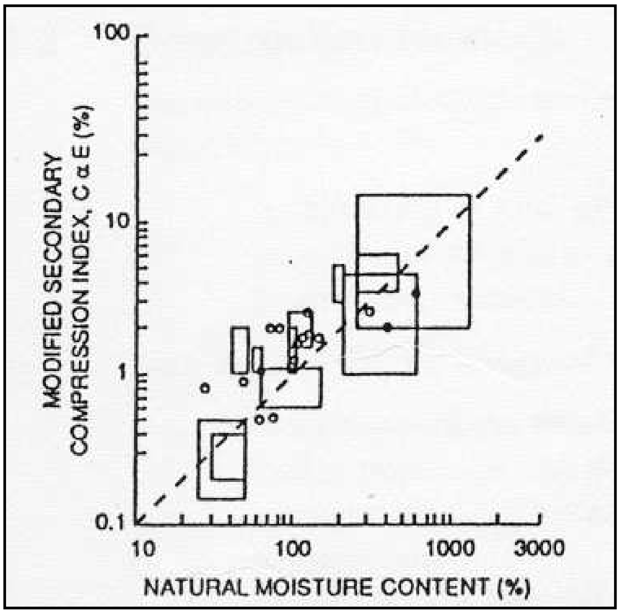


Fig. 5 Relation between Secondary Compression Ration and Water Content (from Mesri, 1973)

4.3.2 Rate of Settlement

For one dimensional consolidation with vertical drainage, the degree of consolidation, U_v is a function of the time factor, T_v where :

$$T_v = c_v\, t / H_D^2$$

Where

c_v = Coefficient of consolidation (m^2/year)

t = Time following application of loading (year)

H_D = Drainage path length (m)

The average degree of consolidation as a function of time factor for Terzaghi's theory of consolidation by vertical flow can be expressed as :

$$U_v = \sqrt{\frac{4T_V}{\pi}}$$

for $T_v = c_v\, t / H_D^2 < 0.2$

$$U_v = 1 - \frac{8}{\pi^2}\exp\left(\frac{-\pi^2 T_V}{4}\right)$$

for $T_v = c_v\, t / H_D^2 \geq 0.2$

The coefficient of consolidation, c_v, can be obtained from oedometer tests at the levels of effective stress similar to those anticipated under embankment loading. Another reliable way to determine c_v is from field in-situ permeability tests together with m_v from laboratory oedometer consolidation tests :

$$c_v = k / (m_v\, \gamma_w)$$

where

k = permeability from field permeability tests (m/sec)

m_v = coefficient of compressibility (m^2/kN)

γ_w = density of water (kN/m^3)

The use of field values of k will give a better representative effects of large scale soil structure and permeability, not able to be reflected in laboratory tests. Since the permeability and compressibility of the soil reduce with increase in effective stress (under embankment loading), the value of c_v should be modified to reflect the state of stress over the period during which settlement rates are being calculated.

5 METHODS FOR EMBANKMENT CONSTRUCTION

From the results of stability analyses results, an engineer will be able to know whether it is feasible or not to construct the embankment in single stage, or multi-stage and combination of other alternative construction methods. Figure 6 shows the flow-chart, outlining the summary on selection of construction methods.

In the cost conscious market of today, usually a cost comparison between the various methods which are technically feasible will be required by an engineer throughout the design. Only by carrying out analysis of the costs and benefits of different methods, will the engineer able to identify where possible modification to the initial constraints can be undertaken.

The following sections of the paper describes some of the commonly used embankment construction methods.

5.1 Modification of Embankment Geometry

Reduction of slope angle or construction of counterweight berms improves the stability of the embankment by increasing the length of potential failure surfaces in the soft soils. The weight of the shallow slope or berm counter-balances the disturbing moment on potential failure surfaces under the embankment.

The fill should be raised equally across the embankment. However this method has a disadvantage of greater land-take and volume of fill materials are needed.

5.2 Excavation and Replacement of Soft Soils (Total or Partial)

This method is very old but still viable. The very soft compressible cohesive soils are excavated out and replaced with better materials (e.g. compacted sand or suitable fill) that provide a stronger and less compressible foundation. The experience on highway construction indicates that the excavation and replacement depth up to a maximum depth of 4.5m is viable in terms of cost and practicability. Usually the excavation should extend to at least to the toe of the embankment and beyond to increase the stability of the embankment.

If the soft material is much deeper than the practical excavation depth, partial excavation and replacement is also possible. However the effect on stability and long term settlement of the remaining soft material should be considered. Sometimes partial excavation and replacement of soft material is used with other ground treatment to overcome the above problems.

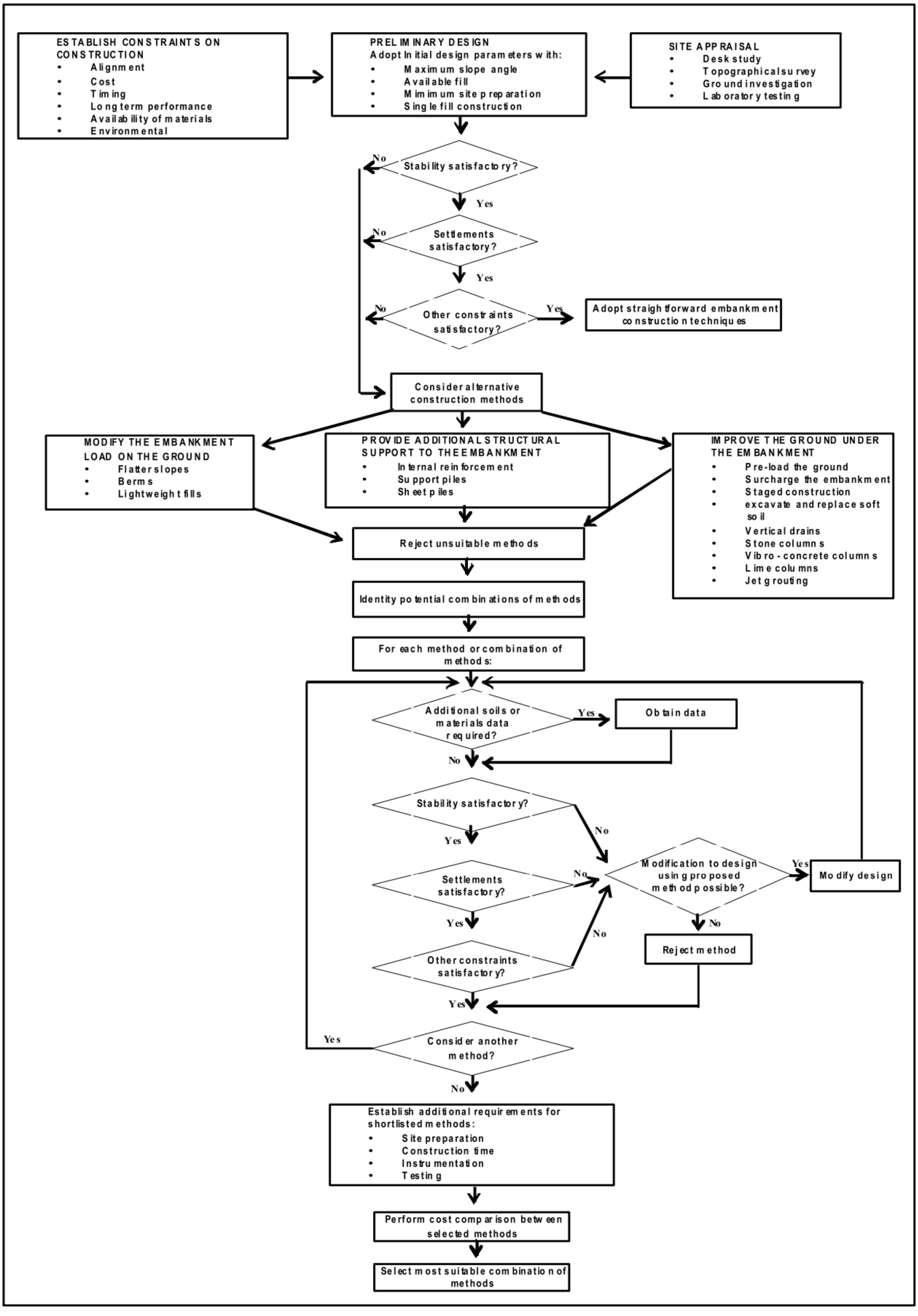


Fig. 6 Procedure for the Selection of Construction Method (from O'Riordan & Seaman, 1993)

This method will be more difficult if the groundwater level is high. If pumping of water not practical, then compacted suitable material cannot be used and underwater replacement materials (granular materials) should be used. These materials shall be of a grading that it is effectively self-compacting. The main disadvantage of the method is the amount of soft soil which needs to be disposed.

There is always some arguments that the top few meters of very soft to soft clay shall not be replaced as the undrained shear strength are relatively higher in view of hard crust. However, the Author opines that with the removal of the top few meters of very soft to soft clay will result in significant reduce in settlement as shown in Figure 7. It also demonstrates that by removed the top 0.5m to 2m very soft to soft clay, the subsoil settlement is reduced by 27% and 60% respectively compared to no E&R. Another reason is the strength of thin top crust is not reliable and will be disturbed and weaken during construction due to movement of construction machineries and soaking.

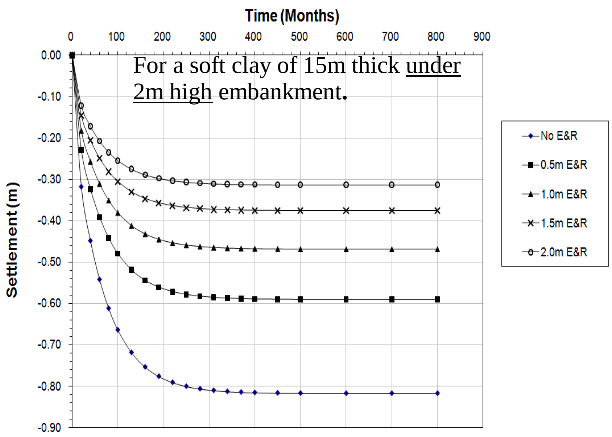


Fig. 7 Effect of E&R on Magnitude of Consolidation Settlement

5.3 Vertical Drains

Vertical prefabricated band-shaped drains are installed through soft clay soils to accelerate the speed of consolidation of the subsoil by reducing the drainage path lengths and utilizing the naturally higher horizontal permeability of clay deposits. Prefabricated drains using corrugated polymeric materials (polyethylene and polypropylene) for the core, and woven or non-woven fabric or fibre for the filter. They are about 100mm wide, about 4mm thick and are installed using a closed-end mandrel and usually to a depth no more than 30m in very soft soil or terminate shorter in stronger materials (SPT'N' ≈ 7 to 10). Pre-boring will be required to penetrate some surface crust or artificial obstructions at the surface.

Vertical drains will only be effective when using in conjunction with another technique, such as pre-loading, surcharging and staged construction (to be discussed in the following sections) and the design is governed by the time allowed in the construction programme for consolidation to occur. The average degree of consolidation for radial consolidation by Barron's theory is given by Hansbo (1981) as :

$$U_h = 1 - \exp\left(\frac{-8T_h}{\mu}\right) \quad \text{for} \quad T_h = c_h\, t / D^2$$

Where $$\mu = \frac{n^2}{n^2-1}\ln(n) - \frac{3n^2-1}{4n^2}$$

Where

n = drain spacing ratio, D/d

D = Diameter of an equivalent soil cylinder influenced by each drain, which is equal to 1.13s for a square pattern and 1.05s for a triangular pattern.

s = drain spacing

d = equivalent diameter of prefabricated vertical drains = 2(b+t)/π (Hansbo, 1981)

The average degree for combined vertical and radial consolidation is obtained from Carillo's theorem (1942) :

$$U = 1 - (1-U_v)(1-U_h)$$

The vertical drains should have sufficient capacity to enable the water to discharge to layers above and below the consolidating layer. Granular materials are laid above the ground surface as platform for the movement of the plant and also as drainage layer. Pre-fabricated drains are usually left about 150mm above the initial drainage layer prior to placing further drainage material.

Another important aspect in PVD construction is discharge outlet, which is always overlooked by the engineers. If the discharge outlet is clogged, the excess pore water pressure will not be able to discharge effectively. Therefore, following are some good construction practices proposed by the Author and implemented at the electrified double tracks railway project to improve PVD efficiency with minimal cost:

a) To install PVD at horizontal direction with spacing of not more than 5m c/c. This is to use cost effective PVD as modified "horizontal" subsoil drains to assist the sand blanket layer to discharge water

b) To provide crusher run at the end of sand layer as shown in Figure 8. This is to ensure there is clear outlet for the water coming out from the subsoil through vertical drains to be effectively discharge out from the embankment. A clearly visible crusher run layer will also prevent contractor from accidentally block it and for easy inspection by supervising engineer.

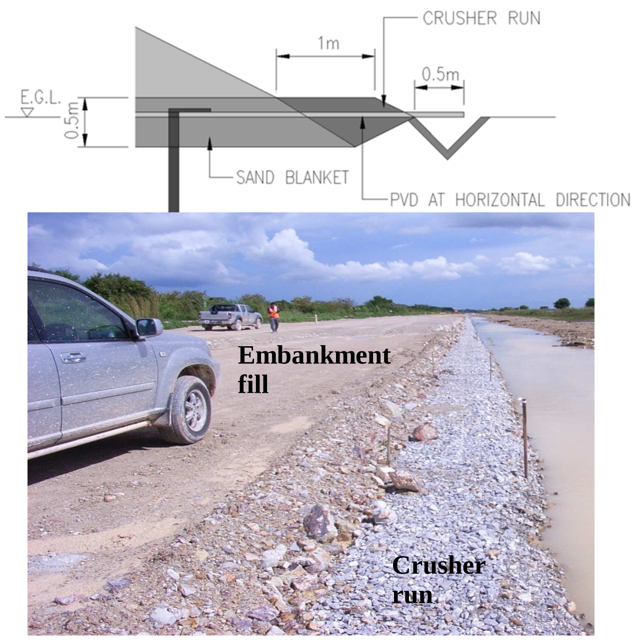


Fig. 8 Details of Discharge Outlet for Sand Blanket Connecting All PVD

5.4 Temporary Surcharging

Temporary surcharging is to subject the ground to higher pressure than that during the service life in order to achieve a higher initial rate of settlement thus reducing long term settlements. Usually this method are used to control both total settlement and differential settlement at the abutments to bridge / flyover and where culverts are crossing beneath the embankment.

Several important design criteria for this method are as follows :

- Stability should be checked with extra temporary surcharge load
- Temporary surcharging should be designed to chosen construction period
- Settlement after construction should be within the range of tolerances
- The option should be economical
- Proper planning of construction programme for cost effective use of materials available
- Does not cause damage to any adjoining structures.

The magnitude and duration of the temporary surcharging will be controlled by the magnitude of total settlement (consolidation and secondary settlement). Usually the extra loading must continue until the effective stress in the subsoil is larger than that from the long term loading from the embankment. This method can also reduce the effects of secondary compression slightly.

Figure 9 shows the concept of surcharging. Usually, surcharging is used together with prefabricated vertical drain (PVD) installed into the low permeability soft compressible layer.

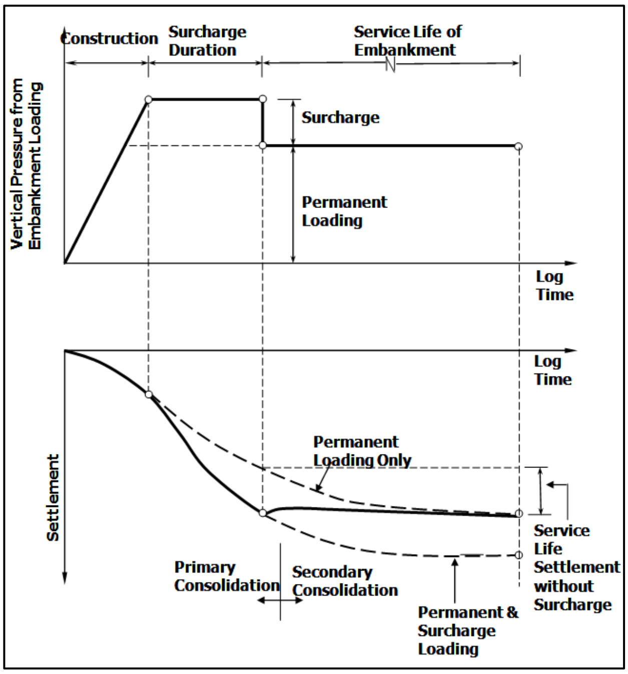


Fig. 9 Surcharging Concept

5.5 Geotextile Basal Reinforcement

Installation of PVD will not increase the subsoil shear strength. The subsoil will only experience gain in strength upon dissipation of excess pore pressure caused by the embankment load during construction (filling) and surcharged period. As the soft compressible subsoil has low undrained shear strength, staged construction is normally adopted to construct the embankments. However, in order to meet the tight construction schedule, geotextile basal reinforcement is used to allow higher embankments to be built without compromising the embankment stability during construction.

5.6 Stone Columns

Stone columns were utilised as ground treatment once the combination of E&R, PVD, temporary surcharge and geotextile basal reinforcement are found not viable. The presence of stone columns creates a composite material of lower compressibility and higher shear strength than the in-situ very soft to soft clay. Therefore, stone columns were adopted to support high embankments. In addition, stone column also act as "giant PVD" to accelerate consolidation settlement and thus shorten the construction period.

Following are several limitation of stone columns:

a) Commonly not recommended for subsoil with undrained shear strength of less than 10kPa in view of low confinement stress in the subsoil. Special care shall be exercised in design and construction if stone columns are to be used for the subsoil with very low shear strength
b) Not suitable for subsoil with high organic contents

Commonly not recommended for embankment height of less than 2.5m. Sufficient embankment height is required to provide soil arching within the embankment fill to distribute more load to the stone columns instead of loading the soft soils between them. This is to prevent long term "mushroom" effect problems (uneven settlements that looks like mushrooms) on the embankment surface. However, thicker crusher run layer with high angle of friction (ϕ') or high stiffness geogrid can be used to overcome this weakness.

5.7 Piled Embankment

At areas of very soft clay where other ground treatments are not suitable to support the railway embankment, piled raft are used as ground treatment and settlement reducer. Piled embankments are designed to provide the required embankment stability and to achieve the specified tracks settlement criteria. Hence, lower global safety factor such as 1.5 is adopted for pile capacity. Whilst, the piled slab supporting the embankments were designed generally in accordance with BS 5400 Part 4 using lower partial safety factors.

5.7.1 Piled Embankment as Transition Zone

Significant differential settlements at bridge approach are still common along highways and railways in Malaysia. Bridge abutment over soft deposits is normally supported by piles. The piles for the abutment are usually installed to a firm/hard layer. The long term settlement of the abutment is hence negligible. However, the embankment adjacent to the abutment would still have some settlement with time. Consequently, this will create a significant differential settlement between bridge abutment and flexible embankment as shown in Figure 10. This will pose high risk to the train in view of the high travelling speed. Hence, piled embankments with different pile lengths as transition area is utilised to provide smooth profile between bridge abutment (rigid structure with pile to set) and embankment. Figure 11 illustrates the innovative solution to mitigate the differential settlement.

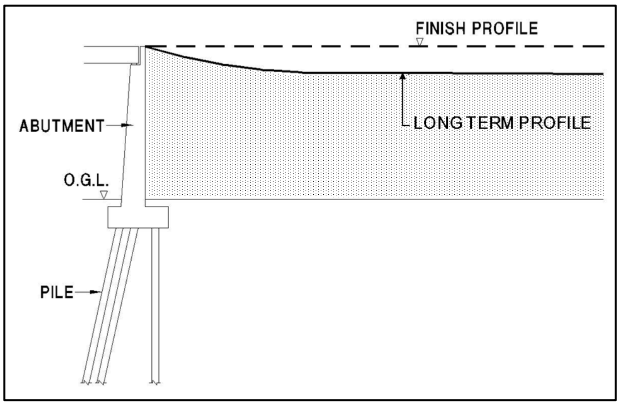


Fig. 10 Settlement Profile at Bridge Approach

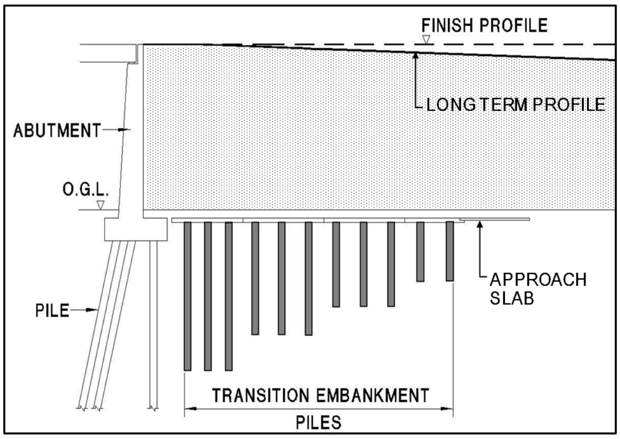


Fig. 11 Transition Pile at Bridge Approach

5.7.2 Ground Treatment for Culvert

Very often, culverts are wrongly 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. 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 flexible embankment.

In order to overcome this problem, the ground treatment and foundation for culverts shall be the same as the embankment. This mean that section with culvert needs to complete the ground treatment (i.e PVD, surcharge etc) prior to construction of the culvert. As such, the differential settlement can be mitigated.

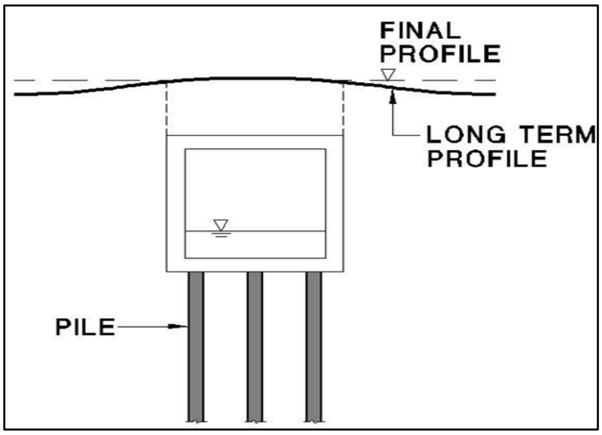


Fig. 12 Culvert Founded on Pile Foundation (not recommended)

5.8 Staged Construction

Staged construction is the method by which the embankment can be constructed on the soft soil such that the rate of filling is governed by the increase in soil strength due to consolidation. Usually vertical drains are used together to increase the consolidation process. Usually the design of the staged construction is carried out using undrained strength method (Ladd, 1991). The stability and degree of consolidation can be related to gain in strength from the tests carried out and observations of excess pore water pressures in the ground or indirect methods stated in Section 6.0.

The use of the staged construction method requires close liaison and communication between the design engineer, contractor and supervising engineer. Instruments like settlement markers, displacement markers, piezometers, etc. need to be placed to monitor the performance of the embankment during construction to prevent failure. In more sensitive cases, confirmation of gain in strength is needed before the application of the next stage of loading.

6.0 CONSTRUCTION MONITORING AND CONTROL

It is important to monitor the performance of an embankment and the subsoil supporting it during and after construction. Table 2 list different types of instrumentation that can be utilized in embankment construction. Figure 13 shows the embankment instrumentation used in the Muar Flat trial embankment by the Malaysian Highway Authority (1989).

Measurement	Types and Location
Vertical Settlement	- Settlement gauges on original ground surface or base of excavation. - Settlement markers on surface of fill or ground outside the embankment. - Full-profile settlement gauges under the embankment. - Subsurface Settlement gauges or extensometers in the subsoil beneath the embankment to measure settlement at different depths of the subsoil.
Horizontal Movement	- Inclinometers in the subsoil at toe of embankment. - Displacement markers at the top and toe of embankment.
Pore Water Pressures	- Piezometers (preferably vibrating wire type) at several depths and locations in the subsoil beneath the embankment.

Table 2 Types of Instrumentation for Embankment

6.1 Embankment Stability Check during Construction

Many literatures reported that failures of an embankment constructed over soft clay are closely related to the magnitude and history of the deformations which took place before failure. Therefore the information obtained from field instrumentations measurements can be used to ensure the safe construction of embankments.
Displacement markers that are relatively cost effective compared to inclinometer, were extensively utilised to monitor the lateral movements of the constructed embankments. With reference to the lateral movements and settlements

(recorded from settlement gauge), the stability of the constructed embankment is monitored based on "modified" Matsuo stability plot as shown in Figure 14.

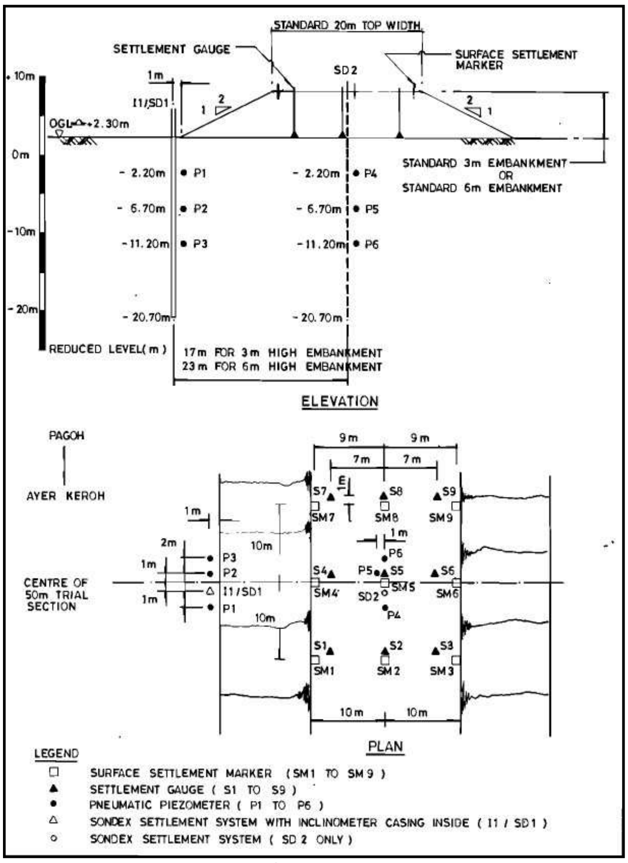


Fig. 13 Layout of Instrumentation Scheme (MHA, 1989)

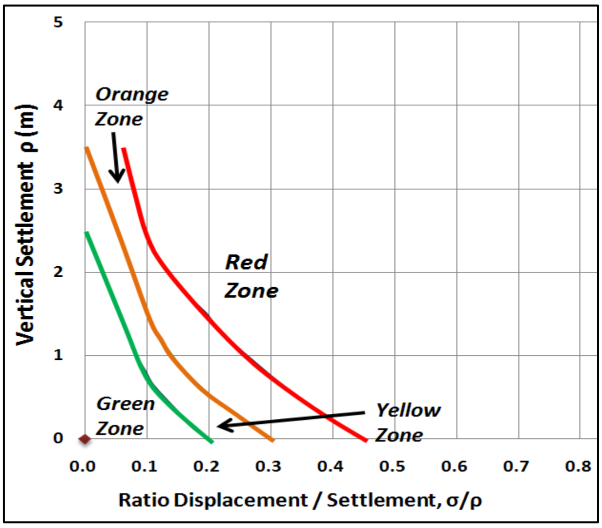


Fig. 14 "Modified" Matsuo's Stability Plot

During construction, Fill height (fill thickness) versus Lateral displacement (FHLD) plot as shown in Figure 15 was developed by the Author as supplementary to the "modified" Matsuo's Method (1997). The FHLD plot was developed based on actual monitoring results of fully instrumented trial embankment constructed in Tokai, Kedah, Malaysia (Tan et al., 2010) and finite element method (FEM) analysis using computer programme "Plaxis".

Both "Modified" Matsuo and FHLD plots were classified to Green, Yellow Orange and Red Zone respectively. Table 3 shows the actions to be taken at site once the monitoring results reach certain colour zone.

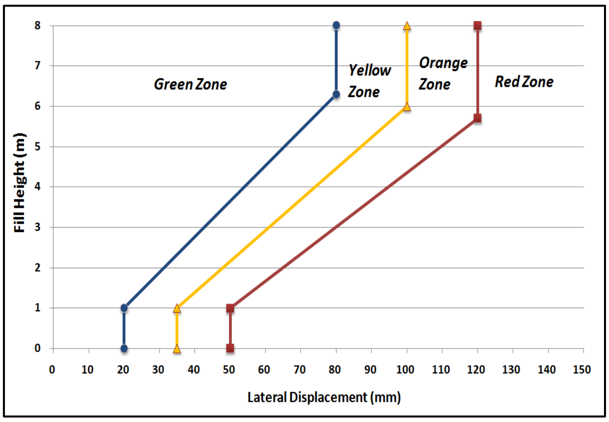


Fig. 15 Fill Height (Fill thickness) versus Lateral Displacement Plot

Zone	Action
Green	Embankment filling can continue.
Yellow	Supervising Engineer to inform design office and embankment filling can continue. For area with PVD, the embankment filling rate reduced to 0.5m/week.
Orange	Supervising Engineer to inform design office immediately. Embankment filling work only can proceed with permission from the design office
Red	Supervising Engineer to stop the embankment filling immediately and inform design office. Design office to review instrumentation data and advise on next course of action.

Table 3 Actions Required During Construction Monitoring

6.2 Control of Embankment Settlement

There are two commonly used methods to interpret the measured settlement. They are :

(A) Hyperbolic Method (Chin, 1970; Tan, 1971 & Tan, 1995)

(B) Asaoka Method (Asaoka, 1978)

6.2.1 *Hyperbolic Method*

This method is usually used to evaluate future settlement based on measured settlement data. This method is based on the assumption that the settlement-time curve is similar to hyperbolic curve and can be represented by the equation shown in Figure 16.

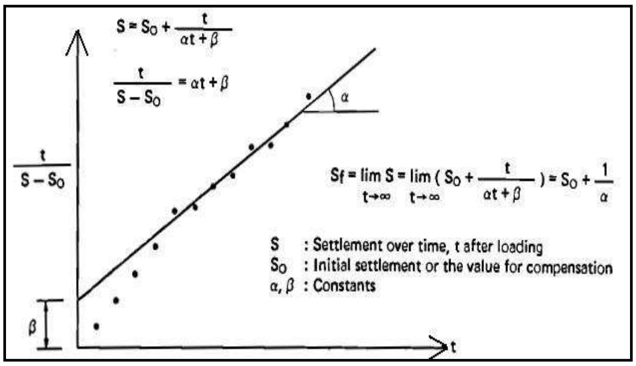
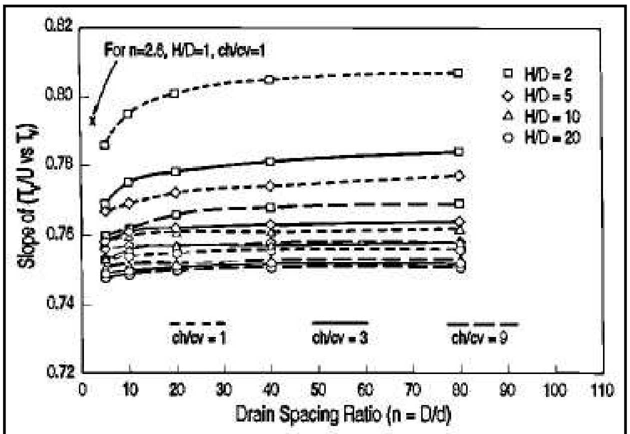


Fig. 16 Hyperbolic Method to Predict Future Settlement.



H = clay drainage depth ;
D = drain influence diameter ;
d = drain diameter ;
T_v = time factor for vertical consolidation
U = degree of consolidation of combined vertical and radial drainage.

Fig. 17 Plot of α_i (from Tan et.al. 1996)

From the figure, we can predict the long term settlement with time. However the final settlement estimated through this method by using the measured data of early period after loading may be on the high side.

Tan (1992) proposes the use of Hyperbolic method for field estimation of total primary settlement of subsoil treated with vertical drains and surcharge. The procedure for use of the method involves four simple steps as outlined below :

Step 1 : Plot the the field settlement data as (t/ρ) vs t, where t is the time and ρ is the settlement from the beginning of surcharge load application. This method is not applicable if the construction time of the embankment with surcharge is more than the time to achieve 60% consolidation.

Step 2 : From the plot, identify the first linear segment immediately after the initially concave downward or humped segment of the curve, and measure its slope, A_i (these corresponds to data between the 60% and 90% settlement points).

Step 3 : From the estimated relevant soil and drain parameters (n, H_D/D and c_h/c_v), determine by interpolation of Figure 17, the applicable theoretical value of the initial linear slope, α_i.

Step 4 : Calculate the slope of the lines from the origin intercepting the 60% and 90% settlement points by Equations (a) and (b). Draw the three lines and obtained the interception points. The total primary settlement can be estimated as either (α_i / S), (ρ_{60} / 0.6) or (ρ_{90} / 0.9). All three estimates should be close as a verification to the correctness of the prediction

$$S_{60} = S_i \frac{\alpha_{60}}{\alpha_i} = (1/0.6) \frac{S_i}{\alpha_i} \qquad \text{(Eqn (a))}$$

$$S_{90} = S_i \frac{\alpha_{90}}{\alpha_i} = (1/0.9) \frac{S_i}{\alpha_i} \qquad \text{(Eqn (b))}$$

Figure 18 shows a typical example of hyperbolic method.

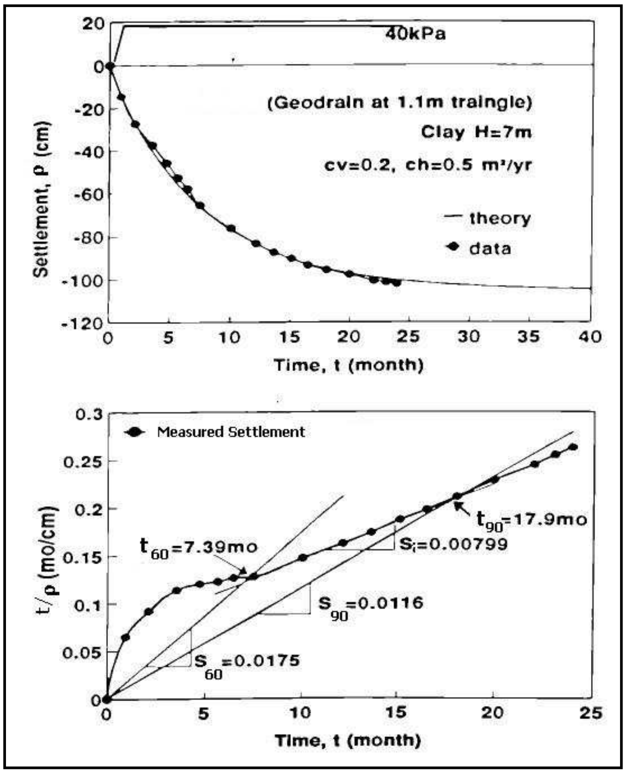


Fig. 18 Example of Hyperbolic Plot Method (adapted from Tan et. al., 1996)

6.2.2 Asaoka's Method (1978)

A graphical approach to estimate final total primary consolidation settlement and settlement rates from settlement data obtained during a certain time period has be proposed by Asaoka (1978). The steps in the graphical procedure are as follows :

Step 1 : The measured time-settlement curve is plotted to an arithmetic scale and is divided into equal time intervals, Δt. Δt can be about 7 to 60 days depending on the available information. The settlements ρ_1, ρ_2, ρ_3, ….. corresponding to the time t_1, t_2, t_3, …. are tabulated as shown Figure 19(a).

Step 2 : The settlement values ρ_1, ρ_2, ρ_3, ….. are plotted as points (ρ_{i-1}, ρ_i) in a coordinate system with axes ρ_{i-1} and, ρ_i, as shown in Figure 19(b). The 45° line $\rho_i = \rho_{i-1}$ shall also be drawn.

Step 3 : A straight line (*I*) is fitted through the points. The point where this line intersects the 45° line gives the final consolidation settlement, ρ_C. The slope β_1 is related to the coefficient of consolidation, c_v and can be used to calculate the rate of settlement as follows :

$$c_v = -\frac{5}{12} h^2 \frac{\ln \beta_1}{\Delta t}$$

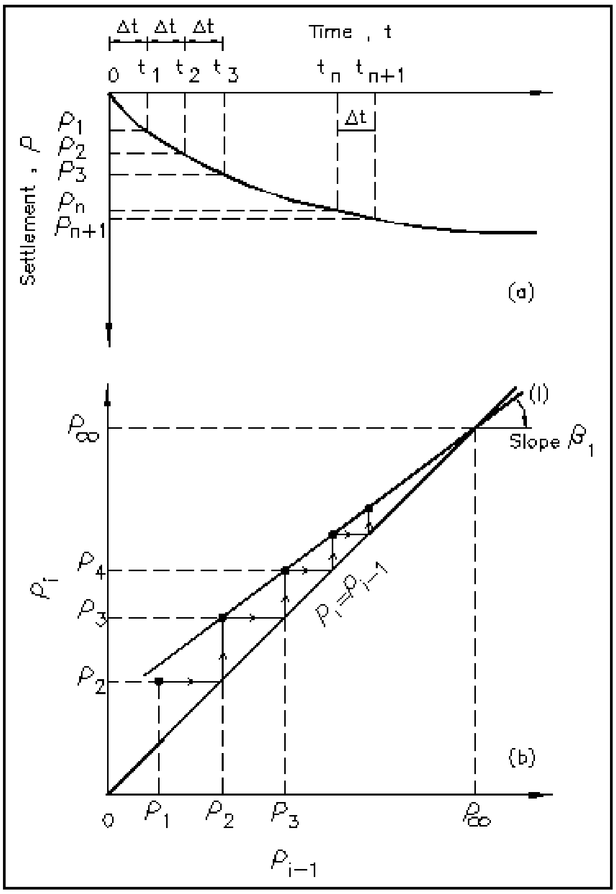


Fig. 19 Graphical Method of Asaoka

The graphical method above is limited to a single layer with one-way or two-way drainage.

6 CONCLUSION

The basic requirements for a successful construction of embankment over very soft compressible alluvial deposits are summarized below:

- Awareness of the project requirements in terms of serviceability criteria (deformation tolerances, bearing capacity, etc.), costs (construction cost and maintenance cost), site constraint and time (construction time, service period).
- Knowledge on the site and subsoil conditions through proper desk study, gathering of geological information and well planned and supervised subsurface investigation and laboratory testing to acquire the necessary reliable parameters for geotechnical designs.
- Proper geotechnical design to address both stability of the embankment and control of deformation.
- Full time proper supervision of the construction works by qualified personnel / engineer.
- Careful and proper monitoring on the performance of the embankment during and after construction through instrumentation scheme.

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