UNIVERSITY OF BIRMINGHAM

# Highway Embankment Design in Bangladesh

**Thesis submitted for the fulfilment of Master of Philosophy in Civil Engineering**

Shyamal Kumar Bhattacharyya
B. Sc. (CE) M. Eng. (WRE)

**School of Civil Engineering**

**College of Engineering and Physical Science**

**The University of Birmingham**

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# Abstract

At present there is no systematic approach for the design of highway embankments in Bangladesh. To address this issue, this thesis presents a methodology which can be used for the design of embankments in countries with similar conditions to those experienced in Bangladesh. Such conditions include: the large seasonal variation in flood water levels, which can reach up to 5 m in height; the soft soils on and from the embankments are mainly constructed, and; the prevailing socio- economic climate.

The large seasonal variation in flood water levels coupled with the underlying soft (clay) soils results in mainly two types of embankment failure. These are broadly associated with excessive amounts and slow rates of embankment foundation settlement and embankment stability. The latter includes slope failure and foundation bearing capacity failure. The chosen methodology to prevent these types of failure took into account the prevailing socio-economic situation. That is the methods selected had to be sustainable, capable of being implemented and economically viable.

Design methods were developed to keep the embankment within the allowable settlement limit and make it stable. These methods include performing settlement and stability analyses respectively. To this end, two techniques described in literature, finite difference settlement analysis and Bishop's method for stability analysis implemented in commercially available software (Settle3D and OASYS SLOPE respectively), were selected from a number of options studied.

In addition to the two methods described above, two other methods were identified where the commercial software is not available. These are less accurate and may not be used under certain conditions are easily implemented in a spreadsheet. For settlement analysis Standard method and to perform stability calculations Low's method were chosen.

For the former and Low's method and developed stability chart were proposed for stability analyses.

Under some conditions the developed design processes may identify that some form of remediation is required. Where it is found necessary to increase the rate of settlement of the embankment and foundation beyond that possible naturally prefabricated vertical drain (PVD) and the addition of a surcharge with a horizontal drainage blanket were identified as

suitable options. Additional methods to improve stability include the addition of geo-fabrics, stage construction, the use of geo-cells and pressure berms.

The top of the embankment, or subgrade, acts as a platform for the pavement and whose properties are the input for pavement design. To assist the pavement design process therefore, a procedure has also been developed in the research to specify appropriate subgrade properties. These included the use of appropriate materials and compacted by suitable amounts so that the fill does not undergo excessive settlement during the life of the pavement. Suitable amounts of compactive effort, layer thickness and construction moisture content were included as part of the specification with appropriate values based on previous research findings.

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# *Dedication*

*The work is dedicated to my parents*

***Namita Bhattacharyya***

***and***

***Chandra Mohon Bhattacharyya***

*Who always help, encourage, think and pray for me*

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# Chapter 1 Introduction

## 1.1 Background

Bangladesh is prone to intensive seasonal rainfall which results in flooding making its road network particularly vulnerable to damage and failure. With meagre resources Bangladesh finds it difficult to maintain its road network under such conditions, let alone carry out necessary improvements. This situation may worsen should the impacts of predicted climate change occur.

To help address these issues, this MPhil research project is part of a larger study which seeks to develop a new road and embankment design process for developing countries which will, when implemented, facilitate the design and construction of new road infrastructure to cope with climatic and physical conditions which are similar to those experienced in Bangladesh.

The focus of this particular MPhil research project is to develop a suitable methodology for the design of embankments which carry the road infrastructure in many parts of Bangladesh. A prime objective of the embankments is to keep the roads they support above the maximum likely flood level (HFL) so that they are not washed away. As a result the height of existing highway embankments in some areas in Bangladesh is more than five metres. Currently however, there is no standard design specification for highway embankments in Bangladesh. Many of the embankments are built on organic soils, in particular peat and black cotton, which have low hydraulic conductivity and high compressibility and have the potential to settle, via consolidation, by large amounts. Furthermore, as the embankments are both high and in many cases poorly compacted, many suffer slope and bearing capacity failure in the wet season as well as heave and consolidation settlement resulting from seasonal water table variation.

## 1.2 Aim and Objectives of the Study

As mentioned above, the aim of this study is to develop a method for the design of road embankment which is sustainable for climate, soil and socio-economic condition found in Bangladesh. To meet this aim, the major objectives of this study are as follows.

1. Research the literature to investigate existing methodologies which are used for conditions similar to those of the investigation
2. Review the existing embankment design process in Bangladesh
3. Identify possible modes of embankment failure
4. Identify the major factors (i.e. input parameters) which need to be considered in the design process
5. Identify the worst case in relation to the seasonal water level variation in embankment design
6. Develop models of embankment stability
7. Build a conceptual model for the rational design of embankments
8. Test the usefulness and accuracy of the model using data obtained from Bangladesh
9. Investigate suitable generic design and construction techniques which can be used to improve embankment stability and keep settlement within tolerable limits.

## 1.3 Usefulness of the Study

It is anticipated that the developed methodology will influence the design of future highway embankments in Bangladesh. In particular, it is hoped that the methodology will be appropriate for use in other countries facing similar problems in terms of climate, hydrogeology and economy.

## 1.4 Novelty of the Research

The innovation of the research concerns the development of a design methodology, with a verified analytical process and failure analysis, for highway embankments in climatic conditions found in countries such as Bangladesh.

The originality in the work lies in the development and application of the process for the first time in Bangladesh.

## 1.5 Layout of the Thesis

The work carried out to achieve the above objectives is described in the following chapters of this thesis:

**Chapter 1** **Introduction**: Discusses the background of the research and describes its aims and objectives.

**Chapter 2** **Literature Review:** Reviews previous related research on the methods of embankment design, modes of failure and their remedial measures, input parameters, and loadings.

**Chapter 3** **Methodology:** Describes the methodology which has been used to carry out the research.

**Chapter 4** **Settlement Analysis:** Describes in detail a number of different methods which can be used to predict embankment settlements due to various loading conditions. The methods are compared and suitable ones for the task in hand are suggested.

**Chapter 5** **Methods of Accelerating Consolidation:** This Chapter describes various methods to accelerate consolidation settlements with a view to recommending those which may be particularly appropriate to conditions found in Bangladesh.

**Chapter 6** **Settlement Modelling:** In this Chapter the behaviour of foundation settlement under various types of loading and settlement acceleration methods are compared and contrasted with a view to selecting an appropriate approach for the task in hand. Using the selected technique a sensitivity analysis is carried out to investigate input parameters which are particularly important to determine settlements accurately.

**Chapter 7** **Stability Analysis:** In this chapter different methods, identified from the literature, which can be used to investigate embankment stability are analysed. Suitable ones for the research project described here are identified.

**Chapter 8** **Stability Modelling:** In this Chapter important factors affecting embankment stability are investigated using the technique identified in the previous chapter. A probabilistic approach to stability analysis, to take into the variability of soil properties, is also introduced.

**Chapter 9** **Subgrade Requirements:** The chapter describes the process used to determine the design formation level (i.e. height) of an embankment subject to flooding. Additionally, the important parameters used to design a suitable surface to support the overlying highway pavement are discussed.

**Chapter 10 Proposed Design Method:** A step by step procedure, incorporating the methods identified in Chapter 4, 5, 6, 7, 8 and 9 for design of a highway embankment is described. A number of examples of the design process are illustrated using data obtained from Bangladesh.

**Chapter 11 Discussion:** This chapter discusses the developed design process in relation to the research objectives.

**Chapter 12 Conclusions and Recommendations:** The final chapter of the thesis draws conclusions from the study and presents recommendations for further work.

# Chapter 2 Literature Review

## 2.1 Introduction

The purpose of this study is to develop a generic methodology for the design of embankments to support roads in Bangladesh, where most embankments are in areas prone to long periods of seasonal flooding since the embankments are often situated in the flood plain and are therefore constructed from locally available materials such as alluvial silts and sand, with varying organic content, or organic clay soils.

The proposed general design will cover all aspects of embankment and their foundations. It will address the issues relating to the stability of the embankment itself, both during and after construction the ability of the existing soils to carry the weight of the embankment together with traffic applied loads (i.e. bearing capacity) and the settlement of foundation soil and embankment materials itself.

To this end, this chapter identifies and describes the probable modes of embankment failure and associated design considerations. Methods described in the literature which may be used to design an embankment taking into account these considerations are identified. Suitable ones are compared and contrasted in subsequent chapters with a view to selecting the most appropriate ones for the task in hand.

The chapter also describes various techniques which may be implemented to reduce the likelihood of embankment failures.

## 2.2 Modes of Embankment Failures

The Failure of a highway embankment is normally associated with two criteria: structural stability and plastic settlement. There are four modes by which the road embankment can be unstable. These are: rotational (slope) failure and sliding failure of the embankment, bearing capacity failure and plastic failure (squeezing out of soil) of the foundation soil (Kanairaj, 1988). Settlement takes place both in the foundation used to support the embankment and the embankment itself (Hsi and Martin, 2005). It may occur during the construction of the embankment and also for some time afterwards.

In practice most of the road embankments in Bangladesh fail due to the settlement of embankment materials. This is primarily due to improper compaction and the use of unsuitable materials from the adjacent land (BRTC, 2005). In addition, some roads constructed on soft soil are not built with subsoil drainage or pre-compression to increase the rate of consolidation and as a result settlement occurs in both the foundation and within the embankment. Embankments which have been constructed with proper subsoil drainage, suitable materials and compaction are performing well. The Gopalganj bypass of Dhaka-Khulna road built under the Southwest Road Network Development Project is an example of this kind where prefabricated vertical drains were used for subsoil drainage and dredged sand was used as fill. The embankment was constructed on nine metres of soft plastic and compressible clay (JOC-NA-BCL, 2000).

From observation, the main modes of stability failure in Bangladesh are found as rotational or slope stability failure. Siddique et al. (2005) found that slope and bearing capacity failure had occurred in a section of the Dhaka-Khulna national highway when it was under construction. Most of the road embankment is constructed across a river or canal where the designed slope was not maintained. In some roads fill materials have been collected from land adjacent to the road to be constructed without considering slope stability issues. Often the materials collected from the toe of the embankment so that the effective height of the embankment was increased resulting in instability in the slope (see photographs in Appendix -IX).

## 2.3 Settlement

The stress on foundation soil increases when the embankment compressing the foundation soil layers through elastic and consolidation settlement. It is usual to classify the resulting settlement as comprising of elastic and a consolidation element. Consolidation is further considered to take places in two phases termed primary and secondary consolidation.

In addition to foundation settlement the embankment materials also compresses due to the effect of the overlying fills and traffic.

### 2.3.1. Elastic Settlement

Elastic settlement starts immediately after the application of load and is associated with the shearing of materials at constant volume without any change in the water content of the foundation soil. The magnitude of the settlement depends on the types of underlying materials in addition to stress on it. The important soil parameters for the calculation of the elastic settlement are the elastic modulus and Poisson's ratio of the foundation soil (Das, 2004; Craig 2005; Bergado et al., 1990). Various methods which may be used to determine elastic settlement are described more fully in Chapter 4.

The value of undrained Poisson's ratio is normally taken as 0.49. The relationships between drained ($E'$) and undrained modulus for soils and the value of drained Poison's ratio are given in Table 2.1.

Table 2.1 Relation between drained and undrained modulus (Bergado et al., 1990)

| **Soil Type** | **Relation between E′ and $E_u$** | **Drained Poisson's Ratio (υ′)** |
|---|---|---|
| Weathered Clay | $E' = 0.36E_u$ | - |
| Very soft clay | $E' = 0.15E_u$ | - |
| Soft clay | $E' = 0.26E_u$ | 0.35 – 0.45 |
| Medium clay | $E' = 0.57E_u$ | - |
| Stiff clay | - | 0.30 – 0.35 |

### 2.3.2. Primary Consolidation Settlement

Primary consolidation settlement is defined as the compression due to the expulsion of water from the pore spaces under the application of load. Primary consolidation settlement is considered to start when the soil has settled elastically and continues until the complete dissipation of excess pore water pressure. When a soil is loaded such as with an embankment, the increase in load is carried by an increase in pore water pressure. This creates a pressure differential, or excess in pore pressure, between the soil affected by the load and the residual ground water pressure. The excess pressure dissipates over time as water flows out from under the loaded area.

Consolidation is time related and its extent in time and magnitude depends on length of the drainage paths in the foundation, the soil type and the stress from the embankment. With time excess pore water pressure generated under the application of the load dissipates. Soil particle needs a certain stable state of compaction after dissipating excess pore water pressure completely. This process is known as primary consolidation (Craig, 2005; Das, 2004).

Primary consolidation settlement can be calculated from Terzaghi's one-dimensional consolidation theory (Terzaghi, 1925; cited in Craig, 2005 and Das, 2002) assuming the compression volume is equal to the change in volume in voids due to expulsion of water. The total primary settlement depends on the compression characteristics and depth of drainage layers of foundation soil. Compression characteristics such as void ratio, compression index and the over consolidation ratio (OCR) are the main parameters for total primary settlement (Bergado et al., 1990)

As the primary consolidation continues with time, the rate of settlement can be found from Terzaghi's one-dimensional consolidation theory (Craig, 2005; Das, 2002) for saturated soils if the consolidation characteristics of soils are known. The consolidation parameters are characterised by the coefficient of consolidation ($C_v$) and coefficient of volume compressibility ($m_v$).

Terzaghi's one-dimensional consolidation theory considers only the instant application of load on the foundation was and has been modified by Olson (1977) to consider ramp loading. In Olson's model, the load on foundation increases with time as the embankment construction proceeds.

### 2.3.3. Secondary Consolidation Settlement

Secondary consolidation settlement is considered to start after the complete dissipation of excess pore water pressure. Under the action of constant prolonged loading soil particles rearrange and reorientation resulting in further compression which is known as secondary consolidation or creep settlement. Organic soils are prone such behaviour (Das, 2004; Craig, 2005; Barnes, 1995).

In reality it takes an infinite time to complete 100% primary consolidations so the secondary settlement phase is assumed to start after completion of 90% primary settlement for practical embankment design (Bergado, et al., 1990).

The coefficient of secondary compression ($C_\alpha$) is one of the most important soil parameters required to calculate secondary compression. If a surcharge is applied to reduce secondary settlement, an additional parameter known as post surcharge secant secondary compression index ($C_{\alpha}''$) is also required. The following paragraphs describe the coefficients and their method of determination.

#### 2.3.3.1. Coefficient of Secondary Compression ($C_{\alpha}$)

$C_{\alpha}$ is generally found from the result of oedometer tests (Craig, 2005). This is the slope of downward part of e-logt curves which starts after the end of primary consolidation. In case of peat soil it is very difficult to find out the end of primary consolidation from an e-logt curve because of the nonexistence of a yield point or inflection point. This happens due to following reasons (Ewers and Allman, 2000).

- The primary consolidation amount decreases with increase of $\Delta\sigma'_{v}/\sigma'_{v}$; as a result the transition from primary to secondary consolidation is not well defined.
- The slope of secondary compression curve increases after the stress equal to pre-consolidation pressure ($\sigma'_{c}$).

Peat soils exhibit significant secondary compression for the following reasons (Mesri et al., 1997).

- They have a high void ratio and  high natural water content
- They have the highest value of $C_{\alpha}/C_{c}$ ratio.
- Primary consolidation occurs relatively quickly due to initially high values of permeability as a result high value of compression index ($C_{c}$). As a result secondary compression stars earlier than as in other soil.

Various values of $C_{\alpha}/C_{c}$ and coefficient permeability ($k_{vo}$) have been suggested for different types of inorganic clay and peat soils as follows (Mesri et al., 1997):

Table 2.2 Value of $C_\alpha/C_c$ (after Mesri et al., 1997)

| Types of soil | Natural water content (w %) | Coefficient permeability ($k_{vo}$), m/s | $C_\alpha/C_c$ | Proposed by |
|---|---|---|---|---|
| Peat | 520 | | 0.06 – 0.10 | Lewis(1956)[1] |
| | 400-750 | $10^{-5}$ | 0.075 – 0.085 | Weber (1969)[1] |
| Fibrous peat | 850 | $4\times10^{-6}$ | 0.06 - 0.10 | Hanrahan (1954)[1] |
| | 605 – 1290 | $10^{-6}$ | 0.052 – 0.072 | Samson and LaRochel (1972)[1] |
| | 613 – 886 | $10^{-6}$-$10^{-5}$ | 0.06 – 0.085 | Berry and Vickers (1975)[1] |
| | 610 – 850 | $6\times10^{-6}$ | 0.052 | Mesri (1997) |
| Amorphous to fibrous peat | 600 | $6\times10^{-6}$ | 0.042 – 0.083 | Dhowian and Edil (1981)[1] |
| Soft alluvial clay | | | 0.04 | Ewers et al. (2000), Mesri et al. (1997) |

[1] (cited in Mesri, et al., 1997)

#### 2.3.3.2. Post Surcharge Secant Secondary Compression Index ($C''_\alpha$)

If the secondary settlement takes longer than required, a surcharge can be applied to reduce secondary settlement. The coefficient of secondary consolidation and is known as post surcharge secant secondary compression index ($C''_\alpha$) is used in the calculation of secondary settlement when surcharge is applied. Mesri et al. (1997) gives a graphical presentation for $C''_\alpha / C_\alpha$ values for Middleton peat as a function of the time, $t_l$, required to restart the secondary compression after removal of surcharge by the time required for primary rebound $t_{pr}$ for various effective surcharge ratios ($R'_s$). He also relates the values of $t_l/t_{pr}$ with the effective surcharge ratio. The effective surcharge ratio can be defined as (Mesri et al., 1997):

$$R'_s = \left( \sigma'_{vsc} / \sigma'_{vst} \right) - 1 \qquad 2.1$$

Where, $\sigma'_{vsc}$ = maximum effective vertical stress attain before removal of surcharge, $\sigma'_{vst}$ = structure stress or effective stress after removal of surcharge.

Relationship between the post surcharge secant secondary compression index ($C_{\alpha}^{''}$) and effective surcharge ratio for Middleton peat are shown in Figure 2.1 (Mesri et al., 1997).

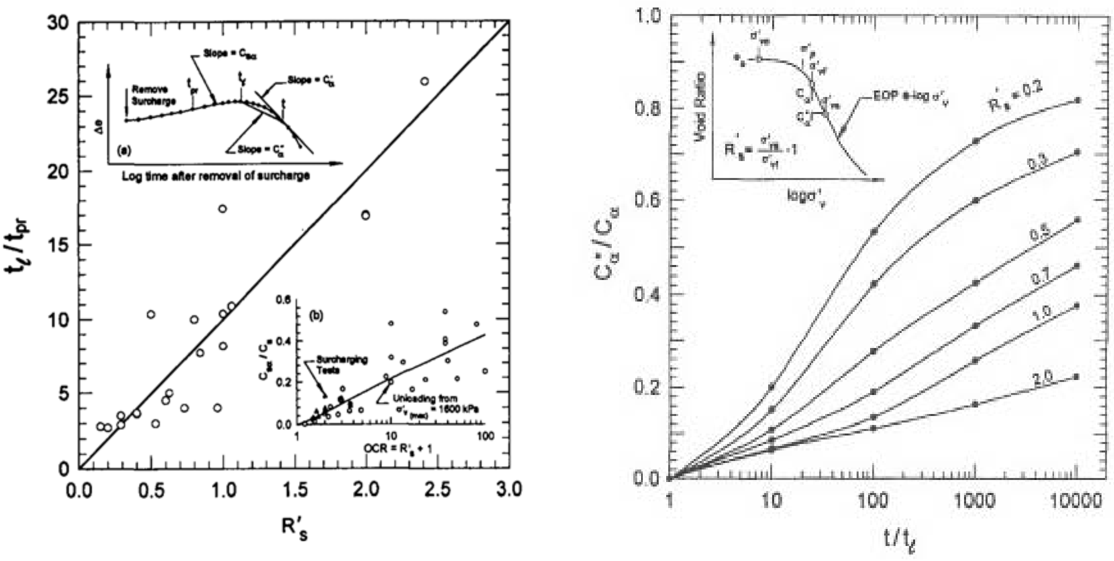



Figure 2.1 Post surcharge secant secondary compression index ($C_{\alpha}^{''}$) and effective surcharge ratio for Middleton peat (after Mesri et al., 1997)

### 2.3.4. Compression within the Embankment

Due to traffic loads and the self weight of embankment long-term settlement also occurs within the embankment itself. This can be considered to arise from a) the deformation of soil particles b) rearrangement of soil particles and c) dissipation of air and water from the inter-particles void spaces. If the embankment is constructed with non-compressible materials and each layer is well compacted, the settlement within the embankment in the long term can be kept within 0.2% of the height of the embankment (Das 2002, Hsi and Martin 2005, Craig 2005).

Different types of foundation settlement (elastic and consolidation) including their methods of prediction and design of remedial measures for safe road construction are described in Chapter 4 and 5.

## 2.4 Tolerable Settlement

Hsi and Martin (2005) suggested the following tolerable limits of residual and differential settlement from their experience of the construction of the Yelgun to Chinderah freeway in Australia.

a) Residual settlement: Maximum allowable settlement of between 100- 160 mm over 40 years.
b) Differential settlement: Maximum differential settlement in lateral (transverse) direction 1% and the longitudinal direction 0.3% for 40 a year period.

Long and O'Riordan (2001) suggested maximum allowable residual settlement of 350 mm and differential settlement should not exceed 50 mm after the operation of 25 years design life.

No guidelines were found regarding residual settlement in respect of roads in Bangladesh.

## 2.5 Accelerating Settlement

The rate of settlement can be accelerated by various means including applying pre-compression and installing vertical drains in the foundation.

### 2.5.1. Pre-compression

There are two main methods of pre-compression (Das, 2004; CIRIA C573, 2002).

- **Preloading:** Pre-loading is the placement of load for a period of time to achieve the desired compression and removing it before actual construction.
- **Surcharging:** Surcharging is the application of load on foundation in addition to the actual structural load. If the loading requirement is greater than the structure load, a surcharge can be applied.

The application of pre-compression minimises immediate and primary settlement and reduces the rate of secondary settlement. After the removal of the surcharge it is necessary to control any rebound or swelling. To control rebound, CIRIA suggests (CIRIA C573, 2002) the final load should not be less than one-third of the surcharge load. The time for removal of surcharge load is determined in such a way so that the degree of consolidation ($U$) is at least

$\sigma_{sc}/(\sigma_{sc}+\sigma_{st})$ or typically 75% (CIRIA C573, 2002) where $\sigma_{sc}$ and $\sigma_{st}$ represent the surcharge and structure load respectively.

### 2.5.2. Vertical Drains

Vertical drains can be used to accelerate the consolidation process so that the anticipated maximum consolidation settlement can be achieved before starting the actual construction work. They achieve this by increasing the hydraulic conductivity of the subsoil in the vertical direction and accelerating consolidation by radial drainage. Two types of vertical drains are normally used (1) sand drains which are constructed by making holes through soft soil layers and backfilling them by sand and (2) prefabricated vertical drains (PVD) which are the combination of a core enclosed by a geo-textile filter fabric (Indraratna et al., 2005; Hansbo, 1997).

### 2.5.3. Performance of PVD

The performance of prefabricated vertical drain (PVD) is regarded to be superior to sand drains and costs less (Kyfor et al., 1988). Kyfor et al. (1988) found that the vertical drains reduced time for 80% consolidation almost to one-third to that of without drains. They also investigated the performance of prefabricated vertical drains (PVD) constructed under a highway embankment and found that a 1.52 m spacing of PVD achieved 50-75% consolidation during the construction period.

The use of performance of prefabricated vertical drains in highway construction in New Orleans is also appreciable. Sarkar and Castelli (1988) report that there are a 1.52 m spacing of PVD was used and they found that 80% consolidation occurred against only 28% where PVD was not used. They used a reinforcement mat on the soft soil to make a working condition for the installation of PVD which was successfully used in that highway project.

The inclusion of horizontal drain on the top of the vertical drain can be used to accelerate the consolidation process. Typically, the horizontal drain is a layer of sand at the surface of the foundation although a geo-textile filter fabric can be used to improve the efficiency of drainage and to get a good working platform.

To construct a high embankment on soft soil, it is necessary to improve the bearing capacity of foundation by stage loading which improves shear strength by consolidation of subsoil. It takes a lot of time to achieve the desired degree of consolidation for improving the bearing capacity. Prefabricated vertical drains accelerate consolidation as well as improve the shear strength which also improves the bearing capacity of foundation. This improvement in bearing capacity will help in faster embankment construction (Sinha et al., 2009).

#### 2.5.3.1. Hydraulic Conductivity Parameters

The hydraulic conductivity parameters needed for the simulation of consolidation with vertical drains are: coefficient of consolidation in vertical ($C_v$) and horizontal ($C_h$) direction and permeability in vertical ($k_v$) and horizontal ($k_h$) direction respectively.

##### *2.5.3.1.1 $C_h/C_v$ ratio*

The value of $C_h$ plays a dominant role as the pore water drains horizontally when vertical drains are installed (Kyfor et al., 1988). As the coefficient of consolidation in the horizontal direction ($C_h$) is difficult to estimate, Kyfor et al. (1988) assumed the value of $C_h$ equal to three times of $C_v$. It was originally proposed by Carlo in 1984 (Carlo, 1984; cited in Kyfor et al., 1988). He also back calculated the $C_h$ from actual settlements and found it was approximately equal to three times $C_v$. However, Sarkar and Castelli (1988) carried out the primary analysis by assuming the value of $C_h$ as two times to that of $C_v$, but after measuring settlement and pore water pressure they found the $C_h/C_v$ ratio as 1.3. The value of $C_v$ can be found form laboratory oedometer tests. To make a reasonable prediction it is necessary to determine the value of $C_h$ directly from piezocone with pore pressure dissipation test (CPTU) if it is available (Sills and Hird, 2005; Chu et al., 2002). The value of $k_v$ and $k_h$ can be calculated from $C_v$, $C_h$ and $m_v$. The value of $m_v$ is a deformation parameter known as the coefficient of volume compressibility.

##### *2.5.3.1.2 Difference between field and laboratory $C_v$*

Row (1968; cited in Barnes, 1995) found errors in the values of $C_v$ based on laboratory oedometer test when compared to that of field rate of settlement. He suggested that the deviations take place due to the variation of fabric arrangements between field and small sample of laboratory oedometer tests. Tan et al. (2003) found that the in-situ coefficient of consolidation for normally consolidated clay (OCR = 1.0) was approximately ten times to that of laboratory oedometer test following the Casagrande method. Bergado et al. (1990) carried out experiments on Bangkok clay and found the value of the field $C_v$ was 26 times that of the laboratory determined $C_v$. The value of $C_v$ is also dependent on the methods of determination such as Casagrande, Taylor, velocity and log ($H^2$/t) – $U$ method (Cortellazzo, 2002). Thus it can be seen that without accurate values of $C_v$ the predicted settlement will bear no means which ever model is used for the prediction of settlements. To avoid these errors, Bishop and Al-Dhair (1969; cited in Barnes, 1995) alternatively proposed the following relationship to estimate $C_v$.

$$C_v = \frac{k_{v(insitu)}}{m_{v(lab)}\gamma_w} \qquad 2.2$$

### 2.5.4. Design of Vertical Drain

When designing PVDs it is necessary to determine a suitable spacing of the drains. A number of methods can be used to this end. The Barron-Kjellman formula can be used but does not taken into account well resistance and smear effects (see Section 2.5.4.3 and 2.5.4.4) (Kyfor et al., 1988). A more accurate method was developed by Hansbo (1997) which considers both well resistance and smear zone effects (Indraratna et al., 2000). Yeung (1997) developed several curves for designing prefabricated vertical drains based on the Hansbo theory. These methods are compared and contrasted in detail in chapter 5.

#### 2.5.4.1. Size of Vertical Drain

Generally prefabricated vertical drains available in the market are of rectangular in cross section. To accommodate them in the design process the rectangular size of PVD is converted into an equivalent circular diameter because of the conventional theory of radial consolidation. To this end equation for calculating the equivalent drain diameter ($d_w$) of vertical drains have been proposed by Hansbo (1997) as follows:

$$d_w = 2(a+b)/\pi \qquad 2.3$$

Where, $a$ = width of prefabricated core, $b$ = thickness of the core.

Long and Covo (1994) suggested the following formula for calculating equivalent drain diameter.

$$d_w = 0.5a + 0.7b \qquad 2.4$$

Of these two equations, that suggested by Hansbo (1979) is widely used as it has proved to be accurate in design (Indraratna et al., 2005).

The usual available sizes of PVD are given in the flowing Table 2.3. The usual size of vertical sand drains are 300, 450 and 600 mm in diameter to suit available installation equipment (Yeung, 1997).

Table 2.3 Materials and dimensions of PVD (after Holtz et al., 1991; cited in CIRIA C573)

| **Type of drain** | **Dimensions** | | **Material** | |
|---|---|---|---|---|
| | **Width a (mm)** | **Thickness b (mm)** | **Filter** | **Core** |
| Kjellman | 100 | 3.0 | Cardboard | Cardboard |
| Alidrain | 100 | 6.1 | Geo-textile | Polyethylene |
| Amerdrain | 92 | 10.0 | Geo-textile | Polyethylene |
| Bando | 96 | 2.0 | Paper | PVC |
| Castleboard | 93 | 3.2 | | Polyolefin |
| Colbond | 300 | 4.0 | Geo-textile | Polyester |
| Desol | 95 | 2.0 | | Polyolefin |
| Geodrain | 98 | 4.0 | Paper | Polyethylene |
| Geodrain | 96 | 3.5 | Geo-textile | Polyethylene |
| Hitek | 100 | 6.0 | Geo-textile | Polyethylene |
| Mebradrain | 95 | 3.2 | Paper | Polypropylene or Polyethylene |
| Mebradrain | 95 | 3.4 | Geo-textile | Polypropylene or Polyethylene |

### 2.5.4.2. Discharge Capacity

The discharge capacity ($q_w$) of a vertical drain is defined as the volume of water flow through it in a year. Depending on lateral confining earth pressure various discharge capacities of PVD have been proposed as given in Table 2.4 (Indraratna et al., 2005).

Table 2.4 Discharge capacity of PVD (after Indraratna et al., 2005)

| Source | Discharge Capacity ($m^3$/year) | Lateral Stress (kPa) |
|---|---|---|
| Kremer et al. (1982) | 256 | 100 |
| Kremer (1983) | 790 | 15 |
| Jamiolkowski et al. (1983) | 10-15 | 300-500 |
| Rixner et al. (1986) | 100 | - |
| Hansbo (1987) | 50-100 | - |
| Holtz et al. (1989) | 100-150 | 300-500 |
| De Jager and Oostveen (1990) | 315-1580 | 150-300 |

#### 2.5.4.3. Well Resistance

The radial consolidation process is retarded when the discharge capacity ($q_w$) of the vertical drain reaches its maximum capacity and is referred to as well resistance. The well resistance is not only controlled by the maximum discharge capacity of the drain but also on the radial permeability of the soil ($k_h$), discharge length of drain ($l_m$) and probable geometric defects on the drain. Mesri and Lo (1991, cited in Indraratna et al., 2005) proposed a well resistance factor ($R$) by the following equation.

$$R = q_w / k_h \, l_m^2 \qquad 2.5$$

Indraratna et al. (2005) in his field study found that well resistance is negligible for the value of $R$ greater than five and hence proposed the minimum discharge capacity to avoid well resistance by the following equation.

$$q_w = 5 k_h l_m^2 \qquad 2.6$$

#### 2.5.4.4. Smear Effect

The smear zone is the disturbed area due to installation of a mandrel and is a function of mandrel thickness. The exact determination of the size of the smear zone diameter is difficult and it is often estimated as two times the mandrel area (Hansbo, 1997). On the other hand Chai et al. (2001) proposed the following formula:

$$d_s = 3d_m \tag{2.7}$$

Where $d_s$ = diameter of smear zone, $d_m$ = diameter equivalent to area of mandrel by which PVD are to be installed.

For simplicity in the design calculations, the effect of the smear zone can be neglected but its inclusion proves accuracy in predicted result when compared with the field performance (Indraratna et al., 2000). The vertical drain installation restricts the lateral displacement and minimizes the risk of shear failure. The effect of smear on vertical drain performance in the long term (>400 days) is negligible and hence it acts as perfect drain (Indraratna et al., 1992).

## 2.6 Stability of Embankment

The stability of an embankment depends on the materials used for its construction, ground water, geometry and the properties of embankment and foundation soil on which the embankment is constructed. Since embankments in Bangladesh are usually constructed on soft alluvial soils in the flood plain of the rivers Ganges and Brahmaputra, emphasis here will be given to the construction of embankments in such areas using low quality (alluvial) materials.

### 2.6.1. Stability Analysis

The stability of embankment slopes (rotational failure) and bearing capacity failure are the predominant types of embankment failure in Bangladesh and are focussed on herein.

#### 2.6.1.1. Slope Stability

An embankment slope may fail in the following ways (Craig, 2005):

1. Rotational slip failure: It may be a circular or noncircular path. Normally circular slips are formed in homogeneous, isotropic soil and non-circular slip formed with non-homogeneous soil
2. Translational and compound slips occur if the underlying stratum possesses significantly different properties and strength.

The slope stability of an embankment can be analysed using the following types of analysis.

a) Total stress

b) Effective stress

c) Coupled analysis

Total stress analysis (undrained conditions) is carried out to predict short term stability. When an embankment is constructed on soft clay of low permeability, excess pore water pressures develop due to the sudden application of load. If the initial shear strength of foundation soil is low and the load is applied within a short period, the problem may be considered to be undrained and the undrained shear strength, $C_u$ of foundation soil is of interest. Short term stability analysis is necessary to check the stability during the construction period. In this analysis the shear strength along the potential failure surface can be given by

$$s_u = c + \sigma_f \tan \varphi \qquad 2.8$$

Under saturated condition the cohesive soil is frictionless i.e. $\phi = 0$ and the factor of safety is given by

$$FS = \frac{s_u}{\tau_m} \qquad 2.9$$

The $S_u$ can be determined from unconsolidated-undrained triaxial compression test (UUC) or from a field vane test and $\tau_m$ is the mobilize shear strength or shear strength at failure.

Effective stress analysis (i.e. drained conditions) is usually carried out for long term stability analysis as any excess pore water pressure will dissipate with time after the completion of the construction. The static pore pressure also becomes zero during the slow rate of shearing when the failure occurs. The variation of the factor of safety against collapse with pore water pressure dissipation with time for an embankment during construction is shown in Figure 2.2 (Bishop and Bejerrum, 1960). In this analysis the available shear strength is determined from effective stress parameters and can be given by the following relations.

$$s = c' + \sigma'_f \tan \phi'$$

or $s = c' + (\sigma_f - u)\tan \phi'$ 2.10

Where, $s$ = resistance to shear along the failure surface, $c'$ = cohesion intercept of soil in drained condition, $\sigma_f$ = normal stress on the failure plane, $u$ = pore water pressure, $\varphi'$= angle of shearing resistance with respect to effective stress.

The value of c′ and φ′ can be obtained from laboratory drained triaxial shear strength test or undrained triaxial test with the measurement of pore water pressure.

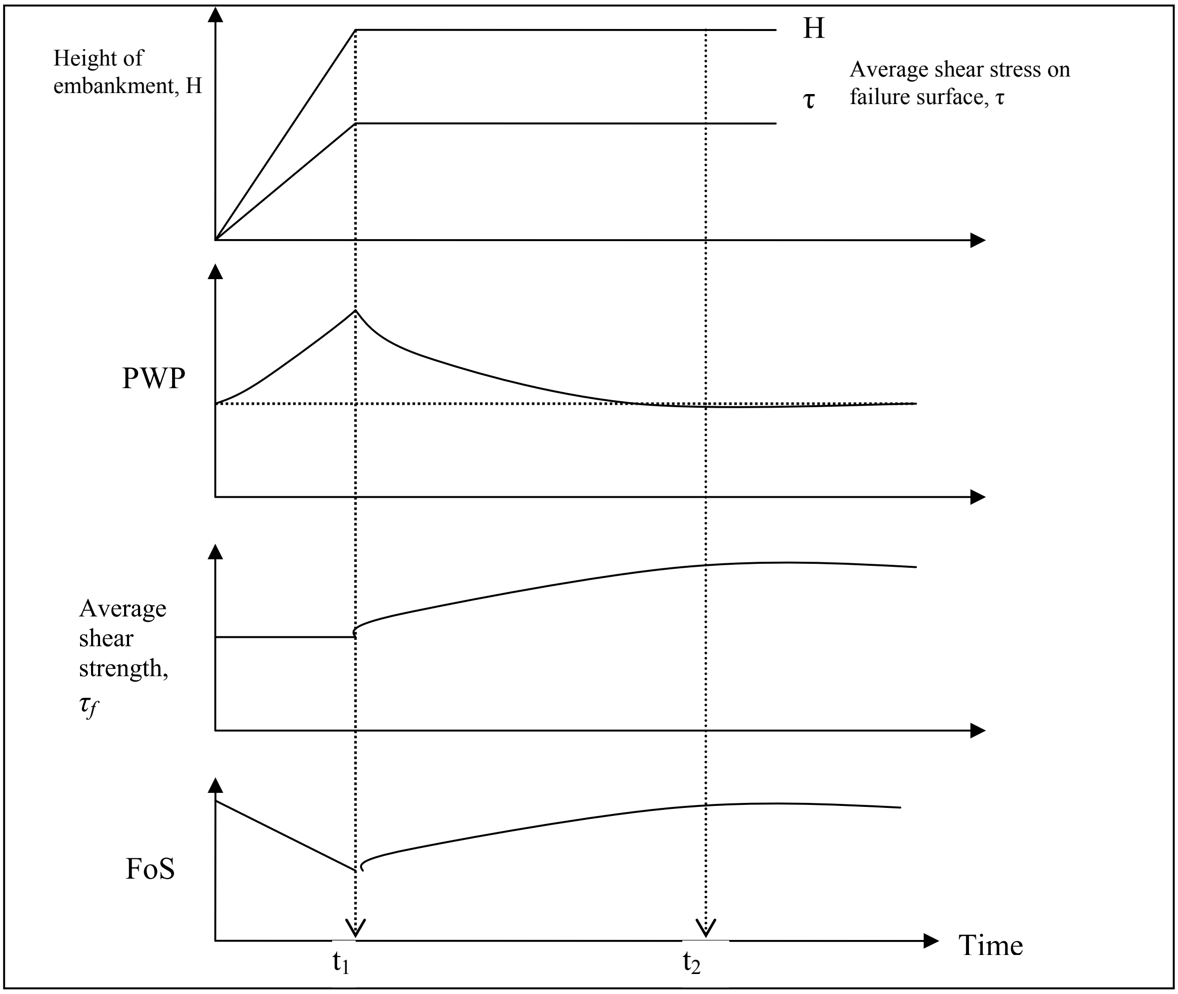



Figure 2.2 Variation of factor of safety with time in respect of embankment construction on soft clay (after Bishop and Bjerrum, 1960; cited in Das, 2004)

Coupled (initially undrained and then drained) analyses are carried out for rapid drawdown cases but in the case of a cohesive soil the rapid drawdown can be considered to be undrained. When the loading generated pore water pressure takes a long time to dissipate, a total stress undrained analysis is appropriate. When the rate of dissipation of pore water is rapid, the situation is considered to be drained and the analysis should be carried out on the basis of effective stress (Berilgen, 2006).

Embankment stability can be analysed on the basis of the rate of dissipation of pore water pressure during lowering of water level and difference of water level on both side of embankment when steady seepage condition is maintained. Drawdown of the water level can occur rapidly or slowly. For the former 1) where pore water in the embankment materials do not get sufficient time to drain out and the later 2) where the internal pore water drains with the lowering of water level (Lane and Griffiths, 2000). The condition (1) is critical and drainage and overall stability depend on the permeability, soil shear strength parameters, unit weight of fill, rate of drawdown, geometry of embankment. Condition (2) does not affect the stability of the embankment.

Considering the above stress conditions, stability can be analysed on the basis of following two broad conditions:

a) Steady seepage: Steady state seepage occurs when an embankment is used as a dam where there is a difference in water level between either sides of the embankment.
b) Rapid drawdown: If the embankment fill material has low hydraulic conductivity, then its stability may be reduced by a sudden drawdown of the water level.

**Methods of Analysis:**

Limit equilibrium and limit state theories are considered in the analysis of stability (Craig, 2005). Limit equilibrium methods consider moment and or force equilibrium for factor of safety calculations. In limit state method partial factors are applied to the shear strength parameters. For example, in a method suggested by Craig (2005) the shear strength parameters $c'$ and $\tan\varphi'$ are divided by a 1.6 and 1.25 respectively. The value of soil parameter $C_u$ is divided by 1.40 and a factor 1.0 is taken for self weight of soil and pore-water pressure.

Among the above two methods limit equilibrium methods are the most commonly used methods for slope stability analysis. The following are considered here:

- Ordinary Method of Slices (Fellenius or Swedish) (Fellenius, 1927; cited in Craig, 2005)
- Bishop's Simplified Method (Bishop, 1955)
- Spencer method (Spencer, 1967)
- Bishop and Morgenstern (1960)
- Janbu Generalised Method (Janbu, 1973; cited in Barnes, 1995)
- Morgenstern and Price (Morgenstern and Price, 1965)
- Low's method (Low, 1989)
- TRL method (ORN 14, 1997).

The methods and their relative merits are described and discussed in Chapter 7 with a view to selecting the most appropriate one for the task in hand.

#### 2.6.1.2. Bearing Capacity

Bearing capacity refers to the value of stress which a foundation can support safely without any sudden catastrophic settlement of the foundation due to shear failure.

The analysis of bearing capacity of soils under an embankment is particularly important where the foundation soil is very weak cohesive soil. The bearing capacity of soil under an embankment is not similar to those of bearing capacity under a footing. The important difference is, the footing is a rigid body and it itself displaced downward with the underneath soil as it is displaced. But in case of embankment the fill materials is displaced downward together with the foundation soil and thus the displacement occurs as a single rigid body. When the foundation soil is very weak, the embankment load pushes the foundation soil laterally and the fill materials also follow this path. This horizontal component of thrust adversely affects the stability of the embankment. Bearing capacity of soft clay foundation under an embankment can be calculated by the equation developed by Radoslaw (1992) with the assumptions of limited thickness of soil layer carrying the embankment load.

**Methods of Analysis**

Two main methods are used to analyse the bearing capacity of soft foundation soils, these are a method of identifying failure mechanisms described by Prandtle (1920) & Hill (1950, cited in Radoslaw, 1992) and via equations given by Jewell (1988). The equations are described in detail in Chapter 7.

### 2.6.2. Methods of Improving Stability

Where the foundation of an embankment is found to be insufficient in terms of strength to support the embankment various methods may be used to strengthen the foundation. A summary of the available techniques is given below and specific techniques which may be applied in Bangladesh are discussed more fully in Chapter 7, with a view to identify those work may be usefully adopted.

#### 2.6.2.1. Use of Reinforcement

Reinforcement under an embankment can be used to reduce vertical and horizontal displacement but in general it is more effective in reducing horizontal displacement. The reduction in displacements obtained is a function of the shear strength and depth of the soft clay layer. Typically, the highest shear strains, and maximum value of shear stress are developed at the centre and shoulder of the embankment. The embankment fill imposes vertical load to the foundation soil equal to its self weight and there is an associated development of outward shear stress. Outward shear stress is the stress acting on soil in a direction from the centre to the toe of the embankment. This shear stress is the cause of the reduction in bearing capacity of the foundation soil. The reinforcement improves the resistance to shear stress and hence improves the stability of the embankment (Sae-Tia et al., 1999).

Three-dimensional geo-synthetic materials have been found to be particularly useful in increasing the bearing capacity of the foundation. In addition they may reduce excessive and differential settlement and lateral deformation (Latha et al., 2006). The properties of a number of such materials have been tested by Latha et al. (2006) in accordance with ASTM D4595 (ASTM, 1996-2009) and the results are given in Table 2.6.

Rowe (1984) developed a method of embankment design on soft soil using reinforcements. In his study he developed some charts from which the reinforcement characteristics can be selected based on the embankment and foundation material properties and the geometry of the embankment.

Kaniraj (1988) proposes a methodology for the design of reinforced embankments on soft soil. In his analysis he considers four failure criteria and suggests geo-reinforcement at the interface of the embankment and foundation soil. Details of the reinforcement suggested are given in Table 2.5. For his analysis the embankment height was 4 m, the angle of internal friction and unit weight of fill respectively were $36^o$ and 17 kN/m$^3$ and the soft foundation depth was 3.7 m.

Table 2.5 Reinforced embankment design details (after Kaniraj, 1988)

| **Foundation soil undrained shear strength, $S_u$ (kPa)** | **Minimum reinforcement force (kN/m)** | **Slope (*1:n*), the value of *n*** |
|---|---|---|
| 9.6 | 90 | 2.6 |
| 10.7 | 82 | 2.3 |
| 11.7 | 74 | 2.0 |

Table 2.6 Properties of geo-grid (after Latha et al., 2006)

| **Properties** | **UX** | **BX** | **NP-1** | **NP-2** |
|---|---|---|---|---|
| Ultimate tensile strength (kN/m) | 40 | 20 | 4.5 | 7.5 |
| Failure strain (%) | 28 | 25 | 10 | 55 |
| Initial modulus (kN/m) | 267 | 183 | 75 | 95 |
| Secant modulus (at 5 % strain) | 200 | 160 | 70 | 70 |
| Secant modulus (at 10% strain) | 95 | 125 | 45 | 50 |
| Aperture size (mm) | 210×16 | 35×35 | 50×50 | 8×7 |

UX = Uniaxial grid, made of polyethylene, BX = Biaxial grid, made of polypropylene, NP -1 and Np-2 = grid made of non-oriented high density polyethylene.

### 2.6.2.2. Stage Construction

Stage construction can be adopted for embankments on soft soil to improve the shear strength of the foundation soil which in turn improves the bearing capacity and overall stability of the embankment (Togrol and Cinicioglu, 1994; Cinicioglu and Togrol, 1991).

After loading, the shear strength of the foundation can increase by an amount which can be predicted from the effective vertical stress after loading, pre-consolidation pressure, rate of shear strength increase under loading. The increased shear strength can be predicted by the following empirical equations (Larsson, 1980; cited in Lechowicz Z, 1996).

$$S_{fu} = K_s . \sigma'_v \qquad 2.11$$

Where, $S_{fu}$ = undrained shear strength after completion of loading, $K_s$ = coefficient of shear strength increase under different loading conditions, $\sigma'_v$ = effective vertical stress after loading, the value of $K_s$ is given by

$$K_s = S(OCR)^m \qquad 2.12$$

Where, $S$ = normalised undrained shear strength = $S_{fu}/\sigma'_p$, $\sigma'_p$ = pre-consolidation pressure, $m$ = slope of relation between $S_{fu}/\sigma'_v$ vs OCR. The values of S and m are given in Table 2.3.

$$OCR = \frac{\sigma'_p}{\sigma'_{vo}} \qquad 2.13$$

Generally, for embankment construction the value of $\sigma'_p$ is same as the value of $\sigma'_{vo}$, so the value of OCR becomes unity.

The value of $K_s$ can be determined from the charts developed by Ladd (1985) and Larsen (1990; cited in Lechowicz Z, 1996). The value of $K_s$ can also be found from effective stress level (ESL) as follows.

$$K_s = S(ESL)^{m_{oc}} \qquad ESL>1 \qquad 2.14$$

$$K_s = S(ESL)^{m_{nc}} \qquad ESL<1 \qquad 2.15$$

where, $ESL = \sigma'_{po} / \sigma'_v$ 2.16

$m_{oc}$ and $m_{nc}$ = slope of $log(\tau_{fu}/\sigma'_v)$ *vs* $log(ESL)$ graph for over consolidated and normally consolidated soil. The parameters $m_{oc}$, $m_{nc}$ and $S$ for organic soil are evaluated from field and laboratory tests by Wolski et al. (1988; cited in Lechowicz, 1996).

Table 2.7 Values of S and m (after Ladd, 1991)

| **Types of soil** | **S** | **m** | **Comments** |
|---|---|---|---|
| Sensitive marine clay | 0.2 with SD = 0.015 | 1.0 | PI < 30% LI > 1 |
| Homogeneous clays of low to moderate sensitivity | S =0.20 + 0.05(PI) or 0.22 | $m = 0.88(1 - C_s/C_c) \pm 0.06SD$<br><br>Or m = 0.8 | PI = (20% - 80%) |
| Northeastern U.S varved clays | 0.16 | 0.75 | |
| Sedimentary deposits of silts and organic soils | 0.25 with SD = 0.05 | $m = 0.88(1 - C_s/C_c) \pm 0.06SD$<br><br>Or m = 0.8 | |

#### 2.6.2.3. Pressure Berms

Where the embankment height is too high to satisfy the stability criterion, pressure berms can be used for stabilisation purposes. The height and width of the pressure berms depend on the density of fill and berm materials, the shear strength of foundation materials and the depth to a firm bottom (Carlsten, 1996). Details procedure for the design of pressure berms are described in Section 7.5.3.

## 2.7 Formation Level

The top level of the subgrade is defined herein as the formation level of the embankment and it is usual for it to be situated a recommended height above high flood level (HFL). The recommended height is called the freeboard and is a function of soil property. The deformation of subgrade is dependent on the moisture content of fill materials under repetitive load conditions which in turn is a function of ground water level. Capillary rise from the ground water table is a function of soil property and compaction effort. Different types of soils show differences in capillary height. Therefore the subgrade height should be selected on the basis of available fill materials in the locality. A comparative study also can be carried out on fill materials from distant locations if the fill height for local material becomes high. For Bangladeshi conditions, the RMSS (1994) study recommended a formation level 1.6 m above the highest flood level.

Elfino and Davidson (1987) carried out investigations on different types of soil in Florida. Table 2.8 shows the variability of resilient modulus with respect to the height of formation level above water table. The seasonal variation of water table is the cause of variation of subgrade modulus.

Table 2.8 Subgrade resilient modulus at various heights above water table (after Elfino and Davidson, 1987).

| **Height above Water Table in metre** | **Soil Type** | | | |
|---|---|---|---|---|
| | **A-3** | **A-2-4** | **A-2-6** | **A-5** |
| | **$M_r$ (Resilient Modulus, MPa) at 10,000 repetition** | | | |
| 0.38 | 194.54 | | | |
| 0.46 | 236.23 | | | |
| 0.61 | 275.60 | | | |
| 0.76 | | 169.95 | | |
| 0.91 | | 213.37 | | |
| 1.22 | | 220.48 | | |
| 1.02 | | | 330.72 | |
| 0.91 | | | | 76.84 |

The Florida Department of Transportation suggested an overall guideline for selecting the fill height over the water table as given in Table 2.9

Table 2.9 Freeboard suggested by Florida Department of Transportation (Elfino and Davidson, 1987)

| **Types of Road** | **Above WT (m)** | **Return period (Year)** |
|---|---|---|
| Four-lane primary and interstate | 0.91 | 50 |
| Two-lane Primary | 0.61 | 50 |
| Secondary | 0.30 | 25 |

The present pavement design guide of Bangladesh Roads and Highways Department proposed the freeboard and the return period for determination of formation level (RHD, 2005).

Table 2.10 Freeboard and return period for different road classes (RHD, 2005)

| Road Type | Freeboard (m) | Return Period for HFL (Years) |
|---|---|---|
| Dual carriage way | 1.00 | 50 |
| 7.3 m carriageway | 1.00 | 50 |
| 6.2 m carriageway | 1.00 | 50 |
| 5.5 m carriageway | 0.90 | 50 |
| 3.7 m carriageway | 0.90 | 20 |
| Minimum in any event | 0.30 | 20 |

## 2.8 Subgrade Rutting during Construction Period

The subgrade is the foundation layer of the pavement structure on which the subbase is laid directly. When this layer is completed after proper compaction, usually a granular layer having coarse granular materials is placed on it. The construction traffic passes over it for a long time until the final bituminous layer is laid which may cause rutting to develop. Powel (1984; cited in Hau and McDowell, 2005) proposed an empirical relationship to determine the minimum granular thickness over subgrade to allow rutting within a permissible limit of 40 mm for a known value of CBR and a given traffic volume.

$$\log N = \frac{l_g (CBR)^{0.63}}{190} - 0.24 \qquad 2.17$$

Where $N$ = number of axel load passes during construction period, $l_g$ = minimum thickness of granular material in mm, $CBR$ = California Bearing Ratio of subgrade material.

## 2.9 General Properties of Embankment and Foundation Soil

### 2.9.1. Embankment Materials

Materials used in the embankment should have a high shear strength that is unaffected by the changes of moisture content. Suitable materials include rocks, gravels and medium to coarse sand. The shear strength of materials with a high organic content, change with moisture content and are therefore not ideal.

Before using any materials consolidation tests should be carried out based on the anticipated density. Soils having different characteristics should not be mixed together (BS 6031, 1981).

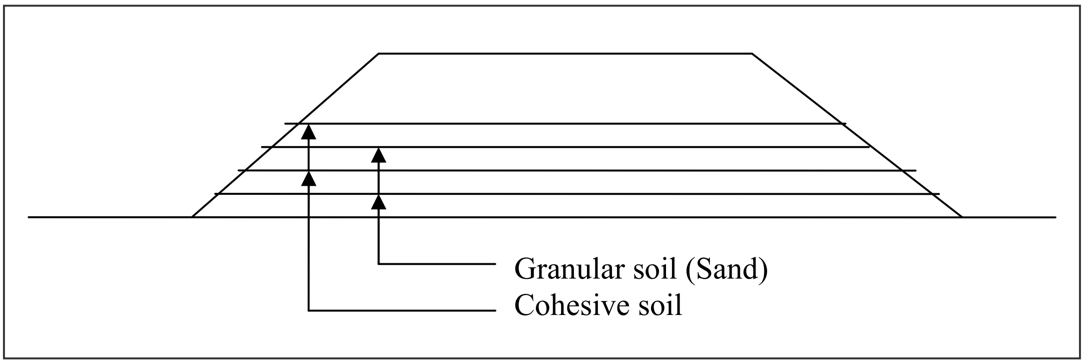
#### 2.9.1.1. Material Characteristics

Different types of materials can be used for embankment fill although those used in practice are often taken from cutting close to the embankment or from river beds. The performances of different types of materials are discussed in the flowing paragraphs (RMSS, 1994 and BS-6031, 1981).

**Granular soils:** Coarse grained granular materials make a very good fill material. The development of excess pore water pressure during construction is low and therefore most of the deformation occurs during embankment loading and strength is gained rapidly by the fill materials. Further, this type of material is very effective against foundation shear failure where the embankment is constructed on soft soil. Problems may arise if the fill material is fine sand and it is in saturated condition.

The granular materials are also helpful in those areas where the foundation soils are soft and marshy.

**Cohesive soils**: This type of soil has low permeability and develops high excess pore water pressure under increased loading. The undrained behaviour of this soil does not allow quick consolidation and therefore during construction there is little consolidation settlement with a risk of future settlement of the embankment soil. The rate of pore pressure dissipation can be improved by providing granular layers in between the cohesive soil which also improve the strength of the embankment fill.




**Silts:** The strength and deformation behaviour of silt soils are very much susceptible under loading.

**Peat:** This is an unsuitable material for embankment fill. If there is any peat soil in the foundation the ground should be improved by removing it or accelerating settlement by applying a surcharge and / or providing vertical drainage.

### 2.9.1.2. Embankment Soils in Bangladesh

In 1994, a study was carried on road materials survey by Roads and Highways Department in Bangladesh (RMSS, 1994). Several numbers of parameters for pavement and embankment material was summarised in that study without major input parameters for embankment design. Especially the parameters for stability and settlement analysis were absent in that study. Parameters relating to subgrade soil properties such as CBR, dry density, optimum moisture content, liquid limit, plastic limit, plasticity index were tabulated in respect of area and types of materials. The geotechnical properties regarding bearing capacity of some source fill materials for embankments are summarised below (RMSS, 1994).

Table 2.11 Geotechnical properties of fill materials (RMSS, 1994)

| Geotechnical Properties | Dredged Sand | Soil in Flat Areas |
|---|---|---|
| CBR | 9 | 7 |
| Std. SCBR | 6 | 3 |
| MDD (modified)in $kN/m^3$ | 18 | 18 |

**Characteristics of River Sand**:

The river bed is a great source of embankment and pavement materials in Bangladesh. The coarsest particles of sand are located in the upstream part (North) and the finest are close to the mouth of the river (down stream/South).

Most of the sands have a CBR of between 7-15% and are used as subgrade material and also as a capping layer.

There are some problems in construction with this river sand. The main problems are its cohesion-less property and difficulties to compact by rollers.

**Characteristics of Clay**:

Two types of clays are found in Bangladesh (RMSS, 1994). They are:

1. Modhupur and Barind clay: This type of clay contains iron and can produce lateritic gravels. It is found in the Northwest of Dhaka, Tangail, Mymenshing and Gazipur districts.
2. Grey Clay: It is found in the southern Bangladesh.

### 2.9.2. Problematic Foundation Soil

Construction of embankment on foundations containing peat, organic and clay soils create problems in terms of both the settlement and stability of the embankment. The geotechnical properties of such soils have been studied by Hsi et al. (2005) in a Sri Lankan highway and are given in Tables 2.12 to 2.14.

Table 2.12 Settlement properties of peat, organic and clay soil (after Hsi et al., 2005)

| **Soil type** | | **Peat** | | | **Organic soil** | | | **Soft clay** | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Parameter | Unit | Min | Max | Mean | Min | Max | Mean | Min | Max | Mean |
| $C_c$ | | 0.044 | 0.633 | 0.310 | 0.043 | 0.529 | 0.211 | 0.048 | 0.310 | 0.147 |
| $C_r$ | | 0.002 | 0.125 | 0.027 | 0.005 | 0.080 | 0.022 | 0.003 | 0.046 | 0.018 |
| $e_o$ | | 0.660 | 11.610 | 4.450 | 0.540 | 3.740 | 1.240 | 0.480 | 2.610 | 1.240 |
| $m_v$ | $m^2$/MN | 0.100 | 7.490 | 2.240 | 0.200 | 3.700 | 1.200 | 0.100 | 1.600 | 0.680 |

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| $C_v$ | $m^2$/year | 0.140 | 9.600 | 1.690 | 0.270 | 8.800 | 1.570 | 0.400 | 7.100 | 2.020 |
| $k$ | $m^2$/year | 3.154 | 31.536 | 17.345 | | | | | | |
| $C_\alpha$ | | 0.100 | 0.650 | 0.375 | 0.076 | 0.184 | 0.130 | | | |
| $C\varepsilon\alpha$ | % | 1.780 | 9.710 | 4.080 | 1.960 | 4.060 | 2.960 | 0.500 | 2.500 | 1.500 |

Table 2.13 Index properties of peat, organic and clay soil (after Hsi et al., 2005)

| **Soil type** | | **Peat** | | | **Organic soil** | | | **Soft clay** | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Parameter | Unit | Min | Max | Mean | Min | Max | Mean | Min | Max | Mean |
| $\gamma_d$ | kN/m$^3$ | 1.28 | 19.91 | 5.69 | 3.34 | 18.05 | 9.91 | 6.28 | 20.50 | 12.07 |
| $LL$ | % | 28 | 110 | 78 | 28 | 140 | 64 | 22 | 113 | 64 |
| $PL$ | % | 13 | 57 | 37 | 21 | 56 | 34 | 13 | 45 | 27 |
| $OC$ | % | 10 | 57 | 37 | 21 | 56 | 34 | 13 | 46 | 27 |
| $G_s$[1] | | 1.3 | 1.9 | 1.6 | | | | 2.7 | 2.9 | 2.8 |

[1] after Das 2004

Table 2.14 Shear strength properties of peat, organic and clay soil (after Hsi et al., 2005)

| **Soil type** | | **Peat** | | | **Organic soil** | | | **Soft clay** | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Parameter | Unit | Min | Max | Mean | Min | Max | Mean | Min | Max | Mean |
| $S_u$ | kPa | 4 | 103 | 21 | 3 | 70 | 27 | 14 | 87 | 40 |
| $c'$ | kPa | 0 | 18 | 6 | 8 | 12 | 11 | 4 | 29 | 13 |
| $\varphi'$ | Deg | 19 | 25 | 23 | 21 | 27 | 25 | 10 | 32 | 24 |

## 2.10 Specification for Highway Embankment in Bangladesh

Quality control specification for earth work, subgrade, and improved-subgrade were described in specification for highway, volume 3 of contract document (RHD, 2001). There is a guide line for slope angle which is 1:2 for all roads irrespective of height and materials. There are options for subsoil drainage but no specific procedures and guideline for its design.

## 2.11 Documents Available in Bangladesh

Available documents in Bangladesh relating to embankment design and constructions were reviewed as a part of this project and are listed below.

- Specification for highway, volume 3 of 4 of contract document
- Road materials and standards study Bangladesh, volume I to IV, 1994
- Geometric design standards for Roads and Highways Department

- Pavement design guide
- Standard cross sections for RHD roads
- Overseas Road Note 31
- Overseas Road Note 14.

## 2.12 Summary

This chapter has reviewed the literature concerning different elements of the embankment design process including various ground improvement work. It found that any design approach must ensure the stability of the embankment and also make sure that the rate and amount of consolidation is within acceptable limits. A number of techniques were identified which can be used to facilitate these processes. These will be further examined in later chapters of this thesis.

No systematic approach to highway embankment design has been identified from previous work. Although specifications for embankment fill and subgrade materials and embankment geometry are exists.

# Chapter 3 Methodology

## 3.1 Introduction

From the review of literature presented in the previous chapter it may be seen that the main objective of highway embankment design is to provide an engineered surface on which the pavement structure may be built. The embankment must be stable, not prone to any type of failure and the majority of settlement should occur within a specific period of time and it should be kept within tolerable limits. The formation level on which embankment is to be built must provide a platform with an adequate bearing capacity and acceptable amount of settlement.

In terms of the embankment construction, soft fine-grained foundation soils, such as those found in Bangladesh are particularly problematic since they are prone to both bearing capacity failure and excessive settlement. In the case of clayey soils the settlement period may be long however the time to achieve an adequate amount of settlement may be reduced using one or more of a number of measures to accelerate consolidation. The embankment should also be designed to protect the subgrade from being softened from capillary rise and from the ingress of water from surface flow. To achieve this, the geometry of the embankment should be designed appropriately and adequate surface and subsurface drainage should also be provided. This chapter describes how these factors have been considered in an overall design methodology for embankments under conditions experienced in Bangladesh where heavy and prolonged seasonal rainfall, soft organic soils and long consolidation periods are major considerations.

## 3.2 Design

The design of a highway embankment should be carried out to satisfy the following criteria (Hartlen and Wolski, 1996; Hsi and Martin, 2005; Yeung, 1997; Lee et al., 2007).

1) Limit settlement to an acceptable amount

2) Limit the time taken for an acceptable amount of consolidation settlement to occur

3) Provide adequate stability

4) Provide a platform for the road pavement.

However, when considering the design of an embankment it is also necessary to take into account the ease of construction i.e. available plant, expertise and other resources.

These design criteria are described more fully below and a process by which each aspect may be considered in an analytical design methodology is presented.

### 3.2.1. Settlement of Embankment and Foundation

Settlements take place both in embankment and in the foundation on which embankment is constructed. Settlement within the embankment can be reduced considerably by using engineered materials and proper compaction. As described in Section 2.3, the resulting settlement of the foundation due to embankment load may be considered to comprise of two components (Barnes, 1995; Das 2002; Bergado et. al., 1990): 1) elastic deformation 2) consolidation settlement.

Elastic settlement takes place immediately after application of the embankment load and has no prolonged impact on the pavement structure. Mainly consolidation settlements were considered in the design process to achieve the design requirements. The predicted amount of elastic settlement was used in the calculation of excess filling to maintain the formation level.

To prevent the pavement from failing, it is usual to specify a limit on the amount of time taken for the foundation to settle a given amount during construction. To keep the consolidation settlement within desired limits, it is often necessary to adopt measures in the design to accelerate the settlement process. Such methods which may be suitable for Bangladeshi condition are described and contrasted in Chapter 5.

**Settlement criteria**

Settlement of embankment can be considered to be both residual and differential. Residual settlement refers to settlement in a specified area after construction of the pavement whilst differential settlement is associated with the difference in settlement longitudinally or laterally. Differential settlements occur due to variability of soil properties of foundation in longitudinal and lateral direction of road alignment. As the elastic settlement takes place immediately after placing embankment load on the foundation soil, the remaining settlement of foundation occurs as a result of consolidation.

Suitable values for both residual and differential settlement are investigated and described in Chapter 10.

**Methods of analysis**

For the purpose of this work, the settlement of the foundation is analysed by taking into account the soil parameters both for the embankment and foundation under a number of assumed embankment geometries. In order to carry out the analysis several methods were identified from the literature. These were compared against a number of criteria given in Section 3.3 to identify a suitable one for the task in hand.

**Accelerating settlement**

If the predicted degree of settlement, such as that determined using the techniques described above, before the start of construction of pavement structure is not within an allowable limit methods may be used to accelerate the consolidation settlement process. A number of different methods are used for this process and some of these are described in Chapter 5 with an emphasis on those most suitable for conditions in Bangladesh.

### 3.2.2. Embankment Stability

An embankment is said to be stable if it can maintain its original slope during its design life and can withstand against bearing capacity of foundation. A stable embankment doesn't penetrate into the foundation and should maintain its original geometry (Kanairaj, 1988).

To ensure that an embankment is stable it is usual to determine a minimum factor of safety against failures. A factor of safety is defined as the ratio of the available shear strength of soil to the shear stress developed along a critical failure surface. It is determined by knowing the properties of foundation and fill materials, water level and geometry of the embankment and by identifying potential failure surfaces from which the minimum factor of safety can be determined.

**Stability criteria for embankment**

Stability criteria are usually specified on the basis of short and long term behaviour of the embankment (Hsi and Martin, 2005).

- Short term stability requires the embankment to be stable during construction period.
- The long term performance of an embankment is associated with the period after construction and up to the end of its design life.

Suitable factor of safety for both short and long term stability are investigated and described in Chapter 10.

**Methods of analysis**

Numerous methods are available for assessing the stability of slopes in the long and short term. As mentioned in Chapter 2, they include methods developed by Bishop, Janbu, Spencer, Low, Fellenius, Bishop and Morgrnstern and TRL (Bishop, 1955; Janbu, 1973; Spencer, 1967; Low, 1989; Fellenius, 1927; Bishop and Morgenstern, 1960; ORN 14, 1997). In each of these methods it is only possible to analyse stability if soil profiles are known, together with their engineering properties. In addition to this it is necessary to identify plane or planes of weakness along which a failure surface may exist.

**Stability improvement**

If the stability criteria are not satisfied even after the improvement of the shear strength by consolidation processes, measures can be taken to improve the stability. As described in Chapter 2, such methods include using geo-reinforcement, adopting stage construction, use of berms and vegetation. A number of these techniques are compared in the Chapter 7 and their suitability for conditions experienced in Bangladesh determined.

### 3.2.3. Platform for Road Pavement

The subgrade which is the top layer of the embankment serves as the platform of the pavement structure. For the purpose of this work the formation level is defined as upper most part of the subgrade on which the road pavement is constructed as shown in Figure 3.1.

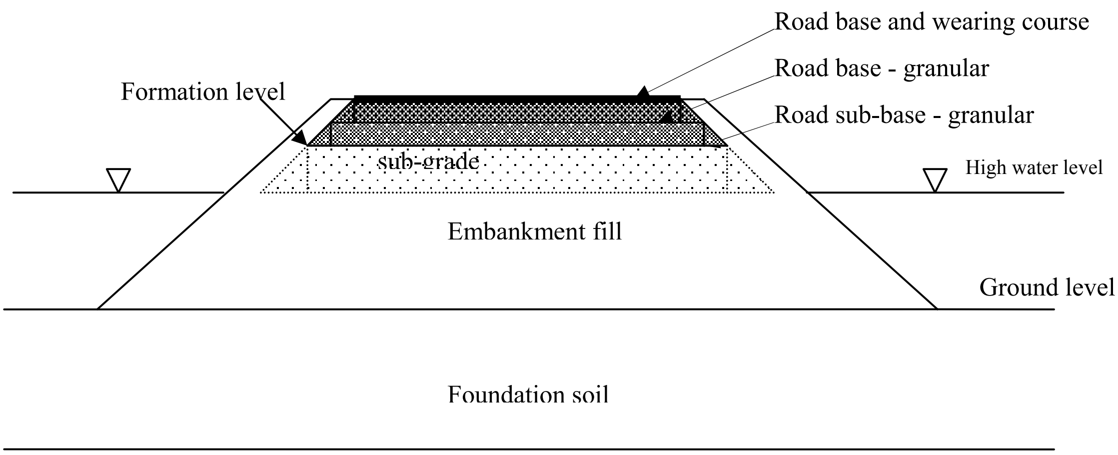



Figure 3.1 Description of road embankment along with pavement layers

Typically the soil formation consists of compacted soil on a stabilized layer of sub-grade material. The road sub-base may be placed directly on this layer.

In Bangladesh the formation level which is the top of subgrade is designed to be approximately 1 m above the highest 1 in 50 year flood water level. However during the monsoon season, the water levels may typically rise to 1m or less. As a result the lower portions of the embankment become inundated and due to capillary actions the moisture content of material above the water level also increases. One of the aims of this project is to design the embankment so that the formation level and subgrade materials properties are also taken into considerations.

The Sub-grade as defined herein is the foundation layer of a highway pavement. Two of the main input parameters for the design of a pavement structure are the sub-grade resilient modulus and Poisson's ratio.

A parallel study to the one described herein is investigating a suitable design methodology for road pavements in Bangladesh and for the parallel study it is necessary to specify achievable sub-grade resilient moduli and Poisson's ratio. From the work presented herein these two parameters have been determined from the embankment material, moisture content and degree of compaction using established testing procedures of well known correlations. To facilitate this process a database is used for subgrade and fill materials properties for different regions in Bangladesh developed by the RMSS of Bangladesh.

If an embankment is constructed with material having low values of strength, materials having higher values of strength may be considered.

## 3.3 The Conceptual Design Process

Four different design criteria have been identified and described above. A number of methods and techniques given in the literature may be used in order to design an embankment according to these design criteria. It is therefore necessary to develop a design framework which considers each criterion as part of an overall design process and which also identifies suitable methods of techniques which may work to design engineers in Bangladesh. The logical design framework is given in Figure 3.2 and 3.3.

It may be seen from Figure 3.3 that settlement criterion is to be satisfied first before going to satisfy stability criterion. Remedial measures required to satisfy the criteria of residual settlement also helps to satisfy stability criteria and that is why settlement criteria has been shown in flowchart before stability criteria.

The methods and techniques which may be used by an engineer in the design process are discussed more fully in Chapter 4, 5, 6, 7, 8 and 9 with a view to selecting one in each case, a suitable technique for the task in hand. The following criteria were used to identify the suitable techniques.

1. Accuracy: The method which considers all the factors influencing the predictions.
2. Ease of use / training: Considering the computation procedure and make it understandable to Engineers.
3. Availability and cost of software: To keep the cost to a minimum and make available the software to all RHD Engineers for computing stability and settlement following any methods.
4. Availability of input parameters: Whether the input parameters for the analyses are achievable by the available field and laboratory testing equipment.
5. Previous field performance with predicted results: The results from each model were compared with observed settlement and real slope failure of some embankments.

## 3.4 Hydrologic Analysis

Ideally as mentioned above, the formation level of road should be above the highest expected high flood level (HFL) for a given design return period. The design return periods for different types of road in Bangladesh are given in Table 2.10.

For this work the design HFL was determined using Chow's (1988) equation for a given return period as mentioned in Chapter 9. The return period was selected on the basis of present Roads and Highways guidelines. To facilitate this process water level data have been collected from Institute of Water Modelling (IWM) and Bangladesh Water Development Board (BWDB). The detailed hydrological analyses undertaken in this work were included in Chapter 9.

## 3.5 Design Input Parameters

The analyses were based on information from other available source of Bangladesh and from literature. To carry out the verification of stability model and understating the field settlement behaviour under seasonal water level variation, field work containing of boreholes and laboratory works were undertaken. Soil parameters were collected form the selected sites of N302 (Tongi-Ashulia road) by a researcher of parallel study. Three bore-hole samples were collected and tested for required soil parameters. The sections were selected in such a way so that one falls in the failure area of embankment and others in good conditions. The soil parameters those were not possible to determine were estimated or taken from other available source of similar soil. For hydrological analysis water level data were collected from IWM. To facilitate the process a large of database is being prepared by a parallel study of this project. Key geotechnical parameters were needed to facilitate the design are as follows: 1) settlement and consolidation parameters to predict settlement and time of consolidation, 2) soil strength parameters to assess stability and 3) Subgrade parameters for the assessment of strength in terms of resilient modulus.

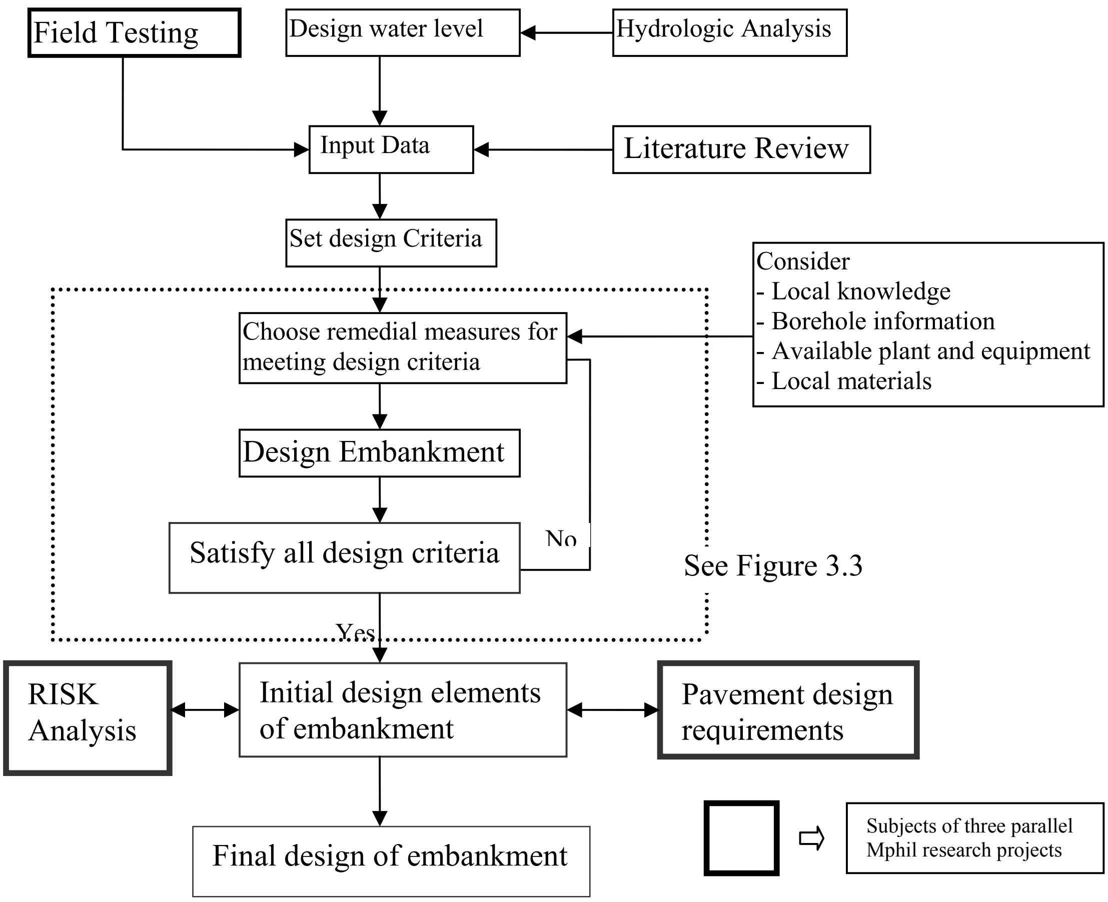



Figure 3.2 Generic Embankment Design

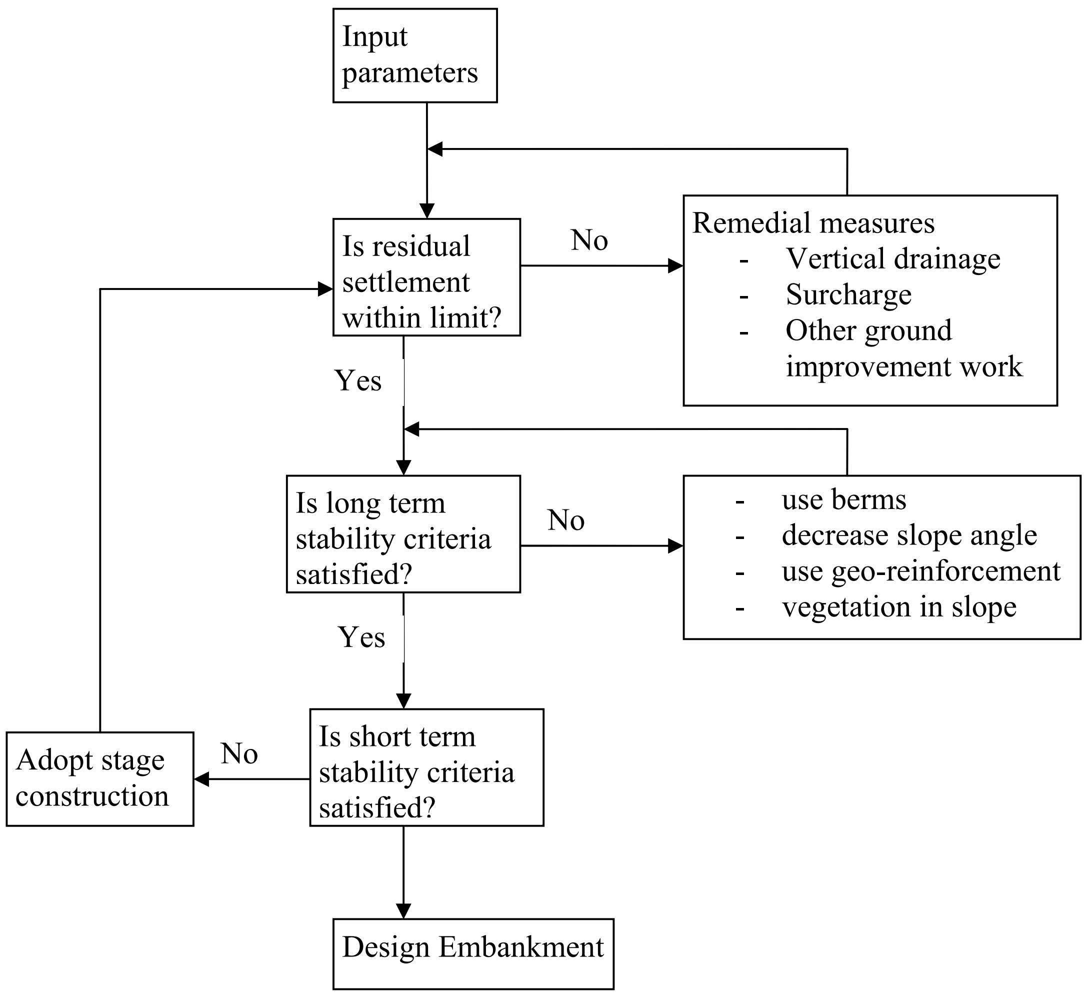



Figure 3.3 Embankment Design Process

## 3.6 Summary

A systematic and rational method of embankment design developed in this research and outlined in this chapter, which considers climatic conditions, available fill materials, the condition of the foundation soil, available construction practices, and the availability of necessary expertise. The components of the embankment design process are described more fully in Chapters 4, 5, 6, 7, 8 and 9. The approach is illustrated using data collected from the field in Chapter 10.

# Chapter 4 Settlement of Foundation

## 4.1 Introduction

As described in Chapter 3, the settlement of an embankment is a combination of the settlement of the foundation under embankment loading and settlement within the embankment itself. If the embankment is constructed with engineered soil and appropriate compactions, settlement within the embankment can be reduced except for long term secondary compression.

This chapter considers various modes of foundation settlement occurred due to embankment and traffic loading and their methods of prediction in view of selecting a suitable method for the task in hand.

## 4.2 Types of Settlement

Settlements in the foundation may be considered to consist of the following types (see Section 2.3):

- Immediate elastic settlement
- Primary consolidation settlement
- Secondary consolidation settlement

### 4.2.1. Immediate Elastic Settlement

The undarined elastic settlement at a corner of foundation from the load of the embankment can be given by Ueshita and Meyerhof's equation (1968; cited in Bergado et. al., 1990).

$$S_e = \frac{qBI}{E_u} \qquad 4.1$$

Where, $q$ = the applied load per unit area, $B$ = width of the loaded area (taken at half height), $E_u$ = undrained elastic modulus and $I$ = influence factor which is a function of $H/B$ and $L/B$ and can be found from Figure 1 in Appendix XIII. Where, $H$ is the depth of soft layer, $L$ is the length of structure and in case of embankment it is taken as $\infty$.

By superposition, the elastic settlement at the centre of the embankment is four times that of corner. The value $E_u$ varies with depth but the equation is applicable for homogeneous soil stratum having an average $E_u$. So the soil having different $E_u$ at different stratum, the settlement can be determined for each stratum individually and summed to get the total settlement.

The value of $E_u$ can be expressed in terms of undrained shear strength ($S_u$) and is given by the following relationship (Bergado, et al., 1990):

$$E_u = \alpha S_u \qquad 4.2$$

Where $\alpha$ is a coefficient depending on the soil characteristics. The value of $\alpha$ can be taken as 150 for clay (Bergado, et al., 1990). The undrained modulus for normally consolidated soils can also be determined from the following empirical equation (Foott and Ladd, 1981).

$$E_u = 215 S_u \ln(FS) / PI \qquad 4.3$$

Where, $FS$ = calculated factor of safety against shear failure and $PI$ = plasticity index of soil.

Considering local yielding the immediate elastic settlement can be given as

$$S_i = \frac{qBI\left(1-\nu_u^2\right)}{E_u \times SR} \qquad 4.4$$

$$SR = f\left(f, \frac{q}{q_{ult}}, geometry\right) \qquad 4.5$$

Where $SR$ = Settlement ratio. It can be found by knowing pore pressure coefficient ($A$) and width of the continuous foundation ($B$), depth of clay layer ($H$), and foundation type (in this case continuous) from Figure 2 (Appendix-XIV). $f$ = initial shear-stress ratio, $q/q_{ult}$ = applied stress ratio, $q$ = the applied load per unit area (stress), $\nu_u$ = undrained Poisson's ratio.

**Pore pressure coefficient (*A*)**: The response of pore water pressure due to change in total stress along the major principal axis under undrained condition is expressed by a coefficient known as the pore pressure coefficient (A) which is defined as the ratio of increased pore pressure to the change in total stress along major principal axis of load application. If the increase in applied stress along the major principal axis $\Delta\sigma_1$ and the subsequent increase in pore water pressure is $\Delta u_1$, the pore pressure coefficient is given by (Craig, 2005):

$$A = \frac{\Delta u_1}{\Delta \sigma_1} \qquad 4.6$$

In fully saturated conditions, the pore pressure coefficient is one which implies the same amount of increase in pore pressure to that of applied load

### 4.2.2. Primary Consolidation Settlement

The amount of primary consolidation is given by the following equations for different consolidation states (Das, 2004).

For normally consolidated clay

$$S_p = \sum_1^n \frac{H}{1+e_0}\left( C_c \log \frac{\sigma'_{vo} + \Delta\sigma'_{av}}{\sigma'_{v0}} \right) \qquad 4.7$$

For over consolidated clays with $\sigma'_0 + \Delta\sigma_{av} < \sigma'_p$

$$S_p = \frac{H}{1+e_0}\left( C_r \log \frac{\sigma'_{vo} + \Delta\sigma'_{av}}{\sigma'_0} \right) \qquad 4.8$$

For over consolidated clays with $\sigma'_0 + \Delta\sigma > \sigma'_p$

$$S_p = \frac{H}{1+e_0}\left( C_c \log \frac{\sigma'_{vo} + \Delta\sigma'_{av}}{\sigma'_p} + C_r \log \frac{\sigma'_p}{\sigma'_{v0}} \right) \qquad 4.9$$

After applying the Skempton-Bjerrum (1957) modification

$$S_p = S_{p(oed)} \times SR \qquad 4.10$$

Where, $\sigma'_{vo}$ = insitu vertical effective stress, $\Delta\sigma'_{av}$ = average increase in vertical stress due to embankment loading, $\sigma'_p$ = maximum past pressure or pre-consolidation pressure, $C_r$ = recompression index, $C_c$ = compression index, $SR$ = settlement ratio = $f(A, H/B$, $A$ = pore pressure coefficient (defined below), $B$ = bottom width of embankment and $H$ = drainage path of clay layer undergoing consolidation. If the depth of clay layer is $H_c$, and if there is two-way drainage $H = H_c/2$, and if one way drainage $H = H_c$.

### 4.2.3. Secondary Consolidation Settlement

The amount of settlement due to secondary consolidation is given by (Das, 2004; Craig, 2005):

$$S_{sec} = \sum_{1}^{n} \frac{H_p}{1+e_p} \left( C_\alpha \log \frac{t_e}{t_p} \right) \qquad 4.11$$

Although $C_\alpha$ is constant over the time its magnitude is directly related to $C_c$ with various consolidation pressures. To include the effect of compression index ($C_c$) into secondary compression, a concept of $C_\alpha/C_c$ was used by Mesri and Castro (1987) and in that case the amount of secondary compression is given by the following equation:

$$S_{sec} = \sum_{i=1}^{n} \frac{C_{c(i)}}{1+e_{0(i)}} \frac{C_{\alpha(i)}}{C_{c(i)}} H_i \left( \log \frac{t_e}{t_p} \right) \qquad 4.12$$

The subscripts stand for the layers having different settlement properties.

#### 4.2.3.1. Secondary Compression after Surcharge

The effect of applying a surcharge to reduce secondary consolidation settlement Mesri et al. (1997) introduced the concept of a secant secondary compression index ($C''_\alpha$) to predict the amount secondary settlement using the following equation:

$$S_{sec} = \sum_{1}^{n} \frac{C_c}{1+e_0} \frac{C_\alpha}{C_c} \frac{C''_\alpha}{C_\alpha} H_o \left( \log \frac{t_e}{t_l} \right) \qquad 4.13$$

Where, $H_o$ = initial thickness of the layer to be consider for settlement, $H_p$ = thickness of the layer to be consider after primary settlement, $t_p$ and $t_e$ = time required for primary consolidation and the design period of the road respectively, $t_l$ = time required to restart the secondary compression after removal of surcharge, $C_\alpha$ = secondary compression index or coefficient of secondary consolidation, $C''_\alpha$ = post surcharge secant secondary compression index. The method of determination of $C''_\alpha$ was described in Section 2.3.3.2.

The total settlement is the sum of the above three phases of settlement and can be expressed as $S_{tot} = S_i + S_p + S_{\sec}$ 4.14

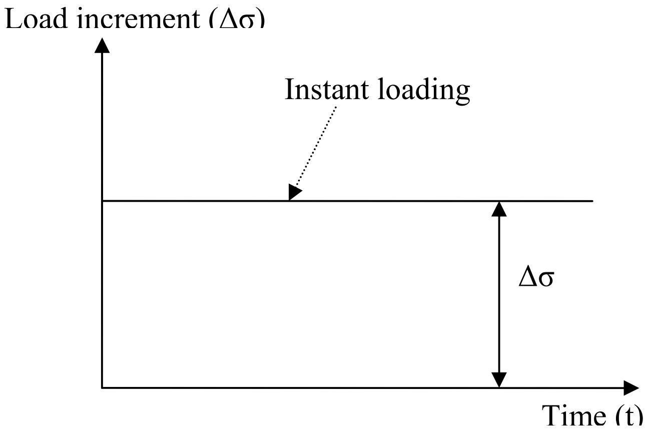
## 4.3 Consolidation with Time

Terzagi's equations for one-dimensional consolidation which is expressed by the degree of consolidation ($U$) for saturated clay soil was developed by Terzaghi in 1925 assuming **instant load** application on the foundation. Later on the equation was modified by Olson (1977) incorporating **ramp loading**. The equation relates degree of consolidation ($U$) with time of drainage from which degree of settlement can be determined for any desired loading period. The mathematical derivation of the time related consolidation equation for instant and ramp loading is described in the following sections based on the following assumptions: (cited in Das, 2005; Craig, 2005).

- The soil-water system is homogeneous
- The soil is fully saturated
- Negligible compression in water
- Darcy's law is valid
- Flow of dissipated water is one-dimensional

### 4.3.1. Instant Loading

In addition to the above assumptions one more important assumption in this case is that the load is applied instantly on the ground. The average degree of settlement ($U$) is given by the following equation:




$$U = \frac{S_t}{S_p} = 1 - \frac{\left(\frac{1}{2H_{dr}}\right)\int_0^{2H_{dr}} u_z dz}{u_0} \qquad 4.15$$

Where, $U$ = average degree of consolidation, $S_t$ = settlement at time t, $S_p$ = maximum settlement in primary consolidation, $u_o$ = initial excess pore water pressure at the time of load application, $H_{dr}$ = length of drainage path. For two way drainage $H_{dr} = H/2$ and one way drainage $H_{dr} = H$.

Excess pore water pressure ($u_z$) at time $t$ can be can be given by the following equation

$$u_z = \sum_{m=0}^{m=\infty}\left[\frac{2u_0}{M}\sin\left(\frac{Mz}{H_{dr}}\right)\right]e^{-M^2T_v} \qquad 4.16$$

Putting the value of $u_z$ in the above equation 4.15, the combined equation can be found as following:

$$U = 1 - \sum_{m=0}^{m=\infty}\frac{2}{M^2}e^{-M^2T_v} \qquad 4.17$$

Here $T_v$ is defined by time factor and is given by the following relations

$$T_v = \frac{C_v t}{H_{dr}^2} \qquad 4.18$$

where, $M = \frac{\pi}{2}(2m+1)$ 4.19

m = 0, 1, 2, 3 …………. until the sum of all remaining terms is insignificant.

The solution of the Equation 4.17 is approximately given by the following equations which were developed by assuming instant load application (cited in Das 2004). The value of *U* can also be determined from the Figure 4.1 which was developed on the basis of Equation 4.20 and 4.21.

$T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2$ For U = 0 to 60% 4.20

For U > 60%

$$T_v = 1.781 - 0.933\log(100 - U\%) \qquad 4.21$$

For U >60%, the solution of Equation 4.17 by parabolic isochrones (Atkinson, 2009) is as follows.

$$U_t = 1 - \frac{8}{\pi}\left\{\exp\left(-\frac{\pi^2}{4T_v}\right) + \frac{1}{9}\exp\left(-\frac{9\pi^2}{4T_v}\right) + \frac{1}{25}\exp\left(-\frac{25\pi^2}{4T_v}\right) + .......\right\} \qquad 4.22$$

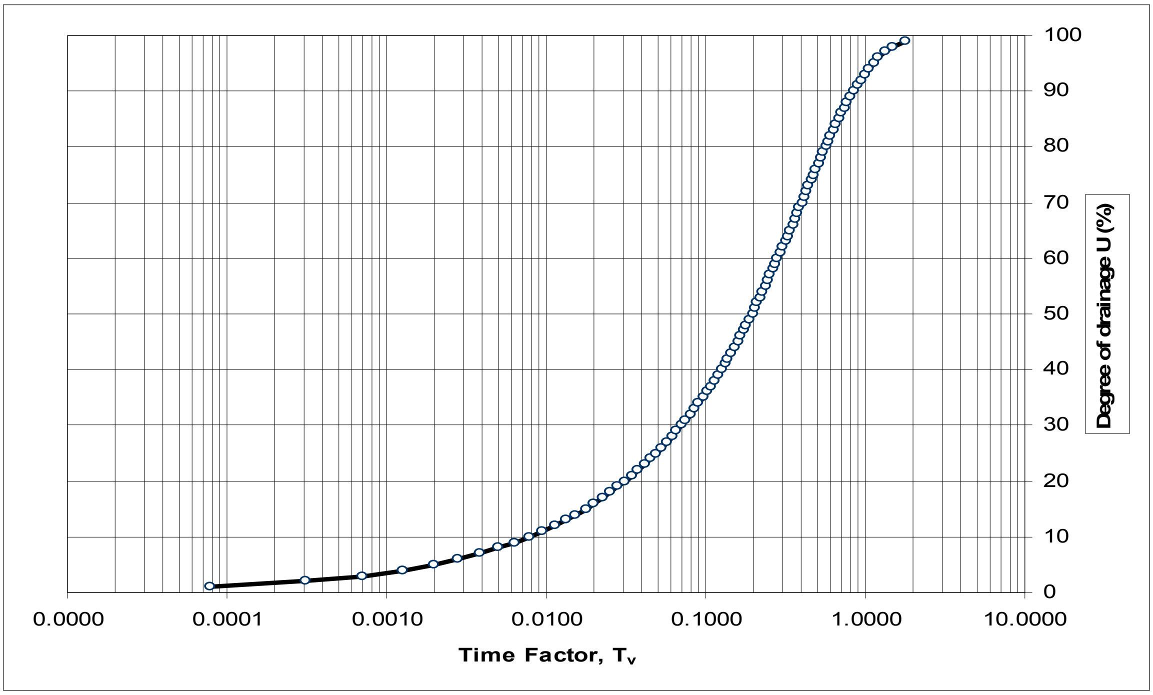



Figure 4.1 Degree of drainage vs time factor for instant loading

### 4.3.2. Ramp Loading

Olson (1977) developed consolidation equations for loading which varies with time (ramp loading) and he introduced an additional parameter, construction time ($t_c$). This is the usual case for embankment loading.

In most of the cases, the embankment loading ($\Delta\sigma$) increases with time ($t$) up to the end of construction ($t_c$) and then it remains constant. The degree of consolidation ($U$) considering ramp loading is given by Olson (1977) as follows:

For $T_v \leq T_c$, $$U_v = \frac{T_v}{T_c}\left[1 - \frac{2}{T_v}\sum_{m=0}^{m=\infty}\frac{1}{M^4}\left\{1 - \exp\left(-M^2 T_v\right)\right\}\right] \qquad 4.23$$

For $T_v \geq T_c$, $$U_v = 1 - \frac{2}{T_c}\sum_{m=0}^{m=\infty}\frac{1}{M^4}\left\{\exp\left(M^2 T_c\right) - 1\right\}\exp(-M^2 T) \qquad 4.24$$

where, $T_c = \dfrac{C_v t_c}{H_{dr}^2}$ 4.25

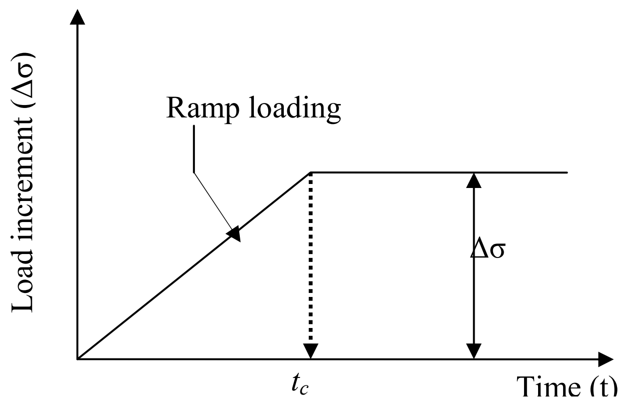



Figure 4.2 Ramp Loading

The degree of drainage ($U_v$) can be found by solving Equation 4.22 and 4.23. The value of $U_v$ also be found directly from Figure 4.3 for time factor $T_v$ and a known construction time factor ($T_c$).

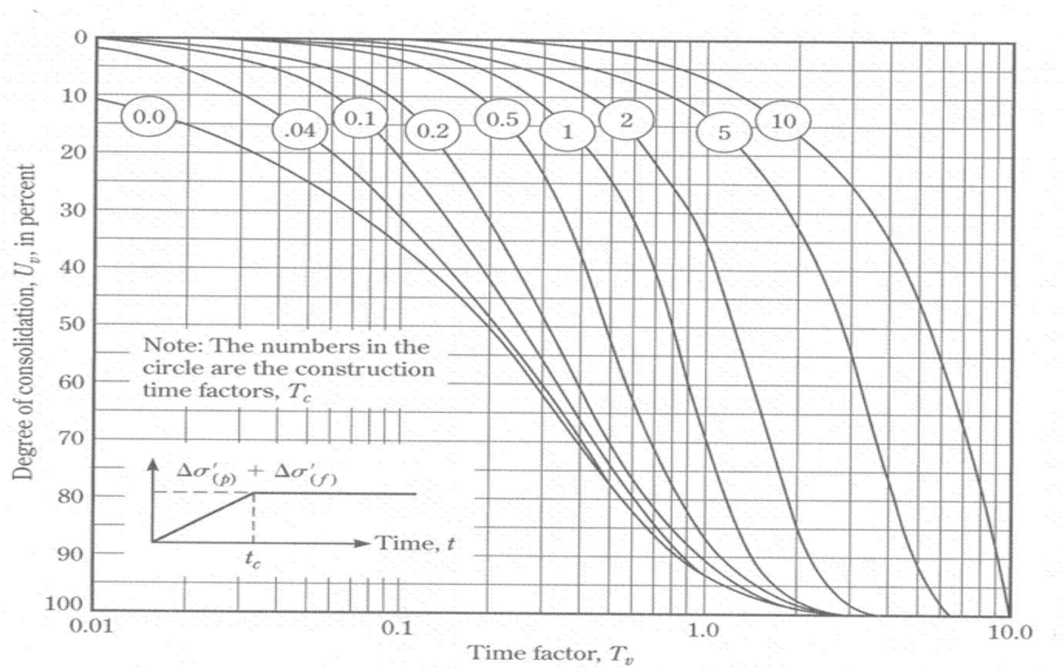



Figure 4.3 Degree of consolidation for ramp loading (after Olson, 1977)

## 4.4 Stress Increase in Subsoil due to Loading

To calculate the anticipated primary consolidation settlement, it is necessary to determine the increase of stress with depth of foundation soil due to an incremental load. In the case of a highway embankment stresses are developed as a result of the embankment and traffic loads.

### 4.4.1. Embankment Loading

The increase of vertical stress to a depth $z$ from the ground surface due to embankment loading can be determined from Osterberg's equation (Das, 2004) as follows:

$$\Delta\sigma_{av} = \frac{q_0}{\pi}\left[\left(\frac{B_1 + B_2}{B_2}\right)(\alpha_1 + \alpha_2) - \frac{B_1}{B_2}(\alpha_2)\right] \qquad 4.26$$

Where, $q_o = \gamma H$, $\gamma$ = unit weight of embankment soil, $H$ = height of embankment.

$$\alpha_1 = \tan^{-1}\left(\frac{B_1 + B_2}{z}\right) - \tan^{-1}\left(\frac{B_1}{z}\right) \qquad 4.27$$

$$\alpha_2 = \tan^{-1}\left(\frac{B_1}{z}\right) \qquad 4.28$$

Where, $\alpha_1$ and $\alpha_2$ are in radians. The simple form of the above equation can be written as

$$\Delta\sigma_{av} = q_0 I \qquad 4.29$$

As loading is symmetrical the vertical stress at the centre of embankment is twice that of half of the section and is given as:

$$\Delta\sigma_{av} = 2q_0 I \qquad 4.30$$

where, $I = f\left(\frac{B_1}{z}, \frac{B_2}{Z}\right)$ 4.31

To determine the stress increase at any point under the embankment it is necessary to find the influence factor for the sections separately and sum them.

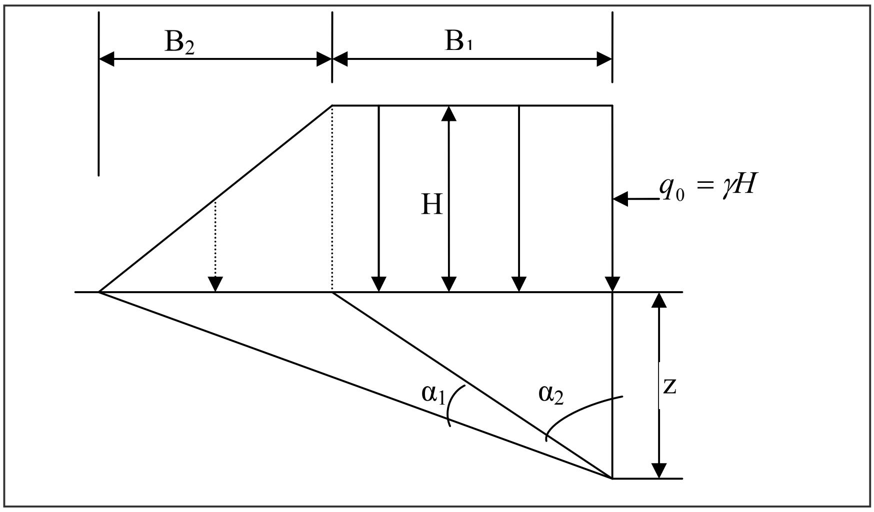



Figure 4.4 Computation elements for embankment loading

The value of the influence factor at centre of the embankment can also be found for specific embankment heights from Figure 4.5 which was produced in this research from above formulae as described in Appendix- XVI.

The variations of settlements along the cross section are not significant from centre to the edge of the embankment although there are significant differences in settlement between the edge of the shoulder and the end of slope (as shown in Figure 4.6). If the anticipated settlement is achieved before the construction of the pavement layers, the differences in settlement in the transverse directions can be compensated for by filling with earth suitably compacted. The analysis of settlement in this thesis therefore focused on the centre of embankment.

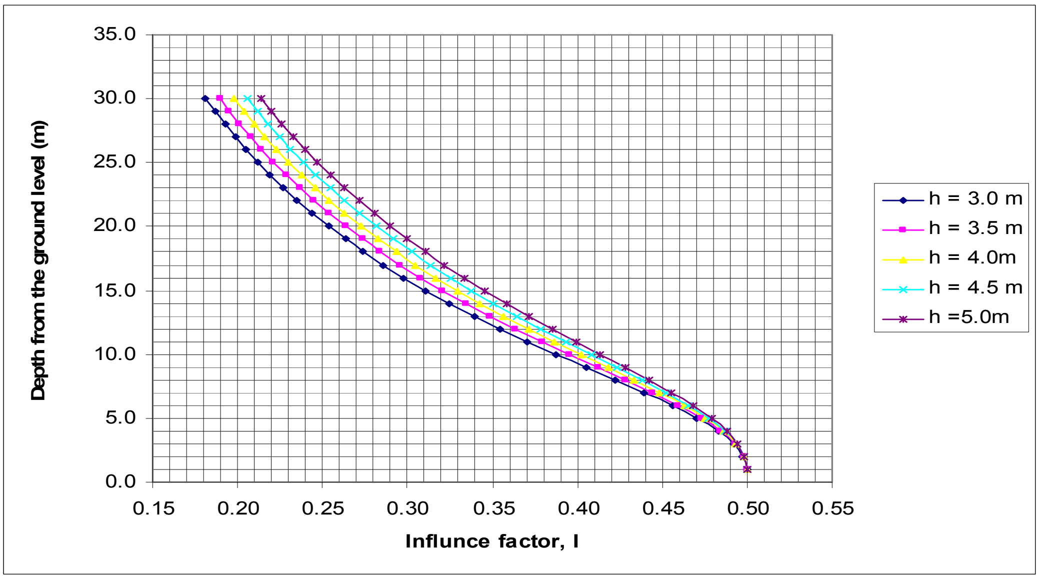



Figure 4.5 Influence factor at centre of the embankment

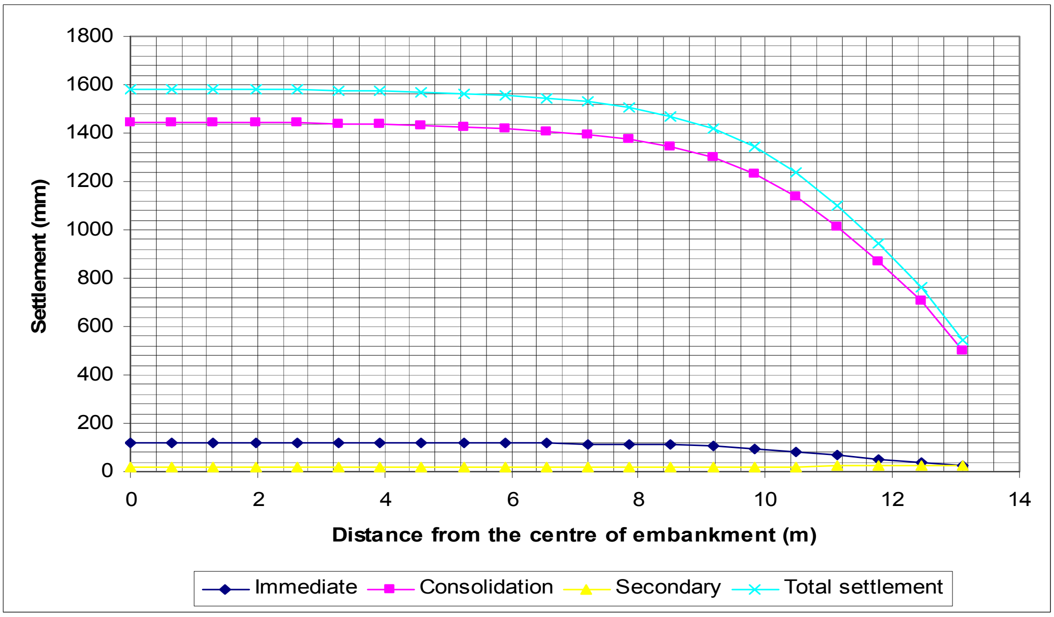



Figure 4.6 Settlements along the X-section of road

### 4.4.2. Traffic Loading

The prediction of traffic load induced settlement can be carried out by the following methods.

#### 4.4.2.1. Fujikawa et al. Equivalent Static Load Method

In this method it is assumed that the stress increase in subsoil layers due to traffic loading is a function of embankment height (h) and depth from the ground surface (Z). The increment of static stress equivalent to the traffic load at the surface of the foundation is given by the following equation (Fujikawa et al., 1996; cited in Shen et al., 2007).

$$\Delta\sigma_T = EXP(4.62 - 0.47h) \qquad 4.32$$

The above stress has a triangular distribution to a depth ($Z_T$) where the stress is assumed to zero and maximum ($\Delta\sigma_T$) at the top of ground surface as shown in Figure 4.7. The depth is given by the following relationship.

$$Z_T = EXP(2.31 - 0.47h) \qquad 4.33$$

Where, $h$ = height of embankment in metres, $\Delta\sigma_T$ = increase of stress due to traffic loading at the top of the ground surface (kPa), $Z_T$ = depth of influence for traffic loading where stress increment is zero (m).

Thus the increase of stress due to traffic loading at a depth Z from the top of the ground can be given using the formula of triangular ratio as follows:

$$\Delta\sigma_{T(Z)} = \frac{\Delta\sigma_T (Z_T - Z)}{Z_T} \qquad 4.34$$

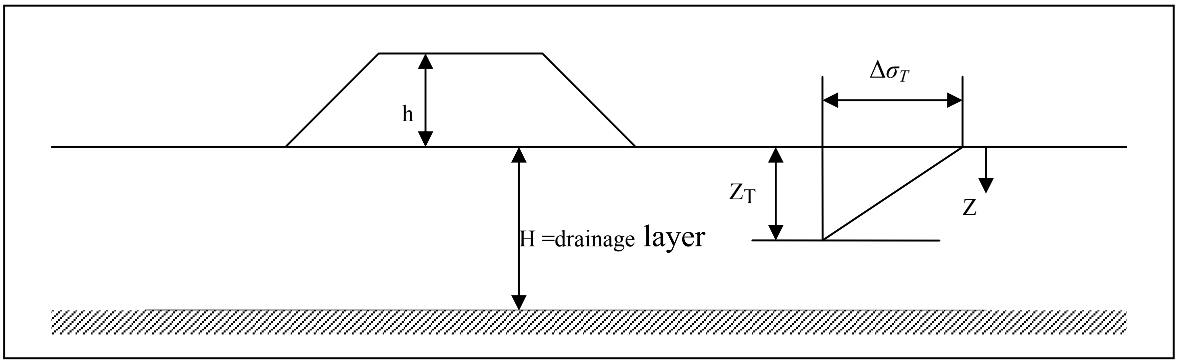



Figure 4.7 Traffic load induced stress distribution in Fujikawa et al. (1996) method

#### 4.4.2.2. Chai and Miura's Empirical Method

Chai and Miura's (2002) method enables traffic load induced settlement due to consolidation and plastic deformation to be predicted. To achieve this, the properties and thickness of pavement structural layers including subgrade and fills are required. The equation for calculating cumulative strain for soft cohesive soil under repeated traffic loading is

$$\varepsilon_p = a\left(\frac{q_d}{q_f}\right)^m \left(1+\frac{q_s}{q_f}\right)^n N^b \qquad 4.35$$

The definitions and methods of determination of the terms and coefficients of the above equation are described in the following paragraphs:

**Dynamic deviator stress ($q_d$):** The stress on the subsoil due to tire load of traffic is termed as dynamic deviator stress and can be determined by the Burmister's multilayer elastic theory (Burmister, 1945; cited in Chai and Miura, 2002).

**Initial deviator stress ($q_s$):** The stress developed in the subsoil due to embankment loading and the load from the upper soil layers and can be expressed by the following equation:

$$q_s = \sum \gamma_i h_i + \Delta\sigma'_v \qquad 4.36$$

Where $\Delta\sigma'_v$ = the stress due to embankment loading which can be determined by the Osterberg's formula as described in Section 4.4.1.

**Static failure deviator stress ($q_f$):** This relates to the properties subsoil and can be determined directly from the shear strength of the soil if it is known. The static failure deviator stress is given by

$$q_f = 2S_u \qquad 4.37$$

Where, $S_u$ is the undrained shear strength of soil.

**Number of repeated load ($N$):** The cumulative number of standard load application through out the design life.

**Constant $a$:** The parameter "$a$" depends on the consolidation properties of subsoil and influences the magnitude of strain of subsoil. It can be determined from the compression index ($C_c$) of the subsoil using the following relationship (Chai and Miura, 2002):

$$a = \alpha C_c \tag{4.38}$$

Where, $\alpha$ is a constant, Chai and Miura (2002) suggested $\alpha$ =8.0

**Constants *b* and *m*:** Parameter ***b*** relates to soil properties and is not sensitive to the dynamic deviator stress. The choice of ***m*** influences both the magnitude and distribution of plastic strain with depth. Chai and Miura (2002) suggested the following values:

$b$ = 0.18 and $m$ = 2.0

**Constant *n***: It is a factor which takes into account the effect of the initial deviator stress. Chai and Miura (2002) proposed $n$ = 1.0

The values of a, b and m suggested by Li and Selig (1996) for his equation which are same as that of Chai and Miura's equation as in the following Table 4.1.

Table 4.1 Constants of the Equation 4.35 (after Li and Selig, 1996)

| **Soil type** | ***a*** | ***b*** | ***m*** |
|---|---|---|---|
| High plastic clay | 1.2 | 0.18 | 2.4 |
| Low plastic clay | 1.1 | 0.16 | 2.0 |
| Elastic silt | 0.84 | 0.13 | 2.0 |
| Silt | 0.64 | 0.10 | 1.7 |

**Example:** An example of traffic load induced settlement prediction for an embankment 2.75 m high is given in Appendix-X. The resultant settlement as a function of depth of subsoil is shown in Figure 4.8. From Figure 4.8 it can be seen that settlement at the top of the foundation due to traffic loading is approximately 65 mm. The total settlement at the top of the subgrade (at 0.75m depth) is approximately 352 mm after repetition of one million standard axles.

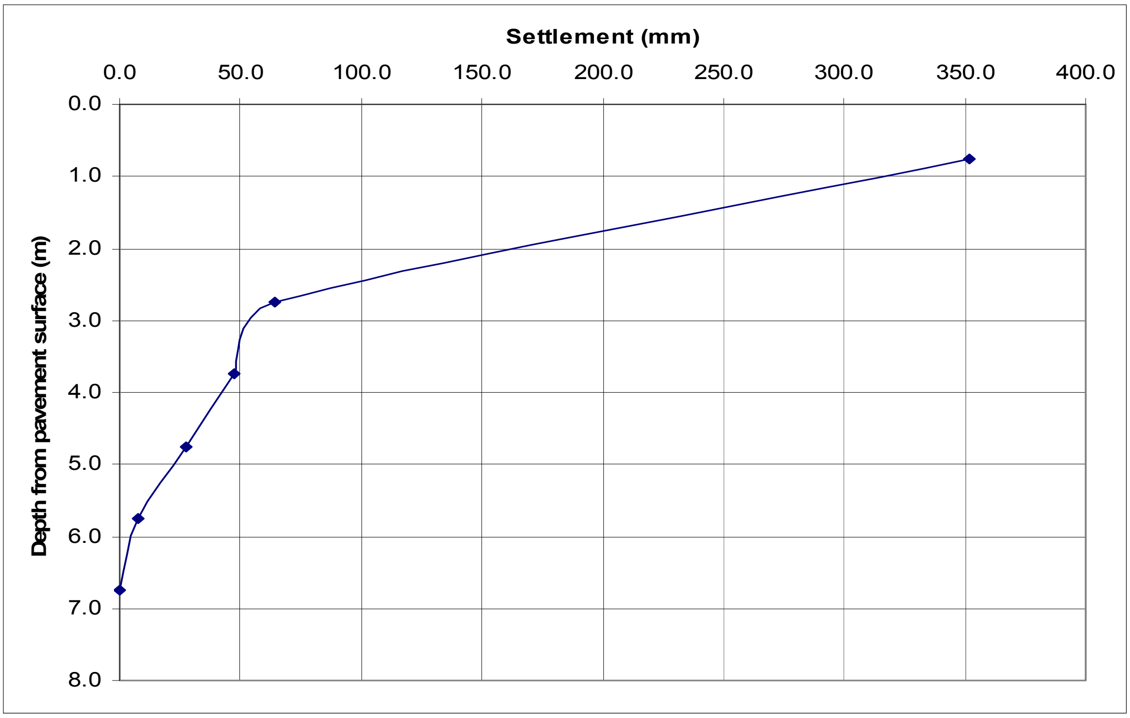



Figure 4.8 Settlement with depth of foundation for traffic loading

It is seen from the analysis that for low embankments, traffic load induced subsoil settlement can be reduced by increasing the thickness or modulus of granular layers. The settlement of the subsoil, fill and subgrade layers can also be reduced by increasing the modulus of subgrade and fill layers. The modulus can be improved by using good quality materials in the upper layers of embankment and stabilizing it mechanically and or chemically.

#### *4.4.2.2.1 Influence of subgrade resilient modulus on traffic induced settlement*

The influences of subgrade resilient modulus on subgrade and foundation settlement are shown in Figure 4.9 and 4.10 which have been determined using the Chai and Miura model described above. It is seen from the Figure 4.9 that settlements at the surface of the foundation are 72, 51, 43 and 38 mm for subgrade modulus 30, 60, 80 and 100 MPa respectively. This demonstrates, as would be expected that increasing the subgrade elastic modulus helps to reduce settlements under the layers beneath it. The settlement of foundation can also be reduced by increasing the thickness of granular subbase or base layers.

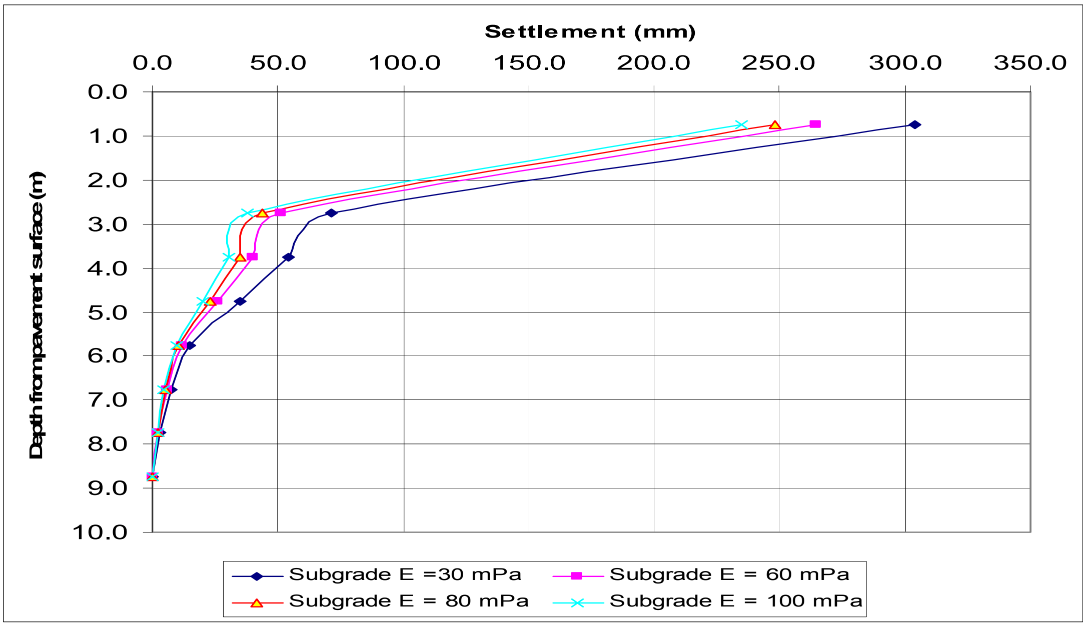



Figure 4.9 Settlement with depth for various subgrade moduli

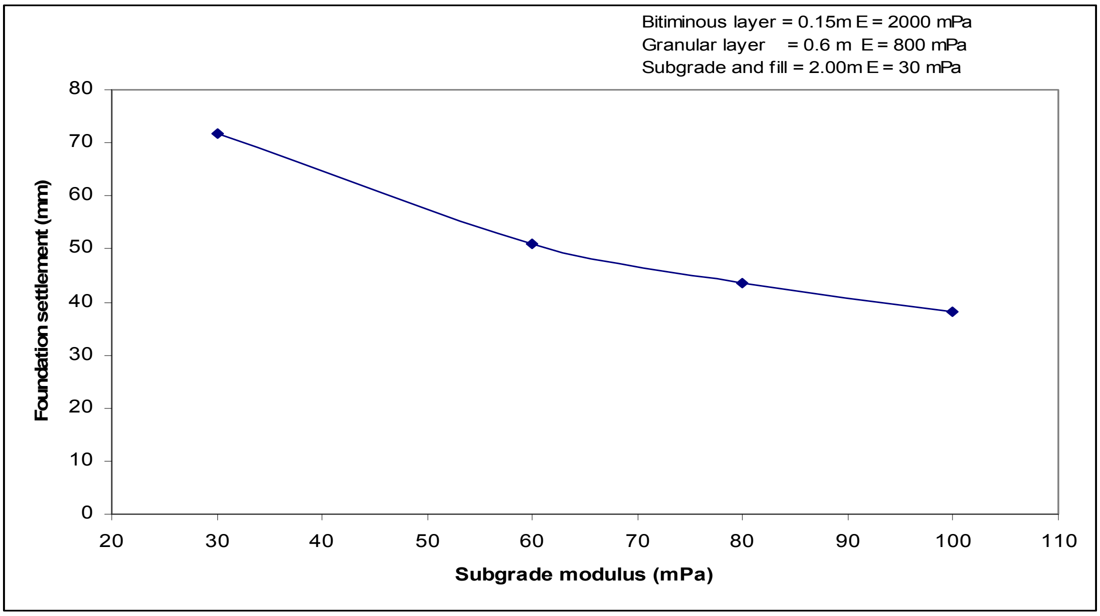



Figure 4.10 Foundation settlements due to variation of subgrade modulus

#### *4.4.2.2.2 Influence of embankment height on traffic induced settlement*

Subsoil settlement due to traffic loading is dependent on the embankment height as shown in Figure 4.11. The settlements were predicted based on the construction traffic volume of 1000 ESA as proposed by Hardman (1976; cited in Hau and McDowell, 2005). Four embankment heights were considered (2.0, 2.5, 3.0, and 3.5 m respectively) and to consist of only one layer (subbase) of pavement. The subsoil and subgrade parameters considered (as shown in Table 4.2) are similar to those found in Bangladesh.

Table 4.2 Parameters used for the prediction of construction traffic load induced settlement

| $Z$ | $S_u$ | $C_c$ | $q_d$ | $q_f$ | $q_s$ | $N$ | $a$ |
|---|---|---|---|---|---|---|---|
| metre | kPa | | kPa | kPa | kPa | ESA | |
| 0.00 | 22 | 0.3 | 2.44 | 44 | 40.0 | 1000 | 2.4 |
| 1.00 | 22 | 0.3 | 1.31 | 44 | 47.0 | 1000 | 2.4 |
| 2.00 | 22 | 0.3 | 0.82 | 44 | 54.0 | 1000 | 2.4 |
| 3.00 | 22 | 0.3 | 0.56 | 44 | 61.0 | 1000 | 2.4 |
| 4.00 | 22 | 0.3 | 0.41 | 44 | 68.0 | 1000 | 2.4 |
| 5.00 | 22 | 0.3 | 0.31 | 44 | 75.0 | 1000 | 2.4 |
| 6.00 | 22 | 0.3 | 0.24 | 44 | 82.0 | 1000 | 2.4 |
| 7.00 | 22 | 0.3 | 0.20 | 44 | 89.0 | 1000 | 2.4 |
| 8.00 | 22 | 0.3 | 0.16 | 44 | 96.0 | 1000 | 2.4 |
| 9.00 | 22 | 0.3 | 0.13 | 44 | 103.0 | 1000 | 2.4 |
| $\gamma_f$ $(kn/m^3)$ | $\gamma_m$ $(kn/m^3)$ | $\alpha$ | $b$ | $m$ | $n$ | $E_{subbase}$ $(MPa)$ | $E_{subgrade}$ $(MPa)$ |
| 17.00 | 20 | 8.0 | 0.18 | 2.0 | 1.0 | 80 | 30 |

The lower the embankment height the larger the settlement and vice versa as the traffic load is distributed with depth as expected. It should be noted that the construction traffic helps accelerate the foundation settlement reducing the residual settlement which takes place before construction of the upper pavement layers.

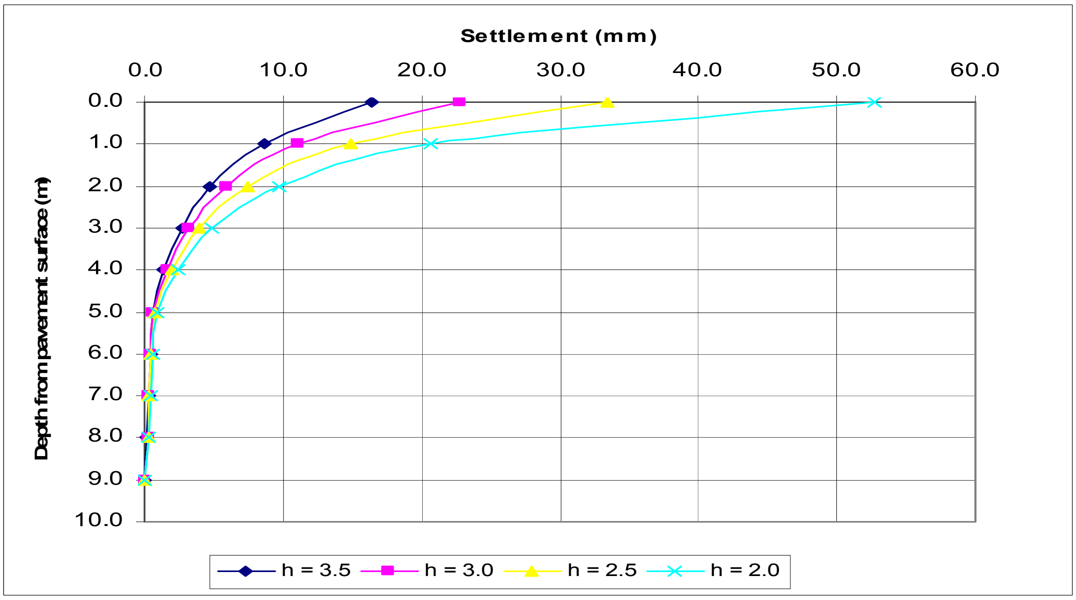



Figure 4.11 Settlement of subsoil foundation due to construction traffic for various embankment heights

## 4.5 Shear Strength Increase due to Consolidation

As described in Section 2.6.2.2, the shear strength of the foundation soil increases during the consolidation process. Hayashi et al. (2002) suggested the following equation for the increase in shear strength with degree of consolidation achieved.

$$C = C_o + m\Delta PU \quad 4.39$$

Where $C$ = cohesion after consolidation (kN/m$^2$), $C_o$ = initial value of cohesion (kN/m$^2$), $m$ = rate of increase in cohesion, $m = tan\varphi'$, $U$ = degree of consolidation achieved, $\Delta P$ = stress increase (kN/m$^2$), $\Delta P = \gamma h \frac{\overline{B}}{B'}$, $\gamma$ = unit weight of embankment soil, $h$ = height of embankment, $\overline{B}$ = width of embankment at crest, $B'$ = width of embankment at base.

An alternative empirically founded model has been suggested by Stamatopoulos et al. (1985; cited in Moh and Lin, 2005) where the increase in shear strength $\Delta S_u$ is given by:

$$\Delta S_u = \left(\frac{1 + w_n G}{0.434 C_c}\right) S_u \frac{\delta}{h} \quad 4.40$$

Where, $w_n$ = natural water content, $G$ = specific gravity, $C_c$ = compression index, $\delta$ = settlement, $S_u$ = undrained shear strength, $h$ = thickness of layer consolidated.

## 4.6 Methods of Settlement Analysis

A number of methods were found in the literature by which the total settlement of an embankment constructed on soft soil can be determined. These are:

- Standard method
- Finite element method
- Finite difference method
- Empirical Method

### 4.6.1. Standard Method

This method of settlement analysis makes use of Taylor's (1942) and Terzaghi's (1925) procedures (see Sections 4.2.2 to 4.3.2) modified by Olson's (1977) ramp loading. The loading of an embankment can not be done in a single step. In case of short construction time and if the construction time is much less than the consolidation time, the loading can be approximated by instant loading (Olson, 1998). In that case the settlement can be calculated following the Taylor's and Terzaghi's process. To calculate the degree of drainage for embankment loading, the construction time needs to be included. This may be achieved using Olson's (1977) equation. This allows the weighted average degree of consolidation (*U*) at any time to be estimated for each step of loading separately from Equation 4.41 (Olson, 1998).

$$U = \sum_{i}^{N} \frac{\sigma_i}{\sigma_{tot}} U_i \tag{4.41}$$

Where $\sigma_i$ = stress applied by load $i$, $\sigma_{total}$ = total stress from embankment load at infinity time, $U_i$ = degree of consolidation for the $i^{th}$ loading.

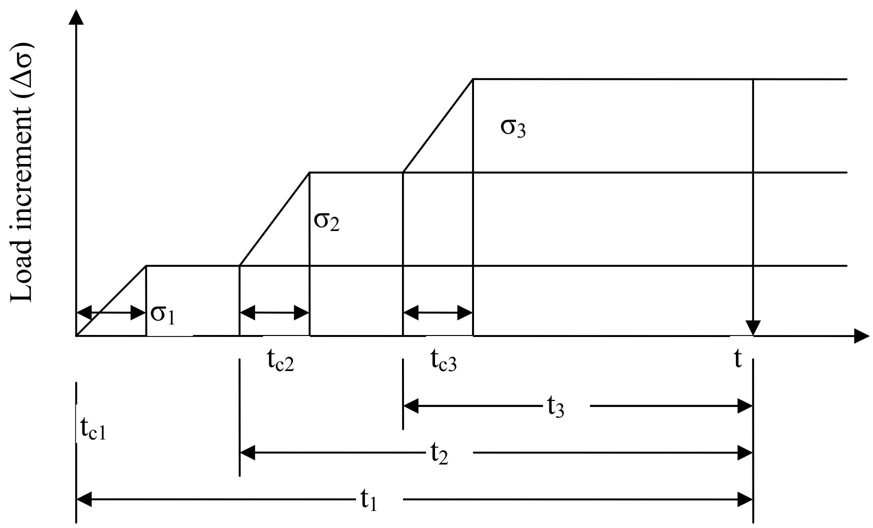



Figure 4.12 Loading stages of embankment construction with time

As per the above construction schedule the average degree of drainage can be written as follows:

$$U = \frac{\sigma_1}{\sigma_{tot}} U_1 + \frac{\sigma_2}{\sigma_{tot}} U_2 + \frac{\sigma_3}{\sigma_{tot}} U_3 \tag{4.42}$$

$$\sigma_{tot} = \sigma_1 + \sigma_2 + \sigma_3 \tag{4.43}$$

$U_i$ is determined from Equation 4.23 and 4.24 assuming the above construction time ($t_c$) and consolidation time (t).

### 4.6.2. Finite Element Method

The Finite Element approach can be used to find the settlement under applied stress. *CRISP* is an example of finite element software based on the modified Cam-Clay model developed by University of Cambridge (Loganathan et al., 1993). Using CRISP settlement analysis considering purely drained or undrained or coupled conditions can be carried out (Indraratna et al., 1992).

### 4.6.3. Finite Difference Method

Calculation of settlements can be determined using the Finite Difference Method (FDM) (Abbasi et al., 2007; Olson, 1998). Commercially available software such as *UTconsol* (Olson, 1998) and Settle3D (Rockscience, 2009) with friendly user interfaces have been developed to incorporate the FDM. Settle3D has been used in this study. It determines pore water pressures in foundations soils using FDM and effective stresses using Terzaghi's one-dimensional consolidation equation. As the method considers water table variation with time, it gives more accurate result in calculating settlements and can consider the influence of vertical drains.

### 4.6.4. Empirical Methods

A number of empirical methods have been developed for organic soils especially for peat soils. For example Carlsten (1988) developed the following empirical equation for free drainage both top and bottom of the organic soil layer (Carlsten, 1988; cited in Szymanski et al., 1996).

$$U = 1 - 0.6\exp\left\{-\left(\frac{0.52(w)^{0.75}}{H^2 q^{0.5}}\right).t\right\} \qquad 4.44$$

Where, $H$ = thickness of the organic soil layer, $w$ = natural water content and $t$ = time in days

He also developed a similar equation where the bottom layer is impermeable or drainage is in one direction only:

$$U = 1 - 0.6\exp\left\{-\left(\frac{0.13(w)^{0.75}}{H^2 q^{0.5}}\right).t\right\} \qquad 4.45$$

In developing his equations, Carlsten made the following assumptions:

- The thickness of the peat ranges from 2 to 6 m.
- Water content from 900 – 1500%
- Applied load q < 50 kPa

Total settlement can also be determined by empirical equations developed by Ostromecki (1956; cited in Szymanski et al., 1996) for peat soil having a maximum thickness 4.5 m with a load range from 10 to 50 kPa.

$$S = 1.08 \times C \times t_o \qquad 4.46$$

Where, $S$ = Total settlement in metres, $C$ = a coefficient which is a function of $H/t_o$ and $\gamma_d$. The coefficient $C$ is given by Ostromecki in Figure 3 (Appendix XIII) for $H/t_o$ and for different values of $\gamma_{d.}$, $t_o = q/7.55$ and is defined as normalized depth of dewatering (m), $H$ = thickness of peat layer (m), $\gamma_d$ = dry unit weight of peat (kN/m$^3$) and $q$ = applied stress on subsoil (kPa).

Similarly Drozd-Zajac (1968; cited in Szymanski et al., 1996) developed the following equation to estimate the total settlement of organic clays:

$$S = 3.6 \times H \frac{(e_0 - 0.36)^{1.1}}{1 + e_o} \log(3.12 \times q \times \sqrt{e_o}) \qquad 4.47$$

Where, $S$ = total settlement in mm, $H$ = thickness of layer to be compressed (m), $e_o$ = initial void ratio and $q$ = increase of stress due to embankment loading (bar).

In general the empirical methods should only be used for preliminary estimation of settlements as there is a question regarding their reliability and transferability conditions other than which they were developed (Szymanski et al., 1996).

## 4.7 Selection of an Appropriate Method

Among the four methods, the Standard method is selected for this study for its convenience in computation in Excel spreadsheet, easily achievable input parameters for analysis, shows reasonable agreement with observed and predicted settlement in FDM analysis (see Chapter 6) and it has reasonable performance in previous projects.

Standard method as called hereby is a combination of Taylor's and Terzaghi's one dimensional consolidation theory, modified by Olson's ramp loading. In this method, stress increase in subsoil due to embankment loading is calculated by Osterberg's (1957) equation. Prediction of immediate elastic settlement due to embankment loading is performed by Ueshita and Meyerhof's (1968) equation. In addition to the proposed method, the FDM computer software Settle3D (see above) which requires similar parameters but is able to analyse time dependent consolidation for layered soil applying both one and three-dimensional stresses, is also recommended as a suitable tool. The software is described more fully in Chapter 6.

## 4.8 Summary

The settlement of the foundation on which an embankment is built occurs due to embankment and traffic loading. Traffic load induced foundation settlement is negligible for embankment height more than three metres although it is significant for low embankments. Prediction of traffic load induced settlement is briefly described in this chapter and a suitable method identified from literature. The settlement of the foundation due to embankment loading can be predicted by a number of methods. From these the following methods were recommended for the task in hand: Standard method and finite difference model Settle3D.

In some cases the acceleration of settlements during embankment construction and before pavement construction may be necessary to reduce the time and amount of residual settlement. Methods of accelerating such settlements are described in Chapter 5 where appropriate methods for Bangladeshi conditions are selected. In addition to the standard method, finite difference modelling of settlement was performed with a view to identifying the most significant factors affecting settlement so that emphasis can be given on methods used to determine them as part of an associated MPhil research project.

# Chapter 5 Methods of Accelerating Consolidation

## 5.1 Introduction

As described in the previous chapter, consolidation can be accelerated to achieve a desired amount of consolidation in a certain time to reduce the residual settlement. This is often necessary when it is not possible to achieve the desired settlement before starting the construction of pavement. The following two main methods for achieving this have been identified in the literature and will be discussed further in this Chapter.

- Pre-compression
- Using vertical drains

A methodology for accelerating consolidation is shown in Figure 5.1

## 5.2 Pre-compression

When soil is highly compressible, such as peat soil, a surcharge load can be applied on top of the embankment before the pavement is constructed.

The basic principle of pre-compression by surcharge is to apply surcharge loads ($\Delta\sigma'_{sc}$) in addition to the embankment load ($\Delta\sigma'_{st}$). The primary consolidation due to the embankment is given by:

$$S_{p(st)} = \frac{C_c H}{1+e_o} \log \frac{\sigma'_o + \Delta\sigma'_{st}}{\sigma'_o} \qquad 5.1$$

If a surcharge load $\Delta\sigma'_{sc}$ is added the consolidation settlement becomes:

$$S_{p(st+sc)} = \frac{C_c H}{1+e_o} \log \frac{\sigma'_o + (\Delta\sigma'_{st} + \Delta\sigma'_{sc})}{\sigma'_o} \qquad 5.2$$

It can be shown that the time ($t_2$) required for consolidation will be shorter if a surcharge load ($\Delta\sigma'_{sc}$) is applied in addition to embankment as shown in Figure 5.2.

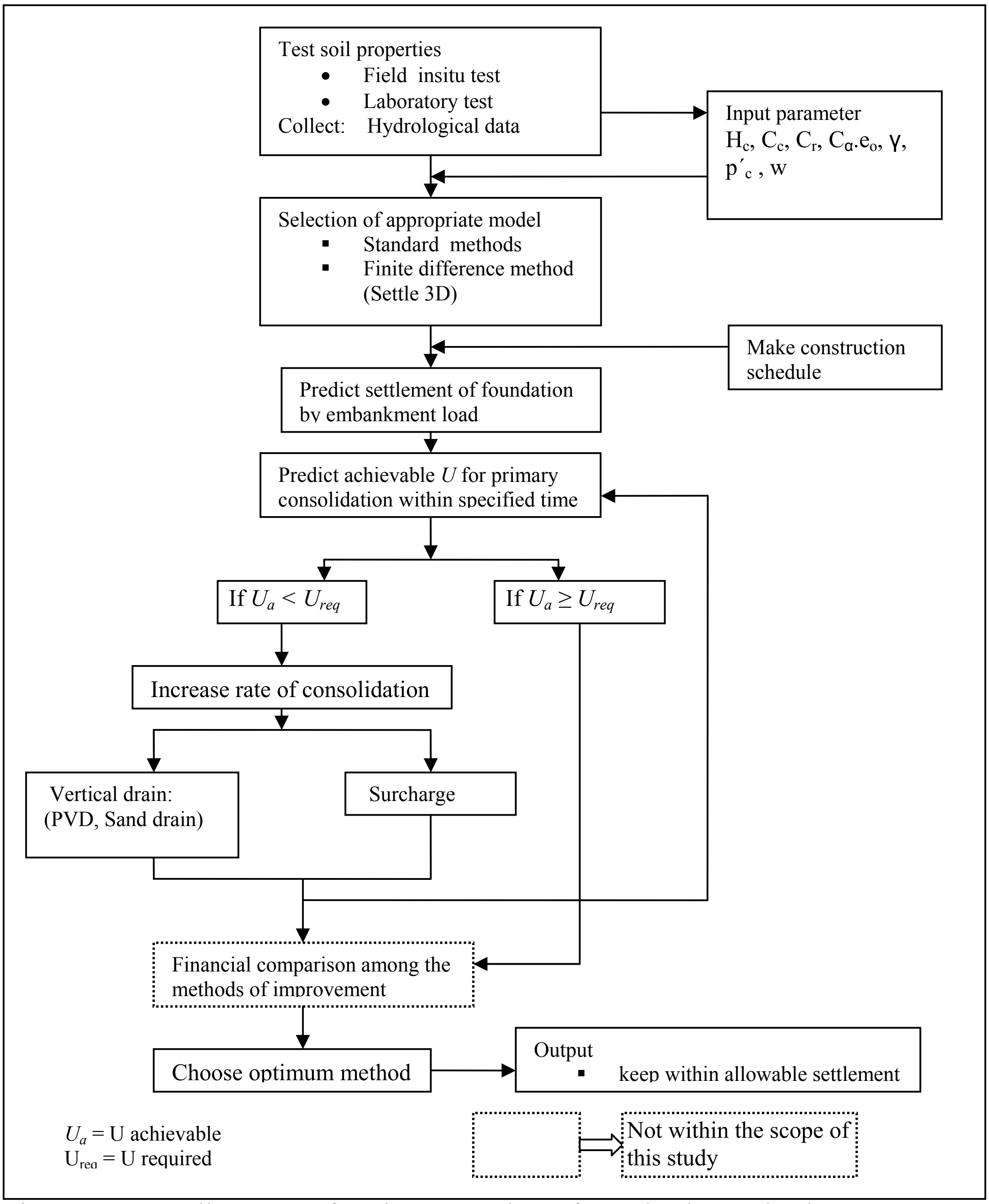



Figure 5.1 Overall process of settlement analyses for embankment load

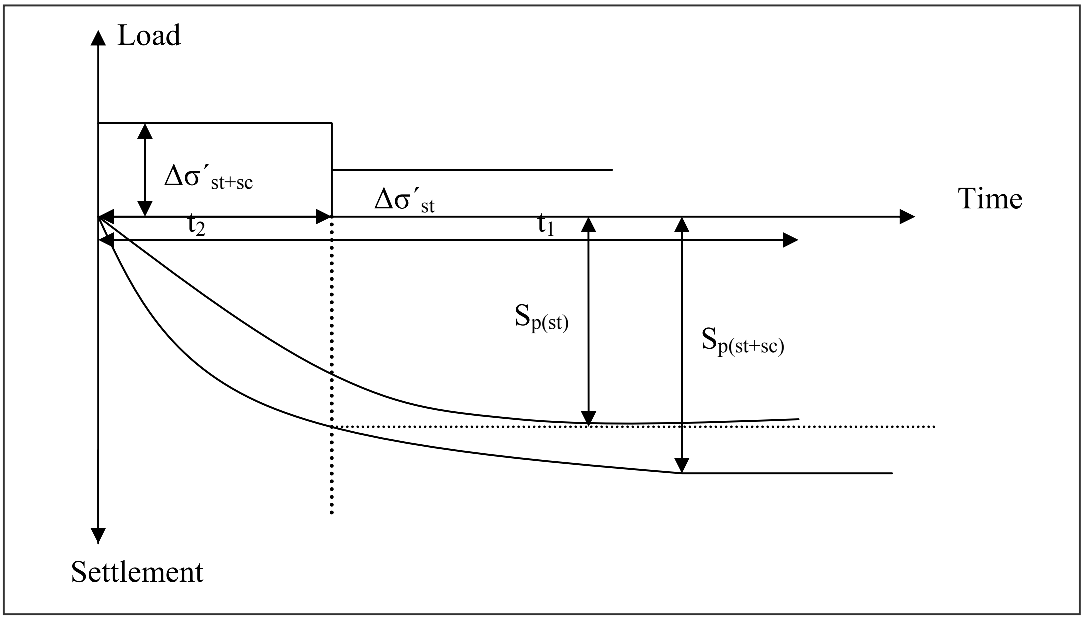


Figure 5.2 Behaviour of settlement with surcharge

**Determination of $\Delta\sigma'_{sc}$ and $t_2$**

The degree of consolidation, *U*, at time $t_2$ after the application of a surcharge is given by (Das, 2004):

$$U = \frac{S_{p(st)}}{S_{p(st+sc)}} = \frac{\log\left(1+\dfrac{\Delta\sigma'_{st}}{\sigma'_0}\right)}{\log\left(1+\dfrac{\Delta\sigma'_{st}}{\sigma'_0}\left(1+\dfrac{\Delta\sigma'_{sc}}{\Delta\sigma'_{st}}\right)\right)} \qquad 5.3$$

The degree of consolidation expressed in the above equation is the average degree of consolidation. However, near the drainage surface a clay soil may swell when the surcharge load is to be removed. With this *U*, the calculation of $t_2$ may lead to a smaller value of $t_2$ than it actually needed to overcome the swelling problems. To overcome this problem we can determine $t_2$ corresponding to the value of *U* in the middle of the clay layer (Johnson, 1970; cited in Das 2004).

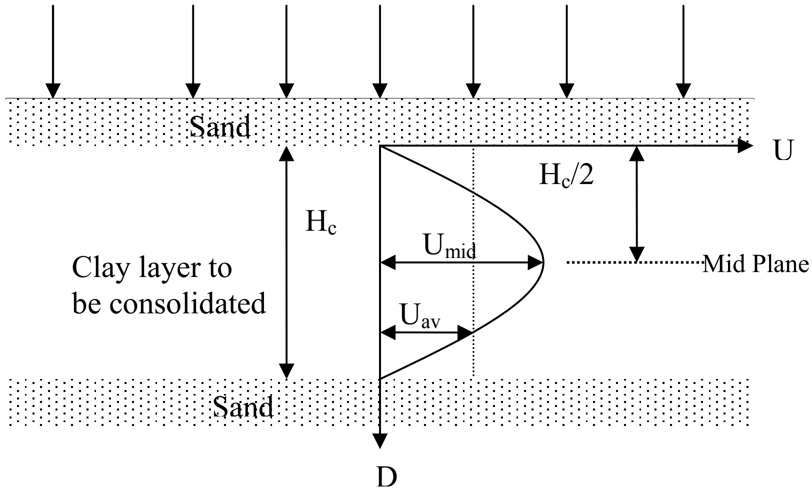



Figure 5.3 Mid-plane degree of drainage

To obtain the pre-compression parameters, two cases may be considered as follows:

**Case 1:** The value of $t_2$ is known but the value of $\Delta\sigma'_{sc}$ is to be obtained.

1. Calculate $T_v = \dfrac{C_v t_2}{H^2}$

↓

2. Find Middle Plane $U$ from Figure 5.6 $U_{mid}$ *vs.* $T_v$

↓

3. Find $\Delta\sigma'_{st+sc}/\Delta\sigma'_{st}$ from Figure 5.7 $\Delta\sigma'_{st+sc}/\Delta\sigma'_{st}$ *vs.* $U$

↓

4. Calculate $\Delta\sigma'_{st+sc}$

Figure 5.4 Procedure for estimating surcharge requirement for a given time (Case1)

**Case 2:** The value of $\Delta\sigma'_{sc}$ is known but the value of $t_2$ is to be obtained.

1. Estimate $\sigma'_o$ and. $\Delta\sigma'_{st}$

↓

2. Find $U$ from Equation 5.3

↓

3. Obtain $T_v$ from Figure 5.6 $T_v$ vs. $U$

↓

4. Calculate $t_2$ from equation

$$t_2 = \frac{T_v H^2}{C_v}$$

Figure 5.5 Procedure for estimating time for a given surcharge (Case 2)

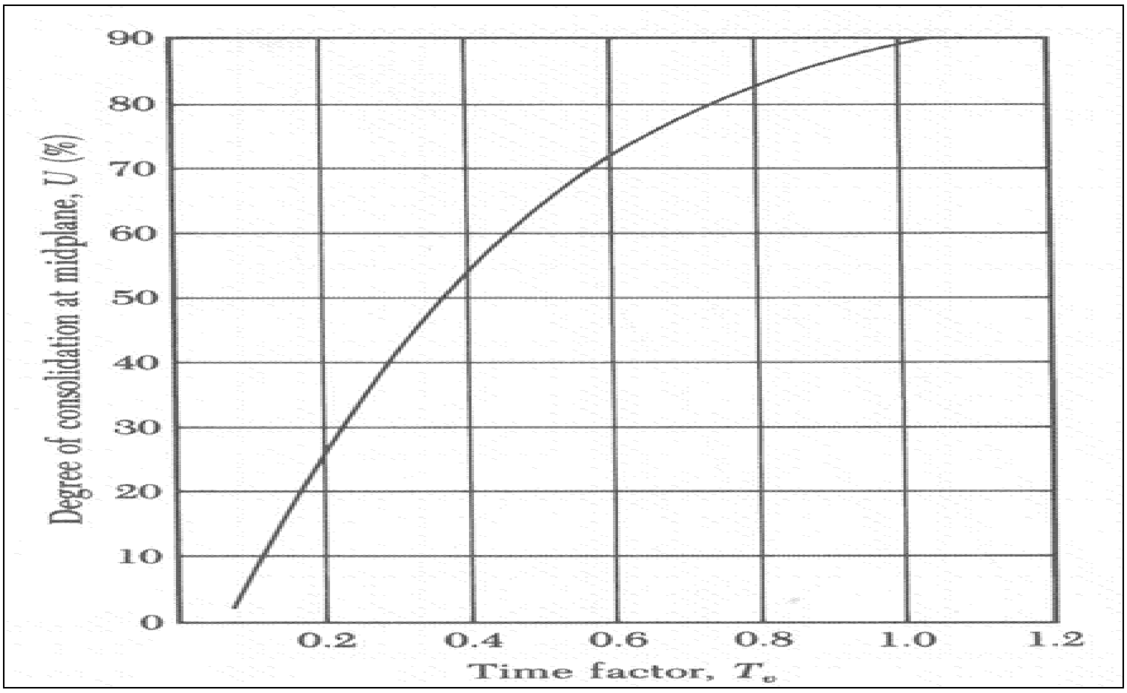



Figure 5.6 Mid plane degree of consolidation against $T_v$ (after Das, 2004)

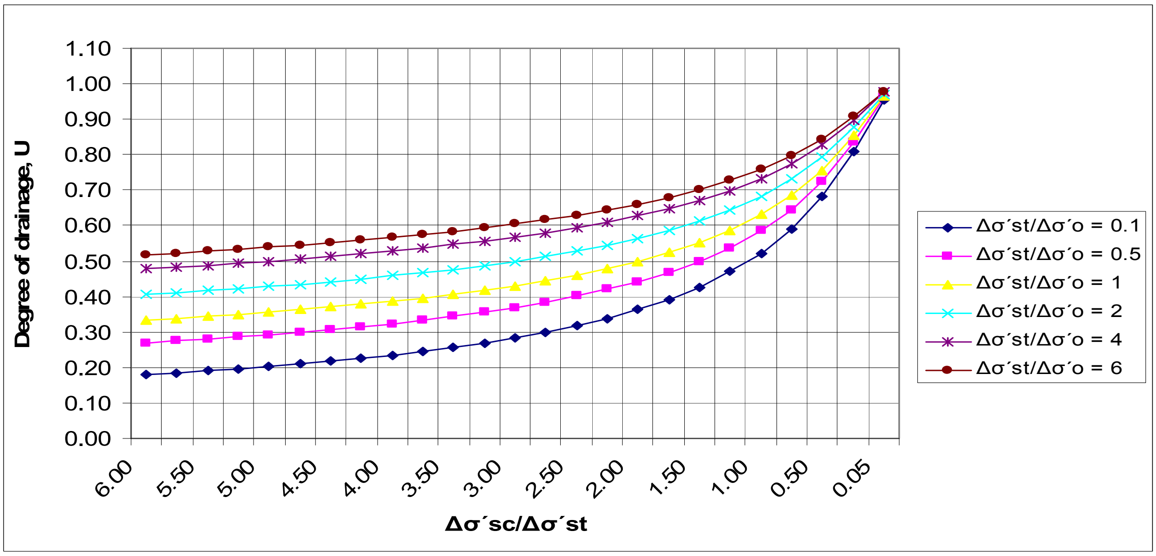



Figure 5.7 $\Delta\sigma'_{sc}/\Delta\sigma'_{st}$ against $U$ for different values of $\Delta\sigma'_{st}/\Delta\sigma'_{o}$

### 5.2.1. Suitability of Surcharge

The amount of surcharge required is a function of the depth of any soft clay layers and its drainage properties (one or two-way). Soft clay layers with one-way compared to two-way drainage require a different surcharge to achieve the same amount of consolidation in a given time (see Figure 5.8 and 5.9). For peat soil the application of a surcharge as a means of accelerating consolidation is particularly effective as the peat undergoes a significant amount of secondary compression as described in Section 2.3.3.2.

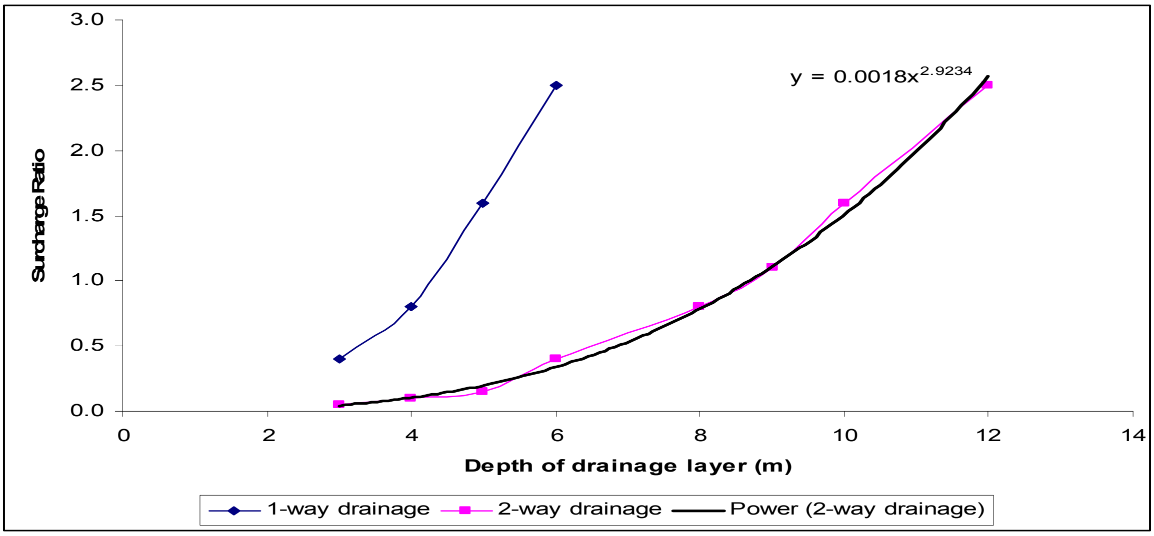



Figure 5.8 Surcharge requirements for various depths of soft clay layers for a two-year consolidation time and 3 m high embankment

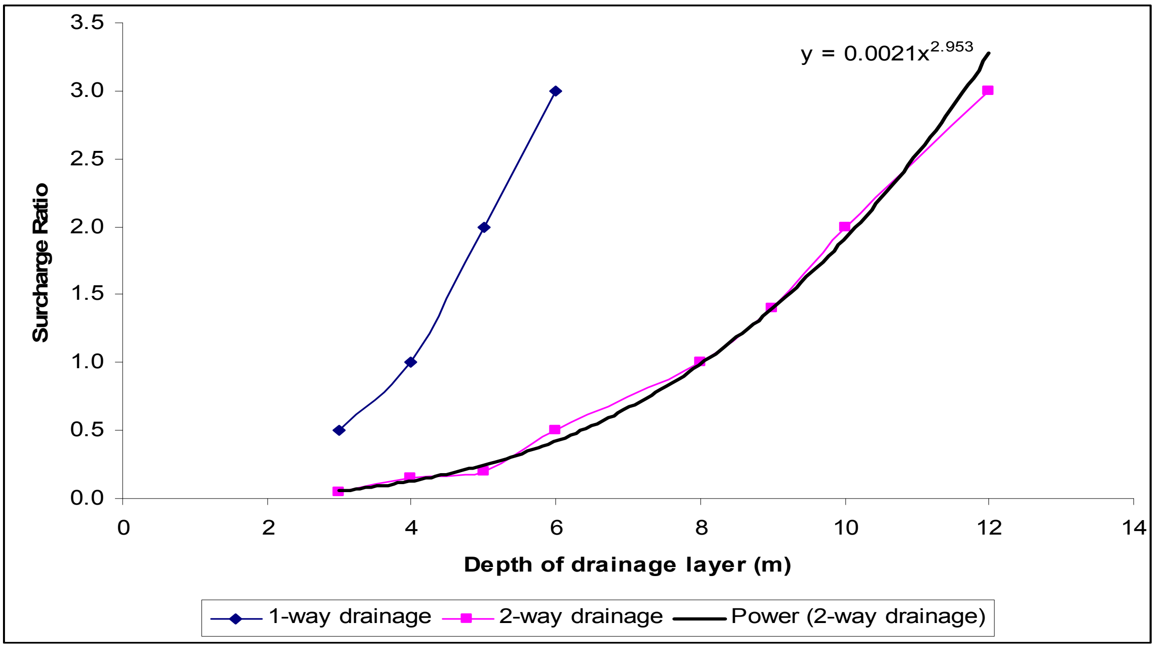



Figure 5.9 Surcharge requirements for various depths of soft clay layers for a two-year consolidation time and 5 m high embankment

## 5.3 Vertical Drains

Vertical drains contribute to subsoil drainage by adding radial drainage paths in addition to vertical ones. Two types of vertical drains are commonly used to accelerate the consolidation settlement process. These are:

- Sand drains
- Prefabricated Vertical Drains (PVD)

Sand drains are constructed by making holes and filling them with sand, in layer(s) to be consolidated. Prefabricated vertical drains (PVD) are manufactured from synthetic polymers. The installation process of a PVD is much faster than for a sand drain as it does not require drilling. Typically they can be installed at a rate of 0.1 to 0.3 m/sec and as a result are becoming more popular than sand drains (Das, 2004).

### 5.3.1. Design of Vertical Drain

Several formulae have been developed for the design of vertical drains and their spacing which take into account both instantaneous and ramp loading.

#### 5.3.1.1. Barron-Kjellman Formula

Barron-Kjellman developed a formula for consolidation time due to radial drainage which takes into account the zone of influence, drain diameter and the degree of drainage (Kyfor et al., 1988).

$$t_h = \frac{D_e^{\,2}}{8C_h}\left(\frac{\ln(D_e / d_w)}{1-(d_w / D_e)^2} - \frac{3-(d_w / D_e)^2}{4}\right)\ln\frac{1}{1-U_r} \qquad 5.4$$

Where, $t_h$ = time required to achieve $U_r$, $D_e$ = diameter of the effective zone of drainage, $d_w$ = effective diameter of the PVD, $C_h$ = coefficient of consolidation in horizontal (radial) direction and $U_r$ = average degree of consolidation by radial drainage.

#### 5.3.1.2. Hansbo Formula

Hansbo (1981) developed the following equation for the degree of radial drainage to determine the drain spacing (*S*) for under conditions when the load may be considered to be applied instantaneously.

$$U_h = 1-\exp\left(-\frac{8C_h t}{\mu D_e^2}\right) = 1-\exp\left(-\frac{8}{\mu}T_h\right) \qquad 5.5$$

where, $\mu = \ln\left(\frac{D_e}{d_s}\right) + \frac{k_h}{k_s}\ln\left(\frac{d_s}{d_w}\right) - \frac{3}{4} + \pi z(2l-z)\frac{k_h}{q_w}$ 5.6

$\mu$ is parameter account for well resistance and smear effect as defined in Section 2.5.4.3 and 2.5.4.4.

The drainage time (***t***) considering well resistance and smear effect can be given as follows

$$t = \frac{D_e^2}{8C_h}\left[\ln\left(\frac{D_e}{d_s}\right) + \frac{k_h}{k_s}\ln\left(\frac{d_s}{d_w}\right) - \frac{3}{4} + \pi z(2l-z)\frac{k_h}{q_w}\right]\ln\frac{1}{1-U_r} \qquad 5.7$$

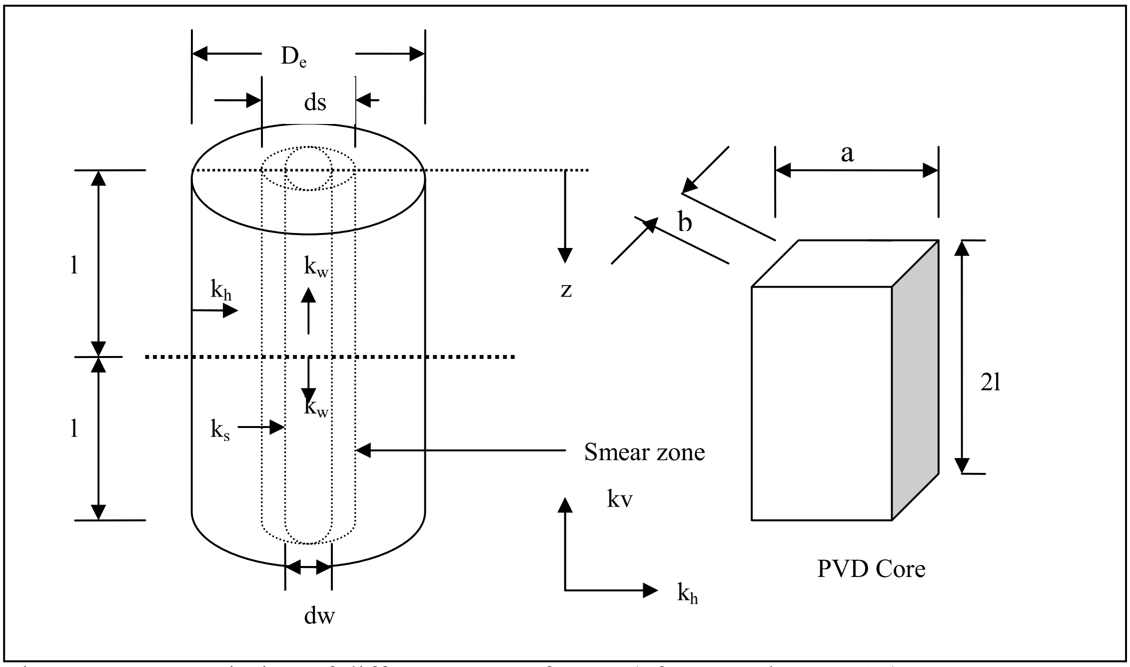



Figure 5.10 Description of different parts of PVD (after Hansboo, 1997)

Without well resistance Equation 5.6 becomes

$$\mu = \ln\left(\frac{n}{s}\right) + \frac{k_h}{k_s}\ln(s) - \frac{3}{4} \qquad 5.8$$

If the smear effect is neglected, μ can be approximated as

$$\mu = \ln(n) - \frac{3}{4} + \pi z(2l - z)\frac{k_h}{q_w} \qquad 5.9$$

The value of *t* for no smear then can be found directly from the following equation.

$$t = \frac{D_e^2}{8C_h}\left[\ln(n) - 0.75 + \pi z(2l - z)\frac{k_h}{q_w}\right]\ln\frac{1}{1 - U_r} \qquad 5.10$$

If both well resistance and smear effect are neglected the value of μ becomes

$$\mu = \ln(n) - \frac{3}{4} \qquad 5.11$$

Neglecting both well resistance and the smear effect t is given by:

$$t = \frac{D_e^2}{8C_h}\left[\ln(n) - 0.75\right]\ln\frac{1}{1-U_r} \qquad 5.12$$

Where, $d_w$ = equivalent drain diameter for prefabricated vertical drain (defined in Section 2.5.4.1), $d_s$ = diameter of smear zone, $2l$ = depth of the vertical drain, $z$ = depth coordinate, $k_h$ and $k_s$ and are hydraulic conductivity of natural soil and smear zone respectively, $k_w$ = permeability of drain materials in vertical direction, $q_w = A_w k_w$ = specific discharge capacity of vertical drain. Specific discharges for various types of drains were given in Table 2.4, $A_w$ = cross-sectional area of vertical drain and

$$n = \frac{D_e}{d_w} \qquad s = \frac{d_s}{d_w} \qquad 5.13$$

The spacing of drains is a function of placing pattern and can be determined form the Equation 5.14 and 5.15 for triangular and rectangular pattern respectively.

$$S = D_e / 1.05 \qquad 5.14$$

$$S = D_e / 1.128 \qquad 5.15$$

The average degree of consolidation ($U$) of PVD improved subsoil can be estimated using Carrillo's equation (Carrillo, 1942; cited in Chai et al., 2001 and Craig, 2005) as follows:

$$U = 1 - (1 - U_v)(1 - U_r) \qquad 5.16$$

Where $U_v$ and $U_r$ are the average degree of consolidation due to vertical and horizontal (radial) drainage and $U$ is the combined degree of drainage.

#### 5.3.1.3. Olson's Method

To take into account the effect of construction time ($t_c$), Olson (1977) developed a set of equations which considers the embankment to be constructed over time (i.e. ramp loading). These are as follows:

For $T_r \leq T_{rc}$ $$U_r = \frac{T_r - \frac{1}{A}\left[1 - \exp\left(-AT_r\right)\right]}{T_{rc}} \qquad 5.17$$

For $T_r \geq T_{rc}$ $$U_r = 1 - \frac{1}{AT_{rc}}\left[\exp\left(AT_{rc}\right) - 1\right]\exp\left(-AT_r\right) \qquad 5.18$$

where $$A = \frac{2}{\mu} \qquad 5.19$$

The value of μ can be given as

$$\mu = \frac{n^2}{n^2 - s^2}\ln\frac{n}{s} + \frac{k_h}{k_s}\left(\frac{n^2 - s^2}{n^2}\right)\ln\left(s\right) - \frac{3}{4} + \frac{s^2}{4n^2} \qquad 5.20$$

For the no smear case $$\mu = \frac{n^2}{n^2 - 1}\ln(n) - \frac{3n^2 - 1}{4n^2} \qquad 5.21$$

The time factor, $T_h$ and $T_v$ for horizontal and vertical drainage respectively are given by the following equations

$$T_r = T_h = \frac{C_h t}{D_e^2} \qquad 5.22$$

$$T_v = \frac{C_v t}{H^2} \qquad 5.23$$

Where $T_{rc}$ stands for factor for construction time, $C_h$ and $C_v$ are coefficients of consolidation for horizontal and vertical direction, $D_e$ = diameter of a unit cell, $d_w$ = equivalent diameter of the drain, $d_s$ = diameter of smear zone; and, $k_h$ and $k_s$ and are the hydraulic conductivity of natural soil and the smear zone respectively.

#### 5.3.1.4. Yeung's Method

Yeung (1997) developed several curves for designing prefabricated vertical drains which are based on Equation 5.5. His proposed procedure is given by Equation 5.24 to 5.26 below and summarised in Figure 5.12.

$$U_r = 1 - \exp\left(-\frac{8C_h t}{d_w^2}\frac{d_w^2}{D_e^2 \mu}\right) = 1 - \exp\left(-\frac{8T_r'}{\alpha'}\right) \qquad 5.24$$

$$T_r' = -\frac{\alpha'}{8}\ln(1 - U_r) \qquad 5.25$$

or $\frac{T_r'}{\alpha'} = -\frac{\ln(1-U_r)}{8} = (T_r')_1$ or $\alpha' = \frac{T_r'}{(T_1')_1}$ (Assume), where $T_r' = \frac{C_h t}{d_w^2}$

$$\alpha' = \left(\frac{D_e}{d_w}\right)^2 \mu = n^2 \mu = \frac{n^4}{n^2 - s^2}\ln\left(\frac{n}{s}\right) - \left(\frac{3n^2 - s^2}{4}\right) + \frac{k_h}{k_s}(n^2 - s^2)\ln s \qquad 5.26$$

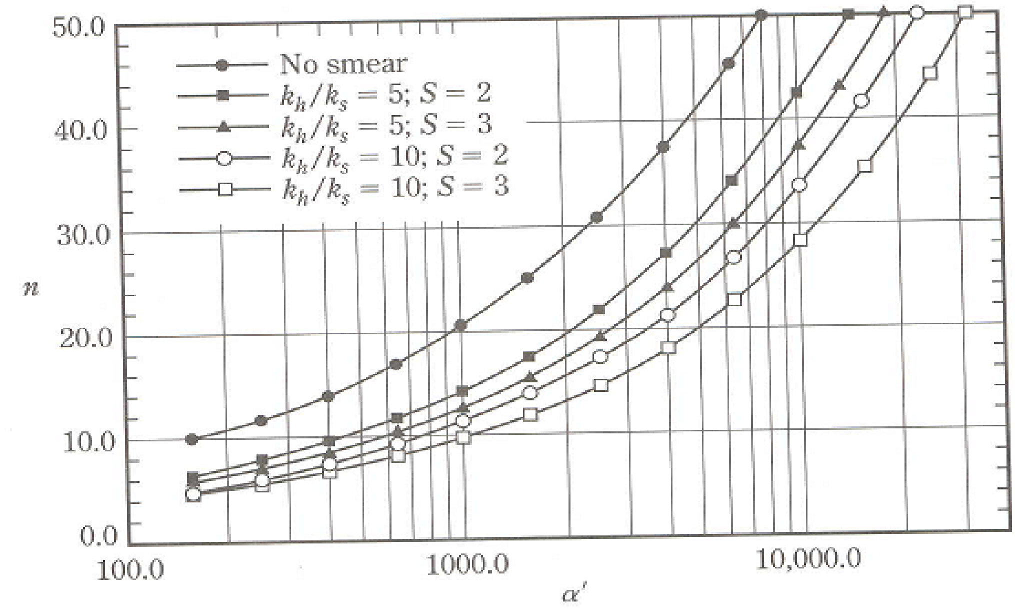



Figure 5.11 Relation between n and α′ (after Yeung, 1997)

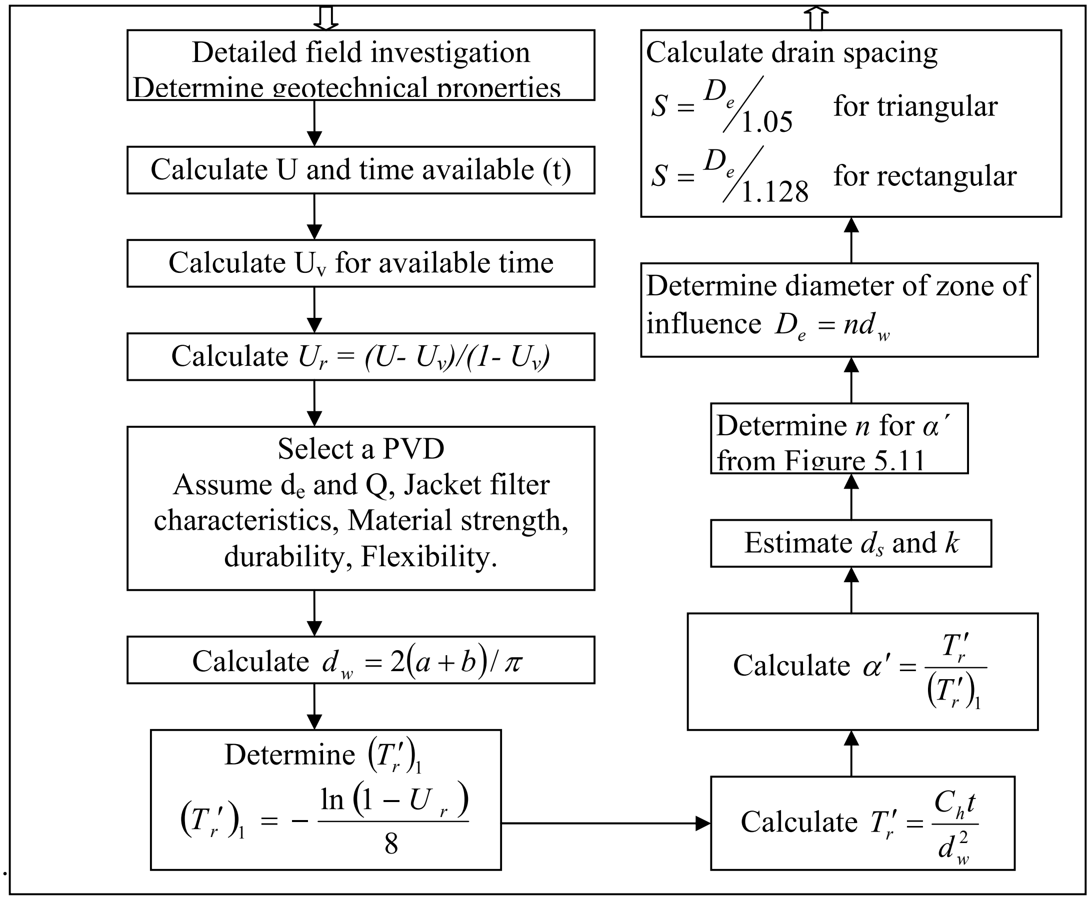



Figure 5.12 Flow chart for the design of PVD (Yeung's method)

### 5.3.2. Comparison of PVD and the Application of Surcharge

A comparative study of using PVD and a surcharge to increase the rate of consolidation was carried out and the results are shown in Figure 5.13. The application of 1m and 2 m surcharges are compared with the use of PVD with a spacing of 1.8 m. Data were used from an existing embankment in Bangladesh, the Tongi-Ashulia Road. Settle 3D (described in Chapter 6) was used to perform the analysis. From Figure 5.13 it can be seen that maximum consolidation settlement is achieved after 30 months by PVDs. The financial costs including all items (labour, machine and materials) of applying the two different methods is given in Table 5.1. This shows that the cost of PVD compares favourably with the application of a surcharge. Whilst this demonstrates that the use of PVDs may be more effective in some circumstances than the application of a surcharge, it should be borne in mind that a surcharge is particularly effective for peat soil foundations to achieve rapid secondary compression in addition to primary consolidation settlement.

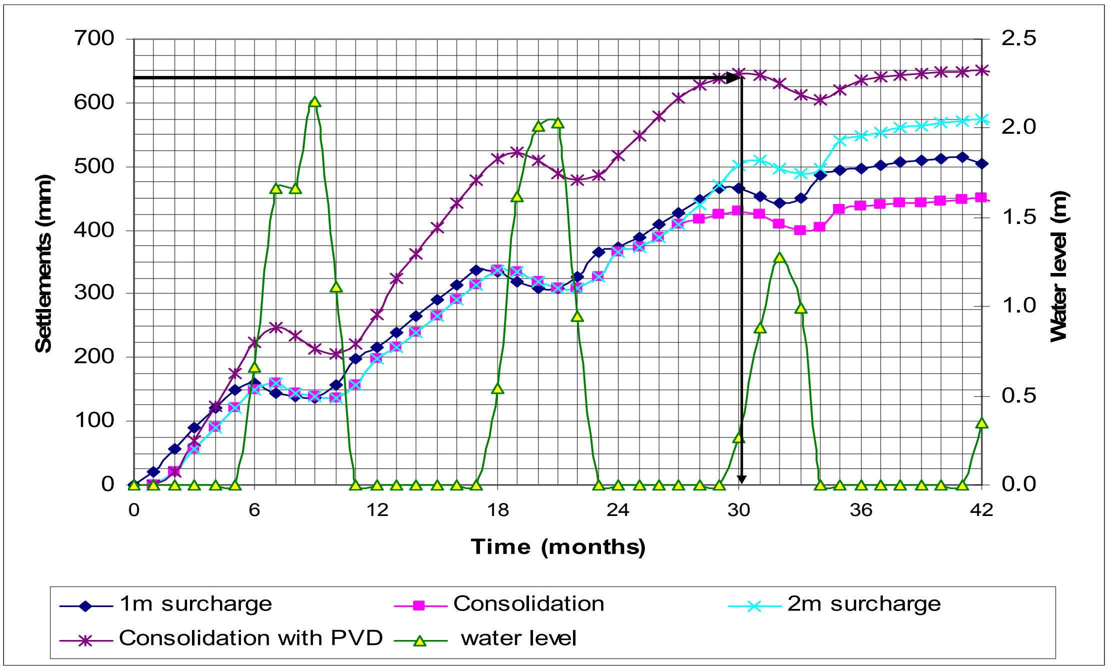



Figure 5.13 Predicted consolidation settlements with surcharge and PVD: an example of Tongi-Ashulia Road.

Table 5.1 Comparison between cost of PVD and surcharge for 1km of embankment of 3.5 m height

| **Methods** | **Cost (BDT)** | **Surcharge height/ PVD spacing** |
|---|---|---|
| Surcharge | 6,124,400.00 | 2m surcharge height |
| | 9,223,200.00 | 3m surcharge height |
| PVD | 9,825,000.00 | Spacing 1.8m square |

## 5.4 Design of Horizontal Drainage Blanket

The performance of a drainage blanket depends on the transmissibility of the drainage section and the permeability of materials used to the drainage blanket. According to Darcy's law the average discharge velocity is given by the following equation:

$$v = ki \qquad 5.27$$

Equation 5.27 can be written as follows:

$$Q = kAi \qquad 5.28$$

Where, $Q$ = $q_w$ = minimum discharge capacity of a drain = $5k_h l_w$, $k_h$ = permeability of soft soil in horizontal direction, $l_w$ = length of vertical drain, $S$ = spacing of vertical drains, $t$ = thickness of horizontal drainage blanket, $A$ = cross-sectional area of blanket = $S \times t$, $i$ = hydraulic gradient = $t/(W/2)$, $W$ = bottom width of embankment, $k$ = permeability of drainage blanket materials.

Replacing the term $A$ and $i$ in Equation 5.28, minimum thickness of a horizontal drainage blanket, $t$ is given by the following equation:

$$t = \sqrt{\frac{QW}{2kS}} \qquad 5.29$$

Typically, $Q = 3.88\times10^{-3}$ m$^3$/day, $W$ = 14.1m, $k$ = permeability of medium sand =$10^{-3}$ to $10^{-4}$ cm/sec (Alam, 2004) = 0.47 m/day, $S$ = 1.5 m.

Hence

$$t = \sqrt{\frac{3.88\times10^{-3}\times14.1}{2\times0.47\times1.5}} = 0.196m \approx 200mm$$

Woodward (2005) proposes that a drainage blanket of thickness 300-500mm should be used to collect the vertical drain discharge.

## 5.5 Summary

Prefabricated vertical drain is the most suitable method of accelerating settlement considering economy and rapid project execution. However surcharge is very much effective in pre-compression for foundation containing peat soil. Combination of two also gives good results. In this Chapter it is found that Carrillo's equation in association with Hansbo's method and Olson's ramp loading is suitable for the analysis of radial consolidation. In the next Chapter the behaviour of settlement under various loading conditions will be modelled.

# Chapter 6 Settlement Modelling

## 6.1 Introduction

The previous two chapters have described various techniques to determine the settlement of the foundation under an embankment load. Chapter 4 identified two methods considered worthy of further consideration for the conditions found in Bangladesh. These are the finite difference technique implemented in some available software, known as Settle3D and a combination of methods, termed the Standard method. The former technique is recommended as the first choice; however it is practically implemented via commercially available software. In the event of such software being unavailable (and unsustainable) in Bangladesh the standard method is also recommended for consideration. To demonstrate the suitability and accuracy of the methods, this chapter compares the results obtained using both methods for a number of real and hypothetical cases.

As mentioned previously a research project is being undertaken in parallel to this one to investigate suitable techniques which may be used in Bangladesh to determine values of various properties of materials required for the design of road pavement and embankment. In order to be able to provide information to the parallel project on the material properties which may be considered to be most important for embankment design, a sensitivity analysis has been undertaken and is described in this chapter.

## 6.2 Settle3D

Settle3D is a piece of software which has been developed to analyse the settlement of foundations under the application of circular, rectangular, embankment or excavation loads (Rocscience, 2009). Settle3D calculates stresses in three dimensions using Boussinesq's equations and the 2:1 method (assumes load distribution varies with depth as 2H: 1V) and settlements in one dimension using Taylor's (1942) and Terzaghi's (1925) theories. The software is convenient for settlement analysis with prefabricated vertical drains and surcharge applications in which the input parameters are easily achievable and understandable. Time related consolidation analysis can be carried out with the model for any time units and for any hydraulic conductivity parameters. The impact of seasonal water level variation as a time series can be accommodated for time dependent settlement analysis. The model is widely used and its accuracy has been verified by ROCSCIENCE, the producer of the software. The software developers have verified the contents of Settl3D by comparing the stresses, elastic and consolidation settlement and pore pressure dissipation with time and depth with a variety of situation presented in the literature. Further detail may be found in Rocscience (2009).

## 6.3 Comparison between Settle3D and Observed Data

For this work, the settlements predicted by the software were compared with the observed values of the following embankment.

### 6.3.1. Limavady Embankment

Settle3D was used to predict the total settlement reported by Kelln et al. (2009) as occurring in an embankment constructed in Limavady, Northern Ireland. Prefabricated vertical drains of a spacing 1.3 m square were included in the model to take into account the actual field conditions. The height of the embankment was 5m; its width was 12 m at the top and 32 m at the bottom. The unit weight of fill materials was 20 kN/m$^3$. The same loading schedule (shown in Figure 6.1) which occurred in the field was used in the model. The initial void ratio and coefficients secondary compression are not given by Kelln et al. and so for modelling purposes there were assumed to be similar to those given in the literature for the same kind of soils. The elastic moduli were also assumed and the simulation was carried out for two values, 3000 kPa and 6000 kPa respectively. The parameters used to simulate the total settlement are shown in Table 6.2. The observed and predicted settlements are compared in Figure 6.2.

Table 6.1 Settlement parameters for Limavady embankment (after Kelln et al., 2009)

| Depth | $v$ | $\gamma$ | $\gamma_{sat}$ | $C_c$ | $C_r$ | $e_o$ | $C_\alpha$ | $C_{\alpha r}$ | $k_v$ | $k_{Vr}$ | $OCR$ | $C_h/C_v$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| m | | kN/m3 | kN/m3 | | | | | | (m/d) | (m/d) | | |
| 0.0-2.0 | 0.3 | 16 | 16 | 0.9212 | 0.069 | 1.50 | 0.03 | 0.03 | 0.001 | 0.001 | 3.5 | 1 |
| 2.0-3.5 | 0.3 | 16 | 16 | 0.4606 | 0.161 | 1.58 | 0.03 | 0.03 | 0.001 | 0.001 | 2.3 | 1 |
| 3.5-6.5 | 0.3 | 16 | 16 | 0.9212 | 0.069 | 1.58 | 0.03 | 0.03 | 0.001 | 0.001 | 1.5 | 1 |
| 6.5-9.0 | 0.3 | 16 | 16 | 0.7105 | 0.138 | 1.58 | 0.03 | 0.03 | 0.001 | 0.001 | 1.0 | 1 |

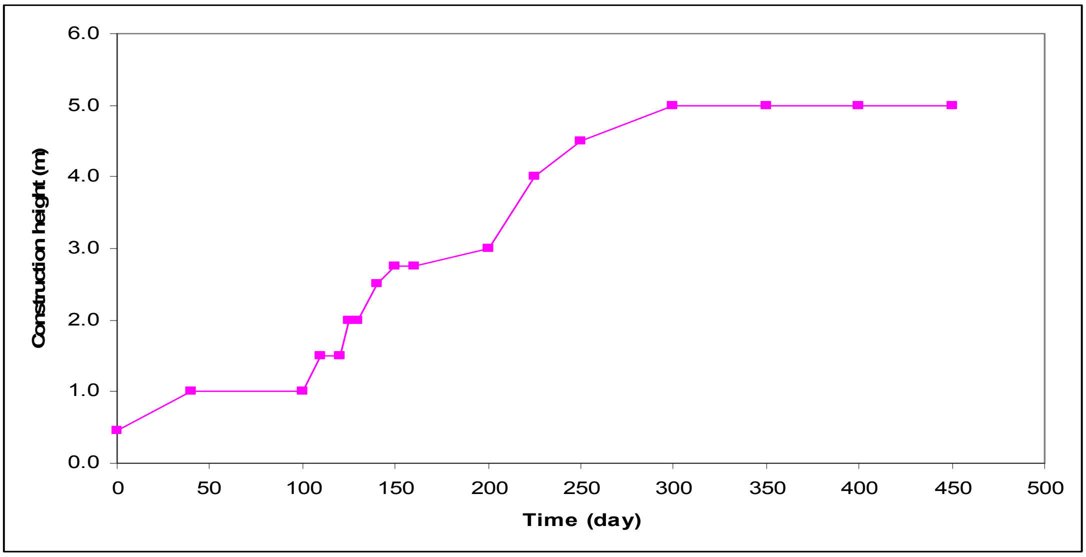



Figure 6.1 Construction schedule of Limavady embankment

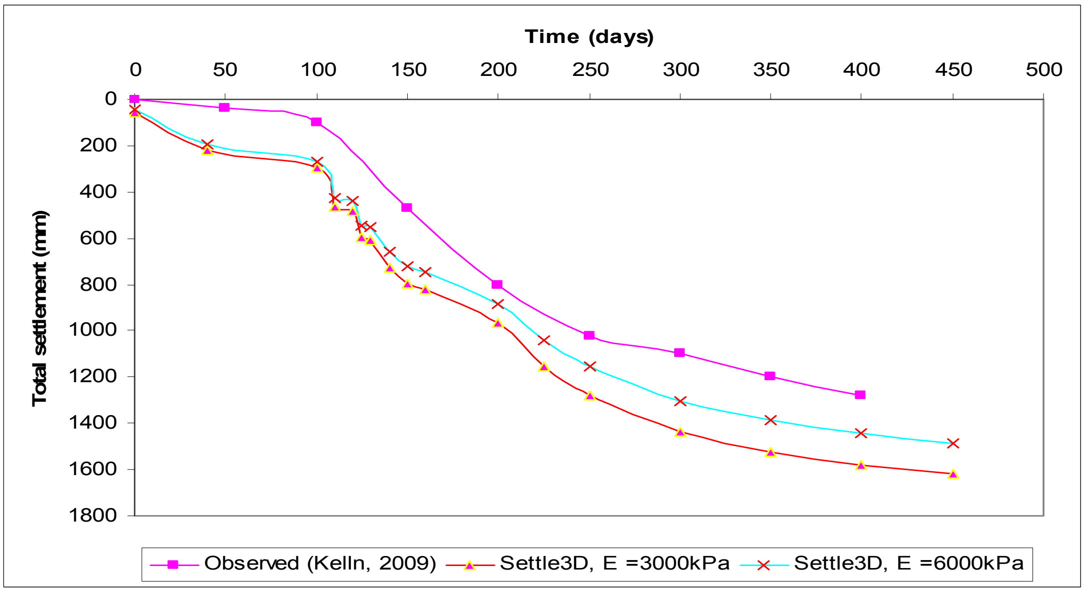



Figure 6.2 Comparison between observed and predicted settlement of Limavady embankment

**Discussion:** It is seen from the Figure 6.2 that the shape of observed and predicted time settlement curve is similar and there is reasonable agreement between them. However Settle3D consistently overestimates settlements by approximately 150mm. A paired difference test (Henry and Edward, 1964) was carried out to test the hypothesis that Settle3D overestimates settlement by an amount 150mm. It found that the mean difference between the settlements calculated by each method was 148mm at a five percent level of significance. This may be a result of the assumed void ratio and secondary compression parameters.

## 6.4 Comparison between Settle3D and Standard Method

The proposed standard method was also compared with settle3D for total settlement, time dependent primary consolidation, degree of settlement and settlement with depth after a particular period. Comparisons were also made for the inclusion of prefabricated vertical drains. Predictions of total settlements were carried using the Limavady embankment data as given in Table 6.1. The other comparisons were carried out using data collected from the Tongi-Ashulia road in Bangladesh as given in Table 6.2. The associated boundary conditions are given in Figure 6.3 and 6.4.

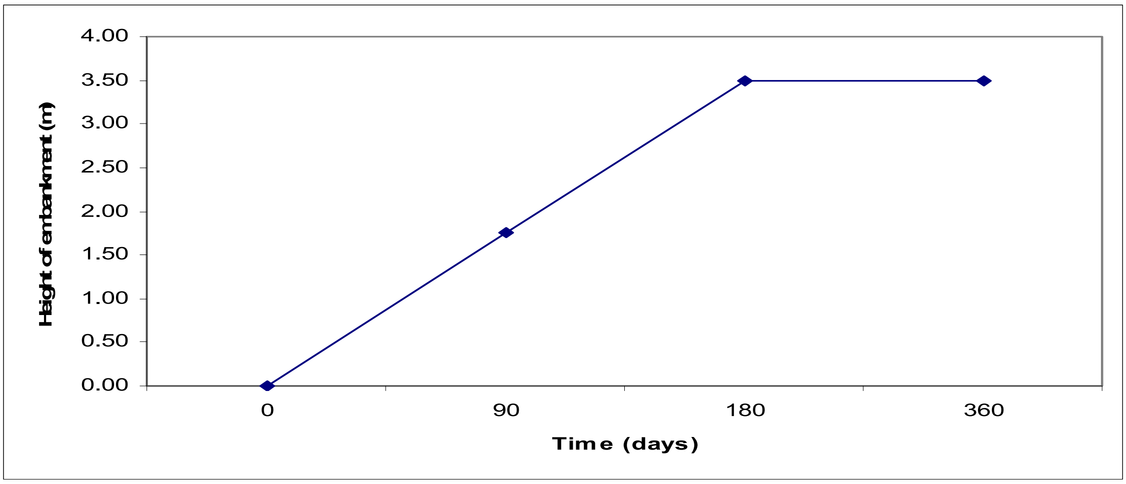



Figure 6.3 Assumed embankment construction schedule for the analyses

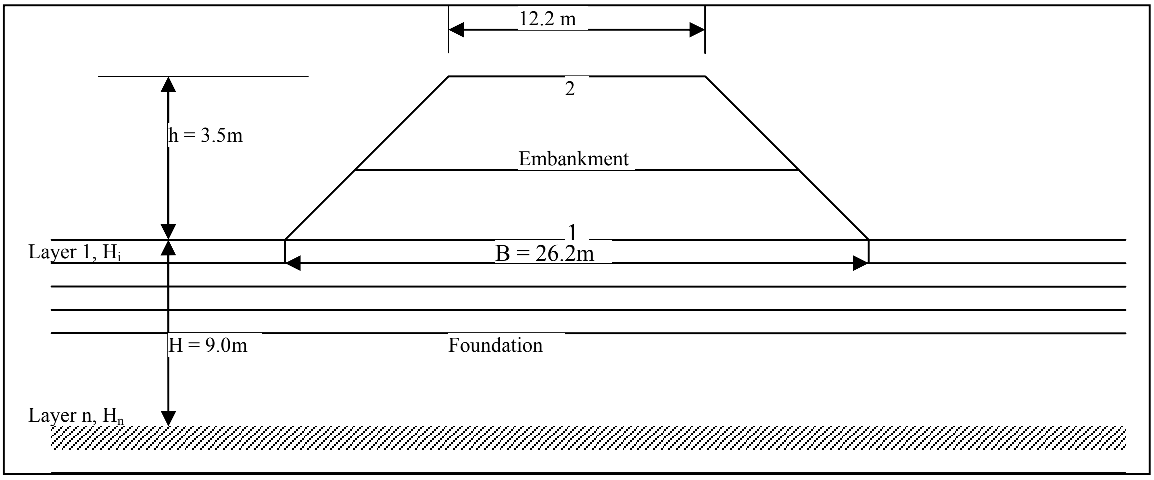



Figure 6.4 Embankment geometry and foundation layers assumed for the analyses

Table 6.2 Parameters used for the comparison between Settle3D and Standard method

| ***Depth*** | $E_s$ | $E_{ur}$ | $\gamma$ | $\gamma_{sat}$ | $C_c$ | $C_r$ | $e_o$ | $C_v$ | $C_{vr}$ | $C_\alpha$ | $C_{\alpha r}$ | ***OCR*** | $C_h/C_v$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| m | kPa | kPa | kN/m3 | kN/m3 | | | | $m^2$/day | $m^2$/day | | | | |
| 0.0-1.0 | 3000 | 9000 | 18.38 | 18.38 | 0.391 | 0.135 | 1.08 | 0.0022 | 0.0022 | 0.0037 | 0.0037 | 1.0 | 2 |
| 1.0-2.0 | 3000 | 9000 | 15.08 | 15.08 | 0.311 | 0.109 | 1.10 | 0.0033 | 0.0033 | 0.0031 | 0.0031 | 1.0 | 2 |
| 2.0-3.0 | 3000 | 9000 | 15.08 | 15.08 | 0.330 | 0.120 | 0.89 | 0.0013 | 0.0013 | 0.0610 | 0.0610 | 1.0 | 2 |
| 3.0-4.0 | 3000 | 9000 | 15.08 | 15.08 | 0.376 | 0.132 | 1.26 | 0.0700 | 0.0700 | 0.0020 | 0.0020 | 1.0 | 2 |
| 4.0-9.0 | 3000 | 9000 | 19.08 | 19.08 | 0.270 | 0.069 | 3.09 | 0.0194 | 0.0194 | 0.0490 | 0.0490 | 1.0 | 2 |

### 6.4.1. Total Settlement

The total settlement predictions for the Limavady embankment (as described in Section 6.3.1) using the standard method and Settle3D are shown in Figure 6.5.

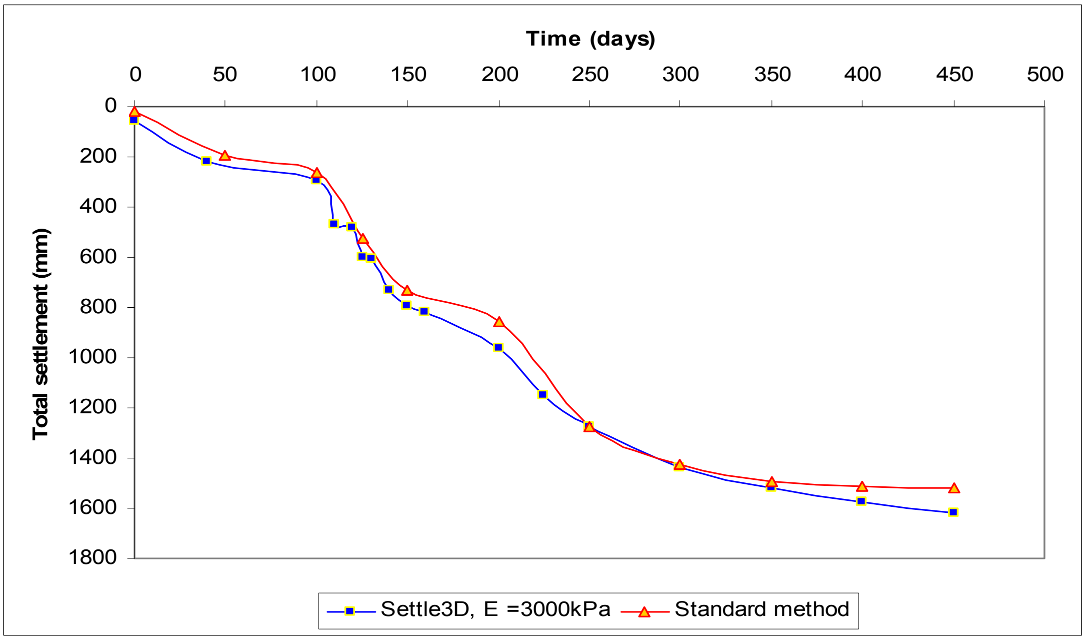



Figure 6.5 Comparison between total settlements predicted by standard method and Settle3D of Limavady embankment

The shape of time-settlement curve is similar and by inspection there is reasonable agreement between the results predicted by both methods. However the settlement predicted by Settle3D is greater by approximately 50mm. A paired difference test was carried out which shows that the settlements calculated by Settle3D are on an average 45mm more than standard method at a five percent level of significance.

### 6.4.2. Consolidation Settlement without Vertical Drain

The time related primary consolidation and degree of settlements computed by Settle3D and the standard method are shown in Figures 6.6 and 6.7 respectively.

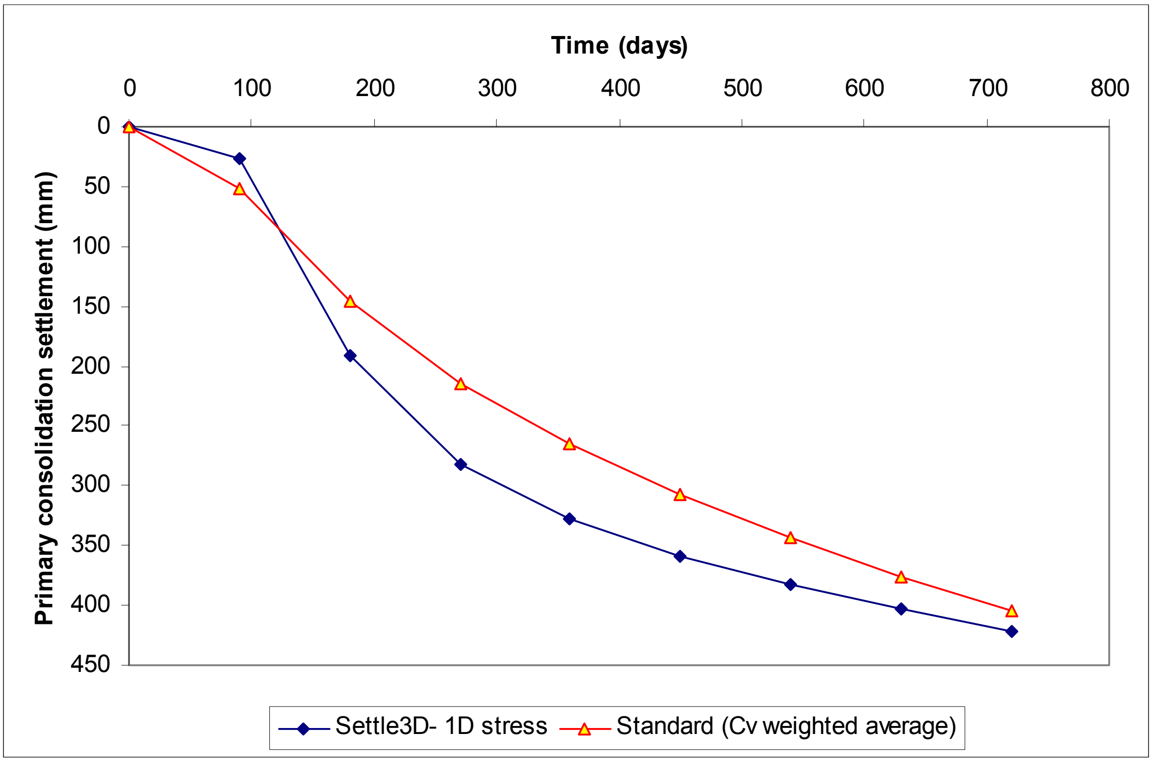



Figure 6.6 Time related primary consolidation without PVD

From the Figure 6.6 it may be seen that Settle3D gives higher amounts of primary consolidation settlements after 100 days by approximately 50mm although it gives a lower result before that time period. The minor deviations occurred due to assumptions of weighted average value of $C_v$. In the standard method the value of $C_v$ was assumed as a depth-weighted average where as in Settle3D the exact values of $C_v$ for each layer were used. The degree of consolidation determined using the two methods is shown in Figure 6.7. This figure shows similar results to that for the analysis of primary consolidation settlement above. Namely the values determined using Settle3D is higher than with the standard method.

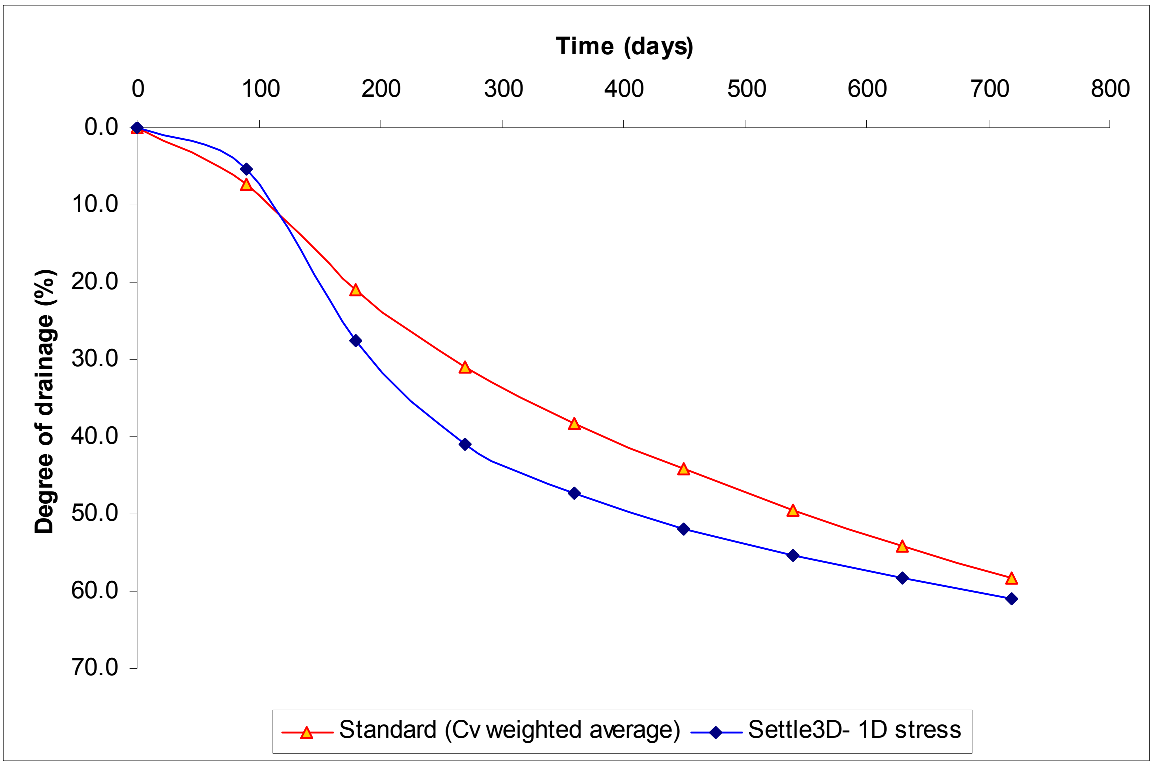



Figure 6.7 Degree of consolidation without PVD

### 6.4.3. Consolidation Settlement with Vertical Drains

The primary consolidation settlement with prefabricated vertical drains was modelled with two methods and the results are shown in Figure 6.8, 6.9 and 6.10. In these analyses both the well resistance and smear effect were neglected. The properties of the prefabricated vertical drains used in the analysis are given in Table 6.3.

Table 6.3 Dimensions and spacing of PVD used in the comparison

| **Dimensions** | **Notation** | **Unit** | **Quantity** |
|---|---|---|---|
| Thickness of core | *a* | mm | 3 |
| Width of core | *b* | mm | 100 |
| Length of PVD | *2l* | m | 9 |
| Spacing of PVD | *S* | m | 1.75 |
| Type of spacing | | | Square |
| Installation time | | day | 0 (before start of construction) |

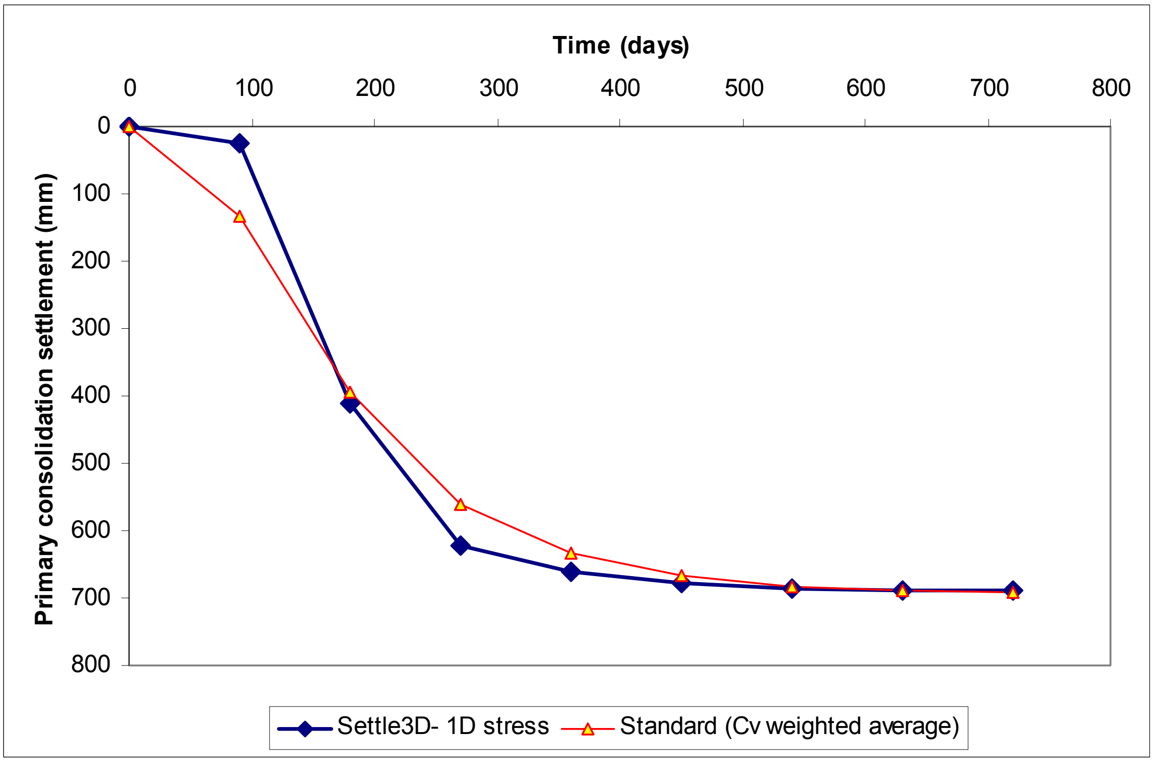



Figure 6.8 Time related primary consolidation settlement with PVD (S =1.75m, square)

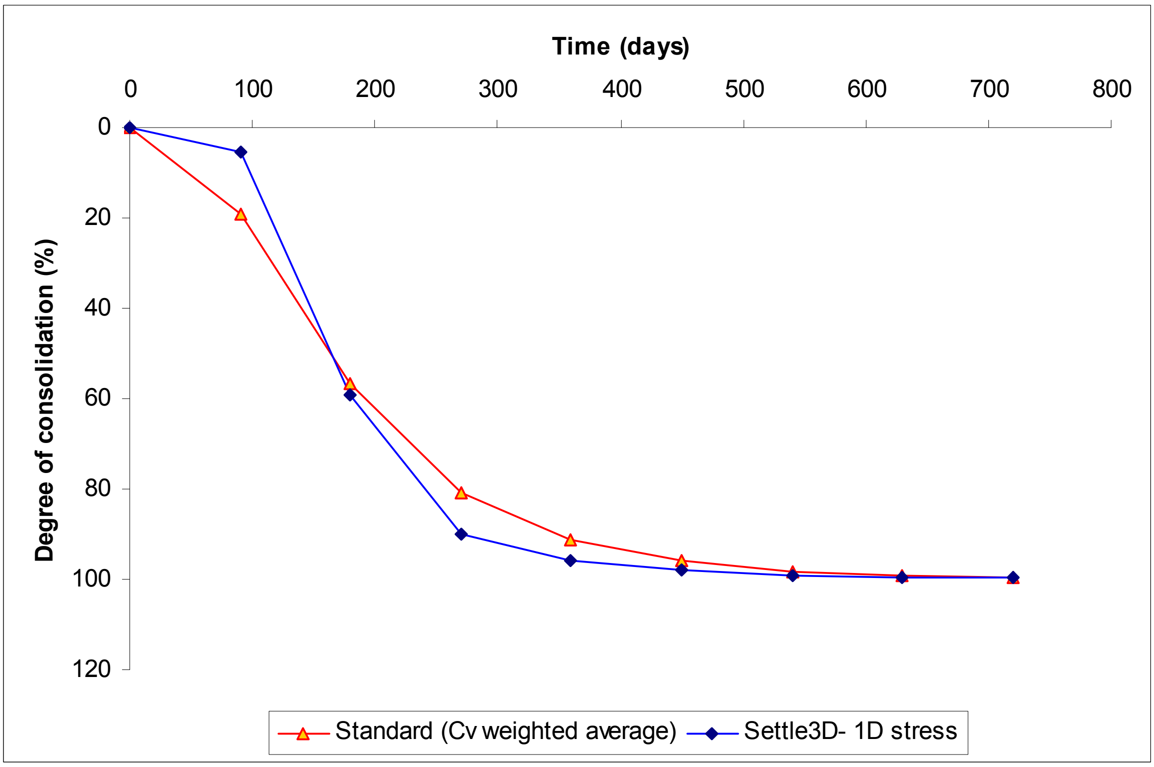



Figure 6.9 Degree of consolidation with PVD (S = 1.75m, square)

From the Figure 6.8, it can be seen that the two methods show reasonable agreement in the predicted amount of consolidation before 150 days but after that it gives lower result until 500 days. After 500 days both the methods show good agreement. A paired difference test was carried out and found that Settle3D predicted on average 7mm more primary settlement than that of the Standard method.

As mentioned in the above paragraph, similar acceptable agreement observed in the prediction of degree of consolidations by the two methods (as shown in Figure 6.9). A paired difference test was also carried out in this case and showed that on average 2.5% greater degree of settlement was calculated using Settle3D than of the Standard method at a ninety five percent confidence level. As stated earlier, the slight deviations occurred due the approximation of the $C_v$ value and by assuming all layers as a single layer in Standard method where as Settle3D calculates time related settlement for each individual layers.

Primary consolidation settlements with depth after 700 days with PVD were determined by both the methods and the results are shown in Figure 6.10. As the primary consolidation settlement in the Standard method was calculated using a compression index of individual layers similar to that of Settle3D the results shows good agreement between them.

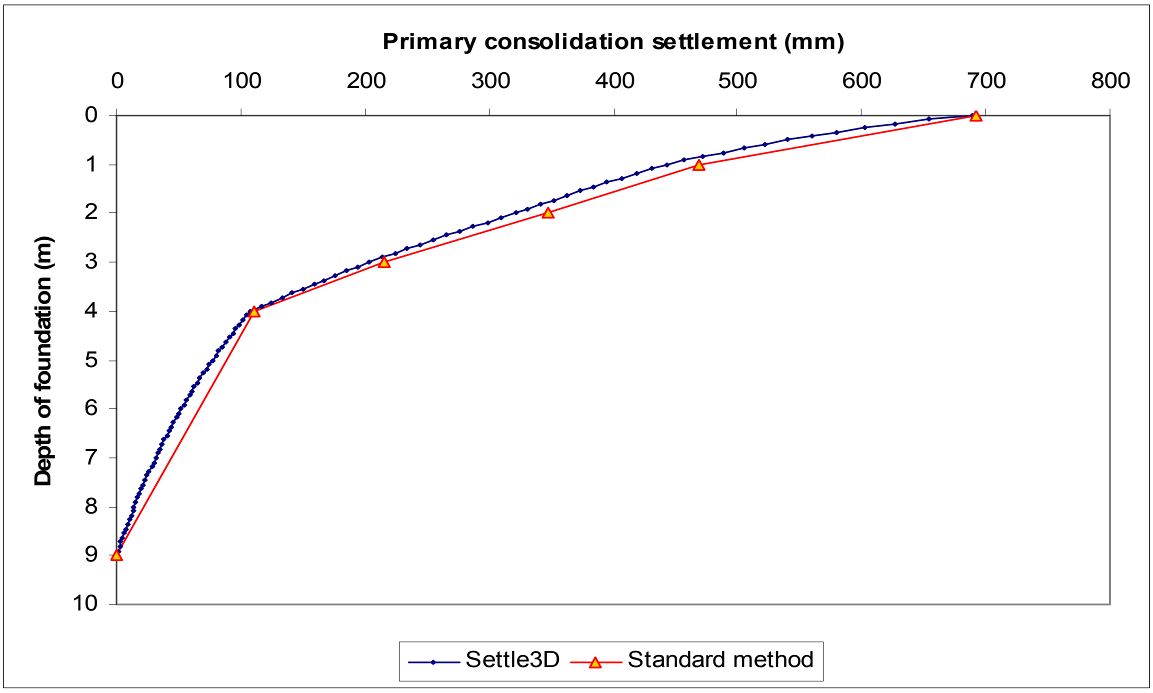



Figure 6.10 Primary consolidation settlements with depth after 720 days with PVD

### 6.4.4. Discussion

In spite of some deviations in settlement calculations at initial time stages, the Standard method shows acceptable agreement with finite difference model Settle3D for the overall settlement analysis. Therefore it is evident from the analysis stated that in the event of unavailability of finite difference software Settle3D, the proposed standard method can be used with reasonable accuracy for settlement analysis.

## 6.5 Impact of Water Level on Settlements

A parametric study has been carried out with Settle3D to understand the behaviour of settlements and pore pressures due to variation of water level above the ground surface. Parameters used for this study were collected from the virgin soil of Tong-Ashulia road and the same embankment loading was used for settlement prediction. The parameters used in this study are given in Table 6.4. It was assumed the soil was normally consolidated (OCR = 1) as the pre-consolidation pressures were not available for this study. The ratio of coefficients of consolidation in radial to vertical direction was also assumed from those available in the literature.

Table 6.4 Parameters used for settlement analysis

| Depth | $E_s$ | $E_{ur}$ | $v$ | $\gamma$ | $\gamma_{sat}$ | $C_c$ | $C_r$ | $e_o$ | $C_\alpha$ | $C_{\alpha r}$ | $C_V$ | $C_{Vr}$ | OCR | $C_h/C_v$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| m | kPa | kPa | | kN/m3 | kN/m3 | | | | | | (m2/month) | (m2/month) | | |
| | 3340 | 10020 | 0.45 | 15.34 | 17.60 | 0.391 | 0.135 | 1.19 | 0.0037 | 0.0037 | 0.06476 | 0.06476 | 1 | 2 |
| 2 | 3250 | 9750 | 0.45 | 18.45 | 18.79 | 0.311 | 0.109 | 1.13 | 0.0031 | 0.0031 | 0.09863 | 0.09863 | 1 | 2 |
| 3 | 3600 | 10800 | 0.45 | 14.69 | 14.76 | 0.330 | 0.120 | 1.11 | 0.0610 | 0.0610 | 0.04007 | 0.04007 | 1 | 2 |
| 4 | 2400 | 7200 | 0.45 | 15.39 | 15.39 | 0.376 | 0.132 | 1.35 | 0.0020 | 0.0020 | 2.19070 | 2.19070 | 1 | 2 |
| 9 | 2240 | 6720 | 0.45 | 13.69 | 13.69 | 0.270 | 0.069 | 0.94 | 0.0490 | 0.0490 | 0.58275 | 0.58275 | 1 | 2 |

### 6.5.1. Response of Stresses and Pore Water Pressures

**Case 1: Water table at ground surface:** Excess pore water pressures (EPWP) develop when the embankment load is applied and dissipate with time under consolidation. As the pore water pressure dissipates, the effective stress and degree of consolidation increases. Pore water pressure which is the sum of excess pore water pressure and static pore water pressure also decreases with the dissipation of excess pore water pressure. Total stress which is the sum of all stresses (loading, pore water and gravity) also decreases with dissipation of EPWP as shown in Figure 6.11.

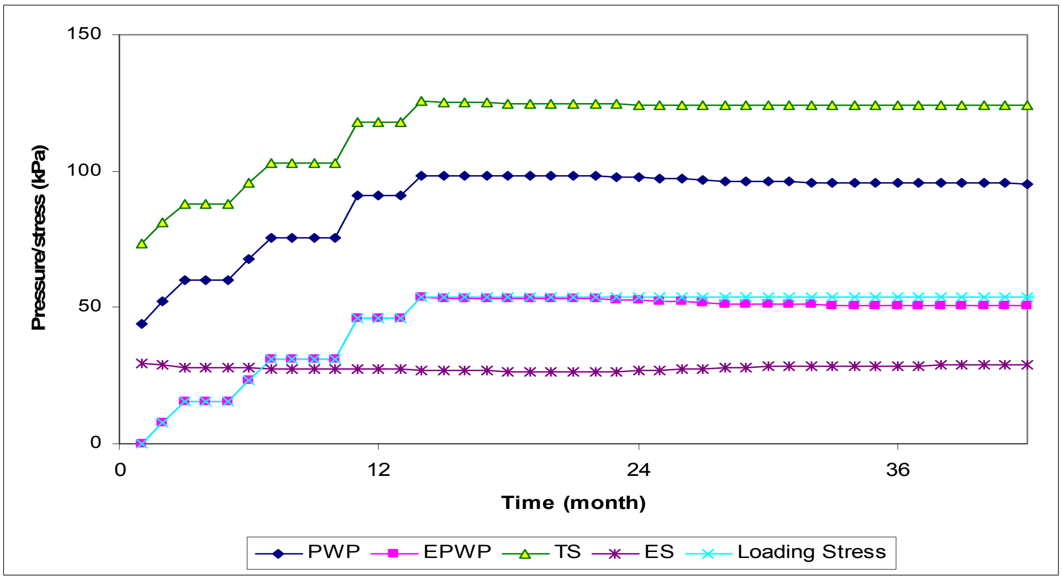



Figure 6.11 Variation of pore pressures and stresses with time and loading keeping water level at GS

**Case 2- Water level variable:** The behaviour of pore pressures and stresses acting on a soil particle in the embankment due to a variation of seasonal water level are shown in Figure 6.12.

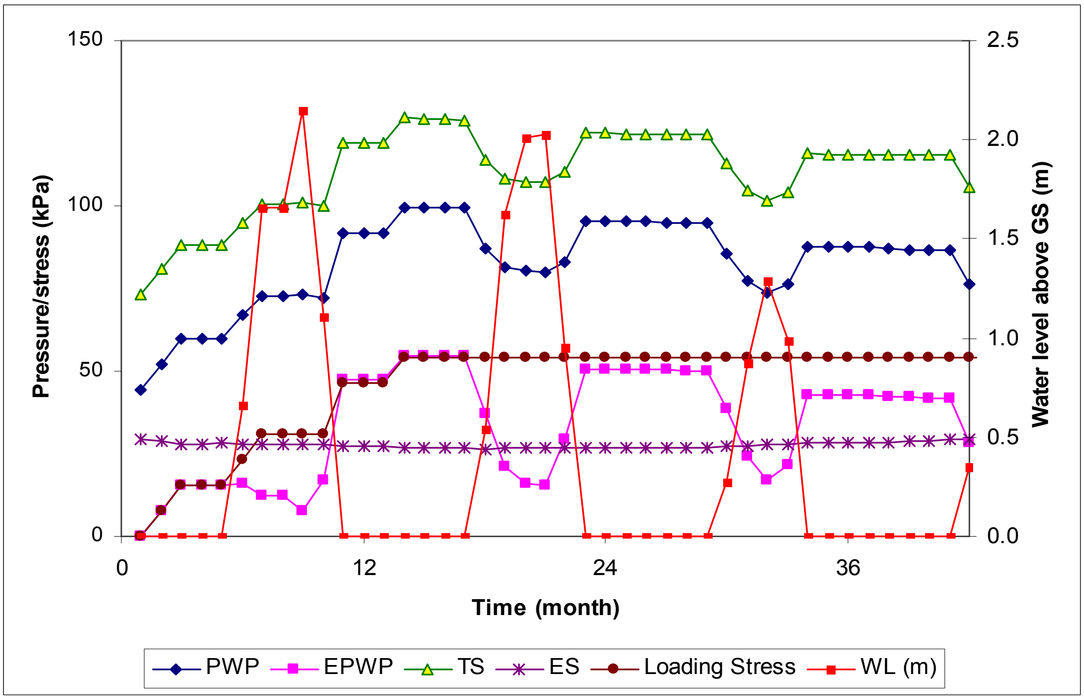



Figure 6.12 Variation of pore pressures and stresses with actual water level variation

As the water table rises, the static pore water pressure increases but the excess pore water pressure decreases due to a decrease in effective stress (as expected). Pore water pressure decreases during the rise of flood water. The total stress also decreases during the rise of flood water but increases when the flood water recedes. The pore water pressure and excess pore water pressure decreases and increases respectively as the water level rises above the ground surface and falls.

### 6.5.2. Impact on settlement

Typically, in Bangladesh the water level rises from 2 to 5 metre above the ground surface and these seasonable variations were used to perform a settlement analysis using Settle3D. Properties taken from the Tongi-Ashulia (N302) road were used for the analysis. Comparisons were made between two cases of water table locations. In first case the water table was assumed at the level of ground surface and in the second case it varied with the lowest level at the ground surface.

**Elastic settlement:**

When the water table is at the ground surface the elastic settlement occurs immediately after the application of the load. As described previously, the total stress changes with the variation of water level and this results in swelling and cycles of elastic settlement through out the life of the embankment. Although maximum portion of the elastic settlement takes place immediately after the application of load, some swelling and resettling occurs when the water table rises and falls as shown in Figure 6.13. Most of the areas in Bangladesh where the monsoon water table remains above the ground surface for two to three months in a year and has a small but prolonged impact on elastic settlement but in areas where maximum water level is at ground surface the elastic settlement takes place immediately and has no prolonged impact.

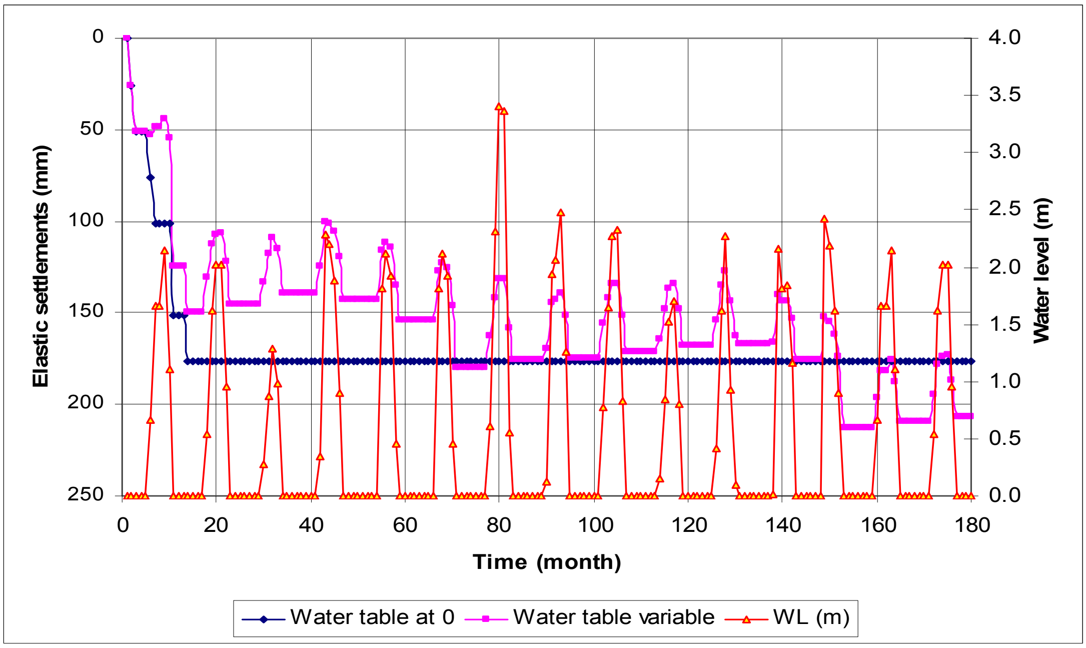



Figure 6.13 Elastic settlements with and without water table variation

**Primary consolidation:**

It has been seen from the time-settlement chart in Figure 6.14 that less primary consolidation settlement takes place when the water table remains above the ground surface compared to that keeping at ground surface. As the water level rises above the ground surface, the effective stress developed due to embankment loading decreases. It is difficult to quantify the additional time requirement to achieve the same degree of settlement keeping water table at ground surface. However, from the above analysis on average two months per year can be deducted from the actual elapsed time. So in the proposed embankment design method as described in Chapter 10, consolidation time can be adjusted to be equal to ten months instead of 12 months as a conservative measure. If the software is available to accommodate water table variation, the calculation can be done without any time adjustment.

**Primary consolidation with PVD:** It may also be seen that using PVD with a spacing of 1.8 m c/c, that the maximum settlement is achieved within 20 months, using the water table variation given above. So using this PVD spacing, construction of pavement can be started after 20 months. If more time is available for construction, the number of PVDs can be reduced by increasing their spacing thereby reducing the construction cost.

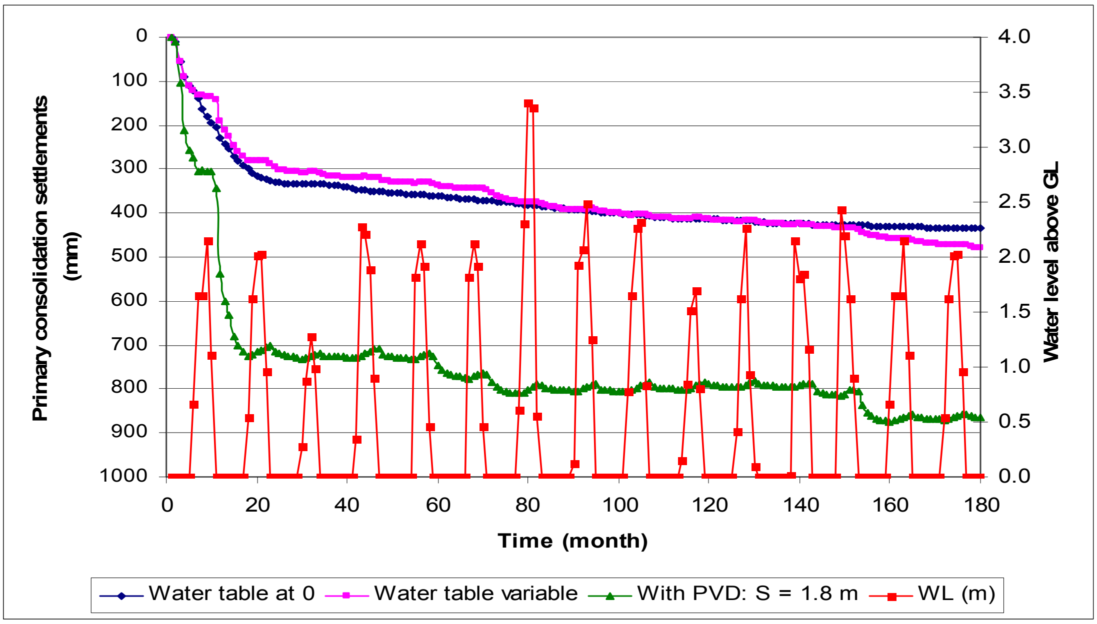



Figure 6.14 Primary consolidation settlements with and without water table variation

From the settlement-depth chart in Figure 6.15, it may be seen that after 42 months, primary consolidation settlements is slightly lower in case of actual water table above the ground surface to that of at ground surface and no settlement takes place below a depth of 2 m. When PVD is used, due to radial drainage consolidation settlement is accelerated.

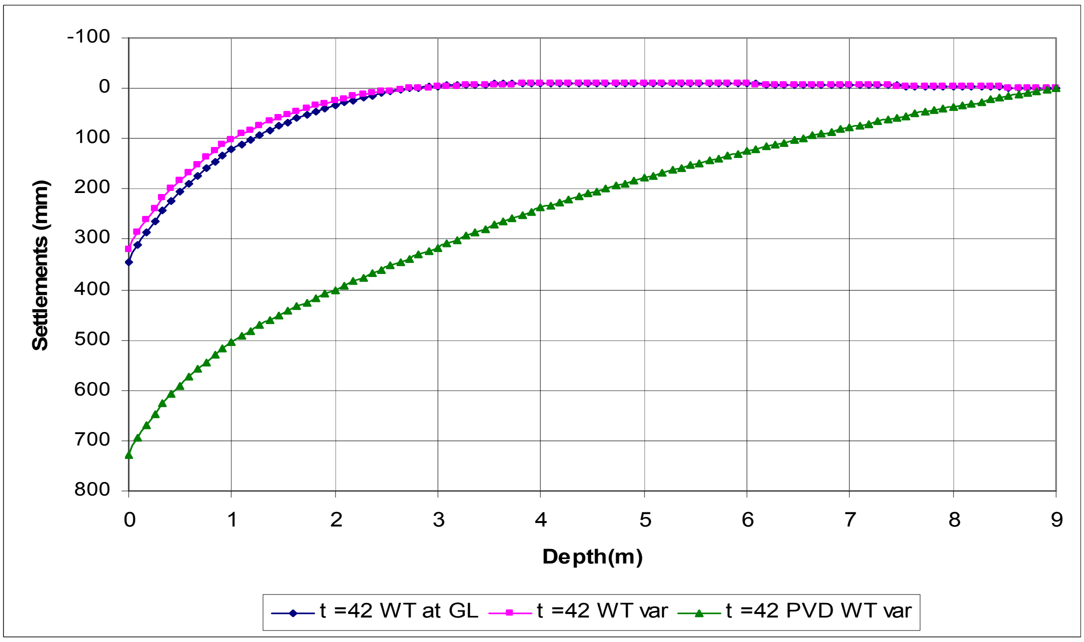



Figure 6.15 Consolidation settlements with depth after 42 months

### Secondary Consolidation:

Although the acceleration of primary consolidation by PVD has no direct impact on secondary compression, its effect indirectly influences secondary compression. It is assumed that secondary compression starts after the completion of primary consolidation and as primary consolidation is accelerated due to the use of PVDs, secondary compression starts earlier and attains a significant amount as shown in Figure 6.16.

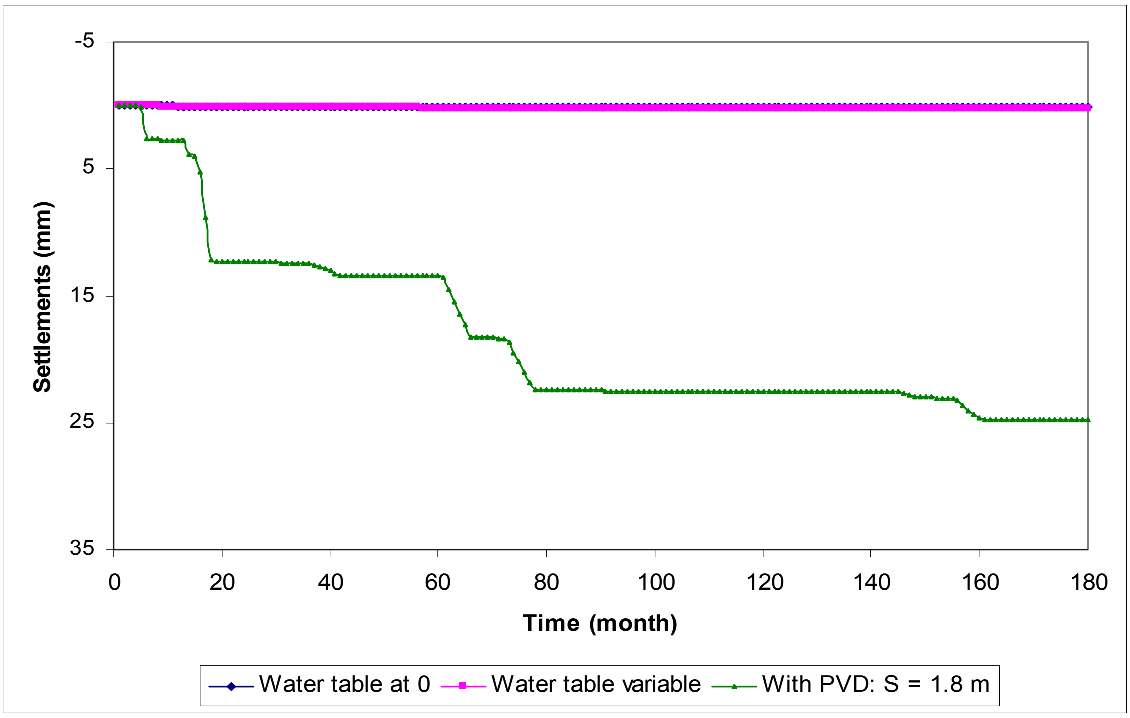



Figure 6.16 Secondary consolidation with and without water table variation

### Total Settlement

Total settlement is the sum of all settlements as explained earlier and for the analysis described here is shown in Figure 6.17. Total settlement is slightly less in case when the water table is above the ground surface compared to when it is at ground level. Using PVD the maximum total settlement is achieved within the shortest possible time as primary and secondary compression accelerates in spite of the rise in the water level above the ground surface.

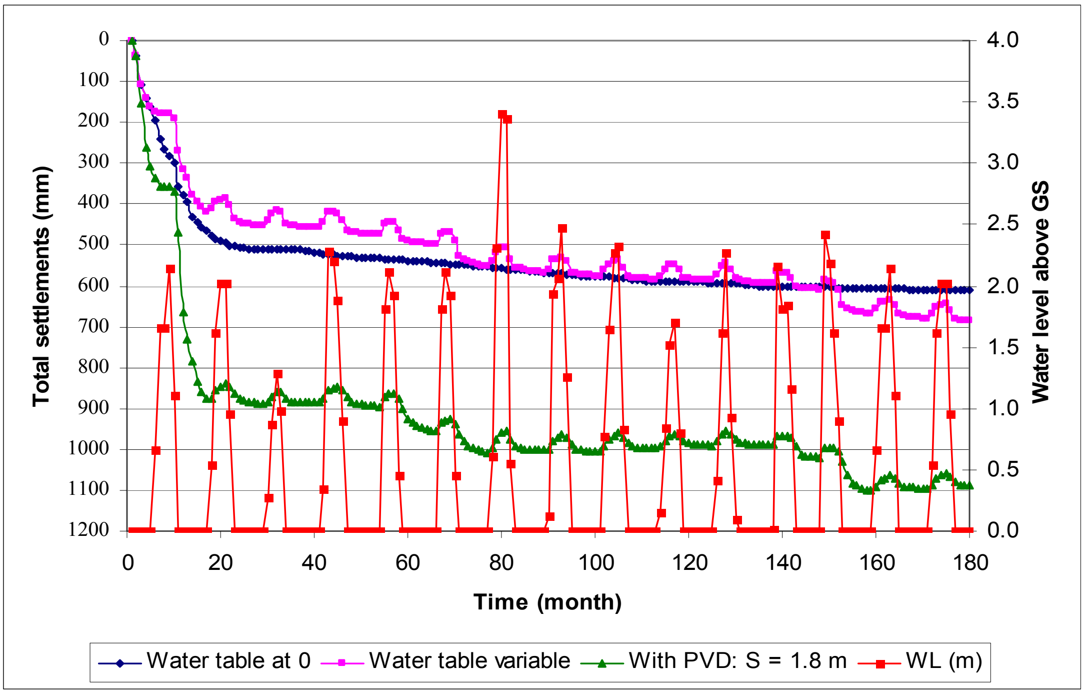



Figure 6.17 Total settlements with and without water table variation

**Discussion:** The water level variation, especially when the level exceeds the ground surface has a small impact on primary consolidation settlement and on elastic settlement. Adjusting the consolidation time when the water level is above the ground, the Standard method can be used with reasonable accuracy if Settle3D is unavailable. Approximately 10 months of consolidation time can be assumed instead of 12 months in the Standard method.

## 6.6 Sensitivity of Settlement Parameters

Analyses have been carried out to determine the sensitivity of the outputs (settlement) obtained from Settle3D as a function of input parameters. In this study following input parameters were varied.

*i.* Elastic parameters: Elastic modulus, unload-reload modulus and Poisson's ratio

*ii.* Consolidation parameters: Coefficient of consolidation, compression index, initial void ratio, over consolidation ratio (OCR).

### 6.6.1. Elastic Parameters

Variations in elastic settlement were studied due to the change of three elastic parameters; initial elastic modulus ($E_s$), unload-reload modulus ($E_{sur}$) and Poison's ration ($\nu$).

**Initial elastic modulus:** Elastic strain reduces non-linearly with the increase of elastic modulus as shown in Figure 6.18. The analysis has been carried out for a single parameter keeping other parameters at their mean values. The ranges of parameters with their mean values used in these analyses are given in Table 6.5.

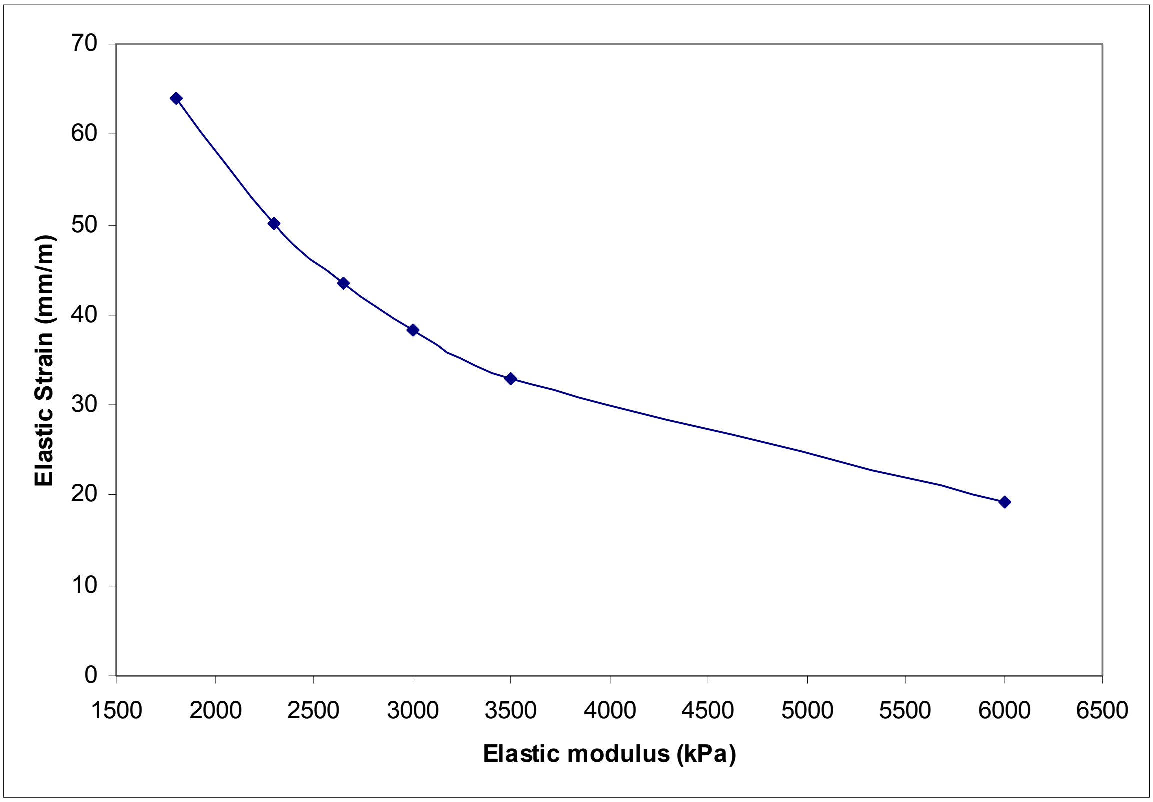



Figure 6.18 Influence of elastic modulus on elastic strain

**Unload-reload modulus:** The unload reload modulus ($E_{ur}$) which is used when modelling the rise and fall of water level has the same impact of elastic settlement. Typically unload reload modulus is 3 to 10 times to that of initial elastic modulus (Eng-Tip-Forums, 2009). Figure 6.19 shows the variations of elastic settlement due to change in total stress (see Figure 6.12 and Section 6.5.1) for different ratios of unload-reload and initial elastic modulus.

Table 6.5 Range and mean value of elastic parameters of soft foundation soil

| **Soil type** | | **Soft clay** | | |
|---|---|---|---|---|
| **Parameter** | **Unit** | **Min** | **Max** | **Mean** |
| $E_s$ | kPa | 1800 | 3500 | 2650 |
| $E_{ur}$ | kPa | 7950 | 23850 | 15900 |
| $v$ | | 0.4 | 0.49 | 0.45 |

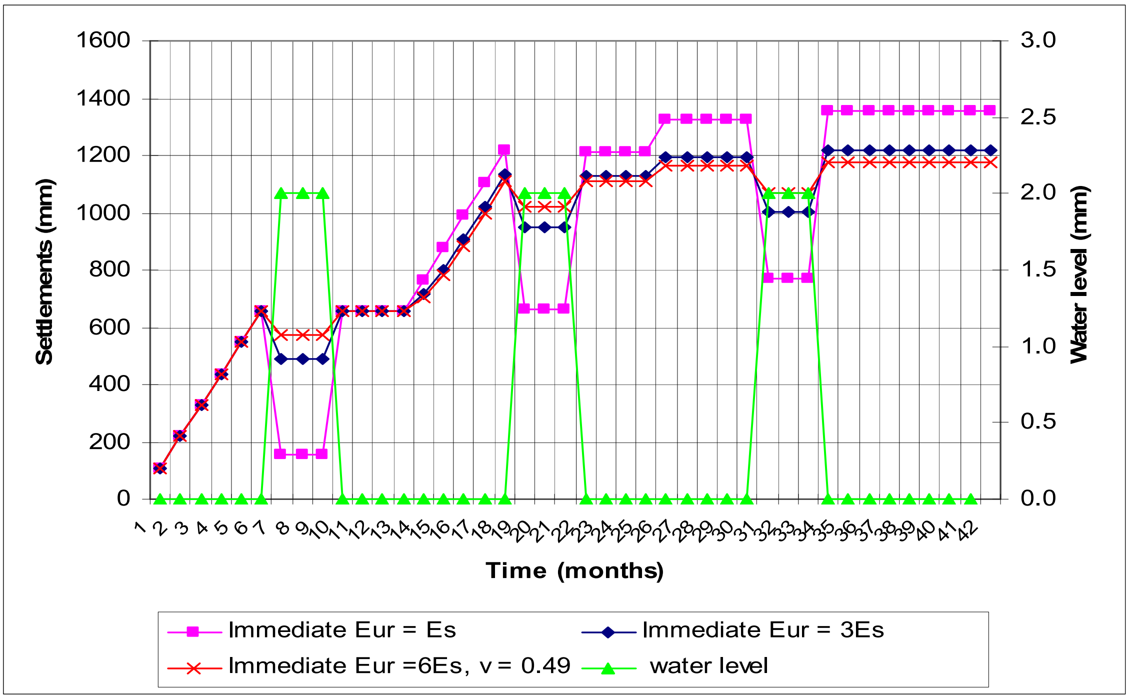



Figure 6.19 Settlements with time for various unload-reload moduli

**Poisson's Ratio:** Figure 6.20 shows the variations of elastic settlement due to change in Poisson's ratio. It is seen that the Poisson's ratio has significant impact on elastic settlement above 0.45 but it is insignificant below 0.35. As the Poisson's ratio for saturated clay varies from 0.4 to 0.5 (Bowles, 1996: Poulos, 1975; cited in Bergado et al., 1990) the average value for the analysis of foundation settlement can be assumed as 0.45 and the analysis in Figure 6.18 was also performed by assuming this value.

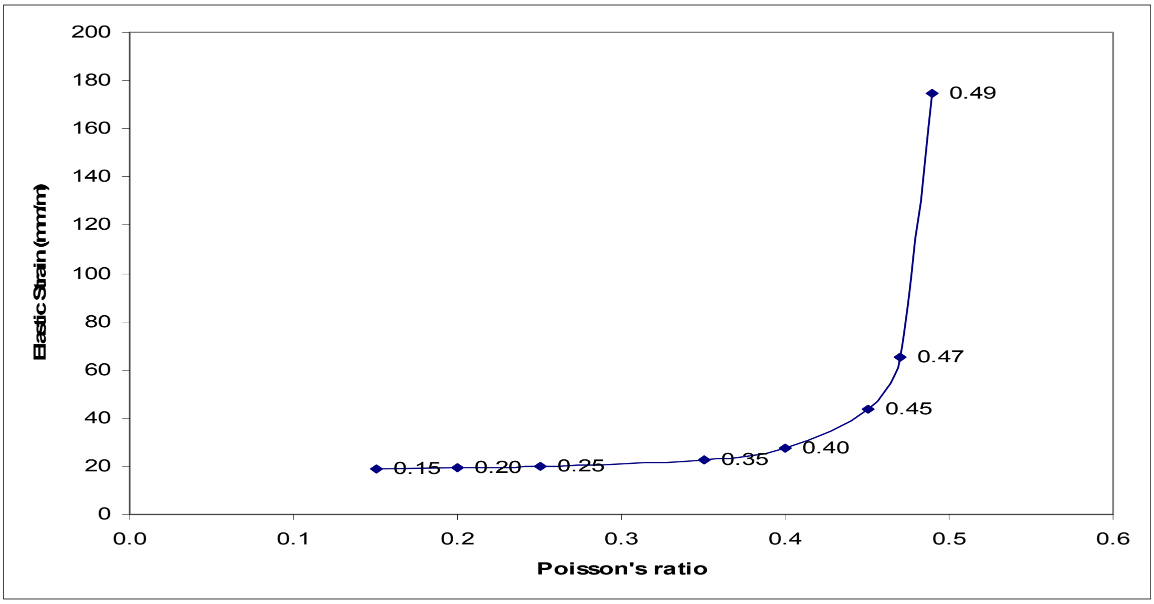



Figure 6.20 Influence of Poisson's ratio on elastic strain

### 6.6.2. Consolidation Parameters

Sensitivity analyses have been carried out on consolidation parameter $C_v$. $C_c$, $e_o$, and OCR which have direct impact on total maximum primary settlement and $C_v$ has the influence on achieving degree of settlement with time for a constant loading.

**Compression index ($C_c$) and void ratio ($e_o$):** In Figure 6.21, 6.22 and 6.23 show the effects of variations in the compression index, void ratio and OCR respectively. The analysis has been carried out for a single parameter keeping other parameters at their mean values. The ranges of parameters with their mean values are given in Table 6.6. Consolidation strain (settlement per metre) increases linearly with the increase of compression index where as it decreases in non-linear manner with the increase in void ratio. Settlement is directly proportional to compression index where as void ratio has a logarithmic inverse relationship. The value of settlement decreases nonlinearly with the increase of over consolidation ratio.

Table 6.6 Range of consolidation parameters

| Soil type | | Soft clay | | |
|---|---|---|---|---|
| **Parameter** | **Unit** | **Min** | **Max** | **Mean** |
| $C_c$ | | 0.048 | 0.310 | 0.147 |
| $C_r$ | | 0.003 | 0.046 | 0.018 |
| $e_o$ | | 0.480 | 2.610 | 1.240 |
| $C_v$ | $m^2$/month | 0.033 | 0.592 | 0.168 |

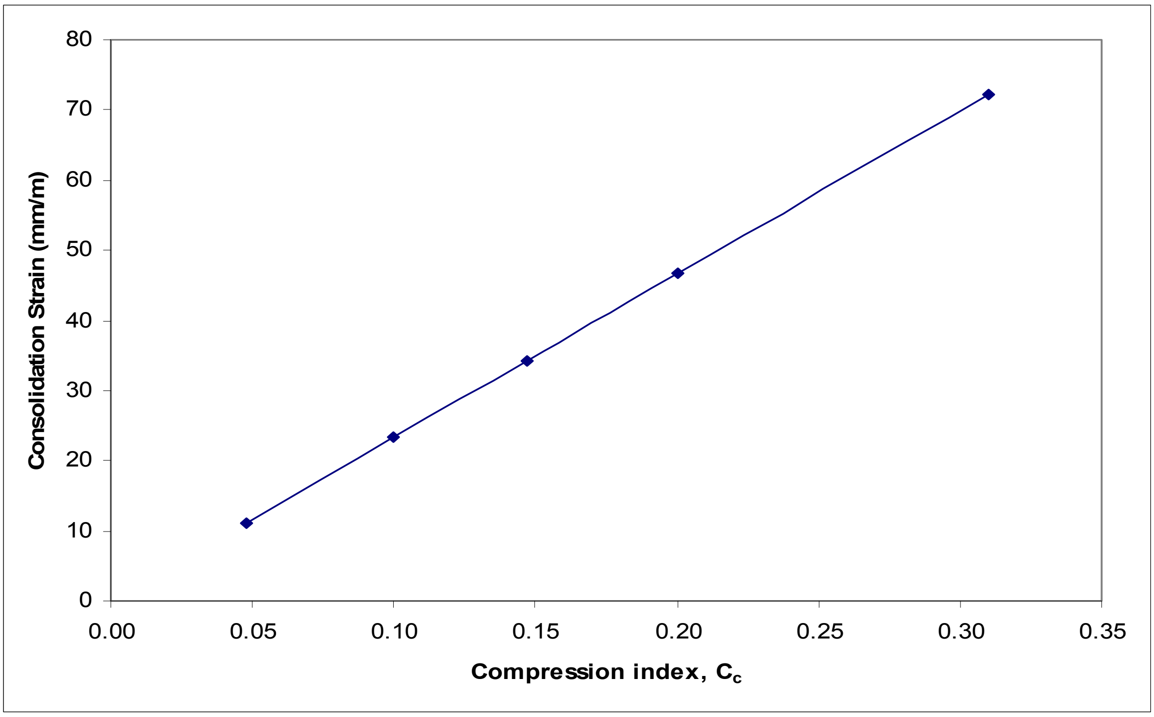



Figure 6.21 Influence of compression index on primary consolidation settlement

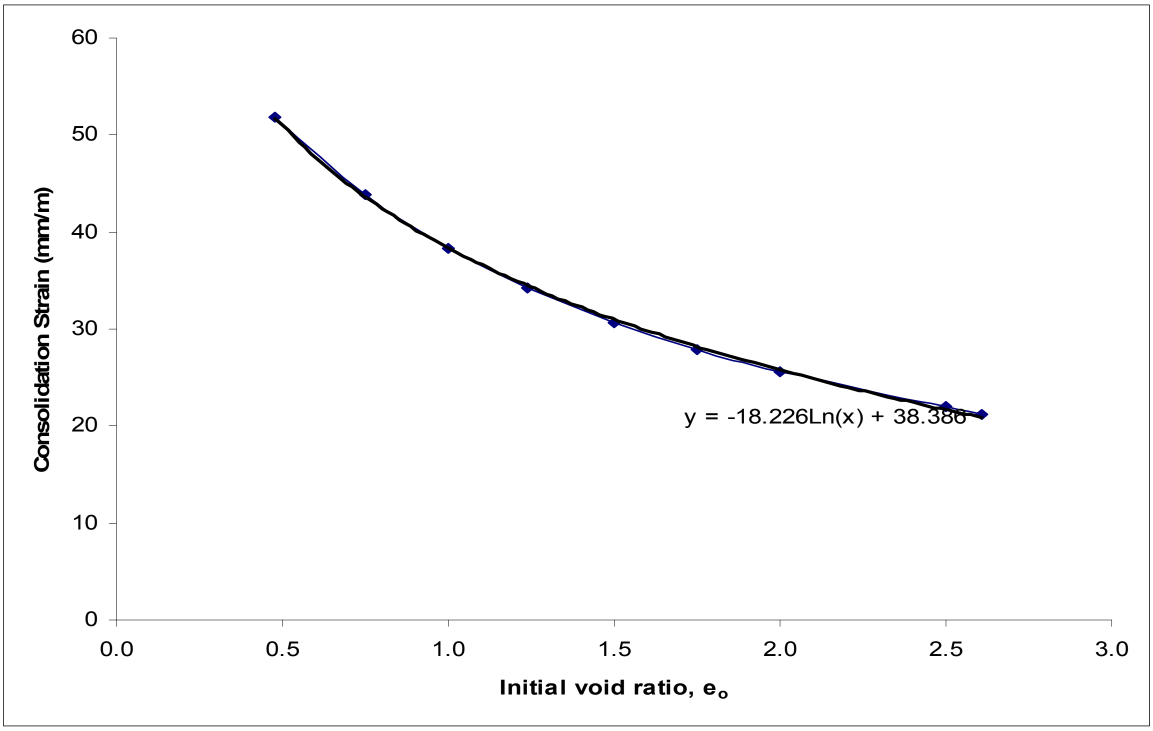



Figure 6.22 Influence of void ratio on consolidation settlement

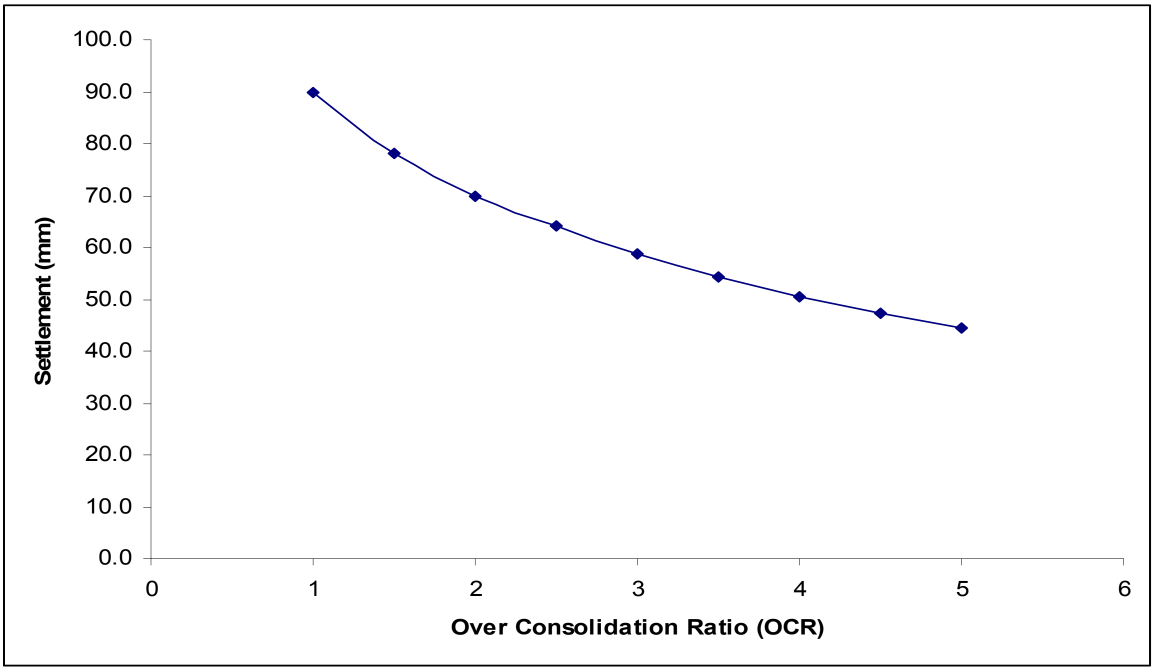



Figure 6.23 Influence of over consolidation ratio on settlement

**Coefficient of consolidation ($C_v$):** Figure 6.24 and 6.25 respectively show the variation of primary consolidation settlement and degree of consolidation as a function of the coefficient of consolidation ($C_v$).

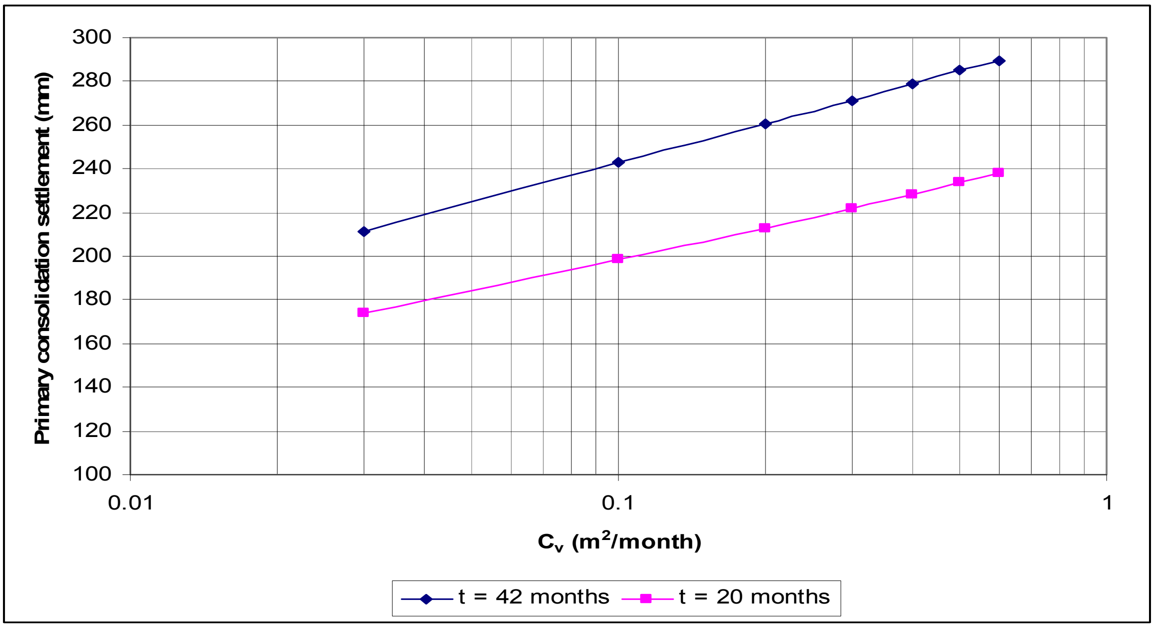



Figure 6.24 Influence of $C_v$ on primary settlement

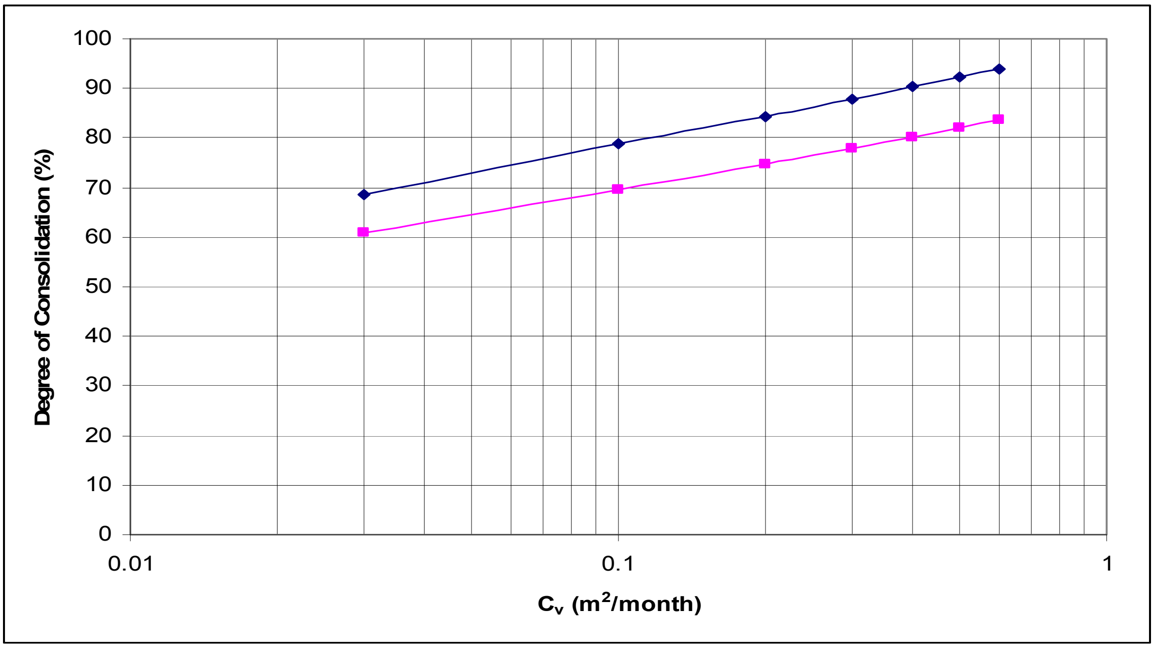



Figure 6.25 Influence of $C_v$ on degree of consolidation

### 6.6.3. Indispensable Parameters for Settlement Analysis

Table 6.7 shows the percentage change in primary consolidation as a function of the changes in the design parameters described above. From this table it may be seen that the compression index and void ratio have more influence on primary settlement where as over consolidation ratio and coefficient of consolidation have comparatively less influence. It is evident that consolidation settlement is most sensitive to compression index and care should be given when determining it. However, all these parameters which can be found from the same test and are essential for prediction with reasonable accuracy of primary consolidation settlement.

Table 6.7 Percentage increase in settlement due to 10% increase in value of input parameters

| Parameter | % of settlement increase due to 10% increase of the value of parameter |
|---|---|
| $C_c$ | 10.0 |
| $e_o$ | -6.0 |
| $C_v$ | 1.3 |
| $OCR$ | -2.7 |

## 6.7 Settlement Prediction of an Existing Road

Settlement analyses were carried out using data from the Tongi-Ashulia road with the water table at ground level and neglecting the impact of its variation. Soil parameters were collected from virgin soils out side of the road embankment and are given in Table 6.8. Using these values and the existing embankment geometry settlements have been predicted for thirty years from the time of construction. It may be seen from Figure 6.27 that only 95% of primary consolidation is achieved within 20 years by which time 813 mm of total settlement is predicted to have taken place (Figure 6.26). It implies that embankment was settling during the period of its service caused failure of pavement. If the precompression measures were taken during the embankment construction, these consequences could be avoided. Differential settlements have also been taken place as the properties of the embankment foundation vary widely spatially. It is assumed for design purposes that secondary compression starts after 95% of consolidation has taken place and thus in this case, as shown in Figure 6.29, the embankment will further settle by about 20 mm due to secondary compression in next ten year period (i.e. 20-30 yrs). The degree of consolidation with depth is shown in Figure 6.28 after 3, 9 and 30 years of consolidation respectively.

Table 6.8 Foundation soil parameters of Tongi-Ashulia Road

| **Depth** | $E_s$ | $E_{ur}$ | $\gamma$ | $\gamma_{sat}$ | $C_c$ | $C_r$ | $e_o$ | $C_v$ | $C_{vr}$ | $C_\alpha$ | $C_{\alpha r}$ | *OCR* |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | kPa | kPa | kN/m3 | kN/m3 | | | | $m^2$/year | $m^2$/year | | | |
| 0.0-1.0 | 3340 | 10020 | 18.38 | 18.38 | 0.391 | 0.135 | 1.08 | 0.803 | 0.803 | 0.0037 | 0.0037 | 1.0 |
| 1.0-2.0 | 3250 | 9750 | 15.08 | 15.08 | 0.311 | 0.109 | 1.10 | 1.204 | 1.204 | 0.0031 | 0.0031 | 1.0 |
| 2.0-3.0 | 3600 | 10800 | 15.08 | 15.08 | 0.330 | 0.120 | 0.89 | 0.4745 | 0.4745 | 0.0610 | 0.0610 | 1.0 |
| 3.0-4.0 | 2400 | 7200 | 15.08 | 15.08 | 0.376 | 0.132 | 1.26 | 25.55 | 25.55 | 0.0020 | 0.0020 | 1.0 |
| 4.0-9.0 | 2240 | 6720 | 13.69 | 13.69 | 0.270 | 0.069 | 3.09 | 7.081 | 7.081 | 0.0490 | 0.0490 | 1.0 |

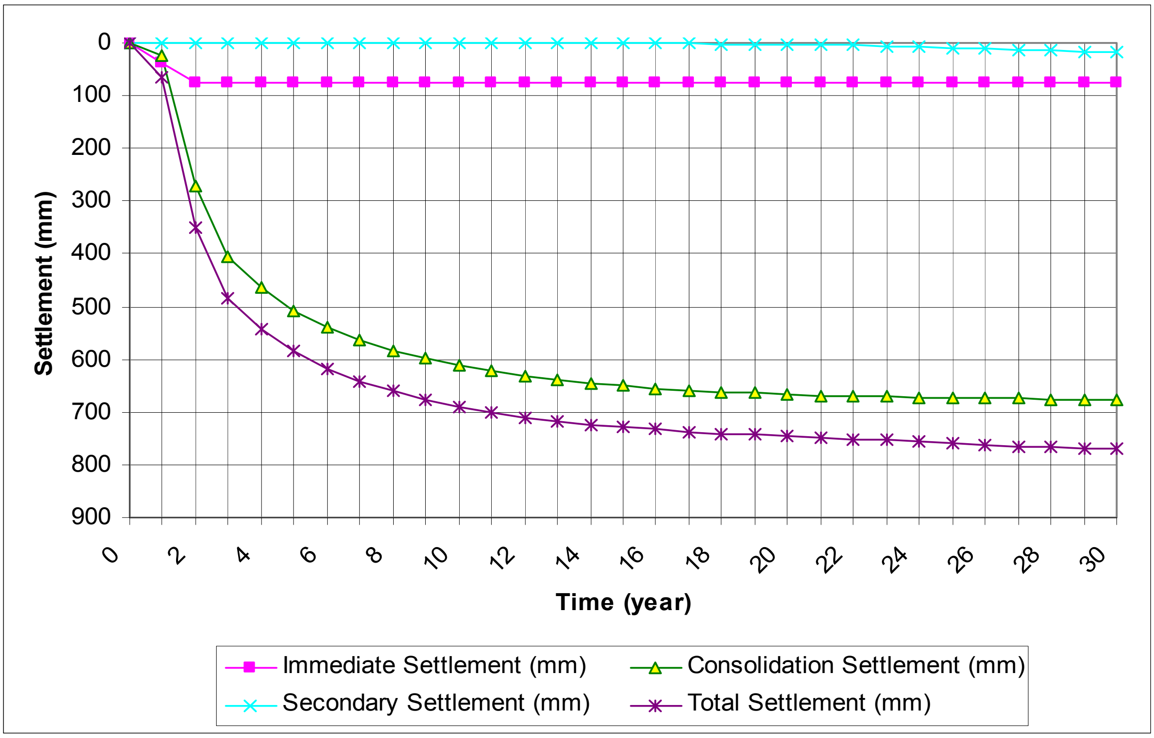



Figure 6.26 Predicted foundation settlements in Tongi- Ashulia road

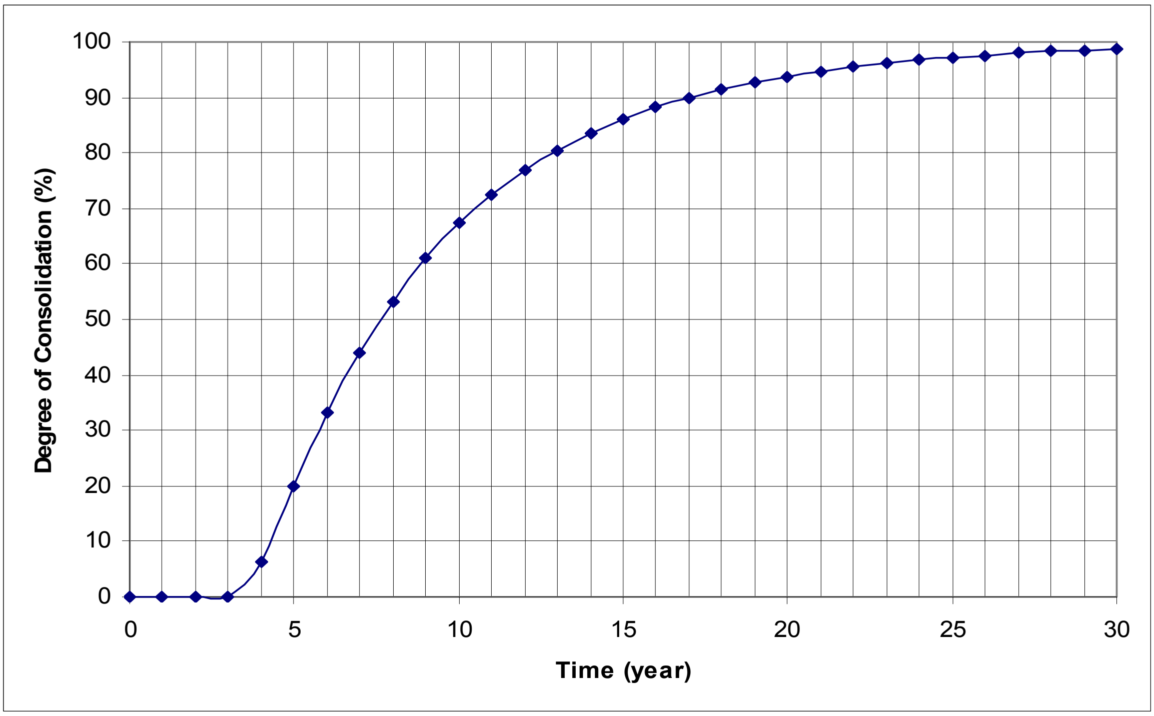



Figure 6.27 Predicted degree of consolidation of Tongi- Ashulia road

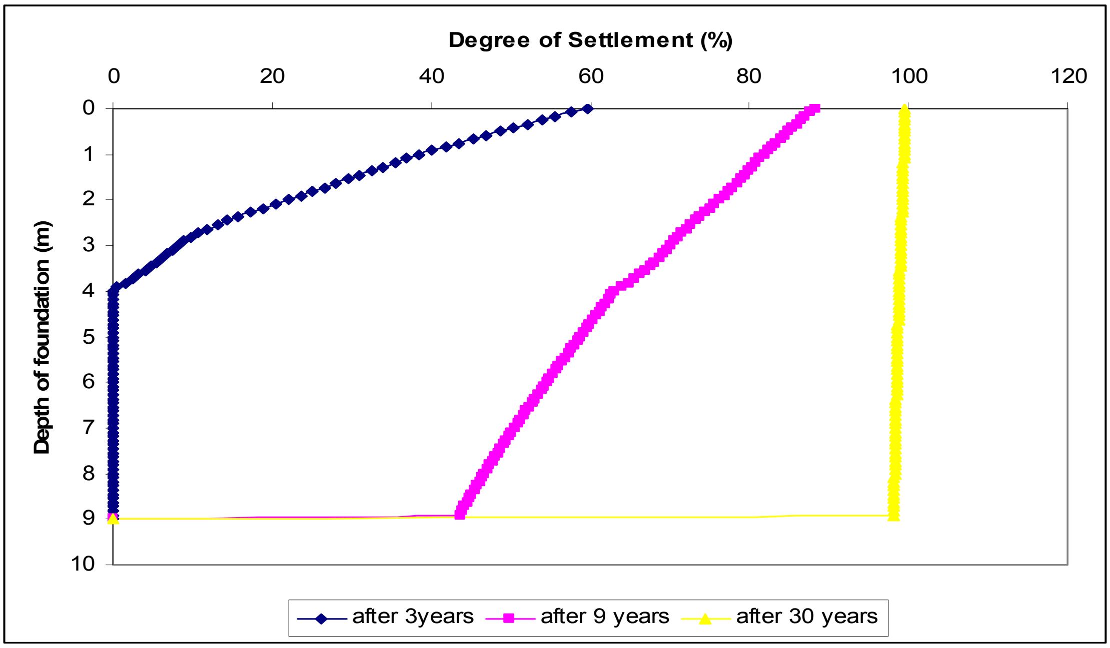


Figure 6.28 Degree of consolidation with depth after specific time periods

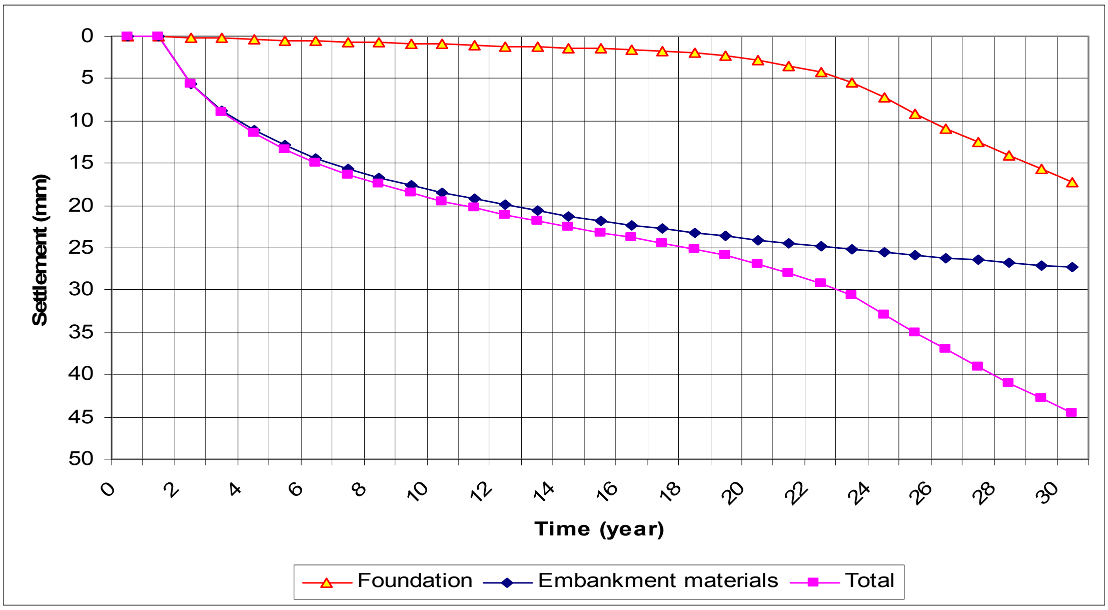


Figure 6.29 Predicted secondary settlement of Tongi-Ashulia road

## 6.8 Summary

Settle3D, a software tool based on the finite difference method was used to carry out a number of analyses to give further understanding into the embankment settlement process and to identify the most important parameters required to model settlement accurately. The software was selected because it was available within the School of Civil Engineering and because it is relatively easy to use and maintain and it is recommended that it be accepted for use in Bangladesh. The analyses carried out by the software as described in this chapter include:

1. A comparison of the output from Settle3D with data from the literature to demonstrate reliability of the software.

2. A comparison of Settle3D with a simpler method (known as the Standard method) which can be implemented in a spread sheet. This was done to demonstrate the suitability of the Standard method, with a view to recommending its use where Settle3D can not be implemented. From the analysis it was found that the Standard method gives comparable result for the cases studied.

3. A sensitivity analysis was conducted by Settle3D to investigate the importance of input parameters by seeing how the outputs from the model vary with inputs. This demonstrated that consolidation settlement linearly increases with the increase of compression index and decreases non-linearly (logarithmic relations) with the increase of initial void ratio. The rate of settlement is only dependent on the coefficient of consolidation and increases linearly with the logarithmic of $C_v$ values if the subsoil profile remains constant. The elastic settlement decreases non-linearly with the increase of elastic modulus and increase non-linearly with the increase of Poisson's ratio. The elastic settlement increases drastically for values of Poisson's ratio over 0.45.

4. A small study was carried out to see the effect of introducing vertical drains on the time taken for consolidation settlement to occur. This demonstrated that the anticipated settlement can be achieved within the desired period of time by using prefabricated vertical drains to reduce residual settlement.

The chapter and its predecessor have considered an important aspect of embankment design, namely the issue of embankment settlement. The following two chapters consider the second issue of design, i.e. stability. The chapters describe a number of methods which may be used for the analyses of embankment stability, with a view to recommending appropriate ones for Bangladesh, and discuss possible measures which may be undertaken to make a stable embankment.

# Chapter 7 Stability of Embankment

## 7.1 Introduction

As mentioned in Chapter 2, the stability of an embankment depends on its geometry, the gravitational and seepage forces and the properties of the embankment and foundation soils. Among the four modes of stability failure identified in Chapter 2, bearing capacity and slope stability failure are the dominant failures in Bangladesh (as described in Section 2.2) and hence these two modes alone are considered herein.

This chapter compares and contrasts the various methods identified in the Literature review of determining embankment stability with a view to selecting the most suitable for the conditions found in Bangladesh.

In addition, the chapter also considers various remedial measures which can be used to improve stability. The most suitable of these for Bangladeshi soils are identified.

## 7.2 Slope Stability Analysis

Slopes may fail by rotational and or translation type failure as described in Chapter 2 and it is usual to analyse failure mechanisms by limit equilibrium methods which assume some limiting conditions for maintaining slope equilibrium (Craig, 2005).

The overall approach developed in this research for determining embankment stability is given in Figure 7.1.

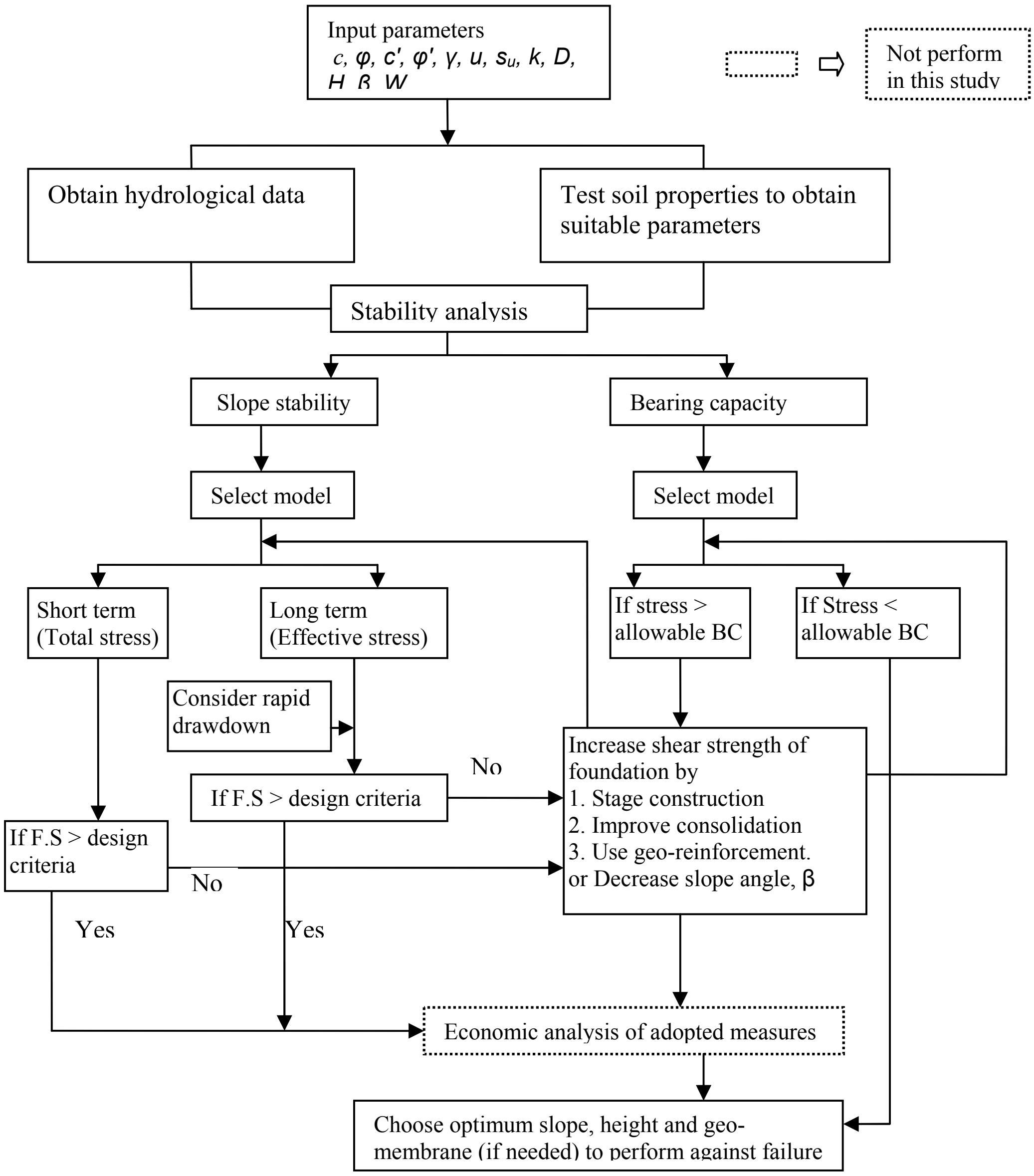



Figure 7.1 Stability analysis process

### 7.2.1. Limit Equilibrium Methods

In these methods a known trial failure surface is conceived and the available shear strength along the surface is compared with the required shear strength to prevent the failure. The ratio of the two values is known as the Factor of Safety and in design it is usual to specify a factor of safety as a means of taking into account uncertainty.

As mentioned in Chapter 2, total and effective stress analyses are usually adopted for short and long term analysis respectively (Ladd, 1991).

As mentioned in Section 2.6.1.1 a number of limit equilibrium methods have been developed. Among these methods Bishop's and Fellenius methods are applicable for **circular slip**. Morgenstern and Price's method and Janbu's method are applicable for any shape (circular or **non-circular**) of failure surface. These methods are discussed below and summarised in Table 7.1.

#### 7.2.1.1. Ordinary Method of Slices

In this analysis the sliding surface is considered as circular to simplify the calculations. The factor of safety (F) for the short term or total stress analysis of stability can be given by the ratio of restoring moment to the disturbing moment. (Kadiali, 2003; BS 6031:1981)

$$F = \frac{\text{Restoring} \quad Moment}{Disturbing \quad Moment}$$

or $$F = \frac{s_u R^2 \theta}{Wx} \qquad 7.1$$

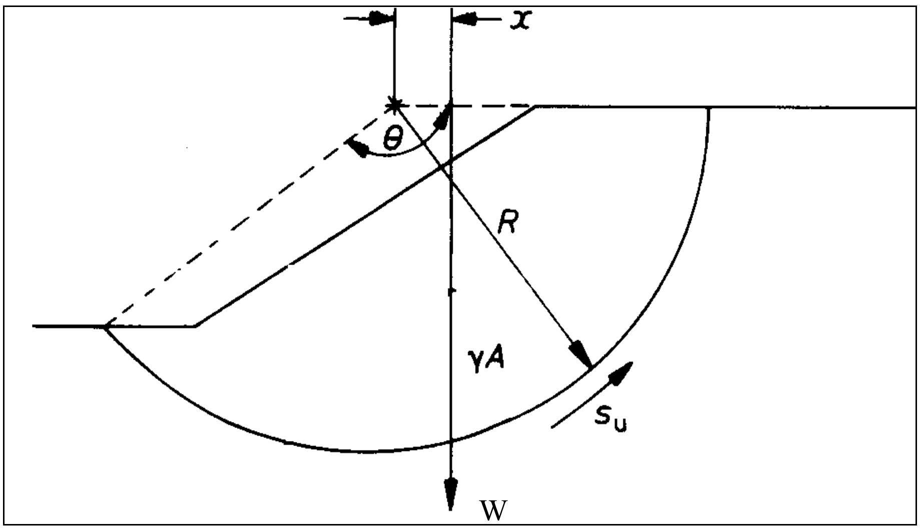



Figure 7.2 Forces acting on a slip circle

Where, $R$ = radius of the circle, $\theta$ = defined in figure and the value will be in radians, $x$ = distance of centre of gravity of soil mass from the centre of circle, $W$ = weight of soil mass = γA, $\gamma$ = unit weight of soil, $A$ = area of soil mass.

If the shear strength of soil varies with the depth, the soil mass above the circular arc can be divided into a number of slices. The factor of safety in this case can be given as

$$F = \frac{R\sum s_u l}{\sum_i^n W_i x} \qquad 7.2$$

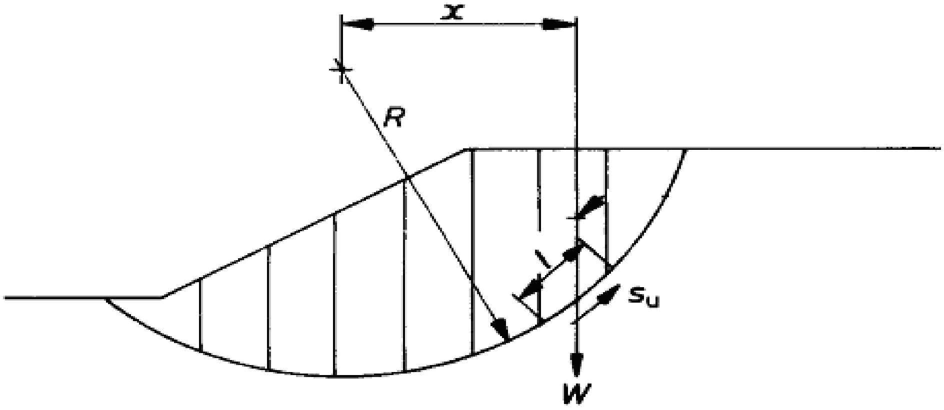



Figure 7.3 Method of slices: slip circle failure

Where, $l$ = length of each slice along failure path, $n$ = number of slices, $W_i$ = weight of each slice.

The intermediate and long term stability should be analysed on the basis of effective stress method. The shear strength in this case can be found from Equation 2.10. The soil mass also divided into number of vertical slices in this case. The factor of safety in Fellenius method can be given as (Fellenius, 1927; cited in Craig, 2005)

For effective stress analysis

$$F = \frac{c'l + \tan\phi' \sum (W\cos\alpha - ul)}{\sum W\sin\alpha} \qquad 7.3$$

For total stress analysis ($\varphi_u = 0, u = 0$)

$$F = \frac{\sum c_u l}{\sum W \sin\alpha} \qquad 7.4$$

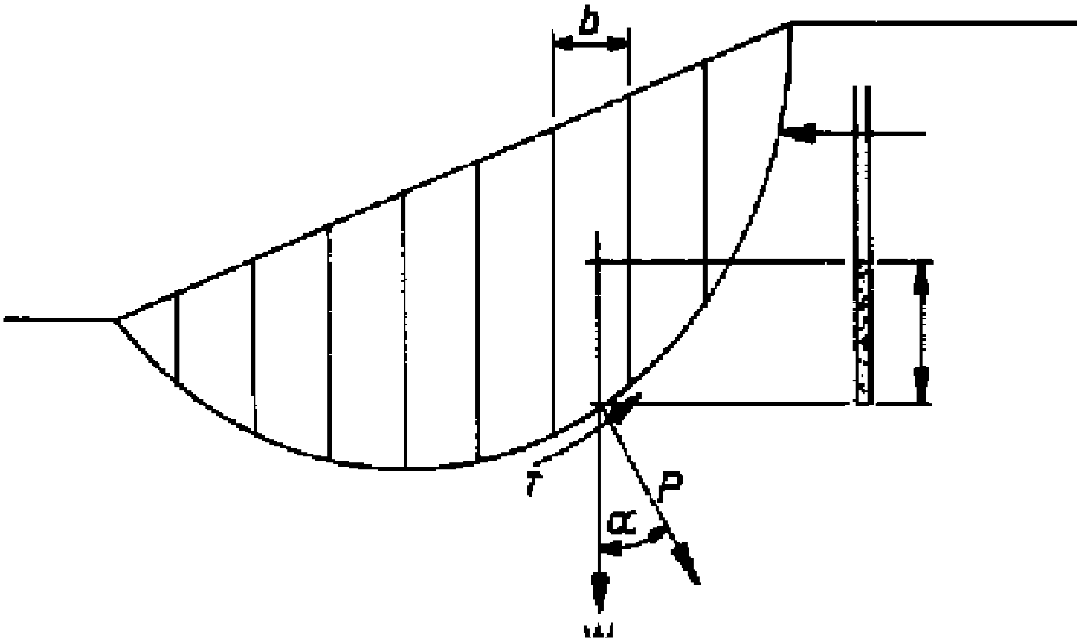



Figure 7.4 Slip circle with slices: force distribution

To obtain the lowest value of factor of safety, a number of trial slip surface are assumed. This method satisfies only the moment equilibrium condition ignoring all interslice forces. Due to simplicity of the method, it is suitable for hand calculation.

#### 7.2.1.2. Bishop's Simplified Solution

Bishop (1955) considered a circular failure surface and divided the soil mass into a number of slices. In his method the inter-slice shear forces are ignored but inter-slice normal forces are considered. This simplified method also satisfies moment equilibrium but it assumes that the resultant inter-slice shear forces acting on the sides of a slice are horizontal i.e. zero or $X_1$-$X_2 = 0$ (Figure 7.5). Factor of safety in this method for effective stress and total stress analyses are given as follows:

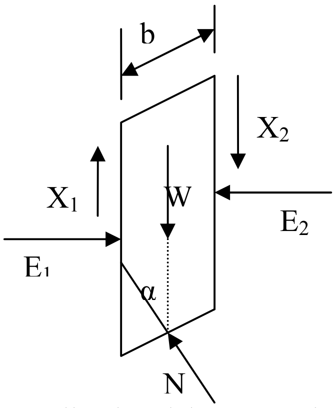


Figure 7.5 Forces acting on a slice in Bishop's method

- Effective stress analysis

$$F = \frac{1}{\sum W \sin\alpha} \sum \left[ \{c'b + W(1 - r_u)\tan\phi'\} \frac{\sec\alpha}{1 + (\tan\alpha \tan\phi')/F} \right] \quad 7.5$$

- Total stress analysis

$$F = \frac{\sum c_u l}{\sum W \sin\alpha} \quad 7.6$$

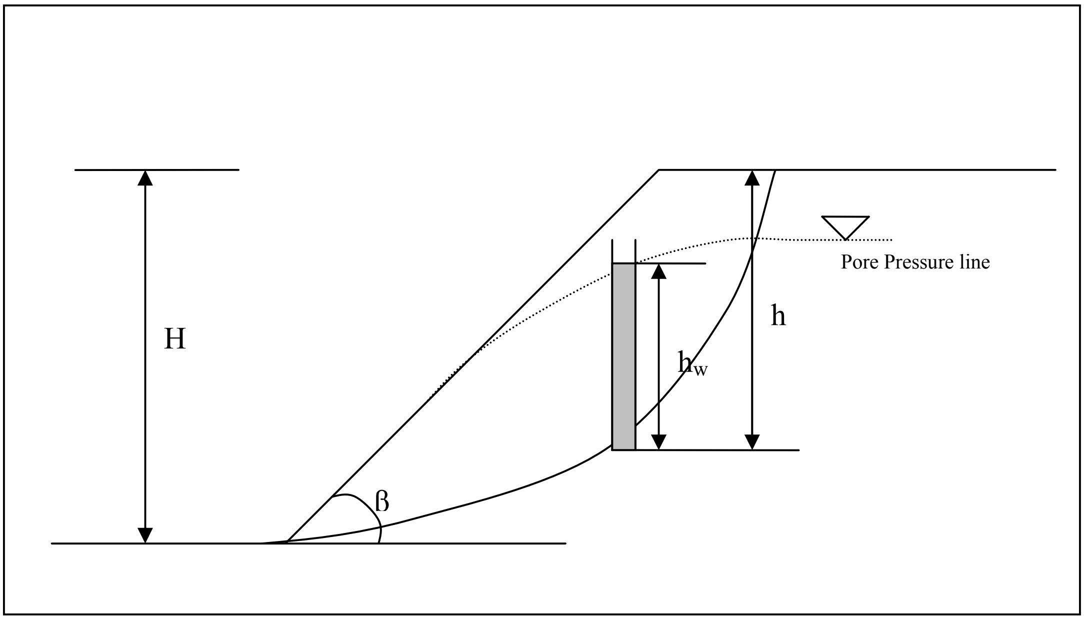



Figure 7.6 Stability analyses with steady seepage

Where, $r_u$ is the pore pressure ratio which defined as the ratio of pore water pressure and earth fill pressure and can be defined by, $r_u = \frac{\gamma_w h_w}{\gamma h} = \frac{u}{\gamma h}$. For each slice it can be given by $r_u = \frac{u}{W/b}$.

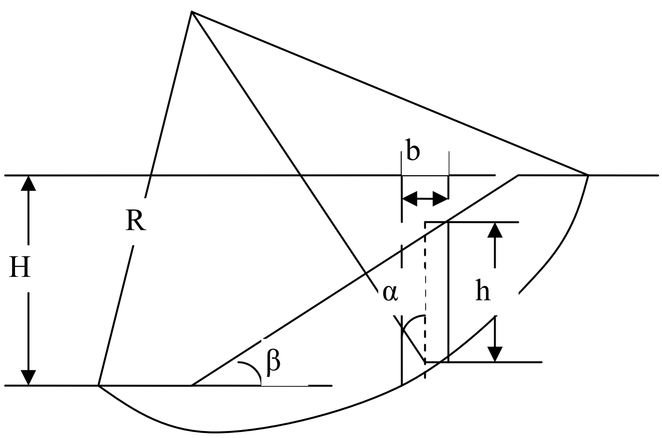
$$\text{or } F = \frac{1}{\sum W \sin\alpha} \sum \left[ \frac{\{c'b + W(1 - r_u)\tan\phi'\}}{m_\alpha} \right] \qquad 7.7$$

$$\text{where, } m = \cos\alpha + \frac{\sin\alpha \tan\phi'}{F} \qquad 7.8$$

The above equations can not be solved directly and therefore the method is best utilised in a computer programme.

#### 7.2.1.3. Spencer's Method

Spencer (1967) modified the Bishop's simplified method by using parallel inclined inter-slice forces (shown in Figure 7.7) instead of horizontal ones. Spencer compared his method with Bishop's for a number of cases and found the methods gave similar results (Spencer, 1967).



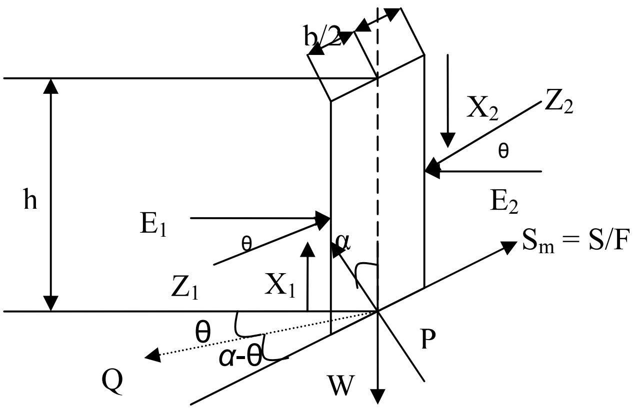



Figure 7.7 Forces acting on a slice in Spencer's method

The factor of safety can be found by solving the following three equations among which Equation 7.9 and 7.10 are for force equilibrium and 7.11 are for moment equilibrium. The value of θ will remain constant for a given slice.

$$\sum[Q\cos\theta]=0 \qquad 7.9$$

$$\sum[Q\sin\theta]=0 \qquad 7.10$$

$$\sum[Q\cos(\alpha-\theta)]=0 \qquad 7.11$$

Where the value of Q (resultant of two inter-slice forces) is given by the following equation and the symbols used in these equations are described by the Figure 7.7.

$$Q=\gamma Hb\left[\frac{\frac{c'}{F\gamma H}+\frac{h}{2H}\frac{\tan\phi'}{F}\left(1-2r_u+\cos 2\alpha\right)-\frac{h}{2H}\sin 2\alpha}{\cos\alpha\cos(\alpha-\theta)\left[1+\frac{\tan\phi'}{F}\tan(\alpha-\theta)\right]}\right] \qquad 7.12$$

### 7.2.1.4. Bishop and Morgenstern

The factor of safety determined using Bishop's method (Equation 7.5) can be solved using a method suggested by Bishop and Morgenstern (1960). In the method it is assumed that the factor of safety for a given slope and given soil properties varies linearly with $r_u$ and can be expressed as follows:

$$F = m' - n' r_u \tag{7.13}$$

Where, $m'$ and $n'$ are stability coefficients and are functions of slope angle ($\beta$), $\varphi'$, and depth factor $D$ and the dimensionless factor $c'/\gamma H$. The Bishop and Morgenstern's value of $m'$ and $n'$ are given in Appendix-II. An example of this method has been shown in Appendix -XI.

#### 7.2.1.5. Janbu's Generalised Method

This method considers horizontal force equilibrium and both components of the inter-slice forces (E and X) are taken into account (Janbu 1973; cited in Barnes, 1995). Both non-circular and circular slip surface can be analysed with this method. The factor of safety for total and effective stress analyses are given by the following equations:

- Total stress analysis

$$F = \frac{\sum c_u l}{\sum W \sin\alpha} \tag{7.14}$$

- Effective stress analysis

$$F = \frac{\sum [c'b + (W + dX - ub)\tan\phi']m_\alpha}{\sum (W + dX)\tan\alpha} \tag{7.15}$$

Where

$$m_\alpha = \frac{\sec^2\alpha}{\frac{1 + \tan\phi' \tan\alpha}{F}} \tag{7.16}$$

and $dX = X_1 - X_2$

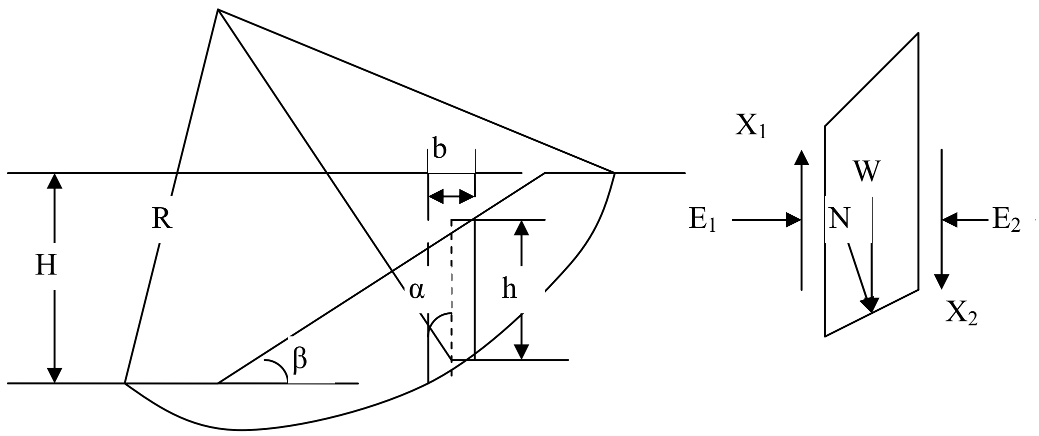



Figure 7.8 Mechanics of Janbu method explaining the symbols

#### 7.2.1.6. Low’s Method

In Low’s method (Low, 1989) the potential failure slip of embankment constructed on soft soil is assumed to be circular. Similarly to Fellenius method this method satisfies only overall moment equilibrium, however the contribution to the factor of safety by the foundation and embankment can be assessed separately. The method assumes that soft foundation (cohesive soil) behaves in an undrained manner (short term) whilst the embankment materials behave in a drained way. This is similar to what happens in embankments in Bangladesh.

According to Low (1989), the factor of safety for slope stability corresponding to a trial limiting tangent at depth D is given by the following equation:

$$(F_s)_D = N_f \frac{C_A}{\gamma H} + N_m \left( \frac{C_m}{\gamma H} + \lambda \tan \phi_m \right) \qquad 7.17$$

Where

$$C_A = 0.35 C_T + 0.65 C_D + 0.35 \left( {D_c}/{D} \right)^{1.1} \Delta C_T \qquad 7.18$$

$C_m$ is the cohesion of embankment fill and $C_A$ is the average undrained shear strength of foundation soil within the depth $D$, $D$ is the depth of trial limiting tangent, $C_T$ is the shear strength of soil at the top of the foundation and $\Delta C_T$ is described in the Figure 7.9, $D_c$ is the depth of foundation to which the shear strength decreases linearly and from which shear strength again increases linearly towards the firm bottom (i.e. at depth $D_{max}$) as shown in Figure 7.9, $D_{max}$ is the depth of firm bottom, $C_D$ is the shear strength at depth D of foundation, $\gamma$ is the unit weight of embankment soil and $H$ is the height of the embankment. The above Equation 7.17 can be written as the following simplified form:

$$(F_s)_D = N_f.S_f + N_m.S_u \qquad 7.19$$

Where, $S_f$ and $S_m$ are the normalized strength parameter for foundation and embankment respectively. $N_f$ and $Nm$ are stability numbers for foundation and embankment respectively and can be determined from stability chart presented by Low (1989). The value of $S_f$, $S_m$, $N_f$ and $Nm$ can also be found from the following relationships:

$$S_f = \frac{C_A}{\gamma H} \qquad 7.20$$

$$S_m = \frac{C_m}{\gamma H} + \lambda \tan\varphi_m \qquad 7.21$$

$$N_f = 3.06\left(\frac{D}{H}\right)^{0.53} \alpha_1^{1.47} / \alpha_2 \qquad 7.22$$

$$N_m = 1.53\left[\left(\frac{D}{H}+1\right)^{0.53} - \left(\frac{D}{H}\right)^{0.53}\right]\alpha_1^{1.47} \Big/ \alpha_2 \qquad 7.23$$

The value of $\alpha_1$ and $\alpha_2$ can be calculated from depth of trial limiting tangent ($D$), height of embankment ($H$) and the slope of the embankment ($\beta$) and can be given by the following relationships:

$$\alpha_1 = 1.564\left(\frac{D}{H}+\frac{1}{2}\right) + 0.1303\frac{\cot^2\beta + 1}{\frac{D}{H}+0.5} \qquad 7.24$$

$$\alpha_2 = \alpha_1\left(\frac{D}{H}+\frac{1}{2}\right)-\frac{1}{2}\left(\frac{D}{H}+\frac{1}{2}\right)^2-\frac{1}{24}\left(\cot^2\beta+1\right) \qquad 7.25$$

$\lambda$ is a function of depth of clay layer (*D*), height of the embankment (*H*) and slope of the embankment ($\beta$) and can be found from the following equation.

$$\lambda = 0.19+\frac{0.02\cot\beta}{D/H} \qquad 7.26$$

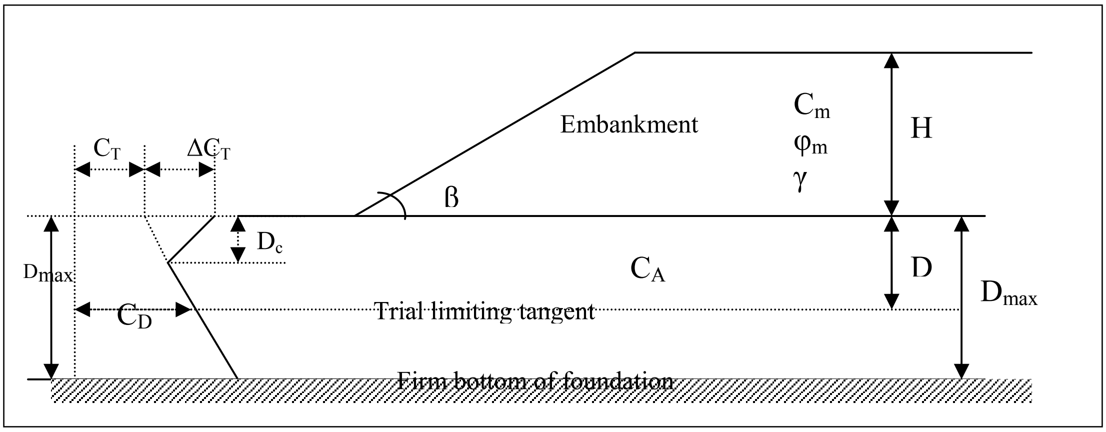



Figure 7.9 Elements of Low's (1989) method

The trial limiting tangent is assumed at various depth of clay layers from which minimum factor of safety can be determined.

### 7.2.1.7. TRL method

In the TRL method (ORN 14, 1997) the factor of safety for slope stability is determined from charts based on the following parameters.

- 24-hour rainfall
- Soil permeability (*k*)
- Soil strength ($S_u$)
- Water table height
- Slope height (*H*)
- Slope angle ($\beta$)

Four intensities of 24-hour rainfall were used for making the charts; these are 250 mm, 350 mm, 450 mm and 550 mm. Soil permeability in the range of $1 \times 10^{-7}$ to $1 \times 10^{-7}$ has been used to represent the tropical condition. There are only some specific values of shear strength parameters in the charts which results difficulties in interpolating the intermediate values. Four heights of water table conditions were assumed in the charts and are expressed as the percentage of the slope height, namely 0%, 25%, 50% and 75% of slope heights. The charts considered slope height from six metre to thirty six metre at six metre interval for slope angles of 2:1, 1:1, 1:1.5 and 1:2 respectively. It is not possible to consider embankments less than six m in height using the method. When considered with the other methods (see Section 7.2.1), the factor of safety determined by the TRL method is grater than that determined using the other available methods (see Section 7.2.2)

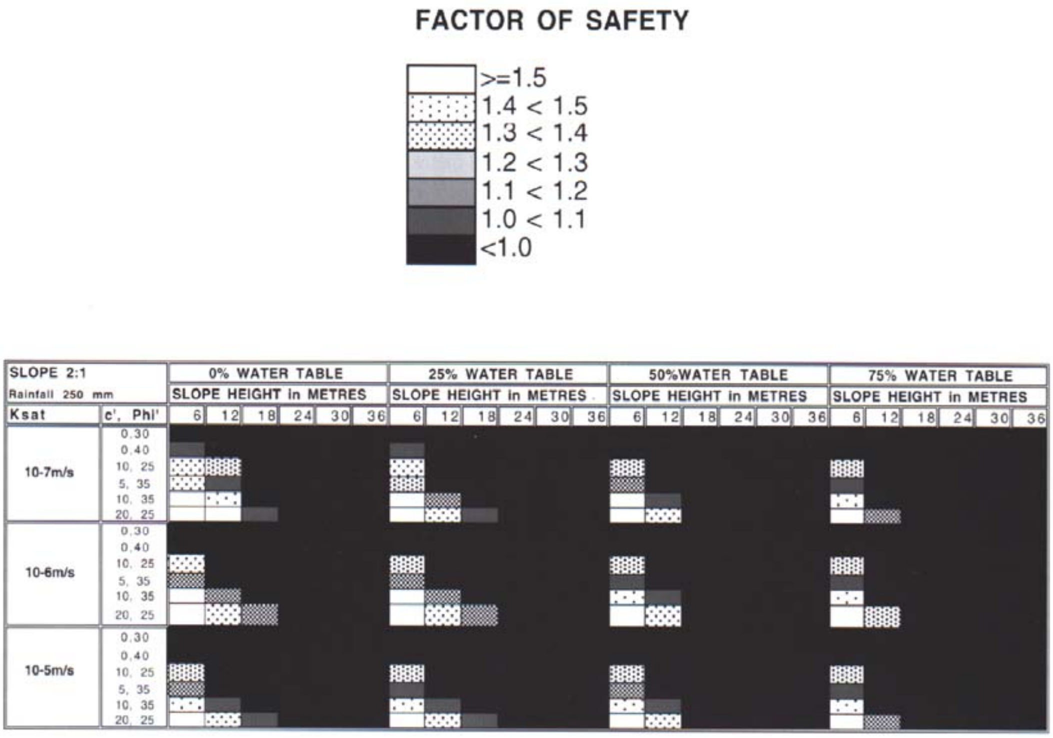



Figure 7.10 Factor of safety in TRL method

Table 7.1 Comparison among various methods of stability analysis

| Methods | Parameters required | Comments |
|---|---|---|

| | | |
|---|---|---|
| Bishop simplified | Shear strength for fill and foundations layers, H, β, depth of foundation layers (D), WT location. | • Consider vertical force equilibrium and moment equilibrium.<br>• Stability can be analysed with reinforcement.<br>• Consider water table |
| Spencer | Shear strength for fill and foundations layers, H, β, depth of layers (D), WT location | • Consider vertical, horizontal and moment equilibrium.<br>• Stability can be analysed with reinforcement.<br>• Consider water table. |
| Swedish or Fellenius method | Shear strength of fill and foundations layers, H, β, depth of layers (D). | • Can not be accommodated submerged slopes.<br>• Stability can not be analysed with reinforcement. |
| Janbu | Shear strength for fill and foundations layers, H, β, depth of layers (D), WT location. | • Can be used for both circular and noncircular slip.<br>• Same as Bishop's modified method (Spencer's) when used with circular slip. |
| TRL | Shear strength of fill, H, β, WT location, permeability of fill, 24 hours rainfall. | • Parameters rainfall and permeability required in addition to the parameters required for other methods.<br>• FoS over is estimated<br>• No consideration for foundation layers.<br>• No consideration for different fill layers<br>• Stability with reinforcement can not be analysed.<br>• Consider slope height only from 6m to 36 m. |
| Low | Shear strength of fill and foundations layers, H, β, depth of layers (D), shear | • Consider shear strength parameters both for fill and foundation.<br>• No consideration of WT |

| | | |
|---|---|---|
| | strength gradient of foundation. | • Applicable for soft soil<br>• Reinforced stability can not be found.<br>• Successfully used in Excel spread sheet. |

### 7.2.2. Comparison

A comparison of each of the above methods was made using a hypothetical embankment with the properties given in Table 7.2.

Table 7.2 Parameters used for the comparison among the slope stability methods

| Parameters | Symbol | Unit | Value |
|---|---|---|---|
| Shear strength | $C'$ | kPa | 6 |
| | $\varphi'$ | Degree | 25 |
| Unit weight of fill | $\gamma_m$ | kN/m$^3$ | 18.0 |
| Permeability of fill | $k$ | m/s | $1\times10^{-7}$ |
| Undrained shear strength of foundation | $S_u$ | kPa | 20 |
| Unit weight of foundation soil | $\gamma_f$ | kN/m$^3$ | 17.5 |
| Height of embankment | H | m | 4.0 |
| Slope angle | $\beta$ | degree | 26.5 |
| 24 hours rainfall | - | mm | 450 |

### 7.2.3. Results

The factors of safety using each method are given in Table 7.3.

Table 7.3 Factor of safety determined by the different methods

| Method | Factor of Safety | | | |
|---|---|---|---|---|
| | WT at 0%H | WT at 25%H | WT at 50%H | WT at 75%H |
| Swedish (Fellenius) | 1.283 | Does not consider water table position | | |
| Bishop simplified | 1.359 | 1.352 | 1.335 | 1.297 |
| Spencer (Bishop's modified) | 1.345 | 1.342 | 1.329 | 1.298 |
| Janbu | 1.345 | 1.342 | 1.330 | 1.298 |
| Low | 1.39 | Does not consider water table position | | |
| TRL | 1.5 | 1.5 | 1.5 | 1.5 |

From Table 7.3 it may be seen that when the water table is considered to be below the bottom of the embankment (WT = 0%), the factor of safety given by Bishop's simplified, Spencer, Janbu and Low methods are in close agreement, where as Fellenius gives a little lower value. Considering various levels of submergence (i.e. WT > 0%), the factor of safety determined by Bishop's method show a slightly higher value than Spencer's and Janbu's. The TRL method gives the highest factor of safety in all cases. This suggests that embankment may be under design by this method.

### 7.2.4. Discussion

Whilst the factor of safety determined from the case study for Bishop's, Spencer's and Janbu's methods are similar, theoretically Bishop's method is the most accurate method of all of them. It takes into account a number of parameters and is also able to include an analysis of reinforcement. However, it requires computer software to implement which may not be available in Bangladesh. Nevertheless, this method is recommended for the task in hand on the premise that the software is available. Among other methods, should the software required to implement Bishop's method not be available, Low's method can be used as it considers soft foundation soils such as those found in Bangladesh and it can be easily implemented in an Excel spreadsheet. As Low's method does not consider the water table on the both sides of embankment, nor reinforcement, a modification has been suggested in Chapter 8 (see Section 8.6) to make it suitable for use in Bangladesh.

## 7.3 Bearing Capacity of Foundation

The analysis of bearing capacity of soils under an embankment is very important where the foundation soil consists of a very weak cohesive soil. The Bearing capacity of soft clay foundation under an embankment can be calculated by the following equation (Radoslaw, 1992):

$$\bar{q} = \gamma_f H_c \left(1 - \frac{H_c}{b \tan \beta}\right) \qquad 7.27$$

Rearranging, the critical height, $H_c$, is given as

$$H_c = b \tan \beta \left(1 - \sqrt{1 - \frac{2\bar{q}}{b \gamma_f \tan \beta}}\right) \qquad 7.28$$

Where $\bar{q}$ = Average vertical component of the limiting stress, $H_c$ = critical height of embankment and can be defined as the height at which failure is occurred due to collapse of the foundation soil, $\beta$ = slope angle of the embankment and $c$ = cohesion or undrained shear strength and can be given with depth (z) for non-homogeneous soil and varies linearly by the following equation.

$$c = c_m + \varepsilon z \qquad 7.29$$

*Where,* $\varepsilon$ = gradient of shear strength with depth and $c_m$ = minimum value of cohesion.

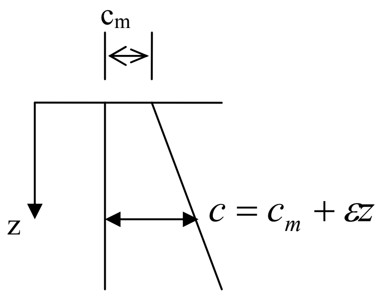



Figure 7.11 Variation of shear with depth

$\chi$ is a coefficient and defined as shear stress factor and is expressed as

$$\chi = \frac{\overline{\tau}}{c} = \frac{\gamma_f H^2 k}{2bc} \qquad 7.30$$

Where $\overline{\tau}$ = tangential component of the stress, $\gamma_f$ = unit weight of the fill material, $H$ = height of embankment, $b$ = half width of embankment at its mid height, $k$ = coefficient of lateral stress and $k \geq \tan^2\left(\frac{\pi}{4} - \frac{\varphi_f}{2}\right)$ and $\varphi_f$ = angle of internal friction of the fill materials.

### 7.3.1. Bearing Capacity with Reinforcement

If reinforcement is used at the interface of the foundation and embankment, a measure of the amount of reinforcement (R) required and the value of shear stress factor, $\chi$, then can be found from the following equation (Radoslaw and Shi, 1993):

$$\chi = \frac{\overline{\tau}}{c} = \frac{\gamma_f H^2 k - 2R}{2bc} \qquad 7.31$$

Where, $R$ = tensile strength of the reinforcement per unit length (kN/m). The value of $\chi$ varies from -1.0 to 1.0. If the calculated value of $\chi$ is equal to -1.0 then the embankment is considered to be fully reinforced which implies the total outward thrust is resisted by the reinforcement and any increase of reinforcement force at this stage will not allow an increase of the slope height. Theoretically the foundation is considered to be at the point of failure and the full thrust is taken by reinforcement, although a positive value of $\chi$ (>0) is recommended for practical design of embankment (Radoslaw and Shi, 1993).

### 7.3.2. Critical Height of Embankment

The critical height of embankment is defined as the height at which failure will occur due to the collapse of the foundation soil. The depth of soft clay strongly influences the deformation after its construction. The critical height of embankment can be determined from the bearing capacity of the foundation soil by the process described in Figure 7.13 using Equations 7.27 to 7.31 (Radoslaw, 1992). An example to determine critical heights of embankment with and without reinforcement is shown in Appendix-VII.

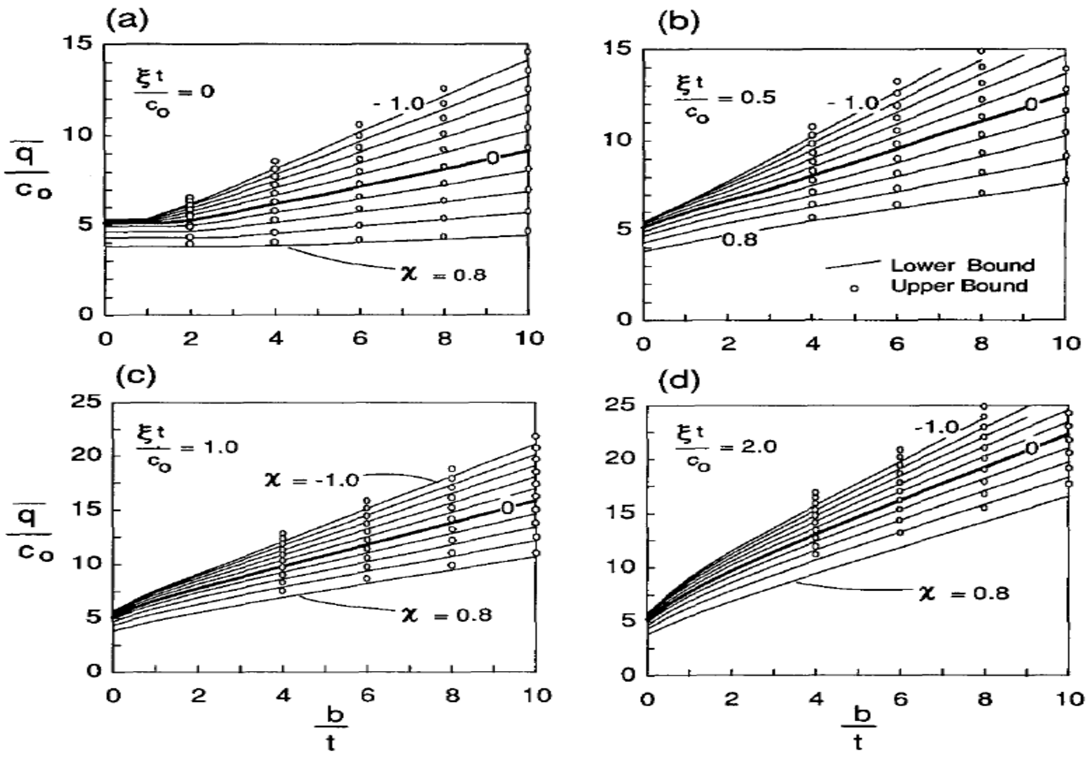



Figure 7.12 Average limit pressures as a function of width-thickness ratio (after Radoslaw and Shi, 1993)

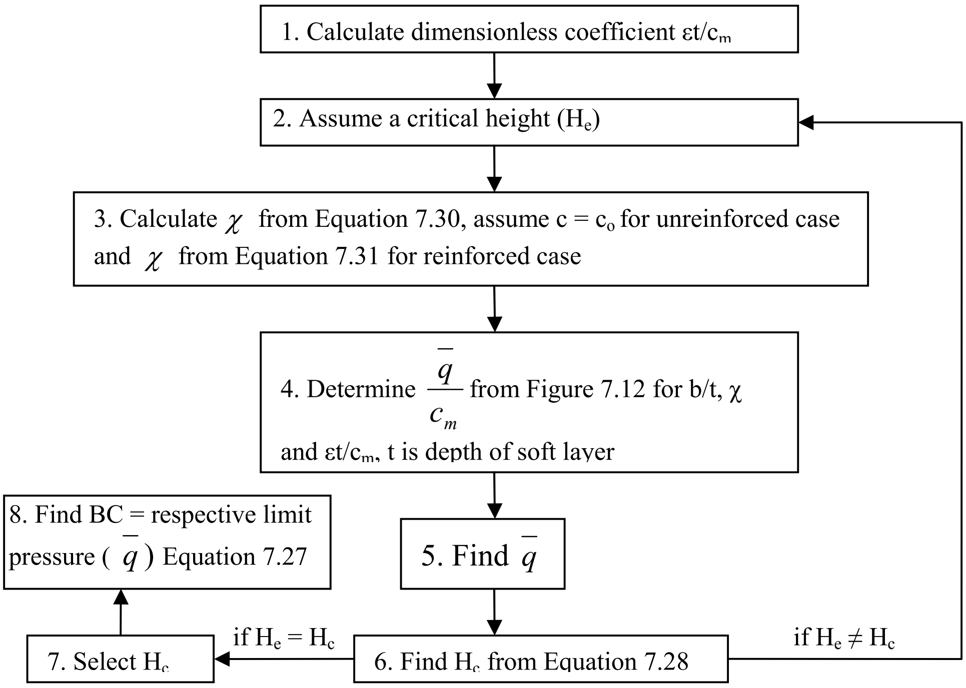



Figure 7.13 Determination of critical height of embankment

## 7.4 Stability Analysis Based on Rapid Drawdown

As stated in Chapter 2 the critical condition in relation to stability arises when the drawdown of water level occurs rapidly in soils of low hydraulic conductivity, such that the pore water in the embankment do not get sufficient time to drain. Such an example, from Tongi-Ashulia road in Bangladesh is shown in Figure 7.14. After the flood a three metre drawdown in water level took place within one month.

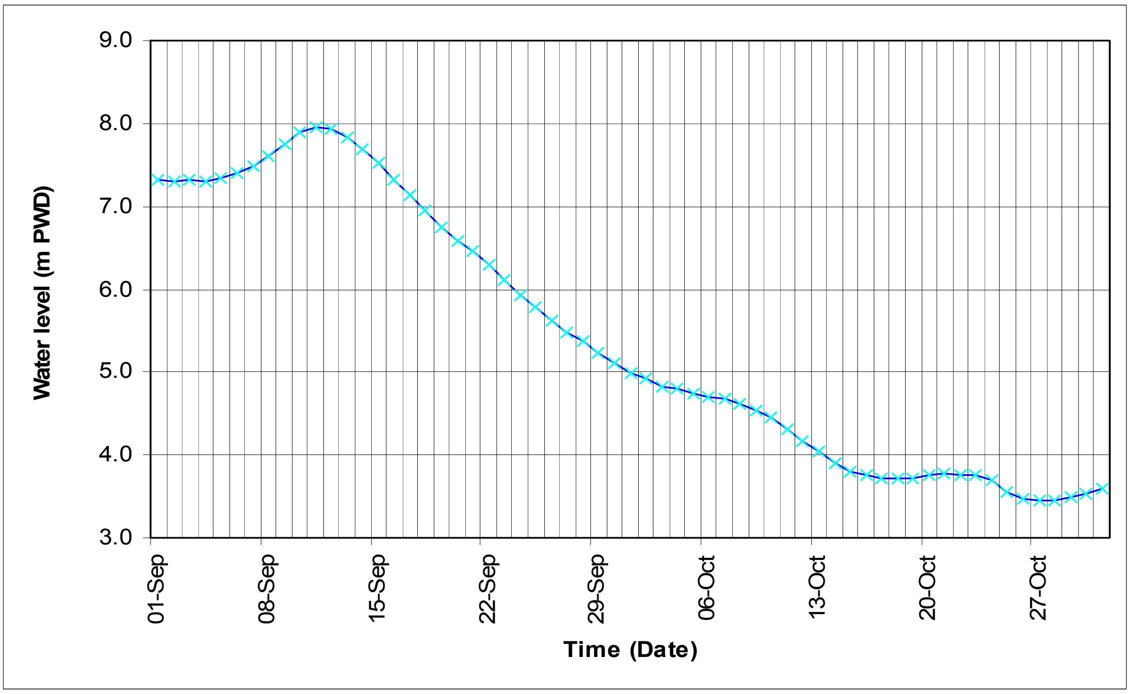



Figure 7.14 Rapid drawdown of water level after 1998 flood at Mirpur station

Embankment stability in such cases can be analysed using charts developed by Morgenstern (1963). Charts shown in Figure 7.17 have been developed for specified slopes (1:2, 1:3, 1:4), specified frictional angle (φ′ = 20, 30 and 40˚), values of *c′/γH* and for the *L/H* (drawdown ratio). The intermediate values can be interpolated (Lane and Griffiths, 2000).

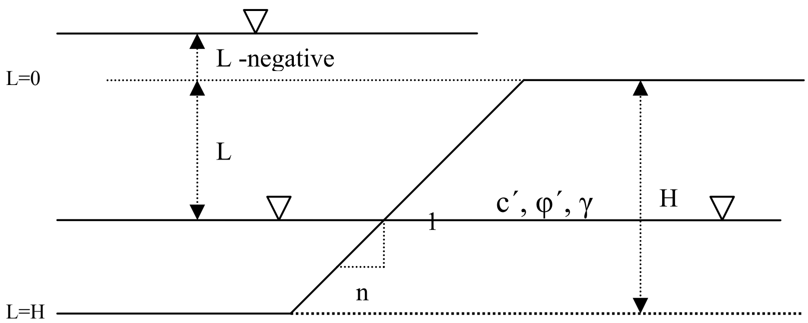



Figure 7.15 Phreatic levels after slow drawdown

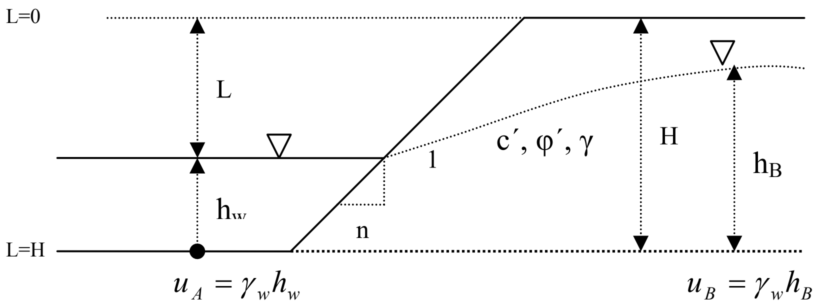



Figure 7.16 Phreatic levels after rapid drawdown

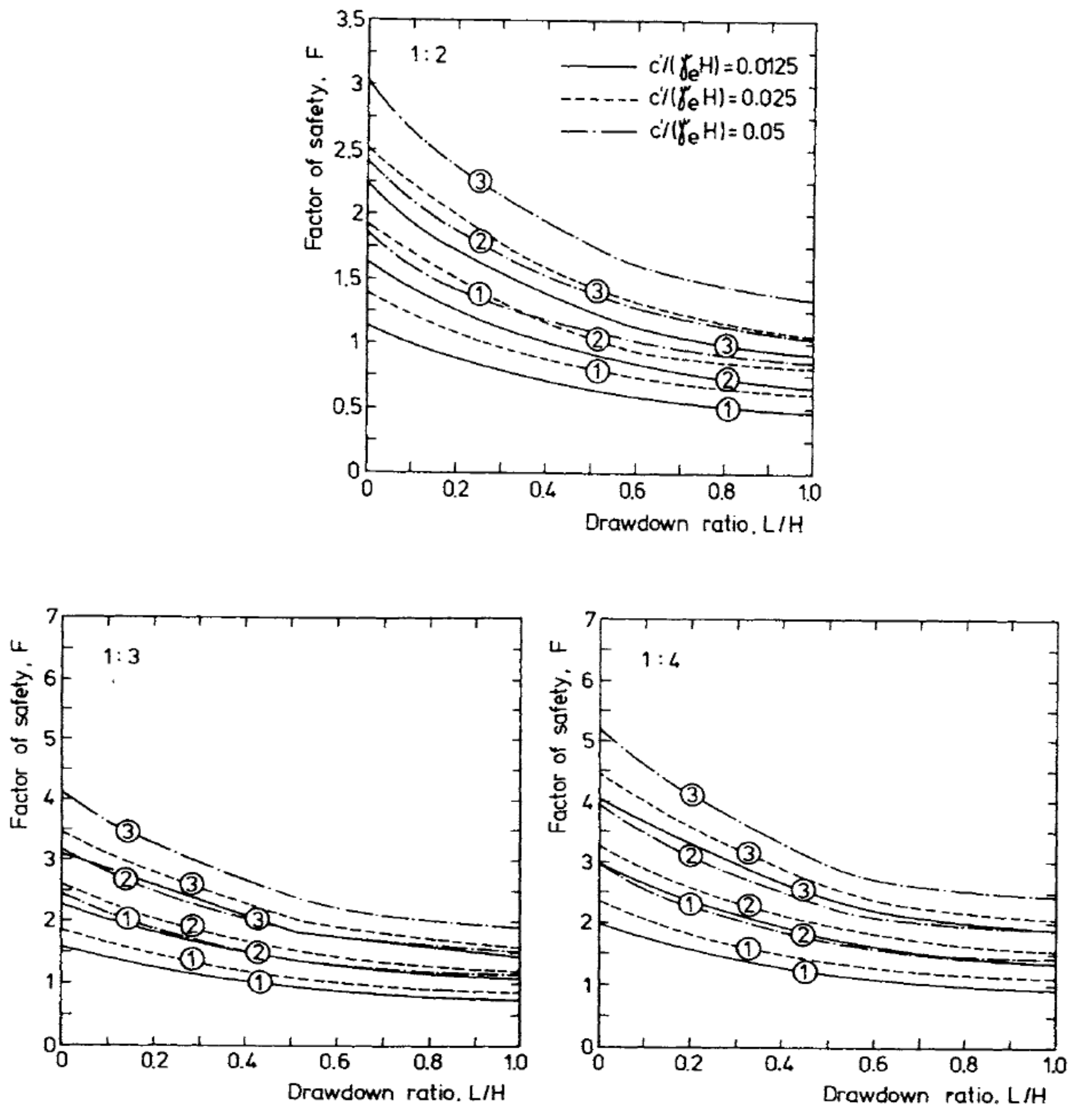



Figure 7.17 Stability charts for embankment in rapid drawdown case for Slopes 1:2, 1:3 and 1:4. Chart (1) $\varphi = 20^0$ (2) $\varphi = 30^0$ (3) $\varphi = 40^0$ (after Morgenstern, 1963).

## 7.5 Methods of Improving Stability

If the factor of safety does not satisfy the design criteria, the stability of an embankment can be improved as described in Section 2.6.2 using the following methods:

- Use of reinforcement
- Adopting stage construction
- Use of pressure berms
- Vegetation

### 7.5.1. Use of Reinforcement

Reinforcement, when it is placed at the interface of the embankment and foundation has negligible impact (shown in Figure 7.18) on improving slope stability but has significant contribution against bearing capacity, sliding and squeezing failure. Figure 7.18 was developed in this study for factor of safety for various undrained shear strength ($S_u$) keeping the other parameters (as mentioned in top of the Figure 7.18) constant following Bishop's method and using OASYS SLOPE.

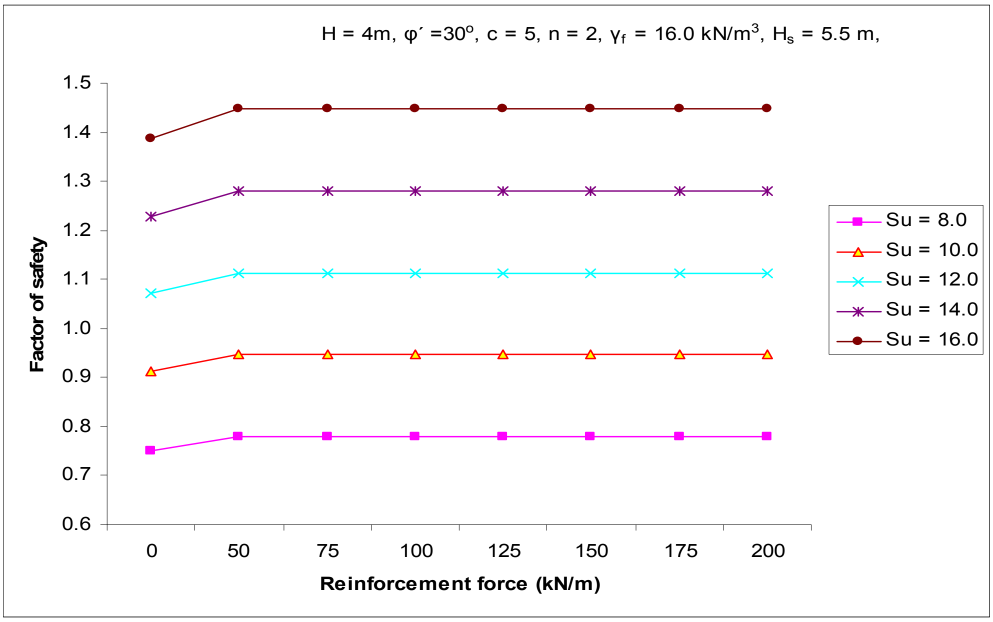



Figure 7.18 Impact of reinforcement on slope stability when it is placed at the interface of embankment and foundation.

From the Figure 7.18, it is seen that factor of safety increases a little amount after using geo-textile at the bottom of the embankment but further increase in reinforcement strength does not help further increase in factor of safety.

The total reinforcement requirement to maintain overall stability (slope stability, bearing capacity, stability against sliding and squeezing) of embankments constructed on soft foundation soil can be calculated by a set of formula proposed by Kanairaj (1988) as follows:

Step 1: Find the minimum value of $n$ ($n_1$) for bearing capacity

$$n > \frac{1}{\pi + 2} \frac{\gamma_f H_s}{S_u} \left( 1.5 - \frac{b}{H_s} \right) \qquad 7.32$$

Step 2: Find minimum value of $n$ ($n_2$) to protect sliding

$$X \tan \varphi \sqrt{1 + n^2} + \left( 1 - X \tan \varphi \sqrt{1 + n^2} \sqrt{1 - \frac{\cos^2 \varphi (1 + n^2)}{n^2}} \right) > 1 \qquad 7.33$$

Step 3: Find minimum value of $n$ ($n_3$) to prevent squeezing out of foundation soil

$$n > \lambda (0.5\alpha - 2) \qquad 7.34$$

Step 4: Find maximum value of n among step 1 to 3

Step 5: Analyse slope stability by Bishop's method (using OASYSYS SLOPE) with reinforcement if required to maintain desired factor of safety for a range of $n$ which satisfy step 4.

Step 6: Select combination of reinforcement and the value of $n$ to satisfy the following condition which was derived for reinforcement requirement assuming maximum sliding and squeezing force acting on the reinforcement.

$$P_R > 0.5 \gamma_f K_a H^2 + n_3 H S_u \qquad 7.35$$

Where, $n$ = slope of embankment (1: $n$), $\gamma_f$ = unit weight of fill, $H_s$ = depth of soft layer, $H$ = height of embankment, $S_u$ = shear strength of foundation soil, $\alpha$ = $\gamma_f H/S_u$, $\varphi$ = angle of internal friction of fill materials.

$$K_a = \text{Coefficient of active earth pressure} = \frac{1 - \sin \varphi}{1 + \sin \varphi} \qquad 7.36$$

An example using this approach is shown in Appendix-XIII.

### 7.5.2. Use of Pressure Berms

Pressure berms are alternative options to achieve desired stability of embankments where fill materials and land are available (Carlsten, 1996). Pressure berms which act as counterweight are particularly useful in making the embankment stable against sliding due to embankment and traffic loading. The associated design process is as follows (Carlsten, 1996):

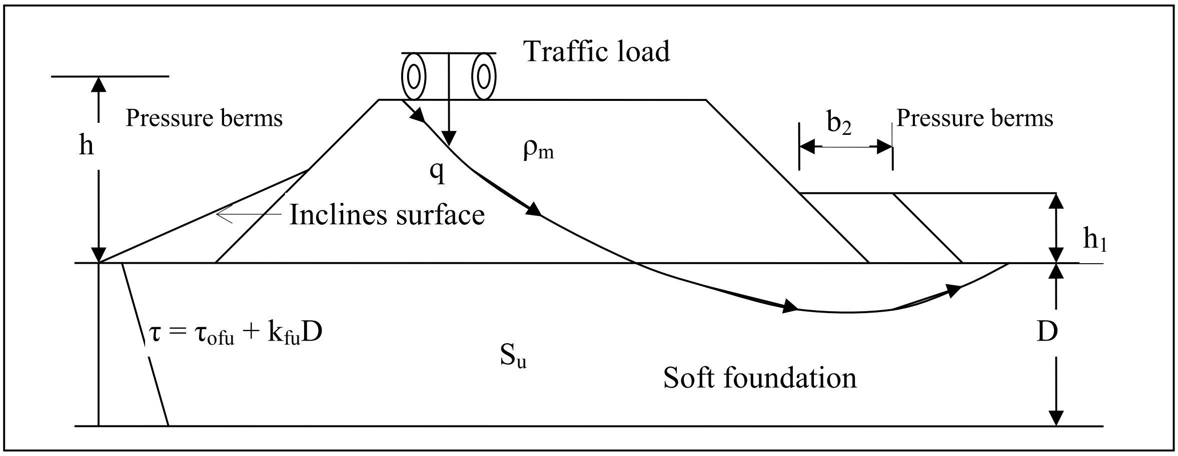



Figure 7.19 Description of embankment with pressure berms

#### 7.5.2.1. Height of Pressure Berms

The height of the pressure berms for both cases (vertical and inclined surface) can be determined from the following equation (Ekstrom et al., 1963; cited in Carlsten, 1996):

$$h_1 = \frac{\left(h\rho_m g + q_{traffic}\right) - \dfrac{5.52 S_u}{F}}{\rho_{pb} g} \qquad 7.37$$

Where, $h$ = height of embankment (m), $\rho_m$ = density of embankment materials (t/m$^3$), $g$ = gravitational acceleration (m/sec$^2$), $q_{traffic}$ = traffic load (kPa), normally 10, $S_u$ = undrained shear strength of foundation soil (kPa), $F$ = factor of safety.

#### 7.5.2.2. Width of Pressure Berms

The width of the pressure berm in this case is determined from the nomograph (Figure 7.20) developed by Ekstrom et al. (1963; cited in Carlsten, 1996) as follows:

Find 1. Allowable shear strength, $\tau_{allow} = \tau_u / F$

2. $\tau_{allow}/q_1$ Where, $q_1$ = load of embankment ($q$) + load of traffic

3. from nomograph (Figure 7.20) value of $b_2/D$

4. $b_2$ = value × D, where $D$ = depth of soft foundation

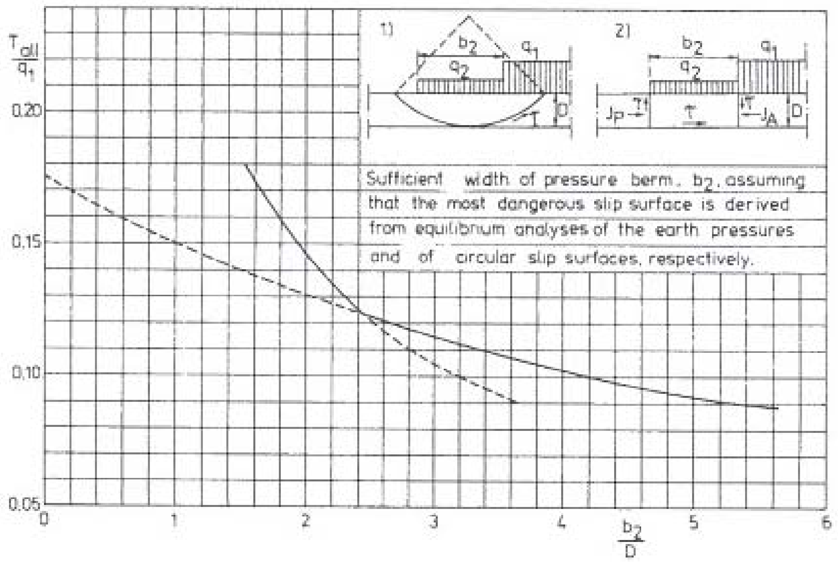



Figure 7.20 Determination of width of pressure berms (after Ekstrom et al., 1963; cited in Carlsten, 1996)

An example for a number of embankment heights is given in Appendix -VIIII

### 7.5.2.3. Inclination of Surface

If the pressure berms are used as inclined surfaces as mentioned in Figure 7.19, the inclination or slope (1: $n$) is determined by the following procedure with the help of Figure 7.21 (Ekstrom et al., 1963; cited in Carlsten, 1996):

Step 1: Find the constant $k$, $k = \dfrac{k_{fu}}{F}$ 7.38

Step 2: Find allowable shear stress $\tau_o$, $\tau_o = \dfrac{\tau_{0fu}}{F}$ 7.39

Step 3: Find the normalised factor, $\dfrac{k.D_1}{\tau_o}$ 7.40

Step 4: Find the normalised factor, $\dfrac{q_1}{\tau_o}$ 7.41

Step 5: Find $k/\sigma$ from Figure 7.21

Step 6: Determine the value of $n$, $n = \frac{k}{\sigma}\frac{\gamma}{k}$ 7.42

Where $k_{fu}$ = shear strength gradient with depth of foundation, $k$ = a constant and defined in Step 1, σ = the increase in load in transverse direction, $F$ = Factor of safety, $D_1$ = depth of soft stratum, $\tau_{ofu}$ = shear strength at the top of the foundation, $\tau_o$ = allowable shear stress at the top of the foundation, $q_1$ = total load = embankment load + traffic load, $\gamma$ = unit weight of fills, $n$ = inclination of the pressure berms.

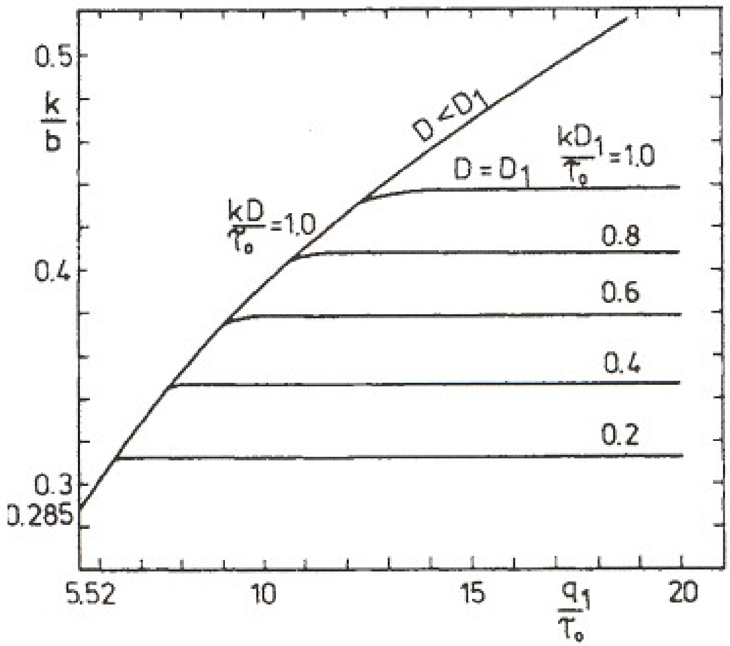



Figure 7.21 Nomogram for the estimation of slope of pressure berms (Ekstrom et al., 1963)

### 7.5.3. Vegetation

The shear strength of upper one metre of soil in an embankment can be improved by using low height vegetation on the soft verge and slopes (Simon and Collison, 2002). The increase of shear strength depends on root depth, density and strength. A study based on large shear box test with and without roots of various plants was carried out by Simon and Collison (2002) from where they provided values of shear strength increase by various vegetations along with the tensile strength, number and diameter of roots as shown in Table 7.4.

However disturbing stresses from the stems and branches reduce the factor of safety to some extent. It was found by Simon and Collison (2002) that the disturbing effect of low height plants is negligible compared to the shear strength contributions by the roots. To reduce the disturbing stress form the plants low height vegetation is encouraged.

Table 7.4 Plantation effect on slope stability (after Simon and Collison, 2002)

| **Parameters/ name of plants** | **Sycam ore** | **River birch** | **Sweetgum** | **Gamma grass** | **Black willow** | **Switch grass** |
|---|---|---|---|---|---|---|
| Shear strength increase ($C_r$), kPa | 7 | 8 | 4 | 6 | 2 | 18 |
| Average root diameter (mm) | 3.1 | 3.2 | 4.8 | 1.4 | 2.5 | 1.3 |
| Number of roots | 117 | 66 | 56 | 76 | 137 | 72 |
| Average age (years) | 7 | 7 | 12 | 5 | 5 | 5 |
| FS increase due to root tension | +0.40 | +0.40 | +0.23 | +0.34 | +0.14 | +1.02 |
| FS increase due to Surcharge effect of stem and branches | -0.07 | -0.08 | -0.06 | 0.00 | -0.03 | 0.00 |

## 7.6 Summary

Two design considerations need to be taken into account for embankments built in conditions which are similar to those in Bangladesh. These are slope stability and bearing capacity failure. For the former, various methods are available to analyse an embankment. This chapter described a number of these and discussed their advantages and disadvantages. A comparison of the outputs from each method was also made using a hypothetical example. From this analysis it was concluded that a method suggested by Bishop (1955) was the most accurate, it considered the most number of input parameters and is suitable for analysing drawdown conditions. However, Bishop's method requires computer software to implement. In the absence of any such software, a modified method given by Low (1989) is recommended for the task in hand as it considers the conditions similar to those experienced in Bangladesh and can easily be implemented in a spreadsheet.

For the analysis of bearing capacity Radoslaw's (1992) method is recommended and it has the advantage that it can easily be implemented in an Excel spreadsheet.

For both types of failure, the shear strength of the embankment and foundation greatly affects the slope stability and bearing capacity of the embankment. A variety of means of improving the shear strength, and thus embankment stability were described in an earlier chapter, whilst it is felt that for the conditions experienced in Bangladesh stage construction may be the most appropriate as it allows for consolidation settlement. To further understand the stability failure in relation to the conditions experienced in Bangladesh the next chapter presents an analysis of rapid drawdown for a number of soil types.

# Chapter 8 Stability Modelling

## 8.1 Introduction

In the previous Chapter several methods for the analysis of slope stability and bearing capacity were compared and two methods were identified as potentially suitable. One of these is based on Bishop's method and because of its complexity requires specially designed computer software to implement it. OASYS SLOPE which was available within the School of Civil Engineering is such a piece of software and was used in this work as described below. A simpler method using solutions suggested by Low, which can easily be implemented in a spreadsheet, was identified as being suitable if software such as OASYS SLOPE is not available and sustainable in Bangladesh.

Using OASYS SLOPE, a number of design charts have been developed to facilitate the design process in the absence of computer based software. In addition, an investigation was carried out to identify the most influential factors which may affect the stability of an embankment.

## 8.2 OASYS SLOPE

OASYS SLOPE is a commercially available piece of software which enables slope stability analyses to be carried out using a variety of methods. These include those suggested by Fellenius (1927; cited in Craig 2004), Janbu (1967) and Bishop (1955) and Spencer (1967). For this work Bishop's method was used to perform stability analysis for the reasons given in Section 7.2.4.

Two comparisons using the outputs from OASYS SLOPE were made as follows:

1. Back calculated shear strength with the observed shear strength at a failed section of an existing embankment in Bangladesh.
2. Outputs from OASYS SLOPE with Mitchell's charts.

### 8.2.1. Comparison by Back Analysis

Data from an observed slope failure of the Tongi-Ashulia road (N302) were used to compare the shear strength of material in a slope which had failed with the shear strength at failure (Factor of Safety = 1) determined using Bishop's method implemented in OASYS SLPOTE.

The average measured shear strength of the upper 2.5 m where the slope failure occurred on the Tongi-Ashulia road was found to be 20.5 kPa (Figure 8.2). For the same conditions Bishop's method gave a shear strength of 18 kPa (see Figure 8.1). Alternatively using the measured shear strength of individual layers the factor of safety was found to be 1.06 (i.e. close to failure when FoS = 1.0). In addition to that a probabilistic stability analysis with a set of measured parameters gives a probability of failure as 90%. These results imply that the outputs from the model are reasonable can be used for further stability analysis in this region with acceptable accuracy.

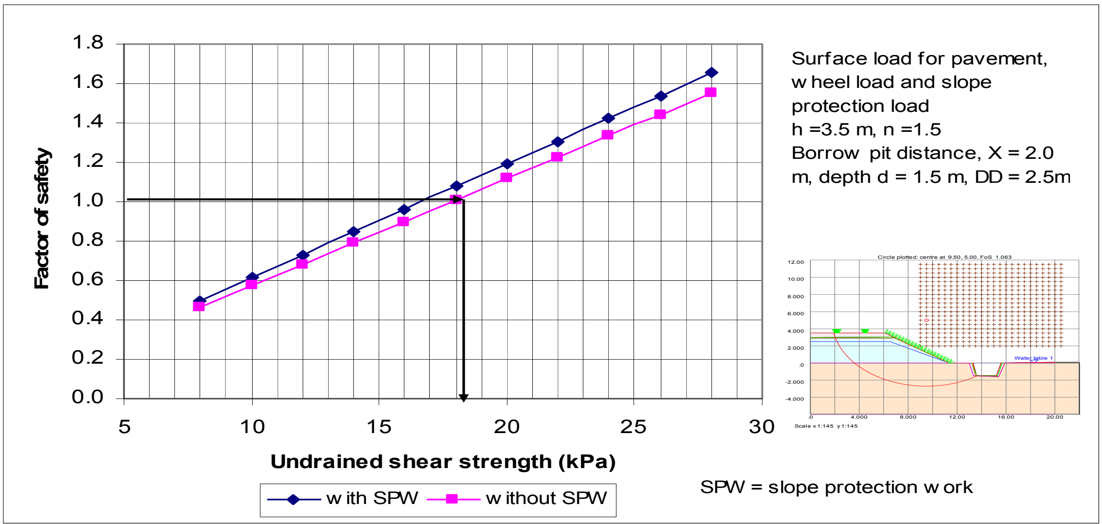



Figure 8.1 Back analysis of shear strength at the failure section of Tongi-Ashulia Road

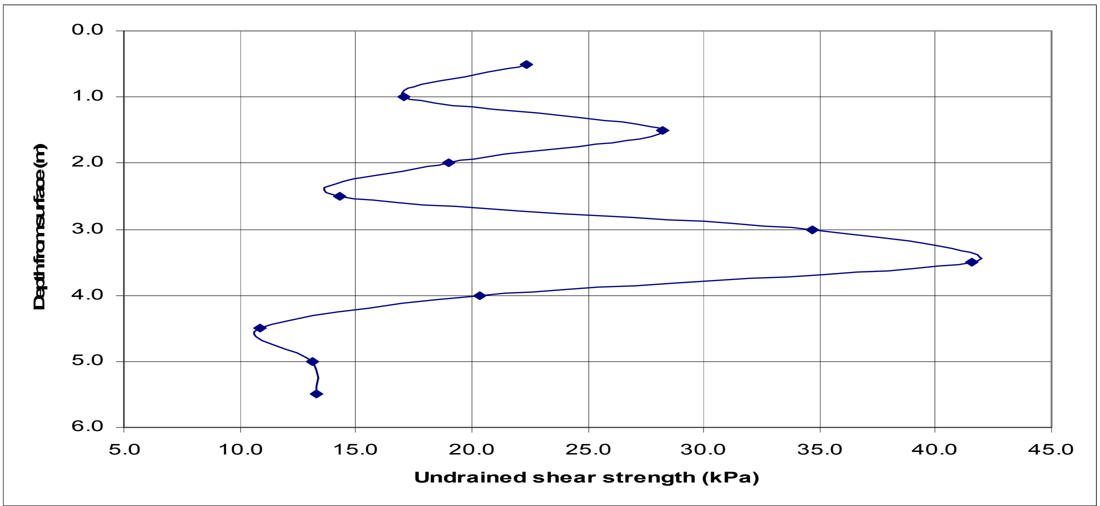



Figure 8.2 Undrained shear strength of soil near failure section of Tongi-Ashulia Road from lab test

### 8.2.2. Comparison with Mitchell's Charts

A comparison was carried out between Bishop's method and Mitchell's chart (Mitchell, 1983) as shown in Figure 8.3. Mitchell's charts provide the value of factor of safety against slope stability for a set of soil properties such as: unit weight, drained shear parameters and $r_u$ (see Section 7.2.1.2). By inspection, the two methods give comparable results providing further evidence of the accuracy of the software.

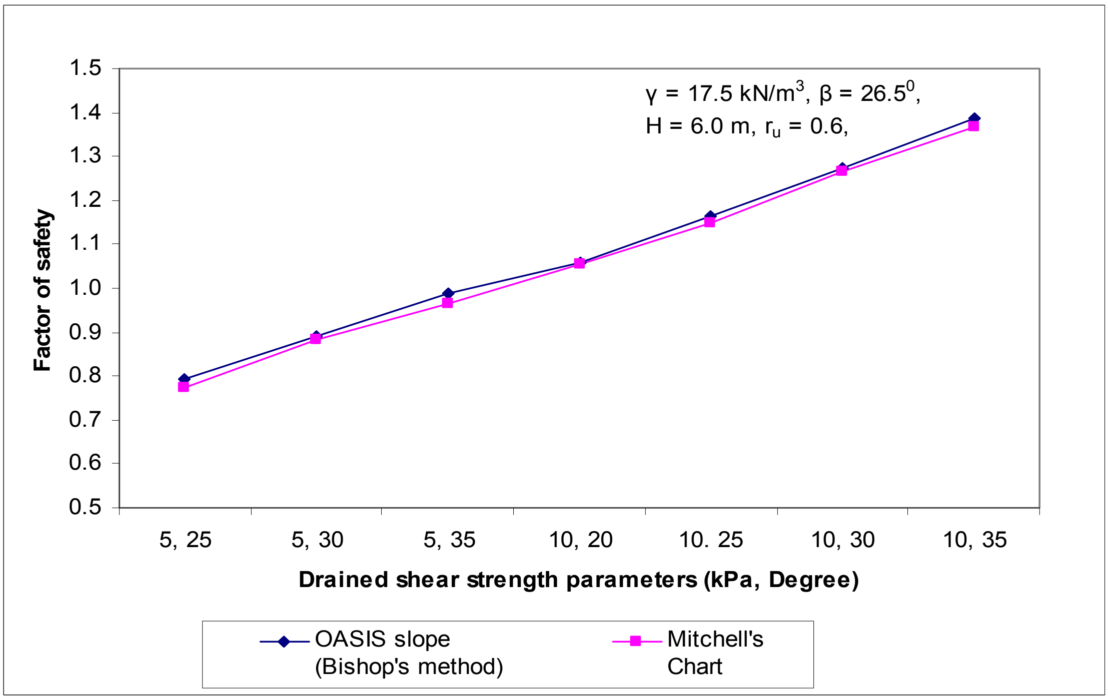



Figure 8.3 Comparison between Bishop's method and Mitchell's (1983) chart

## 8.3 Stability Analysis with Rapid Drawdown

An analysis was carried out to understand the effect of rapid drawdown on slope stability as shown in Figure 8.4. Factors of safety were calculated based on various drawdown ratios for a number of soil shear strengths ($S_u$) keeping other soil parameters and embankment geometry constants. The analyses were carried out assuming undrained conditions using the same soil strength value for both the embankment and foundation. The slope and height of embankment were assumed to be 26.5$^o$ and 4 m respectively and the soil unit weight was assumed to be15.3 kN/m$^3$.

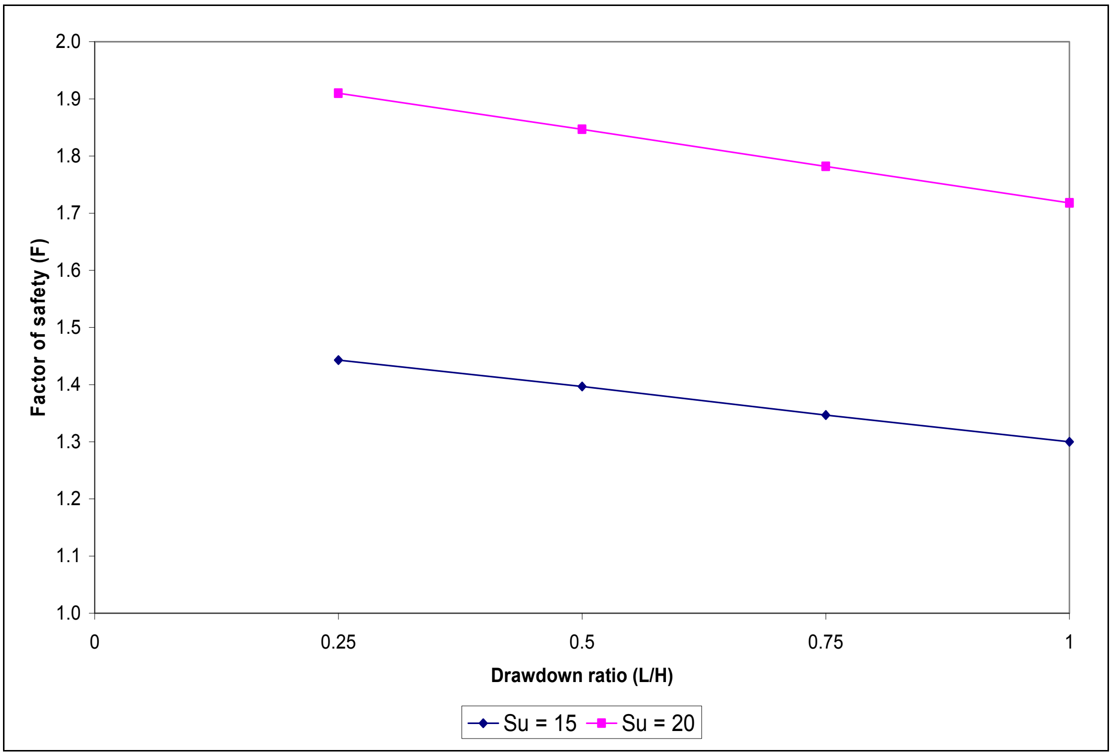



Figure 8.4 Influence of drawdown amount on factor of safety

From the above figure it is seen that factor of safety reduces with the increase of drawdown ratio (L/H) and lowest factor of safety was found for a drawdown ratio equal to one. The symbols L and H stand for the amount of drawdown and embankment height respectively.

## 8.4 Slope Stability Chart

Stability charts have been developed to determine the factor of safety against slope failure. To this end, Factors of safety have been calculated considering full rapid drawdown as a worst case with the embankment constructed from the same soils as that of the foundation. The charts have been developed for embankment heights from 3 to 6 metres with a typical slope angles. Namely, 33.7$^o$ (1:1.5), 26.6$^o$ (1:2), and 21.8$^o$ (1:2.5). Long term stability can be found from these charts as the effective stress parameters (drained parameters) are used. The charts are given in Appendix-III in tabular and in graphical format.

## 8.5 Sensitivity Analyses

As mentioned previously a research project is being undertaken in parallel to this one to investigate suitable techniques which may be used in Bangladesh to determine values of various properties of materials required for the design of the road pavement and embankment. In order to be able to provide information to the parallel project on the material properties which may be considered to be most important for embankment design, a sensitivity analysis was undertaken and to identify the most influential factors which may affect the stability of an embankment. In the analysis the value of each parameter was varied and the factor of safety was calculated for the mean, the lowest and the highest values of each parameter keeping the other parameters at their mean values. The ratio of the percentage change of the factor of safety for each parameter is shown in Figure 8.10. The effects of each parameter on the factor of safety are also shown in Figures 8.5 to Figure 8.9.

As expected, the factor of safety increases linearly with the increase in the value of the parameters $c'$ and $\varphi'$. On the other hand increasing the value of the parameters $\gamma$, $\beta$, $H$ and $r_u$ decreases linearly the factor of safety.

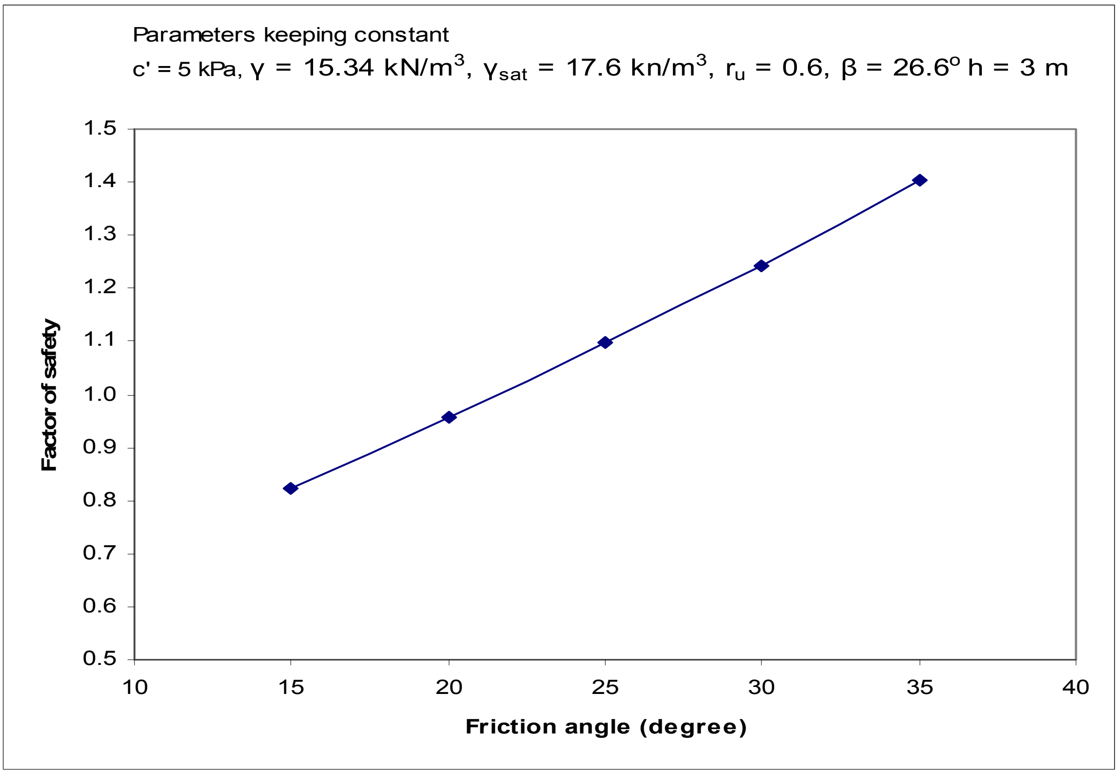



Figure 8.5 Effect of effective friction angle on factor of safety

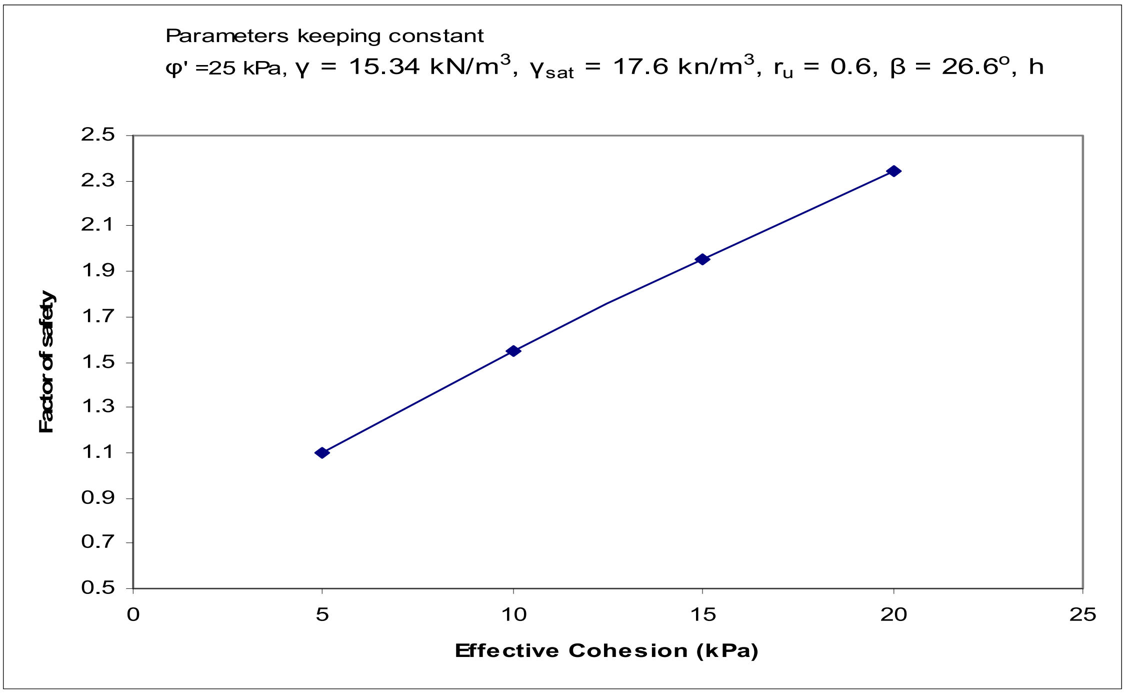



Figure 8.6 Effect of effective cohesion on factor of safety

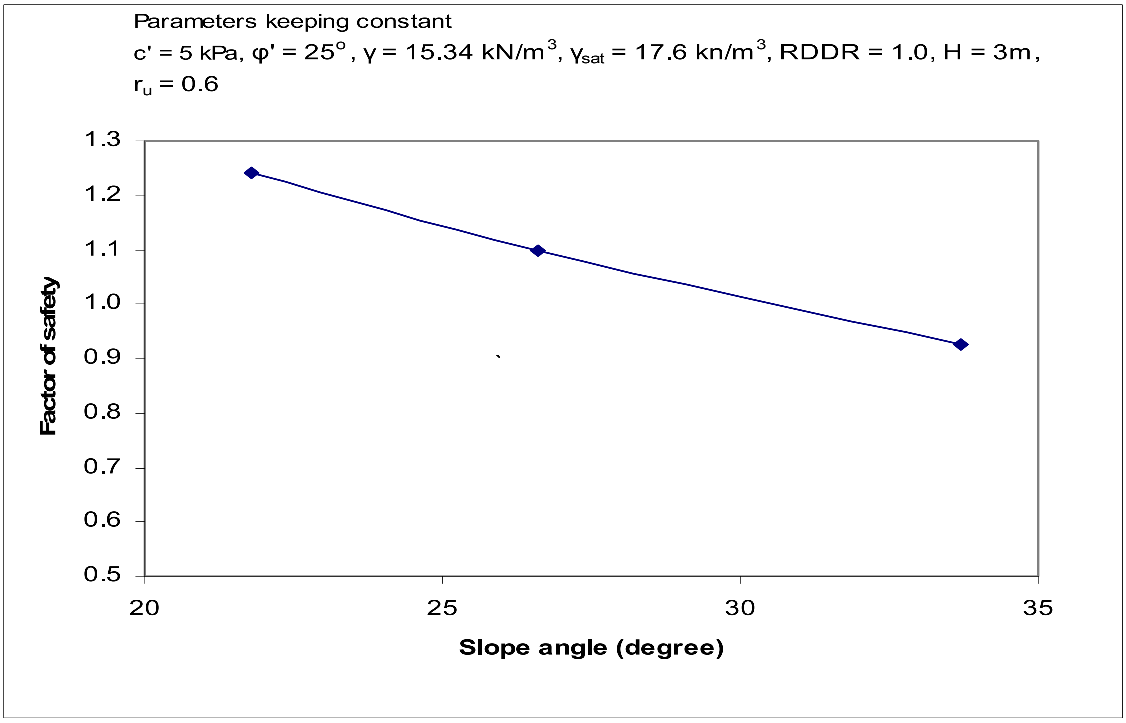



Figure 8.7 Effect of slope angle on factor of safety

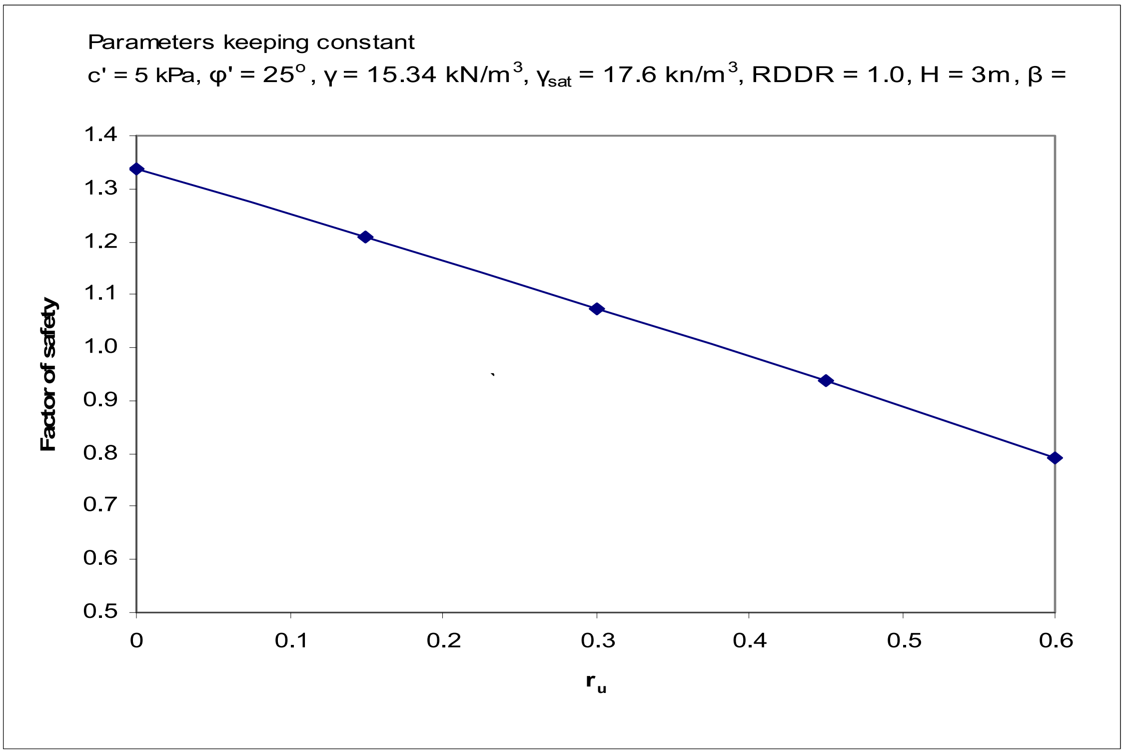



Figure 8.8 Effect of $r_u$ on factor of safety

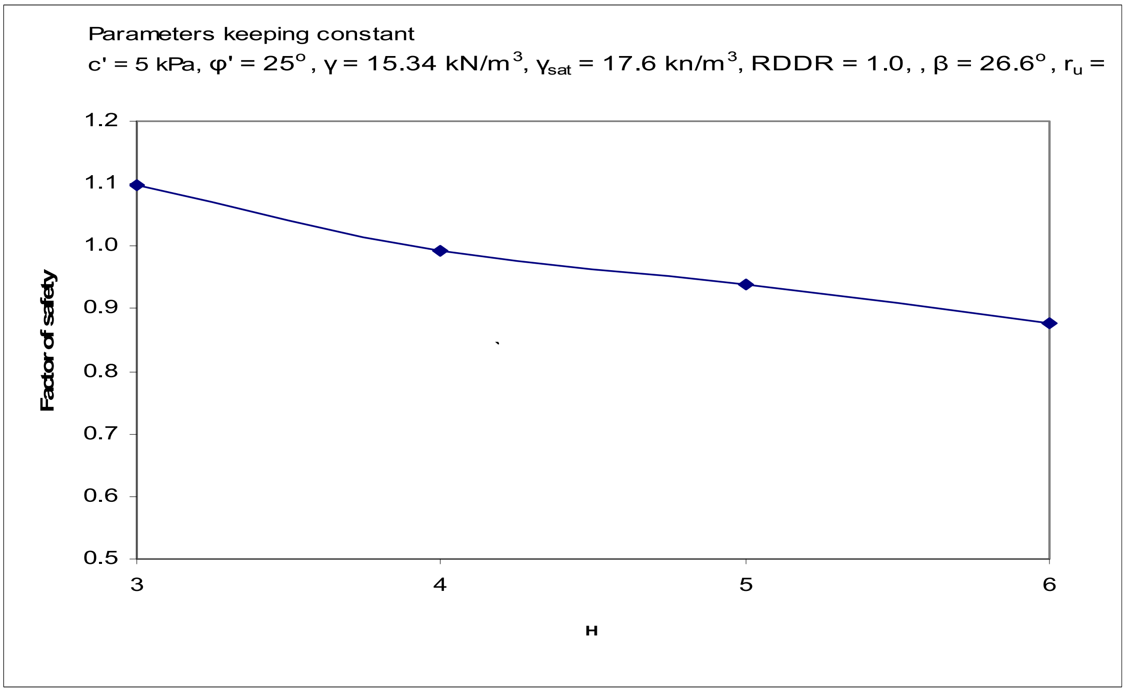



Figure 8.9 Effect of embankment height on factor of safety

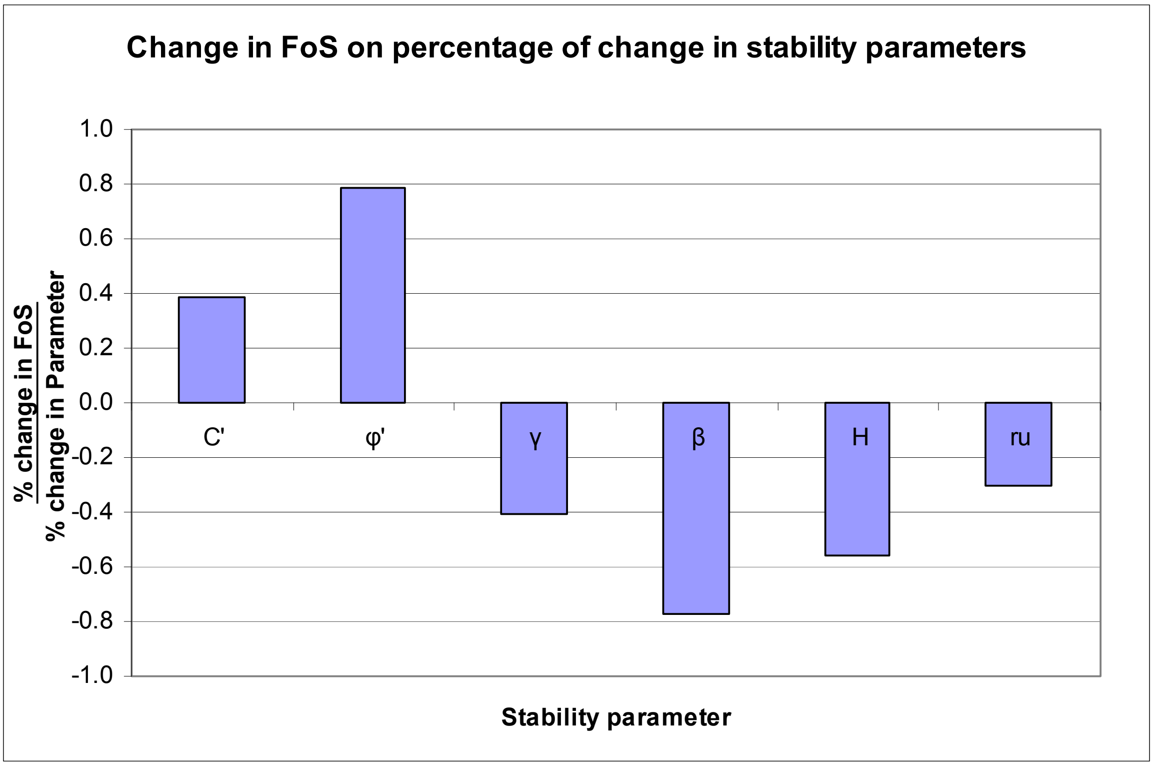



Figure 8.10 Sensitivity of stability parameters

It may be seen from the Figure 8.10 that the stability of embankment is most sensitive to the friction angle ($\varphi'$), slope angle ($\beta$) and height of embankment ($H$). The influence of $\varphi'$ and ($\beta$) are almost double to that of $c'$.

## 8.6 Stability analysis with Low's method

Ideally OASYS SLOPE should be used for the purpose of slope stability analysis. However it is recognised that in Bangladesh and similar countries, the software may not be readily available or supported. For this reason it was felt necessary to investigate the use of a simpler, less accurate method of analysis.

Low's (1989) method was chosen for this work for the following reason.

- It is simpler and can be implemented in a spread sheet.
- The result found by this method show reasonable agreement with other methods described herein (see below).
- The parameters used by this method are easily obtainable.

- The method considers undrained conditions for the foundation and drained ones for embankment materials, which is the usual case of embankment construction in Bangladesh due to the existence of soft soils.

To investigate the accuracy of Low's method, the factor of safety determined by this method for embankments of three different heights (4m, 5m, 6m) on a foundation with a variety of shear strength values have been compared with Fellenius (Swedish), Janbu and Bishop. The factor of safety determined using Low's method were calculated using a spreadsheet whereas the others were determined using OASYS SLOPE. The comparisons are shown in Figures 8.12, 8.13and 8.14. By inspection of the figures it may be seen that the calculated factor of safety are in good agreement for all three embankment heights considered and for the range of foundation shear strength values used. Although Low's method does not consider rapid drawdown of water level, factor of safety can be modified from the relationship between factor of safety with and without rapid drawdown which is prepared with the help of OASYS SLOPE following Bishop's method as given in Figure 8.11.

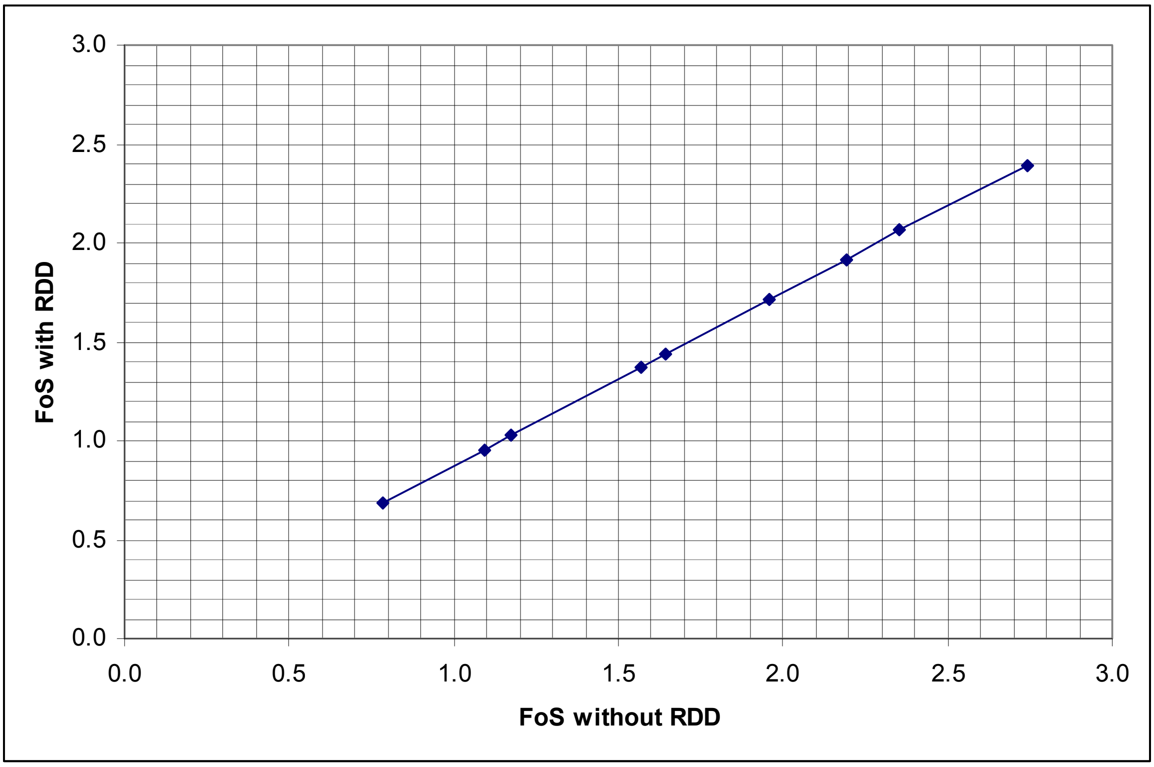



Figure 8.11 Modified factor of safety considering full rapid drawdown (RDD)

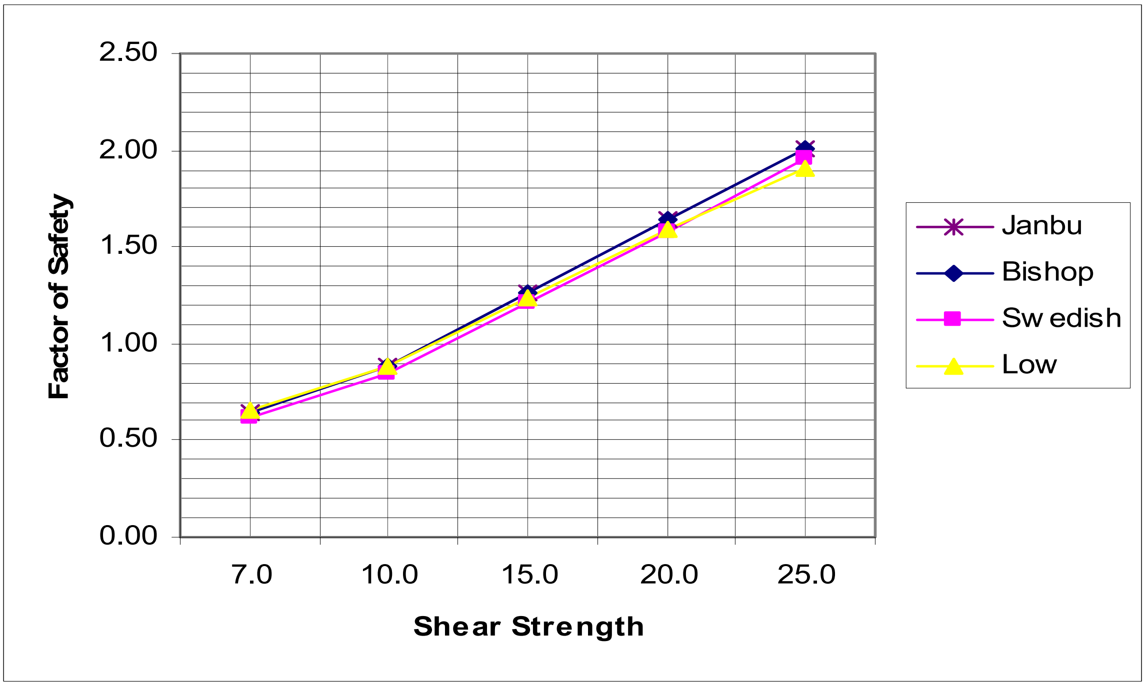



Figure 8.12 Factor of safety variations with shear strength of foundation (h = 4.0m)

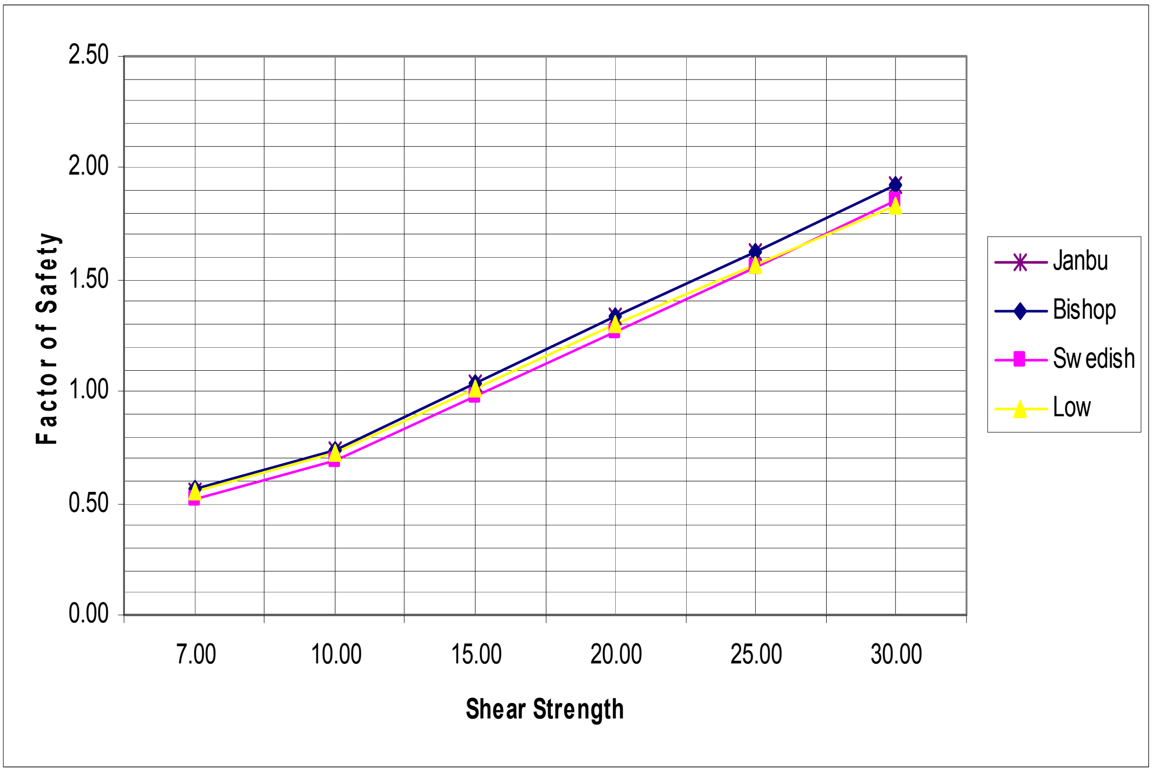



Figure 8.13 Factor of safety variations with shear strength of foundation (h = 5.0m)

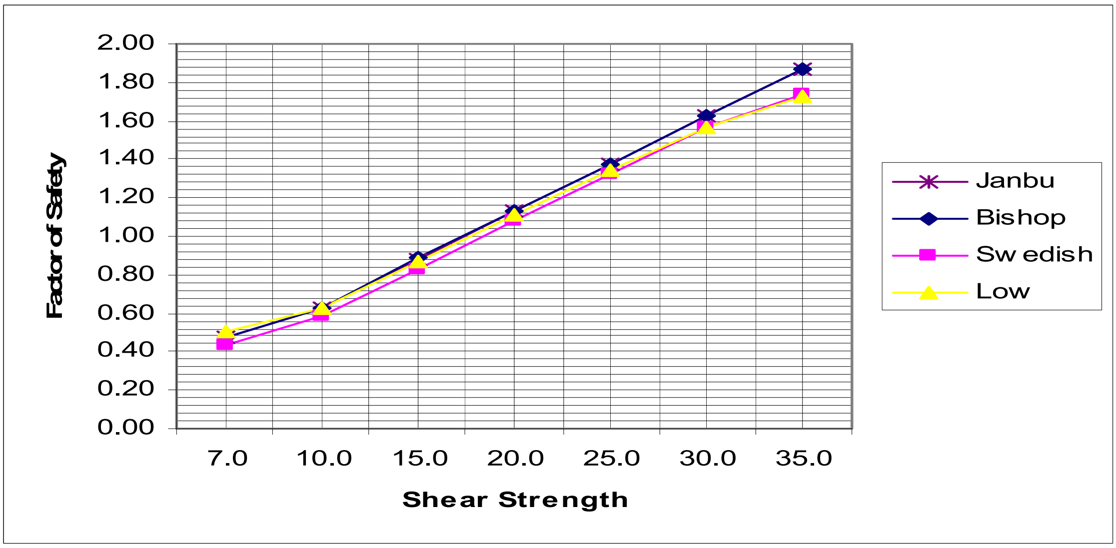



Figure 8.14 Factor of safety variations with shear strength of foundation (h = 6.0m)

## 8.7 Probabilistic Stability Analysis

In the conventional stability analysis the ultimate decision making product is the factor of safety, a unit less number greater than one. However, it does not clearly indicate the reliability of the soil properties in situ which vary spatially. Consequently, a probabilistic approach should be adopted to take into account the certainty or uncertainty in respect of variability of soil parameter. This aspect has been dealt with fully in a research project entitled "Impact of Data Variability and Quantification of Risk in the Pavement and Embankment Design" which is being carried in parallel to this one. The basis for the approach with an example is also described in Appendix-XII of this thesis.

## 8.8 Summary

OASYS SLOPE, a commercially available slope stability software, was used to carry out a number of analyses based on Bishop's method to determine the sensitivity of embankment slope stability to a number of design parameters. The software was selected because it is available within the School of Civil Engineering and because it is relatively easy to use and maintain and it is recommended that it be accepted for use in Bangladesh. The analyses carried out by the software as described in this chapter included;

- Comparing the output from Bishop's method with a real slope failure of an embankment by back analysis and with Mitchell's chart. This demonstrated the usefulness and accuracy of Bishop's method implemented using OASYS SLOPE for such analyses.

- Comparing the output from Bishop's method with a simpler method (known as Low's method) which can be implemented in a spreadsheet. This comparison demonstrated the suitability of the Low's method, with a view to recommending its use where OASYS SLOPE can not be implemented.
- A sensitivity analysis was conducted to investigate the relative importance of design input parameters on slope stability. This demonstrated that the parameters $c'$ and $\varphi'$ have a positive influence on factor of safety, on the other hand increasing the values of $\gamma$, $\beta$, $H$ and $r_u$ reduce the factor of safety. Embankment stability is most sensitive to the friction angle ($\varphi'$), slope angle ($\beta$) and height of embankment ($H$). The influence of $\varphi'$ and ($\beta$) are almost double that of $c'$.
- Stability design charts were developed using Bishop's method. These take into account full rapid drawdown conditions.

The following chapter describes the method for selecting formation level and identifying the design parameters for the subgrade which will ultimately be used as an input for pavement design.

# Chapter 9 Subgrade Requirements

## 9.1 Introduction

The purpose of an embankment is to provide a suitable platform on which the road pavement may be constructed. In order to achieve this it is necessary to ensure that the residual settlements are within acceptable limits; the long term stability of embankment is achieved and the top of the embankment provides the strength and stiffness characteristics dictated by the requirements of the road pavement which it supports. The first requirement has been discussed in Chapter 4, 5 and 6 and the second requirement has been considered in Chapter 7, and 8. This chapter describes how the strength requirement for pavement design may be achieved in practice. It also describes how these properties should be determined for the embankment in practice so that accurate value can be used in the design process.

An additional, nonetheless important function of embankment is to keep the road pavement from being inundated by flood water. This is necessary not only to ensure the use of road pavement dry in rainy season, but also to prevent unnecessary damage or excessive deterioration which may occur in the pavement construction as a result of water ingress. To achieve this, the embankment should be built to exceed the likely height of any flood which may occur during the defined design life of the structure.

## 9.2 Determination of Formation Level

The high flood level (HFL) for a give return period at a specific station can be given by the Chow's equation (Chow et al., 1988).

$$X_T = \overline{X} + K_T \delta \qquad 9.1$$

Where, $X_T$ = design high flood level (m PWD), $\overline{X}$ = mean value of available maximum yearly water level, $\delta$ = Standard deviation of available maximum yearly water level, $K_T$ = Chow's coefficient (Chow et al, 1988) and is given by the following equation:

$$K_T = -\frac{\sqrt{6}}{\pi}\left\{0.5772 + \ln\left[\ln\left(\frac{T}{T-1}\right)\right]\right\} \qquad 9.2$$

$T$ = Return period

The HFL is determined from hydrological analysis of stations surrounding the project area and the maximum one is selected as the design value. The formation level then can be determined by adding an additional height of embankment (known as the freeboard). It is necessary to establish soil moisture characteristics curves (SMCC) for subgrade materials to be used for determination of a reasonable freeboard. Due to unavailability of those curves freeboards recommended by Roads and Highways Department (RHD), Bangladesh for different road classes were used (see Table 2.9).

## 9.3 Subgrade Properties

The subgrade resilient modulus and Poisson's ratio are the main input variables for the analytical design of highway and airfield pavements. Primarily the California Bearing Ratio (CBR) is parameter of subgrade used to measure the strength for its convenience in the method of testing. Due to the unavailability of testing equipment in Bangladesh for the direct determination of resilient modulus, correlations have been developed in a research project carried out in parallel (Shajahan, in print) to determine it from the CBR of subgrade materials. The value of CBR varies with the moisture content of soil. So determination of the design moisture content (DMC) is required initially to estimate the design CBR. The relationship between density of materials, moisture content and CBR, determination of subgrade modulus and poison's ratio and saturation effect on subgrade modulus are discussed in the following paragraphs. The overall process of estimating subgrade properties is described by a flow chart given in Figure 9.1.

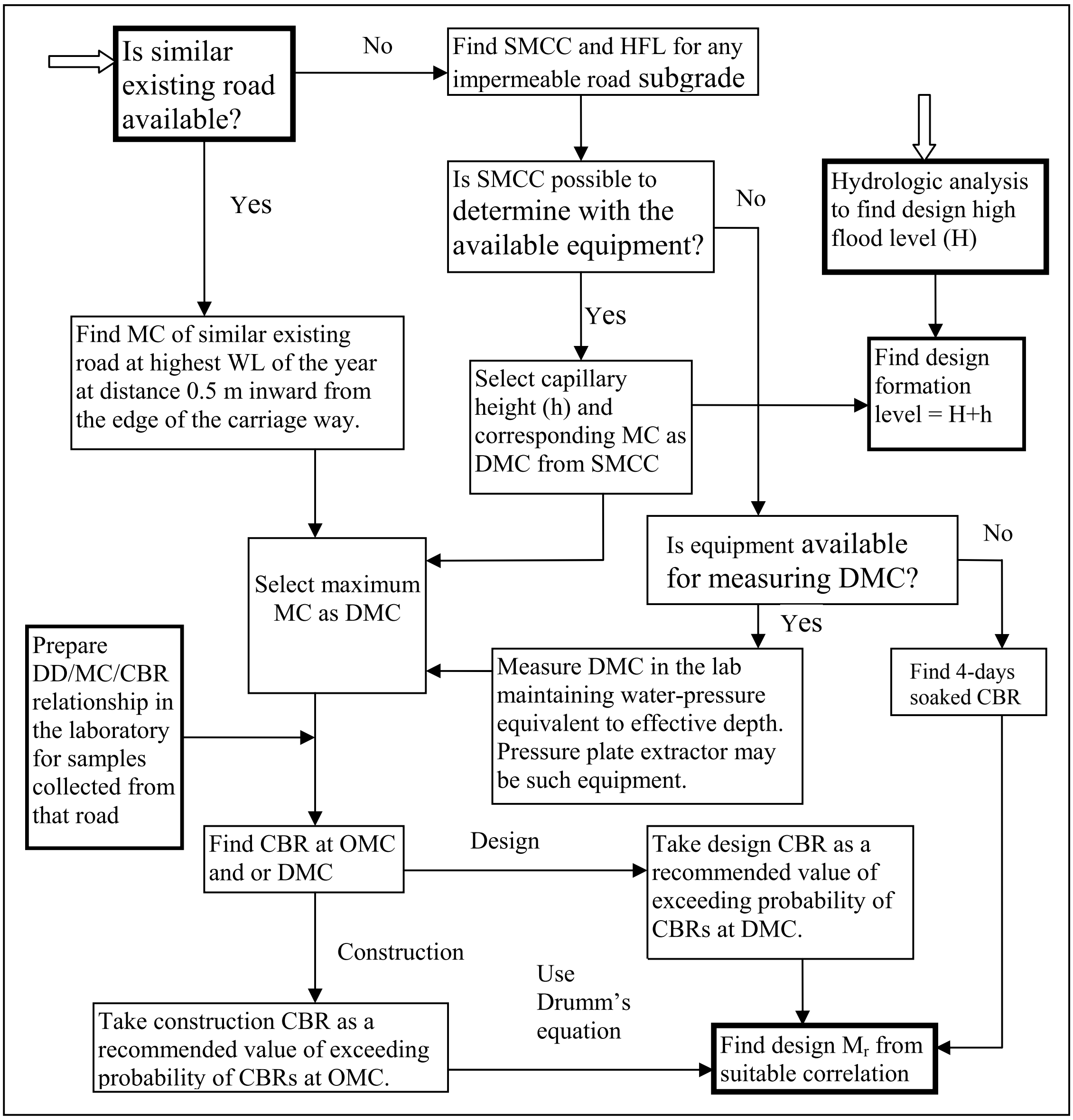



Figure 9.1 Process of estimating subgrade design properties and formation level

### 9.3.1. Density

The density of subgrade materials is largely dependent on the type of materials, moisture content and compaction efforts (Lee et al., 2007; Das 2002; HMSO, 1968; ORN 31, 1993). The maximum dry density and optimum moisture content are functions of grain size distribution, specific gravity, clay content, plastic, liquid limits and plasticity index. For a specific class of soil, the maximum dry density (MDD) can be found for a specific compaction effort at a given moisture content which is known as the optimum moisture content (OMC) for a given compactive effort. Figures 9.2 and 9.3 show typical relationships between dry density, moisture content and CBR and Figure 9.4 shows how dry density increases with the increase of compactive effort. The charts were prepared on the basis of data adopted from BRTC (2005). The process of finding the dry density (DD) at OMC and DMC and corresponding CBR are also shown in Figure 9.2 and 9.3.

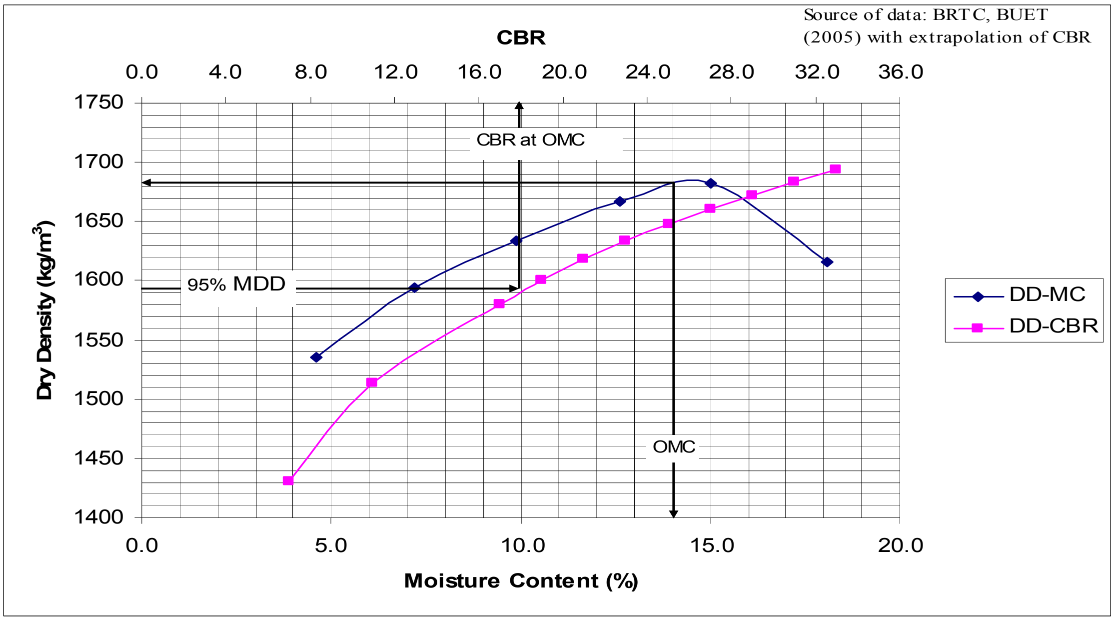



Figure 9.2 Dry unit weight vs. moisture content and determination of CBR at OMC

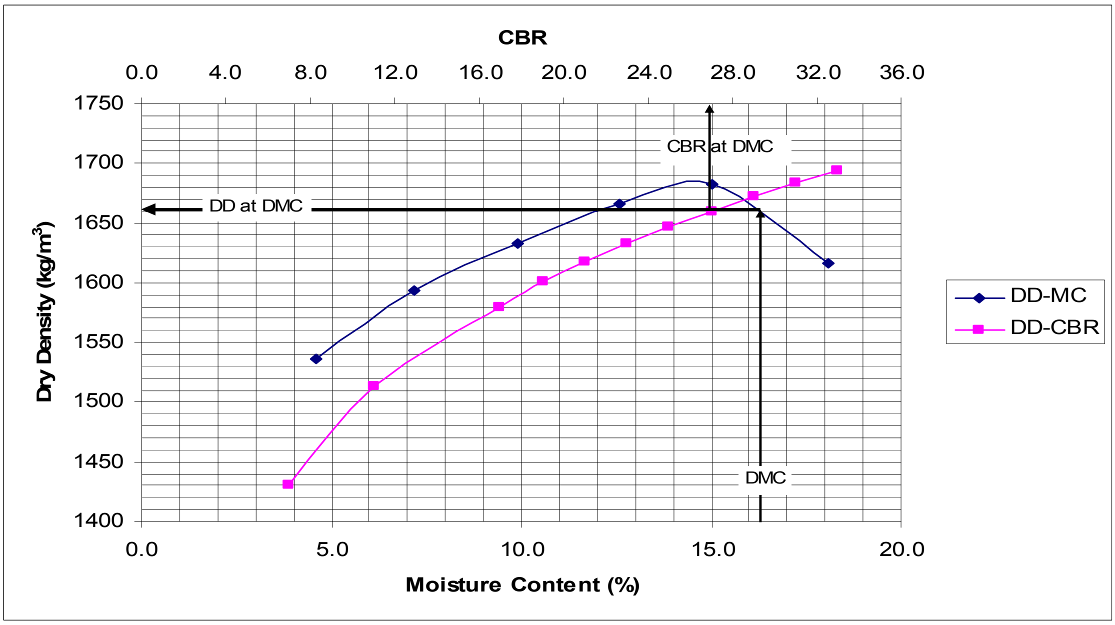



Figure 9.3 Dry unit weight vs. moisture content determination of CBR at DMC

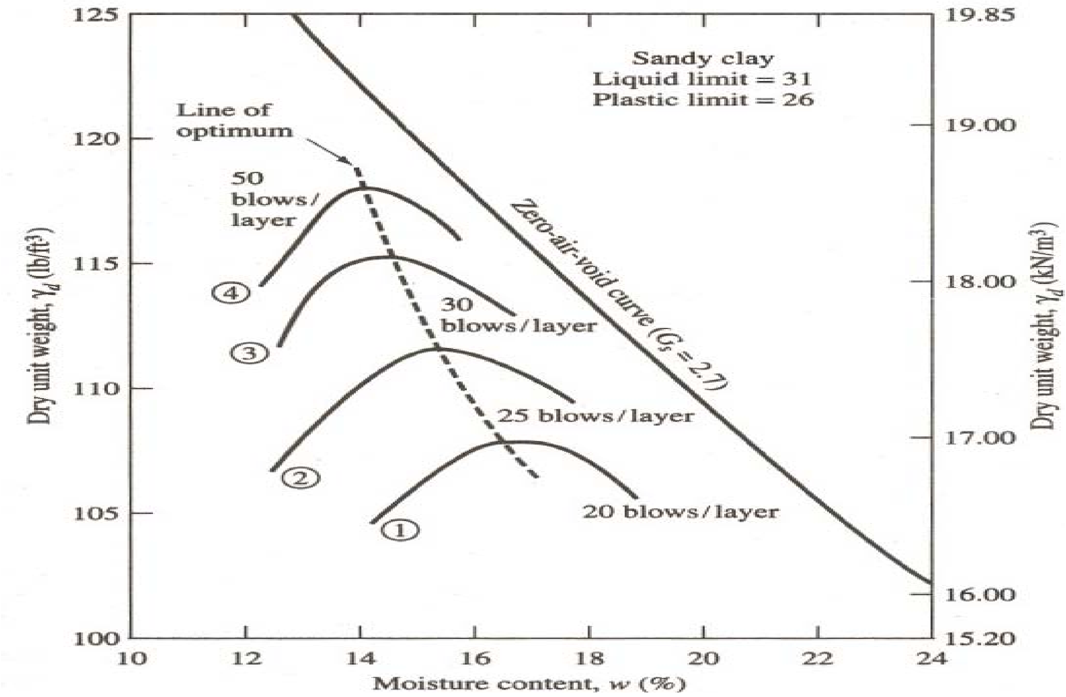



Figure 9.4 Effect of compaction effort on density of soil (after Das, 2004)

The energy required to compact materials at a given compactive effort can be estimated from the following equation (Das, 2004):

$$E = \frac{\begin{pmatrix} Number \\ of \quad blows \\ per \quad layer \end{pmatrix} \bullet \begin{pmatrix} Number \\ of \\ layers \end{pmatrix} \bullet \begin{pmatrix} Weight \\ of \\ hammer \end{pmatrix} \bullet \begin{pmatrix} Height \quad of \\ dropp \quad of \\ hammer \end{pmatrix}}{Volume \quad of \quad mold} \qquad 9.3$$

The energy required complying British (BS 1377-4, 1990) and US (AASHTO, 1982) standards for compaction are shown in Table 9.1 which was prepared in this study using the above equation.

Table 9.1 Energy needed for AASHTO and BS methods of compaction

| **Method** | **Blows per layer** | **No. of layers** | **Weight of hammer (kg)** | **Drop height of hammer (m)** | **Volume of mould ($m^3$)** | **Compaction energy (kN-m/$m^3$)** |
|---|---|---|---|---|---|---|
| AASHTO standard (T 99) | 25 | 3 | 2.50 | 0.3048 | 0.00095 | 594 |
| AASHTO modified (T 180) | 25 | 5 | 4.54 | 0.4572 | 0.00095 | 2696 |
| BS light compaction (2.5kg) | 27 | 3 | 2.50 | 0.3000 | 0.00100 | 596 |
| BS heavy compaction (4.5kg) | 27 | 5 | 4.50 | 0.4500 | 0.00100 | 2682 |

### 9.3.2. Design Moisture Content (DMC)

The convenient way of evaluating the design moisture content (DMC) is to measure moisture content of a similar existing road in the rainy season when the water is at highest level. According to Road Note 31 (ORN 31, 1993), the road where the tests will be undertaken should have a minimum crest width of three metres and the test should be carried out at a distance of 0.5 metre from the edge of the carriageway. The samples are to be collected at a depth of 0.6 metre from the top of the subgrade. In the absence of such information a laboratory test may be conducted on a sample collected from the subgrade beneath an impermeable pavement to establish a soil moisture characteristic curve (SMCC) from which the design moisture content can be estimated for a chosen capillary height (ORN 31, 1993). If the equipment is not available to establish such curves, a simpler test may be performed by maintaining water pressure equivalent to the effective depth (D) of the water table. The effective depth of water table is defined by the following equation (ORN31, 1993).

$$D = WT + (SF \times t) \qquad 9.4$$

Where $WT$ = depth of water table (HFL) below formation level, $SF$ = correction factor from table 9.2 which depend on the plasticity index of subgrade soils, $t$ = pavement thickness in same unit of measurement.

Table 9.2 Correction factors for the calculation of effective depth of water table for soils characterised by PI (after ORN 31, 1993).

| **Plasticity Index (PI)** | **Correction Factor (SF)** |
|---|---|
| 0 | 0.00 |
| 10 | 0.30 |
| 15 | 0.55 |
| 20 | 0.80 |
| 25 | 1.10 |
| 30 | 1.40 |
| 35 | 1.60 |
| >35 | 2.0 |

### 9.3.3. Design California Bearing Ratio (CBR)

The CBR is defined by the ratio of force required to cause a standard plunger to penetrate 2.5 or 5 mm depth into the sample to that of required for a standard material. Variability in soil properties and moisture contents can result in a wide range of CBR values. To deal with such uncertainty it necessary to select the design CBR value at a recommended probability of occurrence. For example, Overseas Road Note 31 (ORN 31, 1993) suggests that the design CBR value should be selected on the basis of it exceeding 90% of the test results. The Asphalt Institute (Huang, 1993) suggests different percentile values for resilient modulus depending on the requirements of the pavement design in terms design equivalent standard axles (ESAL) as shown in Table 9.3. For designs with lower ESAL requirements, the risk is not considered to be as great and the percentile value is correspondingly smaller.

Table 9.3 Design CBR of Subgrade (AI, cited in Huang, 1993)

| Design ESAL | Percentile value (%) for Design CBR |
|---|---|
| $10^4$ or less | 60.0 |
| Between $10^4$ and $10^6$ | 75.0 |
| $10^6$ or more | 87.5 |

### 9.3.4. Design Subgrade Modulus

The design subgrade modulus can be estimated by use of a suitable correlation from CBR values as mentioned in the previous section. Available correlations between subgrade CBR and resilient modulus are as follows:

TRRL method (Lister, 1987; cited in Witczak et al., 1995)

$$E_r(MPa) = 17.6(CBR)^{0.64} \qquad 9.5$$

Shell Oil method (Heukelom and Foster, 1960; cited in Witczak et al., 1995)

$$E_r(MPa) = 10(CBR) \qquad 9.6$$

South African Council on Scientific and Industrial Research (CSIR) method

$$E_r(MPa) = 20.67(CBR)^{0.65} \qquad 9.7$$

US Army method (Green and Hall, 1975; cited in Witczak et al., 1995)

$$E_r(MPa) = 37.26(CBR)^{0.71} \qquad 9.8$$

Among the mentioned method Shell Oil method is widely used in tropical and subtropical countries. In Australia this method is accepted as the standard of measuring resilient modulus from CBR (AUSTROADS, 2004).

Alternatively subgrade modulus can be determined from the back calculation of deflection measured by FWD in combination with geophones on a road having representative moisture content. As FWD is not available in Bangladesh the former procedure is recommended for the estimation of subgrade modulus (Durham et al., 2003)

Subgrade resilient modulus can also be determined from regression equations (Thompson and Robnett, 1979) based on various known parameters which are described in the following paragraphs.

1. Soil parameters are known: If the soil parameter like clay content (%), *PI* (%), silt (%), *OC* (%), *LL* (%) and *GI* (group index) then the resilient modulus can be found by the following regression equation:

   $$E_r(kips/in^2) = a + b_1x_1 + b_2x_2 + b_3x_3 + b_4x_4 + ...... + b_6x_6 \qquad 9.9$$

   Where the intercept "a" and coefficients (b) are found from the table 9.4

2. Soil class and degree of saturation known: If the soil classification as per AASHTO or Unified is known, the resilient modulus can be determined from the degree of saturation by the following regression equation:

   $$E_r(kips/in^2) = a + bS_r \qquad 9.10$$

   Intercept "a" and coefficient "b" can be found from Table 9.5.

   $$E_r(kips/in^2) = 6.89MPa \qquad 9.11$$

3. Laboratory static modulus and unconfined compression stress known: If the static modulus and the unconfined compressive strength of the subgrade material can be determined in the laboratory, the resilient modulus then can be estimated using the following regression equation:

   $$E_r = 3.46 + 1.9E \qquad 9.12$$

   $$E_r = 0.86 + 0.307q_u \qquad 9.13$$

*E* and $q_u$ indicates the static modulus and unconfined stress respectively.

Table 9.4 Intercept and coefficients for Equation 9.9 (Thompson and Robnett, 1979)

| Equation number | a | Clay (%) ($b_1$) | PI ($b_2$) | Organic content (%) $b_3$ | Silt content (%) $b_4$ | Group index ($b_5$) | LL (%) ($b_6$) | Correlation coefficient, R | Standard error of estimate, $S_x$ |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 4.88 | | | | | | | 0.600 | 2.76 |
| 2 | 5.12 | | 0.235 | | | | | 0.581 | 2.80 |
| 3 | 10.71 | | | -2.14 | | | | -0.519 | 2.95 |
| 4 | 15.59 | | | | -0.109 | | | -0.450 | 3.08 |
| 5 | 6.46 | | | | | 0.150 | | 0.434 | 3.10 |
| 6 | 4.32 | | | | | | 0.114 | 0.330 | 3.25 |
| 7 | 4.46 | 0.098 | 0.119 | | | | | 0.630 | 2.70 |
| 8 | 6.90 | 0.0064 | 0.216 | -1.97 | | | | 0.757 | 2.30 |
| 9 | 9.97 | -0.0178 | 0.222 | -1.88 | -0.043 | | | 0.772 | 2.26 |
| 10 | 6.37 | 0.034 | 0.450 | -1.64 | -0.0038 | 0.244 | | 0.796 | 2.18 |
| 11 | 8.58 | 0.0586 | 0.1397 | | -0.0561 | | | 0.611 | 2.68 |
| 12 | 3.63 | 0.1239 | 0.4792 | | 0.0031 | -0.3561 | | 0.721 | 2.49 |

Table 9.5 Intercept '*a*' and coefficient '*b*' for Equation 9.10 for the soil classes (Thompson and Robnett, 1979)

| Soil class (AASHTO) | Horizons | Value of "a" (ksi) | Value of "b" |
|---|---|---|---|
| A-7-5 | ABC | 27.59 | -0.248 |
| | BC | 25.60 | -0.217 |
| A-4 | ABC | 17.33 | -0.158 |
| | BC | 16.76 | -0.146 |
| A-7-6 | ABC | 31.22 | -0.294 |
| | BC | 24.65 | -0.196 |
| A-6 | ABC | 36.15 | -0.362 |
| | BC | 35.67 | -0.354 |

### 9.3.5. Saturation Effects on Resilient Modulus

The subgrade resilient modulus is very dependent on the degree of saturation caused by flooding (Drumm et al., 1997). A number of equations as follows have been developed for resilient modulus considering the effect of saturation.

#### 9.3.5.1. Drumm's Equation

In order to consider the effect of saturation after construction Drumm et al. (1997) proposed an "effective road bed soil resilient modulus" $M_{r\ (wet)}$ which can be calculated from the following equation.

$$M_{r(wet)} = M_{r(opt)} + \frac{dM_r}{ds}\Delta S \qquad 9.14$$

Where $M_{r(wet)}$ = resilient modulus at increased post compaction saturation, $M_{r\ (opt)}$ = resilient modulus at optimum moisture content, $dM_r/dS$ = gradient of resilient modulus with respect to saturation, or slope of the $M_r$ vs. degree of saturation curve, $\Delta S$ = % increase in saturation.

They also developed a statistical model to determine the $M_{r(wet)}$ on the basis of some soil properties and AASHTO soil classification. The soil properties are resilient modulus at MDD and OMC, degree of saturation at OMC, liquid limit (*LL*), plastic limit (*PL*), percent clay, dry density and liquidity index. The statistical formula which was developed for Tennessee soil is as follows:

$$\frac{dM_{r\,model}}{dS} = 1.690 - 194(CLASS) - 11.2(M_{ropt}) \qquad 9.15$$

Where $dM_{rmodel}/dS$ = Slope of the $M_r$ vs. Degree of Saturation curve.

#### 9.3.5.2. Thompson and Robnett's Formula

Thompson and Robnett (1979) developed some empirical relationships between resilient modulus and degree of saturation ($S_r$) to take into account the effect of the degree of saturation on resilient modulus for a particular soil type and a specific compaction effort.

For 100% of AASHTO-T99 compaction

$$E_r = 45.2 - 0.428 S_r \qquad 9.16$$

For 95% of AASHTO-T99 compaction

$$E_r = 35.9 - 0.334 S_r \qquad 9.17$$

From Equation 9.16, it may be seen that for 100% saturation the resilient modulus becomes 2.4 ksi (16.54 MPa), where as for 15% saturation it is 38.78 ksi (267 MPa). More compaction effort also helps increase resilient modulus which can be seen by comparing Equation 9.16 and 9.17.

Thompson and Robnett (cited in Jansen and Dempsy, 1981) developed a relation for determining resilient modulus using volumetric moisture content by the following regression equation.

$$E_{ri} = a + b\theta \qquad 9.18$$

Where, $E_{ri}$ = resilient modulus at a particular volumetric moisture content θ (kips/in$^2$), $\theta$ = volumetric moisture content (in percent) and is given,

$$\theta = w\left(\frac{\gamma_d}{\gamma_w}\right) \qquad 9.19$$

Where $w$ = gravimetric moisture content (%), $\gamma_d$ and $\gamma_w$ are dry unit weight of soil and unit weight of water respectively.

Table 9.6 The values of a and b of Equation 9.18 for *b* and *c* horizon soil (Jansen and Dempsy, 1981)

| **Soil having dry density (kg/m$^3$)** | **Determined as per AASHTO T-99 (% compaction)** | **Value of *a*** | **Value of *b*** |
|---|---|---|---|
| ≤1600 (100 lb/ft$^3$) | 95 | 27.06 | -0.524 |
| >1600 (100 lb/ft$^3$) | 95 | 18.18 | -0.424 |

#### 9.3.5.3. Lytton and Michalak's Equation

The average subgrade modulus can also be determined from the following equation by Lytton and Michalak considering average rainfall that will infiltrate into subgrade in a season (Shang and Lytton, 1984)

$$E_s = \left(E_1 d_1^3 + E_2 d_2^3\right) \Big/ d^3 \qquad 9.20$$

Where, $E_s$ = subgrade modulus considering water infiltration, $d$ = depth of subgrade $= d_1 + d_2$, $E_1$ = modulus under 100% saturation (can be determined from the above Thompson and Robnett's equation), $d_1$ = depth of water (average) that penetrates into the subgrade from base, $E_2$ = dry subgrade modulus and $d_2$ = depth of dry portion (average) of the subgrade.

#### 9.3.5.4. Markow's Formula

Markow (1982) also modelled a relationship for strength reduction factor due to moisture existence in granular bases and subgrades.

$$F_{red} = \left(t_{season} - 0.5t_{wet}\right) \Big/ t_{season} \qquad 9.21$$

$$R' = \left[5t_{wet} + R\left(t_{season} - t_{wet}\right)\right] \Big/ t_{season} \qquad 9.22$$

Where, $F_{red}$ = a reduction factor to be applied to modulus for wetness, $t_{season}$ = length of season (days), $R'$ = AASHTO additional wetness regional factor for cracks in pavement, $R$ = Regional factor and $t_{wet}$ = duration of pavement wetness (days) and can be given as

$$t_{wet} = \left(\frac{r_{season}}{i_{avg}}\right)\left[1 - \exp\left(-9C\right)\right]t_{80} \qquad 9.23$$

#### 9.3.5.5. Recommended Formula

Drumm's equation considers most of the soil parameter which are easily achievable and that is why it is recommended as the suitable equation for the estimation of resilient modulus considering saturation effect. Laboratory study needs to be carried out to justify the use of this equation in Bangladesh after making necessary modifications.

### 9.3.6. Poisson's Ratio

It is usual to estimate Poisson's ratio. For example, the Shell Design manual (Claissen et al., 1977; cited in O'Flaherty, 2002) suggests a value of 0.35, whilst the University of Nottingham (cited in O'Flaherty, 2002) suggest values from 0.4 to 0.35 for cohesive and granular soil. The value suggested by AUSTROADS (2004) for cohesive and granular soils are 0.45 and 0.35 respectively. Das (2004) proposed typical values of Poisson's ratio for following types of soils as mentioned in Table 9.7.

Table 9.7 Typical values of Poisson's ratio for different types of soils (after Das, 2004)

| **Types of Soil** | **Poisson's ratio, ν** | | |
|---|---|---|---|
| | **Minimum** | **Maximum** | **Average** |
| Loose sand | 0.2 | 0.4 | 0.3 |
| Medium sand | 0.25 | 0.4 | 0.33 |
| Dense sand | 0.3 | 0.45 | 0.43 |
| Silty sand | 0.2 | 0.4 | 0.3 |
| Soft clay | 0.15 | 0.25 | 0.2 |
| Medium clay | 0.2 | 0.5 | 0.35 |

## 9.4 Specification for Compaction

### 9.4.1. Field Density and Moisture Content

In Bangladesh it is advised the fill and subgrade materials should be compacted to a minimum 95% and 98% of MDD respectively measured at STP 4.3 which is similar to that of standard proctor or BS light compaction (RHD, 2001). The existing recommended moisture content for field compaction is ±2% of OMC measured in the compacting effort of STP 4.3. Thagesen (1996) suggested that the subgrade should be compacted to a minimum 95% of MDD obtained in the modified proctor test. TRL suggests 100% of BS light compaction or 93% of BS heavy compaction for upper 250 mm of the subgrade (ORN 31, 1993).

The University of Kansas in cooperation with Kansas Department of Transportation (KDOT) carried out a study on compaction of embankment fill and settlement of embankment materials (Parsons et al., 2001). In their study they conducted field tests on eight existing embankments and carried out a survey among fifty states in USA to find out the degree of compaction and moisture content at compaction. The average relative compaction they found was 97% of MDD obtained in standard compaction with a standard deviation ±4%. There were no settlements in the embankments except mild settlement in one embankment. From the survey among the states transportation department, 60% were in favour of 95% of standard compaction for embankment fill. They recommended relative compaction for fill as 95% of standard compaction to avoid settlement of embankment materials.

It is difficult to maintain the optimum moisture content during construction due to the variation in moisture content in natural soil. Water should be added if it is less than OMC and removed (by drying) if it is greater than OMC. The study by University of Kansas found 7.5% moisture content variation from OMC and in most cases it was found on the higher side than OMC. The degree of settlement was found to be low where field moisture contents (FMC) closely matched the OMC but fair to poor where large variations were apparent. The survey result among the states transportation department suggests ±2% of moisture content variation in the field for better performance. Combining the field investigation and survey results they recommended that the difference between FMC and OMC should be from -1% to 3% (Parsons et al., 2001). All these recommendations are only for embankment fill but no guidance is given for the upper layer of the subgrade.

From the above, to achieve a high density and CBR it is recommended that the top meter of the subgrade is compacted at minimum 95% of MDD obtained in BS 4.5 (vibrating hammer) (BS 1377-4, 1990) with a moisture content variation between -2 to 3% from OMC. Similarly 95% of standard relative compaction for fill materials with same moisture content variation is recommended. In this perspective the design subgrade CBR will also correspond to this compaction (95% of MDD in BS 4.5) with moisture content at DMC. The design CBR can be interpolated from the DD-MC-CBR relation ships developed for that soil (AUSTROADS, 2004).

### 9.4.2. Layer Thickness

The survey conducted by the University of Kansas mentioned above, showed that 50% of States compacted material in 200mm thickness, 9% used 150mm and 25% used 305mm. In the last successful project in Bangladesh (JOC-NA-BCL, 2000), compaction was carried out in layers whose thickness varied from 150mm to 300mm were for cohesive and sandy soil with the available equipment.

## 9.5 Summary

This chapter has considered a number of criteria regarding the specification of the properties of the upper layer of the embankment. These needs to be considered in relation to the requirements of the road pavement it support. The finding may be summarised as follows.

- The height of the embankment should be determined using a specified return period of high flood level and a recommended freeboard.
- The design moisture content (DMC) is the primary information for design CBR as well as design resilient modulus. In Bangladesh currently four-day soaked CBR is used in the design of pavement. In the case of unavailability of DMC the four-day soaked CBR can be used but its justification is needed by further study.
- To avoid the settlement within the embankment, a minimum 95% of standard compaction is necessary for fill and 95% of modified compaction for upper most one meter of embankment keeping the moisture content within -1% to 3% from OMC. If the sandy soil is available, albeit at a marginally higher cost, it can be used as fill material as it will avoid primary consolidation and secondary compression settlement within the embankment.

# Chapter 10 Proposed Design Method

## 10.1 Introduction

Methods of settlement and stability analysis were discussed in detail in Chapters 4, 5 6, 7 and 8 and the selection of formation level and subgrade design parameters were discussed in Chapter 9. Suitable methods were identified for the conditions at hand in Bangladesh. In this chapter a systematic and combined approach to the overall design process is presented.

## 10.2 Input Parameters

The input parameters for embankment design are described as follows.

### 10.2.1. Subgrade Design Parameters

With reference to Chapter 9 these are:

- CBR and moisture content (at a number of locations)
- Four-days soaked CBR
- Dry density-moisture content-CBR relationship
- Optimum moisture content (OMC).
- Design moisture content (DMC).
- Soil moisture characteristics curve (SMCC).

Embankment geometry

| Flood Zone/ Area | High Flood Level | Ground Level | Height of embankment (h) | Top width (depend on National policy) |
|---|---|---|---|---|
| | m PWD | m PWD | metre | Metre |

### 10.2.2. Stability Analysis

Parameters needed for stability analyses are as follows (see Chapter 7 and 8):

- Height of embankment (*h*)
- Top width of embankment (*W*)
- Bottom width of embankment (*B*)

- Slope angle ($\beta$)
- Shear strength parameters for fill and foundation($c'$, $\varphi'$ and $c$, $\varphi$)
- Undrained shear strength for foundation layers ($S_u$) with the shear strength gradient ($k$)
- Maximum depth of soft foundation ($D_{max}$)
- Dry, bulk and saturated unit weight of fill and foundation soil ($\gamma_d$, $\gamma$, $\gamma_{sat}$)
- Natural moisture content ($w$)
- Specific gravity of fill and foundation($G$)
- Ground water location

In addition to the above essential parameters it is necessary to determine the Atterberg's limits to estimate the parameters which are not possible determine from field and laboratory testing with the available equipments. The parameters can be estimated by suitable correlations from the literature such as the shear strength parameter ($\varphi$), $s_u/p'$, and saturated unit weight (Das, 2004). The parameters required for stability analysis are tabulated in the Table 10.1 and 10.2 for methodical exploration and testing.

Table 10.1 Foundation parameters for stability analysis

| Depth | $\gamma$ | Atterberg's Limit | | | | $S_u$ | $c$ | $c'$ | $\varphi$ | $\varphi'$ | $u$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | $w$ | $LL$ | $PL$ | $PI$ | | | | | | |
| m | kN/m$^3$ | % | % | % | % | kN/m$^2$ | kN/m$^2$ | kN/m$^2$ | Degree | Degree | kN/m$^2$ |

Table 10.2 Embankment fill parameters for stability analysis

| Types of locally available soil | PSD | $\gamma$ | Atterberg's Limit | | | | $c$ | $c'$ | $\varphi$ | $\varphi'$ |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | $w$ | $LL$ | $PL$ | $PI$ | | | | |
| Description | | kN/m$^3$ | % | % | % | % | kN/m$^2$ | kN/m$^2$ | Deg | Deg |

### 10.2.3. Settlement Analysis

The parameters required for settlement analysis with appropriate remedial measures are given below (see Chapter 4 and 5):

- Height of embankment ($h$)
- Width of embankment ($B$)
- Slope angle ($\beta$)

- Maximum depth of soft foundation ($D_{max}$)
- Dry, bulk and saturated unit weight of fill and foundation soil ($\gamma_d, \gamma, \gamma_{sat}$)
- Natural moisture content ($w$)
- Undrained elastic modulus ($E_u$) and Poisson's ratio ($v$)
- Compression index for the foundation layers ($C_c$)
- Coefficient of consolidation in vertical and horizontal direction ($C_v$ and $C_h$)
- Coefficient of secondary compression ($C_\alpha$) or $C_\alpha/C_c$ and post surcharge secant secondary compression index ($C_\alpha''$).
- Permeability in vertical and horizontal direction ($k_v$ and $k_h$)
- Initial void ratio of foundation soil ($e_o$)
- Location of ground water table ($GWT$)
- Schedule of loading stress.

As stated earlier, in addition to the above essential parameters, it is necessary to determine the Atterberg's limits to estimate parameters which are not possible to determine from field and laboratory testing with the available equipment. The parameters can be estimated by suitable correlations from the literature such as the compression index, coefficient of consolidation, pre-consolidation pressure ($p'_c$) and saturated unit weight ($\gamma_{sat}$) (Das, 2004). For convenience, the parameters are also tabulated in the Table 10.3 and 10.4.

Table 10.3 Foundation parameters for settlement analysis

| Depth | Soil description | $\gamma$ | $G_s$ | $w$ | $E_u$ | $v$ | $C_c$ | $C_r$ | $C_v$ | $C_\alpha$ | $C_s$ | $C''_\alpha$ | $p'_c$ | $e_o$ | $k_h$ | $k_v$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| m | | kN/m$^3$ | | % | kPa | | | | m2/day | | | | kPa | | m/day | m/day |

Table 10.4 Embankment soil and geometry

| Height (h) | Slope angle (β) | Top width (W) | Bottom width (B) | Unit weight (γ) |
|---|---|---|---|---|
| m | Degree | metre | | kN/m$^3$ |

Accurate borehole information is the prime consideration for reasonable settlement prediction. Soil classifications with the depth of individual layers are to be investigated carefully. During the subsoil investigation the presence of any sand seams and organic layers are to be noted carefully for the prediction of settlement and design of PVD and any surcharge loading (Ortigao, 1995).

## 10.3 Design Criteria

**Embankment height and design CBR**

| Road Types | Return period for Design (HFL), years | Free board (m) | Percentile value for design CBR (%) |
|---|---|---|---|
| National highway | 50 | 1.0 | 90 |
| Regional highway | 50 | 1.0 | 75 |

**Settlement**

Based on the literature review (see Chapter 2) and the author's experience in the field the following criteria were considered for the settlement analysis:

- Residual settlement: Maximum allowable settlement of 150 mm over 20 years (Hsi and Martin, 2005, Long and O'Riordan 2001).
- Differential settlement: Maximum differential settlement in lateral (transverse) direction 1% and the longitudinal direction 0.3% for a period of 20 years (Hsi and Martin, 2005).

**Stability**

Stability criteria are usually specified on the basis of short and long term behaviour of the embankment (Hsi and Martin, 2005).

- Short term stability requires the embankment to be stable during construction period. Typically a factor of safety (FOS) of 1.2 is applied to any solution (Hsi and Martin, 2005).
- The long term performance of an embankment is associated with the period after construction and up to the end of its design life. A factor of safety 1.5 is often used in practice (Hsi and Martin, 2005).

It should be noted that the above factors of safety assume good construction practices and adequate supervision and are used as a guide line in this study for embankment stability.

## 10.4 Proposed Design Procedure

To design a highway embankment the following steps are recommended (cf. Figures 3.2 and 3.3).

### A. Formation level and subgrade design parameters

**Step -1:** Collect water level data of stations near the proposed road and carryout statistical analysis using Chow's equation (Equation 9.1 and 9.2) to find the design formation level (Section 9.2).

**Step -2:** Find CBR values from subgrade of a similar nearby road using appropriate methods (see Section 9.3).

**Step -3:** Find the design CBR from statistical analysis (see Section 9.3.3) and use a suitable correlation to find the design $M_r$ (Section 9.3.4).

### B. Settlement and Stability

**Step -4:** Find

a) Embankment fills and subsoil parameters from field and laboratory test (list of parameters as in Table 10.1, 10.2, 10.3 and 10.4).
b) Find slope height from Step-1 and assume slope angle (conventional 1:2 in Bangladesh)
c) Prepare construction schedule

**Step -5:** Set design criteria for factor of safety against slope failure and residual settlement (as in Section 10.3).

**Step -6:** Find short term factor of safety against stability with Low's method (Equation 7.17) with initial shear strength (Section 7.2.1.6) and long term factor of safety from charts in Appendix-III, or use Bishop's method (if OASYS slope software or similar) is available.

Input data: Table 10.1 and 10.2

If short term factor of safety is less than 1.2 adopt stage construction and or use geo-reinforcement.

**Step -7:** Find bearing capacity from Equation 7.27 and 7.30 (Section 7.3.1and 7.3.2). If bearing capacity is less than required, try adding reinforcement to the design process (Equation 7.31 Section 7.3.1).

**Step -8:** Find settlements for embankment fills and the foundation. If the software Settle3D is available use it for improved speed and accuracy of calculation otherwise follow the standard procedure as described below.

**I.** Embankment fills:

Input data $$e_0 = \frac{G_s \gamma_w}{\gamma_d} - 1$$

| $G_s$ | $\gamma_w$ | MDD | OMC | $\gamma_d$ | $\mathbf{e_o}$ | FMC | $\gamma$ | LL | $\mathbf{C_c}$ | **Cα** |
|---|---|---|---|---|---|---|---|---|---|---|
| | kN/m$^3$ | kN/m$^3$ | % | kN/m$^3$ | | OMC+3% | kN/m$^3$ | % | | |

a) Primary consolidation settlement $$S_p = \sum_{i=1}^{n} \frac{H_i}{1+e_{0(i)}} \left( C_{c(i)} \log \frac{\sigma'_{vo(i)} + \Delta\sigma'_{av(i)}}{\sigma'_{v0(i)}} \right)$$

c) Secondary consolidation settlement $$S_{\sec} = \sum_{i=1}^{n} \frac{C_{c(i)}}{1+e_{0(i)}} \frac{C_{\alpha(i)}}{C_{c(i)}} H_i \left( \log \frac{t_e}{t_p} \right)$$

**II.** Foundation:

Input data Table 10.3 and 10.4

Find settlements using Equations 4.1 and 4.8

a) Elastic settlement Equation 4.1

b) Primary consolidation settlement Equation 4.8

To find the above settlement, determine

- $\sigma'_{vo}$ from depth and unit weight of soil layers

- $\Delta\sigma'_{av}$ from Osterberg's Equation 4.27 (describe in Section 4.4.1)

c) Secondary consolidation settlement Equation 4.12 (Section.4.2.3)

**Step -9:** Predict achievable degree of drainage within the time as per construction schedule for $T_v = \dfrac{C_v t}{H_{dr}^2}$ considering construction time factor $T_c$, where $T_c = \dfrac{C_v t_c}{H_{dr}^2}$ and $t_c$ is the construction time of embankment.

Find degree of drainage ($U_i$) for each stage from Figure 4.3 or for more accuracy calculate from Equation 4.23 and 4.24 using the method given in Appendix- XIV. Calculate weighted average degree of drainage from Equation 4.42 as described in Figure 4.12.

**Step -10:** Find desired degree of drainage to be achieved to keep within the limit of allowable settlement.

$$U_{desired} = \frac{S_p - S_{residual}}{S_p}$$

**Step -11:** If the predicted degree of drainage is less than desired degree of drainage introduce vertical drainage.

**Step -12:** Design vertical drains

Use Hansbo (1981) formula to determine the required spacing of the drains and Olson (1977) for degree of drainage considering ramp loading (Equation 5.7 to 5.23, Section 5.3.1.2 and 5.3.1.3).

Find spacing and depth of vertical drains. Specify core thickness, width, Jacket filter characteristics, Material strength, durability, Flexibility.

**Step-13:** If the desired degree of drainage can not be achieved within the project time with only vertical drainages, investigate the use of a surcharge load following the procedure given in Figure 5.4.

**Step -14:** If the secondary compression is more than the allowable residual settlement limit, apply a surcharge to reduce it and modify the vertical drain design to accelerate primary consolidation. Considering surcharge effect, find the secondary compression according to Equation 4.13 (Section 4.2.3.1).

**Step -15:** Find thickness of horizontal drainage blanket based on permeability of the materials to be used for this work according to Equation 5.23 (Section 5.4). Coarse sand is recommended for this purpose.

**Step -16:** Calculate the shear strength increase ($\Delta S_u$) due to consolidation settlement following Equation 4.39. Then determine increased shear strength with the following equation.

$$S_u = S_{u0} + \Delta S_u$$

**Step -17:** Check factor of safety against slope stability with increased undrained shear strength by Low (1989) method (Section 7.2.1.6).

**Step -18:** Check the bearing capacity of foundation as in step-7

**Step -19:** Find the modified factor of safety considering the water level using Figure 8.11.

**Step -20:** If the factor of safety less than 1.5 for long term stability reduce the slope angle of the embankment or investigate the use of pressure berms or geo-reinforcement.

**Pressure berms**

Determine the height, width or surface slope following the procedure described in sections 7.5.2.1, 7.5.2.2, and 7.5.2.3.

**Step -21:** Compare on an economic basis different types of ground improvement available (note this step was considered to be beyond the scope of this thesis).

**Step -22:** Choose final design

## 10.5 Design Parameters

The design method described above has been used to determine the value of the design parameters for an existing highway embankment in Bangladesh. The parameters are height, width and slope of embankment, spacing and length of vertical drains, thickness of horizontal drainage blanket, length and width of pressure berms, tensile strength and stiffness of reinforcement, and height of surcharge load.

## 10.6 Design examples

The parameters used in this example were collected from Tongi-Ashulia road. The subsoil and part of embankment properties were collected by field and laboratory tasting by a parallel study to this thesis (Shajahan, in print). As the preparation of DD-MC-CBR relations and establishment of DMC were not possible, the four-days soaked CBR values were used in this example. The four-day soaked CBR values for different locations were collected by BRRL. The values in each step of the example are given as follows and the calculations in detail are shown in Appendix XVII.

### A. Subgrade design parameters and formation level

**Step 1:** Input parameter: 4-days soaked CBR at different location

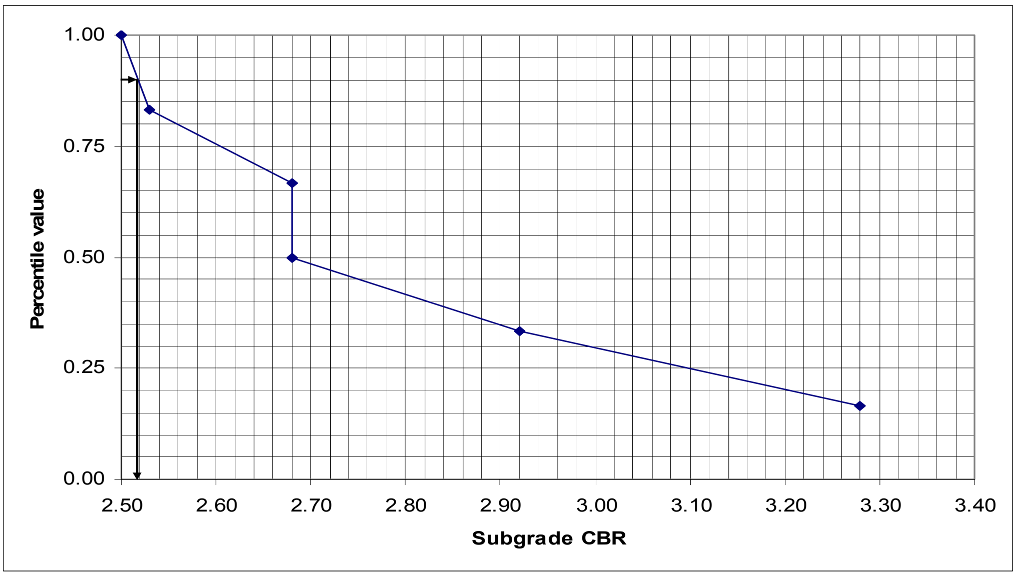



Figure 10.1 Subgrade CBR distribution

From Figure 10.1, Design CBR = 90 percentile value = 2.5

**Step 2:** Design resilient modulus = 10 CBR (Heukelom and Foster, 1960; cited in Witczak et al., 1995) = 25 kPa

**Step 3:**Find formation level

Input parameter: Yearly high water level from near by stations. Here flood level data are available for two stations; Mirpur and Tongi.

a) Determine flood level vs. return period using Chow's equation

$$X_T = \overline{X} + K_T \delta$$

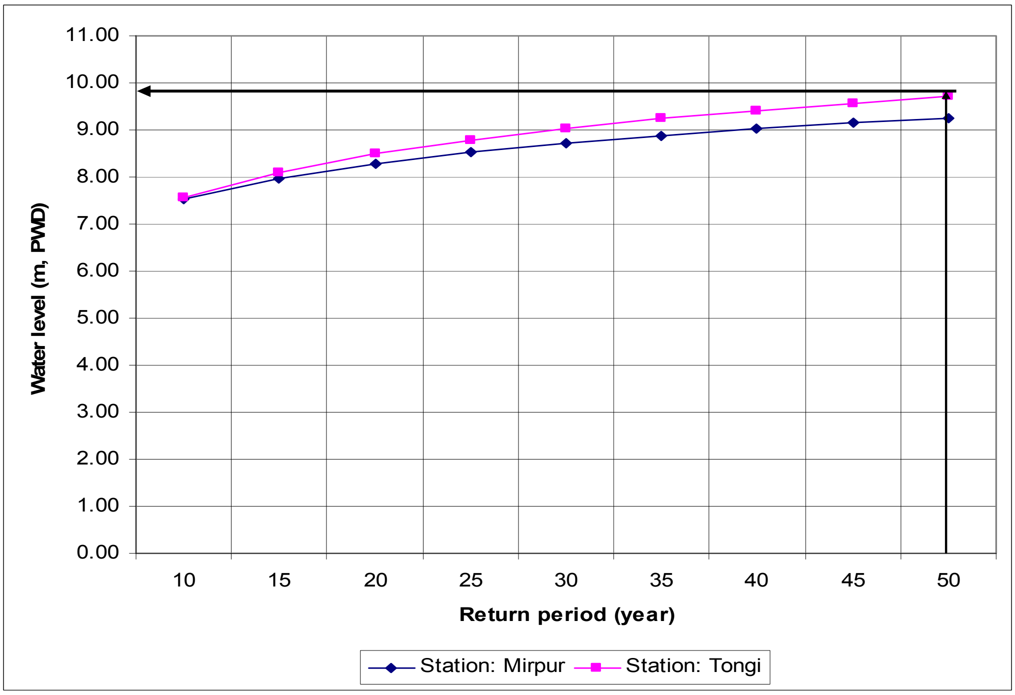



Figure 10.2 Water levels with return periods

b) Find design formation conditions

| | | |
|---|---|---|
| For national highway design return period, T = | 50 | years |
| Water level at this return period, design HWL = | 9.71 | m PWD |
| Freeboard | 1.00 | metre |
| **Design formation level =** | **10.71** | **m PWD** |

As there is no established SMCC curve for the fill materials, the freeboard was taken as that currently recommended by RHD (i.e. 1m).

c) Height of embankment and road level at different sections

| Chainage | GL | Formation Level | Embankment height (h) |
|---|---|---|---|
| 6+220 | 7.22 | 10.71 | 3.49 |
| 6+300 | 7.35 | 10.71 | 3.36 |
| 6+315 | 7.40 | 10.71 | 3.31 |

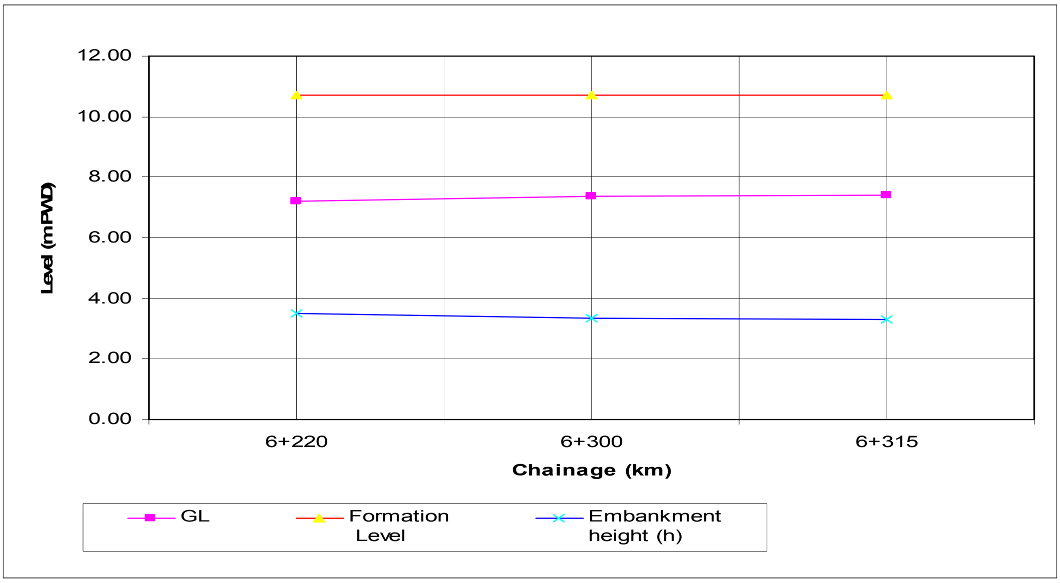



Figure 10.3 Formation level and height of embankment

## B. Settlement and Stability

**Step 4**: Input parameters

a. Embankment fill and geometry

| $G_s$ | $\gamma_w$ | MDD | OMC | $\gamma_d$ | $e_o$ | FMC | LL | $C_c$ | $C\alpha$ |
|---|---|---|---|---|---|---|---|---|---|
| | kN/m$^3$ | kN/m$^3$ | % | kN/m$^3$ | $e_o = G_s\gamma_w/\gamma_d - 1$ | OMC+3% | % | $C_c = 0.009(LL-10)$ | |
| 2.52 | 9.8 | 17.07 | 16.08 | 13.97 | 0.770 | 18.08 | 30 | 0.18 | 0.007 |

| $\gamma$ | c | $\varphi$ | h | $\beta$ | b | BW |
|---|---|---|---|---|---|---|
| kN/m$^3$ | kN/m$^2$ | degree | metre | degree | metre | metre |
| 16.5 | 4.86 | 18.0 | 3.5 | 33.5 | 12.2 | 22.2 |

b. Foundation soil

| Depth (m) | $\gamma$ (kN/m$^3$) | $\gamma_{sat}$ (kN/m$^3$) | $S_u$ (kN/m$^2$) | LL (%) | PL (%) | PI (%) | $w_n$ (%) | $C_c$ | $C_\alpha$ | $C_r$ | $C_v$ (m$^2$/day) | $e_o$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1.0 | 15.34 | 17.60 | 17.0 | 37.90 | 21.82 | 16.08 | 37.4 | 0.391 | 0.0037 | 0.135 | 0.0022 | 1.08 |
| 2.0 | 18.45 | 18.79 | 18.0 | 44.67 | 26.31 | 18.36 | 32.0 | 0.311 | 0.0031 | 0.109 | 0.0033 | 1.10 |
| 3.0 | 14.69 | 14.76 | 14.3 | 42.43 | 23.51 | 18.92 | 77.0 | 0.330 | 0.0610 | 0.120 | 0.0013 | 0.89 |
| 4.0 | 15.39 | 15.39 | 41.6 | 49.33 | 28.50 | 20.83 | 76.0 | 0.376 | 0.0020 | 0.132 | 0.0700 | 1.26 |
| 9.0 | 13.69 | 13.69 | 13.3 | 54.26 | 32.06 | 22.20 | 129.0 | 0.270 | 0.0490 | 0.069 | 0.0194 | 3.09 |
| Average | 15.51 | 16.05 | 20.84 | 44.72 | 26.44 | 19.28 | 70.0 | 0.34 | 0.024 | 0.11 | 0.0348 | 1.483 |

c. Loading schedule

| Stage | Time (d) | h (metre) | Cumulative loading (m) | Loading Stress (kPa) |
|---|---|---|---|---|
| 1 | 0 | 0.0 | 0.00 | 0.0 |
| 2 | 90 | 1.75 | 1.75 | 30.6 |
| 3 | 180 | 1.75 | 3.50 | 61.3 |
| 4 | 360 | 0.0 | 3.50 | 61.3 |

**Step 5:** Design criteria

| | |
|---|---|
| Short term factor of safety (FoS) | 1.2 |
| Long term factor of safety (FoS) | 1.5 |
| Total Residual settlement in 20 years | 100 -150 mm |

**Model description**

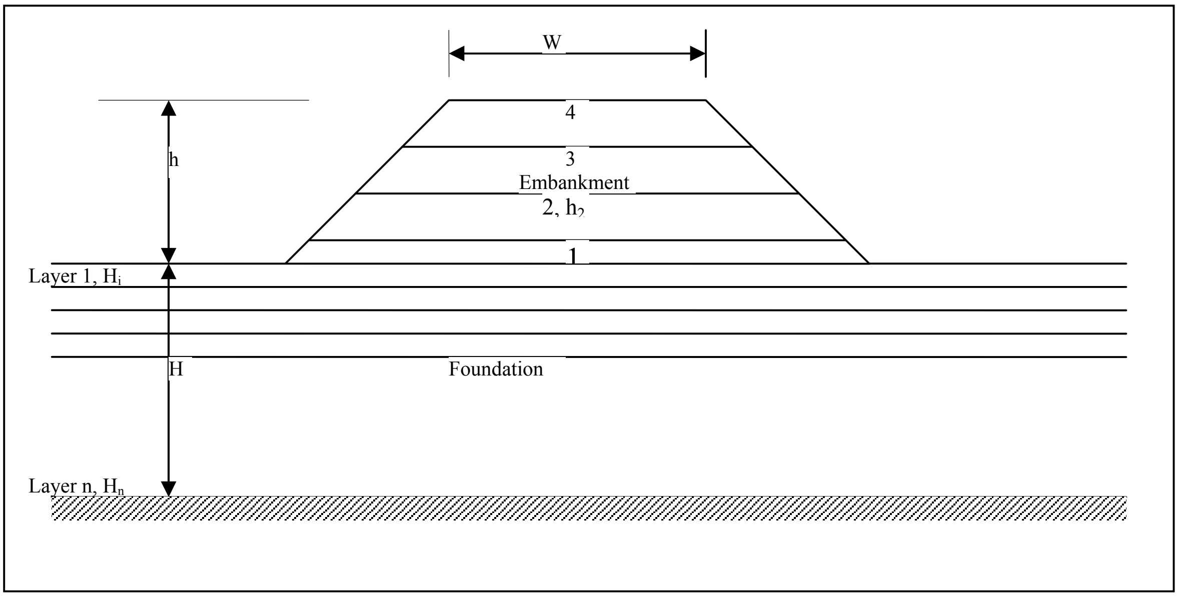



Figure 10.4 Model descriptions for embankment design

**Step 6:** Stability

The factor of safety against stability with initial shear strength = 1.18 and the modified factor of safety considering full rapid drawdown = 1.05 (from Figure 8.11), which is not sufficient for short term stability.

**Step 7:** Settlement

**I. Embankment fills**

a) Maximum Primary consolidation settlement

$$S_p = \sum_{i=1}^{n} \frac{H_i}{1+e_{0(i)}} \left( C_{c(i)} \log \frac{\sigma'_{vo(i)} + \Delta\sigma'_{av(i)}}{\sigma'_{v0(i)}} \right)$$

= 176 mm

b) Secondary consolidation settlement $S_{\sec} = \sum_{i=1}^{n} \frac{C_{c(i)}}{1+e_{0(i)}} \frac{C_{\alpha(i)}}{C_{c(i)}} H_i \left( \log \frac{t_e}{t_p} \right)$

= 19 mm (for 20 years)

Primary settlement achieved after one year for embankment fill = 161 mm

**II. Foundation**

a) Immediate elastic settlement $S_e = 4 * \frac{qBI}{E_u}$ = 57 mm

b) Primary consolidation settlement $S_p = \sum_{i=1}^{n} \frac{H_i}{1+e_{0(i)}} \left( C_{c(i)} \log \frac{\sigma'_{vo(i)} + \Delta\sigma'_{av(i)}}{\sigma'_{v0(i)}} \right)$

= 682 mm for embankment load

= 726 mm for embankment and pavement

(Foundation having n layers of varying settlement and index properties)

c) Secondary compression $$S_{\sec} = \sum_{i=1}^{n} \frac{C_{c(i)}}{1+e_{0(i)}} \frac{C_{\alpha(i)}}{C_{c(i)}} H_i \left( \log \frac{t_e}{t_p} \right) = 248 \text{ mm}$$

**Step 8:** Achievable degree of drainage ($U_v$) within 360 days = 38.2%

**Step 9:** Degree of drainage for primary consolidation required: As the secondary compression is more than allowable residual settlement,

$$U_{desired} = \frac{S_p - S_{residual}}{S_p} = 99\% \text{ (assumed)}$$

**Step 10:** Subsoil drainage is required to achieve 99% the primary consolidation: Design prefabricated vertical drain using Hansbo and Olson's equation (see Figure 10.5). As there is significant secondary compression it is also necessary to incorporate vertical drains which require a minimum 98% radial drainage ($U_r$) to achieve 99% primary consolidation ($U$) to enable the settlement to occur within that specified in the design.

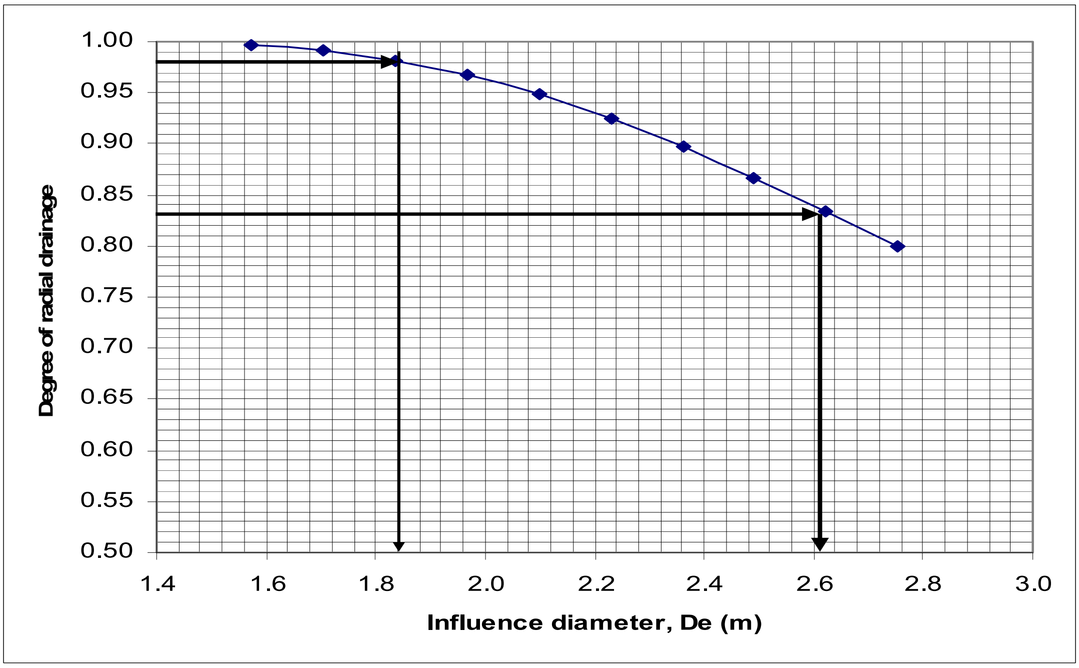



Figure 10.5 Degree of radial drainage with influence diameter

Influence diameter corresponding to 98% drainage = 1.84 m

PVD spacing $S = D_e / 1.128$ = 1.63 (say 1.60) m, square with depth = 9.0 m

**Step 11:** Reducing residual secondary compression:

The secondary settlement is greater than the allowable residual settlement and as there is peat soil in the foundation layers; surcharge should be effective for reducing residual secondary compression.

Design surcharge for secondary compression:

Assume surcharge height = 1m = 17.5 kPa, $t_{pr}$ = 14 days and $t_l$ = 3 months

$$R'_s = \left( \sigma'_{vsc} / \sigma'_{vst} \right) - 1$$

= (17.5+61.5)/61.5 - 1

= 0.28

From Figure 2.1 $t_l/t_{pr}$ = 3, assume $t_{pr}$ = 2 weeks, $t_l$ = 6 weeks = 1.5 months

$$\frac{t}{t_l} = \frac{20 \times 12}{1.5} = 160$$

From Figure 2.1 $\frac{C''_\alpha}{C_\alpha} = 0.2$

$$S_{\sec} = \frac{C_c}{1+e_0} \frac{C_\alpha}{C_c} \frac{C''_\alpha}{C_\alpha} H_o \left( \log \frac{t_e}{t_l} \right) = 42.5$$

Residual settlement due to secondary compression is acceptable if 99% primary consolidation is achieved.

**Step 12:** Check primary consolidation with PVD and surcharge application

Maximum primary settlement with surcharge = 766 mm

Maximum primary settlement for embankment load = 682 mm

Maximum primary settlement for embankment and pavement load =726mm

Degree of settlement required considering only embankment load with surcharge at the end of 360 days

= 682/766 = 88.9%

Degree of settlement required considering both embankment and pavement load with surcharge at the end of 360 days

= 726/766 = 94.7%

**Step 13:** Redesign vertical drain considering surcharge

Degree of drainage achievable without vertical drains = 0.364

Degree of radial drainage required, $U_r = (U- U_v) / (1- U_v)$

= (0.8893 - 0.364)/ (1-0.364) = 83% considering only embankment load

Degree of radial drainage required, $U_r = (U- U_v) / (1- U_v)$

= (0.947 - 0.364)/ (1-0.364) = 92% considering both embankment and pavement load

Influence diameter ($D_e$) corresponding to 83% drainage from Figure 10.5 = 2.60 m

Influence diameter ($D_e$) corresponding to 92% drainage from Figure 10.5 = 2.25 m

PVD spacing $S = {D_e}/{1.128}$ = 2.27 m (say 2.2m) square with depth = 9.0 m

Considering both embankment and pavement load

PVD spacing S = 2.25/1.128 = 1.99m (say 2.0m) square with depth = 9.0 m

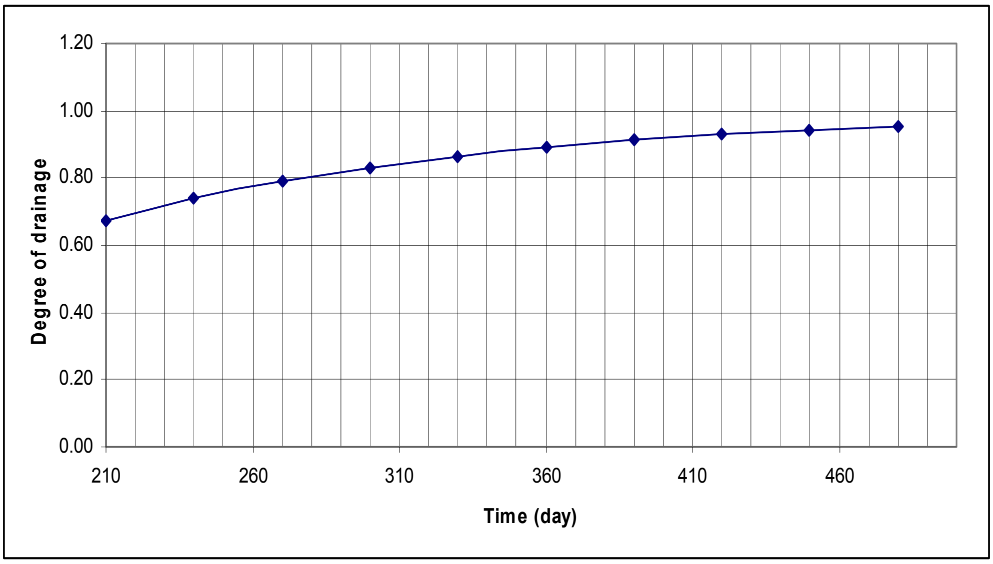



Figure 10.6 Degree of drainage vs. time for PVD spacing = 2.20 metre and surcharge height = 1.0 m

**Step 14:** Approximately 150% shear strength increases after 99% consolidation. With this increased shear strength the factor of safety found as 1.76 and the modified factor of safety considering full rapid drawdown = 1.55 (from Figure 8.11).

**Step 15**: Bearing capacity

Bearing capacity is OK as critical height of embankment is 7 metre (Appendix VII)

**Step 16:** Total residual settlement after 360 days

| | Primary (mm) | Secondary (mm) | Total (mm) |
|---|---|---|---|
| Embankment fills | 16 | 61 | 77 |
| Foundation | 0 | 43 | 43 |
| Total | 16 | 104 | 120 |

Total residual settlement ≈ allowable settlement  Design is OK

**Step 17:** Excess filling due to total foundation settlement

| Immediate (mm) | Primary (mm) | Secondary (mm) | Total (mm) |
|---|---|---|---|
| 57 | 726 | 43 | 826 |

**Step 18:** Excess filling due to immediate and primary settlement of foundation

| Immediate (mm) | Primary (mm) | Total (mm) |
|---|---|---|
| 57 | 726 | 783 |

**C. Horizontal drainage blanket**

Minimum discharge capacity, $q_w = 5k_h l_m^2$ = 0.034992 m$^3$/day where $k_h$ = permeability of subsoil = $8.64$×10$^{-5}$ m/day, $l_m$ = 9.0m.

Hydraulic conductivity of drainage blanket, $k$ = 1.382 m/day (from literature)

Gradient, $i$ = 0.03

Area of section, $A = \frac{q_w/2}{k_i}$ = 0.422 m$^2$

PVD spacing, S = 2.0 metre

Thickness of drainage blanket, t = A/S = 0.21 m say 225 mm.

**Results: Design Parameters**

Height of embankment, h = 3.5 m

Slope of embankment = 1:2

Top width = 12.2 m

Design CBR of subgrade materials = 2.5

Resilient modulus of subgrade materials = 25 kPa

Factor of safety = 1.05 and 1.55 for short and long term

Surcharge = 1 m for 6 months application

<u>PVD</u>

PVD spacing = 2.0 metre

Drainage time from the start of construction = 360 days

Core thickness = 3 mm, width = 100 mm

Depth of PVD = 9.0 metre

Minimum discharge capacity = 0.034992 m$^3$/day

Horizontal drainage blanket = 225 mm with geo-textile at the bottom of the layer

Total settlement = 780 mm

Excess filling due to primary and immediate settlement = 783 mm

Excess filling due to total settlement = 826 mm.

### 10.6.1. Discussion

The Tongi-Ashulia road was constructed in the early 1990s and was supposed to incorporate vertical drains and surcharge in some of the sections to avoid the settlement which has now occurred. As demonstrated in the example given above had these measures been adopted the settlement caused could have been made to occur before the construction of pavement, preventing many of the problems which have occurred to date. As far as slope stability is concerned from the above analysis it may be seen that the short term factor of safety for the Tongi-Ashulia road is 1.05. This is less than that recommended in literature (i.e. 1.2) and helps to explain why there have been numerous slope failures at the embankment (see Appendix IX). However, if the design which incorporates PVD and surcharge was implemented the factor of safety could be increased to 1.55, suggests that the slope failure could have been avoided. The location of the borropit very near the toe of the embankment is another cause of low factor of safety leads slope failure. Further, parts of the embankment were constructed by peat soils. The stability of the embankment could be further improved and the settlement within the embankment could be reduced if the use of such soils were avoided.

## 10.7 Summary

A general procedure for the design of embankment is presented in this chapter with an example. The various elements of embankment design described in the previous chapters are referred to this procedure as a guide to execute every step. The required input parameters and necessary design criteria for the complete design process have been discussed in this chapter. The next chapter will discuss the achievement and limitation of the study with regard to the objectives.

# Chapter 11 Discussion

## 11.1 Introduction

In the literature a great deal of research has been published on stability of embankments and the settlement of the supporting foundation. However, no information was available regarding an integrated design process considering the probable modes of failures experienced in Bangladesh. To address this, the present study established a systematic approach for embankment design considering the prevailing conditions in Bangladesh. The achievement in relation to the aims and objectives of this study as well as perceived limitations are discussed in the following sections.

## 11.2 Aims and Objectives

As mentioned in Section 1.2, the aim of this study was to develop a method for the design of a road embankment which is sustainable for the climate, soil and socio-economic conditions found in Bangladesh. To this end a conceptual model has been developed (cf. Figure 3.2 and 3.3), described and tested using data from an existing embankment in Bangladesh (see Section 6.4 and 8.2).

To meet the aim, the project had the following objects:

1. Research the literature to investigate existing methodologies which are used for conditions similar to those of the investigation: The literature review described in Chapter 2 identified the relevant components and associated techniques for an overall design methodology described in Chapter 3.
2. Review the existing embankment design process in Bangladesh: It was found that there is no such design process in Bangladesh. The available documents relating to embankments have been summarised in Chapter 2.
3. Identify possible modes of embankment failure: From the literature review and the experience of the author the following probable modes of failure are identified

    - Settlement of foundation and embankment materials
    - Slope stability failure of embankment and bearing capacity failure of foundation.

4. Identify the major factors (i.e. input parameters) which need to be considered in the design process: Via the literature review (Chapter 2) and from a sensitivity analysis carried out in Chapters 6 and 8 the input factors have been identified. These are summarised in Table 10.1 to 10.4.
5. Identify the worst case in relation to the seasonal water level variation in embankment design: Modelling was undertaken using software Settle3D and OASYS SLOPE to investigate the settlement and stability of embankment subjected to water level variation in Chapter 6 and 8. This found that the worst case for embankment stability was the full rapid drawdown of flood water.
6. Develop models of embankment stability: Two models were chosen based on the criteria mentioned in Chapter 3 and discussed in Chapter 8.
7. Build a conceptual model for the rational design of embankments: A conceptual model was developed based on knowledge acquired from literature review and given in Figures 3.2 and 3.3 in Chapter 3.
8. Test the usefulness and accuracy of the model using data obtained from Bangladesh: The models chosen for the embankment design were verified in Chapters 6 and 8 and a case study based on an early road was devised in Chapter 10.

The usefulness of this proposed method including the limitations and findings are also discussed in the following sections.

## 11.3 Design Methods

The methodology developed for design a highway embankment in this study is a combination of designs for several interrelated components of the embankment and foundation. Some of the design parts are interrelated directly or indirectly. The whole process depends on intermediate decision making and an iteration of the calculations. By nature the behaviour of soils are not straight forward and as a result any design needs the application of engineering judgement at every step of the process. The design process developed in this work should therefore be regarded as a basis for improved decision making. There are indeed many ways of solving the settlement and instability problems of embankment. In the method described herein a numbers of options were proposed which are considered sustainable in Bangladesh in respect of uptake, construction experience of the contractor, financial capability and success in previous project.

The stability of embankment with respect to slope and bearing capacity failure, settlement of embankment materials and the foundation, the height of embankment considering hydrology, construction specification in view of getting optimum subgrade strength, and subgrade design properties as input variable of pavement design are the elements of whole design process. Methods were presented to determine all the probable modes of embankment failure and their remedial measures. These methods of analysis can be easily carried out within a spreadsheet (cf. Appendices). In addition, two computer software packages, models OASYS SLOPE and Settle3D were used for the analysis of stability and settlement respectively. A number of remedial design options were also considered. An important additional feature of OASYS SLOPE is that an analysis can be carried out for various types of geo-reinforcement. Settlement analysis can also be carried out using Settle3D for the cases when prefabricated vertical drains are incorporated and a surcharge is applied.

## 11.4 Modes of Failures

An investigating of the existing roads in Bangladesh shows that the majority of road embankments fail by excessive settlement and slope stability. As some of the embankments were not adequately compacted using locally available unsuitable materials, settlements have been taken place within the embankment materials as well. This problem could be avoided only following the appropriate compaction specification (see Section 9.4) and using appropriate engineered materials. It is recommended that if the road side materials contain peat or organic soil, material from other locations should be used in their place. Before constructing national and regional highway embankments, subsoil investigation should be carried out to adopt appropriate remedial measures to reduce potential residual settlement and improve bearing capacity. However, for less important roads such as districts roads expensive subsoil investigation and subsoil improvement can be avoided.

In addition to that the above, the improper maintenance of soft verges results water accumulation and leads to slope failure due to the seepage of water through the tension cracks which inevitably develop. Thus appropriate maintenance of earthen slopes is necessary to maintain efficient surface water drainage which is also a requirement for embankment slope stability as well as to control pavement deterioration.

## 11.5 Input Parameters

The method proposed in this research has identified the input parameters for whole embankment design process (see Section 10.2) and via sensitivity analysis carried out in Sections 6.6 and 8.5 respectively have determined the relative importance of each in the design process. This will ultimately guide the collection and testing of the necessary parameters for any highway project. It is not always possible to determine the parameters directly from field and laboratory test as all the necessary equipment are not available in Bangladesh. Where this is the case they can be estimated using suitable correlations as discussed briefly in Section 10.2 and further elaborated in a complementary research project (see below).

## 11.6 Testing Methods

As the input parameters were identified for embankment design, the necessity and usefulness of field and laboratory testing were discussed. However, in the research reported herein the methods of testing the input parameters were not studied in depth as there was a parallel study (Shajahan, in print).

Soil parameters vary to a great extent depending on the method used to determine them and also spatially and temporally. Appropriate representative parameters are therefore needed to carry out the optimum design of a road embankment. To this end, a third research project carried out at the same time as this one is investigating introducing the concept of risk and reliability into the design process (Haider, in print).

## 11.7 Hydrological Loading

The method developed by this study considers the impact of hydrology (flooding) in the stability and settlement analysis of embankment as shown in Sections 6.5 and 8.3. In Bangladesh the embankment is vulnerable to a rapid drawdown of water level after flooding as most of the embankments are constructed through the area where seasonal water level variation is a common phenomenon. In the proposed method this phenomenon is taken into consideration when determining embankment stability. Stability charts have been developed considering this worst case of rapid drawdown. As the charts have been developed considering the same soil for both the embankment and foundation, a more rigorous analysis will be needed for embankments where the fill materials are obtained from other locations. It is recommended that the software OASYS SLOPE or similar be used to perform stability analyses using Bishop's method (Bishop, 1955). Where such software is unavailable, then a method based on Low's procedure (Low, 1989) should be used. However, as Low's method does not consider the impact of rapid drawdown of water, a modification to the process has been suggested in Section 8.6.

Infiltration of rain water is one of the major causes of slope instability of embankments in Bangladesh. However, this factor, though important was considered to be beyond the scope of this work. It is recommended, together with a number of other important factors in the following chapter.

## 11.8 Traffic Load Induced Settlement

The prediction of traffic load induced foundation settlement and its remedial measures were investigated to a limited extent in this study. A number of methods available in the literature for predicting consolidation and plastic settlement for repeated traffic loading were described with examples in Section 4.4.2. The influences of elastic modulus and shear strength of subgrade and foundation soil on settlement induced by traffic repetition were explained with the help of Chai and Miura's (2002) empirical method. Whilst the model was not validated by the present study it has been verified by Chai and Miura (2002) by observing actual settlement in a number of test road sections. However, further studies are needed which take into account the properties of pavement layers, thickness and stabilization of subgrade to accommodate traffic loading in settlement analysis of embankment and foundation. The study could be an integrated approach of pavement design as the settlement of foundation due to traffic loading is dependent on the material properties and thickness of pavement layers.

## 11.9 Improvement of Subsoil and Fill

Two methods of reducing residual settlement were investigated in detail namely the installation of vertical drains and application of surcharge as these were considered to be the most appropriate for the condition at hand in Bangladesh. These improvement measures also improve the shear strength of soil which has the added benefit of improving the stability of the embankment. In the cases where it is necessary to improve the stability of embankment and bearing capacity of foundation geo-reinforcement was also proposed in this study. In addition there are some other options such as the removal of very soft or peat soil from the upper layer of the foundation, use of vibroflotation and the use of chemical stabilizations.

However, their applicability to Bangladeshi conditions requires considerable further investigation.

As the study was limited to the embankment design process, improvements of subgrade strength to withstand traffic loading considering the strength of the pavement structures were not studied. The study concentrated on the process of the selection of subgrade and fill properties to find input variables for pavement design to be used in the parallel study being carried out as part of the overall research project. To reduce the traffic load induced subsoil settlement of low embankments, the pavement subgrade should be stabilised or improved. In this regard further research is needed considering the availability of subgrade or improved subgrade materials in Bangladesh.

# Chapter 12 Conclusions and Recommendations

## 12.1 Accomplished Work

This research project has developed an integrated conceptual and analytical embankment design process. The research programme has focused on developing a methodology (see Figures 3.1 – 3.3) for the design of an embankment which prevents the most probable modes of failure associated with the conditions found in Bangladesh. Namely, high embankments built on soft clays soils subject to large seasonal changes in water levels. The most probable failure modes were identified via a thorough literature review and from observations in the field. The design methodology therefore focused on ensuring: settlements are within acceptable limits and achieved within a realistic time frame, and; the slope of the embankment is stable and its foundation is of adequate bearing capacity.

## 12.2 Conclusions

The main conclusion of the work is as follows:

A systematic methodology, such as that are developed herein, can be used to design embankments to prevent them from failing by the prevalent failure modes applicable to conditions found in Bangladesh.

### 12.2.1. Findings

It was found that the various components of the developed methodology may be achieved successfully as follows:

**Subgrade specification**

- The design CBR should be determined at the insitu, or design, moisture content (DMC).
- It is necessary to determine the soil moisture characteristic curve for subgrade materials so that it can be used for selecting economic capillary height, or freeboard, to avoid moisture susceptibility of the subgrade.
- The bearing capacity of the subgrade can be improved by applying a higher compaction energy as it increases the subgrade's dry density. Increased compactive effort also increases the shear strength of the material which leads to an increased factor of safety against slope stability.

- To achieve higher density and CBR, it is recommended that the uppermost meter of the subgrade should be compacted at minimum 95% of MDD as per BS 4.5 (BS 1377-4, 1990) with a moisture content variation between -2 to 3% from OMC. Similarly 95% of standard relative compaction for fill materials with the same moisture content variation is recommended.
- Considering the type of materials, work volume and availability of equipment the thickness of the compacted layer can be limited to between 150mm - 300mm.

**Stability of embankment and bearing capacity of foundation**

- Bishop's method is a suitable method for determining slope stability with an acceptable degree of accuracy. It also enables analyses to be undertaken which allow for the inclusion of reinforcement and conditions of rapid drawdown.
- OASYS SLOPE is a suitable piece of computer software which can be used to implement Bishop's method.
- A method developed by Low (1989) may be used with reasonable accuracy to determine the short term stability of an embankment. It has the advantage that it can be implemented in a spreadsheet.
- Embankment stability increases with the increase of shear strength, cohesion ($c'$), friction parameter ($\varphi'$) and decreases with the increase of unit weight of fill ($\gamma$), slope angle ($\beta$), height ($H$) and the value of the pore pressure ratio ($r_u$). The stability is most sensitive to the friction angle ($\varphi'$), slope angle ($\beta$) and height of embankment ($H$). The influence of $\varphi'$ and ($\beta$) is almost double to that of $c'$.
- The depth of the foundation has only a small impact on the factor of safety.
- Reinforcement is effective in improving embankment stability on soft soil. It improves the shear strength of the foundation.
- Geo-cells can improve the bearing capacity of the foundation.
- Pressure berms are not suitable for improving embankment stability as they require a large amount of land and associated fill material.
- The use of vegetation in slopes and soft verges can improve embankment slope stability by increasing the factor of safety by up to 20%.

- The shear strength of soft foundations improves after consolidation.
- Consolidation can be accelerated by pre-compression or through the use of vertical drains.
- If the short term factor of safety is less than specified, the adoption of stage construction can improve stability, as the shear strength increases with time as a result of the consolidation process.

**Foundation Settlement**

**Traffic loading**

The impact of cumulative traffic loading on foundation settlement is negligible for an embankment height of more than three metres.

**Embankment loading**

- Two methods for calculating settlement were considered suitable for the task in hand.
    - The Finite difference software package Settle 3D which calculates stress in three dimensions and settlement in one dimension is the preferred option.
    - A method termed herein as the "Standard method" which consists of a number of methods available in the literature. Whilst it is theoretically not as accurate as the Finite Difference method it was considered suitable for the task in hand where software such as Settle 3D is unavailable as it can be implemented in a spreadsheet.
- Primary consolidation can be accelerated and the desired settlement can be achieved by the installation of vertical drains or via the application of a surcharge.
- It is difficult to take remedial measures to reduce the secondary compression although some researchers suggest that a surcharge load can be applied successfully to reduce secondary compression especially for peat soils.
- Hansbo (1981) and Yeung (1997) methods are suitable to determine the vertical drain spacing to increase the rate of consolidation. Both the methods consider smear effect and well resistance which improves the accuracy of the analysis.
- The effect of well resistance is insignificant compared to the smear effect.

- Water level above the ground surface has direct impact on elastic and primary settlement and an indirect impact on secondary consolidation settlement.
- The improvement of shear strength due to consolidation process can be determined by a number of formulae available in the literature. However, the shear strength values obtained using these formulae vary considerably. The empirical formula proposed by Stamatopoulos et al. (1985) is suitable on the basis of having easily measurable parameters.

## 12.3 Recommendations for further study

The following research is recommended to further refine the design methodology developed and described herein:

- Carry out further field investigations in Bangladesh to record characteristic properties of the soil layers in existing embankments including: shear strength values and settlement properties.
- Study the effects of vegetation on shear strength and stability.
- Investigate the shear strength of stabilised soils including the available dredged sand in Bangladesh.
- Study the secondary compression coefficient of problematic soils in Bangladesh including the post surcharge effect on secondary compression giving special attention to peat, organic and soft clay.
- Via field trials, study the effect and suitability of various ground improvement techniques described in the literature.
- Similarly, carry out field trials to determine the suitability of various types of geo-reinforcement to improve the bearing capacity of the foundation and the shearing resistance to the embankment.
- As a trial construct, an embankment of dredged sand with a cohesive blanket layer outside the embankment.
- Carry out further field trials to determine the effect of traffic load induced foundation settlement.

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# Appendix

### Appendix - I Notation

The following symbols are used in this thesis

| Symbol | Description | Unit |
|---|---|---|
| $\gamma$ | Unit weight of soil | kN/m$^3$ |
| $c$ | Cohesion (total stress parameter) | kN/m$^2$ |
| $c'$ | Cohesion (effective stress parameter) | kN/m$^2$ |
| $\varphi$ | Angle of internal friction (total stress parameter) | Degree |
| $\varphi'$ | Angle of internal friction (effective stress parameter) | Degree |
| $u$ | Pore pressure | kN/m$^2$ |
| $w$ | Natural water content | Percent |
| $LL$ | Liquid limit | Percent |
| $PL$ | Plastic limit | Percent |
| $PI$ | Plasticity index | Percent |
| $S_u$ | Undrained shear strength | kN/m$^2$ |
| $PSD$ | Particle size distribution | Percent |
| $G_s$ | Specific gravity of soil | Unit less |
| $E_u$ | Undrained elastic modulus | kN/m$^2$ |
| $E_{sur}$ | Unload-reload modulus | kN/m$^2$ |
| $\upsilon$ | Poisson's ratio | Unit less |
| $A$ | Pore pressure coefficient | Unit less |
| $C_c$ | Coefficient of compressibility or compression index | Unit less |
| $C_r$ | Recompression index | Unit less |

| | | |
|---|---|---|
| $C_\alpha$ | Coefficient of secondary compression | Unit less |
| $C''_\alpha$ | Coefficient of post surcharge secant secondary compression index | Unit less |
| $C_s$ | Coefficient of swelling | Unit less |
| $C_v$ | Coefficient of consolidation | $m^2$/day or $m^2$/year |
| $p'_c$ | Pre-consolidation pressure | $kN/m^2$ |
| $e_o$ | Initial void ration | Unit less |
| $k_h$ | Coefficient of permeability in horizontal direction | m/sec or m/day |
| $k_v$ | Coefficient of permeability in vertical direction | m/sec or m/day |
| $h$ | Height of embankment | meter |
| $\beta$ | Slope angle of embankment | Degree |
| $SR$ | Settlement ratio | |
| $OCR$ | Over consolidation ratio | |
| $\sigma_{sc}$ | Stress due to surcharge | $kN/m^2$ |
| $\sigma_{st}$ | Stress due to structure or embankment load | $kN/m^2$ |
| *SMCC* | Soil moisture characteristics curve | - |
| *DMC* | Design moisture content | - |
| *OMC* | Optimum moisture content | - |
| *DD* | Dry density | - |
| *MDD* | Maximum dry density | |

## Appendix – II Bishop and Morgenstern Values of m´ and n´

**Table 9.2** Bishop and Morgenstern's values of $m'$ and $n'$

*a. Stability coefficients $m'$ and $n'$ for $c'/\gamma H = 0$*

| | Stability coefficients for earth slopes | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Slope 2:1 | | Slope 3:1 | | Slope 4:1 | | Slope 5:1 | |
| $\phi'$ | $m'$ | $n'$ | $m'$ | $n'$ | $m'$ | $n'$ | $m'$ | $n'$ |
| 10.0 | 0.353 | 0.441 | 0.529 | 0.588 | 0.705 | 0.749 | 0.882 | 0.917 |
| 12.5 | 0.443 | 0.554 | 0.665 | 0.739 | 0.887 | 0.943 | 1.109 | 1.153 |
| 15.0 | 0.536 | 0.670 | 0.804 | 0.893 | 1.072 | 1.139 | 1.340 | 1.393 |
| 17.5 | 0.631 | 0.789 | 0.946 | 1.051 | 1.261 | 1.340 | 1.577 | 1.639 |
| 20.0 | 0.728 | 0.910 | 1.092 | 1.213 | 1.456 | 1.547 | 1.820 | 1.892 |
| 22.5 | 0.828 | 1.035 | 1.243 | 1.381 | 1.657 | 1.761 | 2.071 | 2.153 |
| 25.0 | 0.933 | 1.166 | 1.399 | 1.554 | 1.865 | 1.982 | 2.332 | 2.424 |
| 27.5 | 1.041 | 1.301 | 1.562 | 1.736 | 2.082 | 2.213 | 2.603 | 2.706 |
| 30.0 | 1.155 | 1.444 | 1.732 | 1.924 | 2.309 | 2.454 | 2.887 | 3.001 |
| 32.5 | 1.274 | 1.593 | 1.911 | 2.123 | 2.548 | 2.708 | 3.185 | 3.311 |
| 35.0 | 1.400 | 1.750 | 2.101 | 2.334 | 2.801 | 2.977 | 3.501 | 3.639 |
| 37.5 | 1.535 | 1.919 | 2.302 | 2.558 | 3.069 | 3.261 | 3.837 | 3.989 |
| 40.0 | 1.678 | 2.098 | 2.517 | 2.797 | 3.356 | 3.566 | 4.196 | 4.362 |

*b. Stability coefficients $m'$ and $n'$ for $c'/\gamma H = 0.025$ and $D = 1.00$*

| | Stability coefficients for earth slopes | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Slope 2:1 | | Slope 3:1 | | Slope 4:1 | | Slope 5:1 | |
| $\phi'$ | $m'$ | $n'$ | $m'$ | $n'$ | $m'$ | $n'$ | $m'$ | $n'$ |
| 10.0 | 0.678 | 0.534 | 0.906 | 0.683 | 1.130 | 0.846 | 1.367 | 1.031 |
| 12.5 | 0.790 | 0.655 | 1.066 | 0.849 | 1.337 | 1.061 | 1.620 | 1.282 |
| 15.0 | 0.901 | 0.776 | 1.224 | 1.014 | 1.544 | 1.273 | 1.868 | 1.534 |
| 17.5 | 1.012 | 0.898 | 1.380 | 1.179 | 1.751 | 1.485 | 2.121 | 1.789 |
| 20.0 | 1.124 | 1.022 | 1.542 | 1.347 | 1.962 | 1.698 | 2.380 | 2.050 |
| 22.5 | 1.239 | 1.150 | 1.705 | 1.518 | 2.177 | 1.916 | 2.646 | 2.317 |
| 25.0 | 1.356 | 1.282 | 1.875 | 1.696 | 2.400 | 2.141 | 2.921 | 2.596 |
| 27.5 | 1.478 | 1.421 | 2.050 | 1.882 | 2.631 | 2.375 | 3.207 | 2.886 |
| 30.0 | 1.606 | 1.567 | 2.235 | 2.078 | 2.873 | 2.622 | 3.508 | 3.191 |
| 32.5 | 1.739 | 1.721 | 2.431 | 2.285 | 3.127 | 2.883 | 3.823 | 3.511 |
| 35.0 | 1.880 | 1.885 | 2.635 | 2.505 | 3.396 | 3.160 | 4.156 | 3.849 |
| 37.5 | 2.030 | 2.060 | 2.855 | 2.741 | 3.681 | 3.458 | 4.510 | 4.209 |
| 40.0 | 2.190 | 2.247 | 3.090 | 2.993 | 3.984 | 3.778 | 4.885 | 4.592 |

*c. Stability coefficients m′ and n′ for $c'/\gamma H = 0.025$ and $D = 1.25$*

| | Stability coefficients for earth slopes | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Slope 2:1 | | Slope 3:1 | | Slope 4:1 | | Slope 5:1 | |
| $\phi'$ | $m'$ | $n'$ | $m'$ | $n'$ | $m'$ | $n'$ | $m'$ | $n'$ |
| 10.0 | 0.737 | 0.614 | 0.901 | 0.726 | 1.085 | 0.867 | 1.285 | 1.014 |
| 12.5 | 0.878 | 0.759 | 1.076 | 0.908 | 1.299 | 1.098 | 1.543 | 1.278 |
| 15.0 | 1.019 | 0.907 | 1.253 | 1.093 | 1.515 | 1.311 | 1.803 | 1.545 |
| 17.5 | 1.162 | 1.059 | 1.433 | 1.282 | 1.736 | 1.541 | 2.065 | 1.814 |
| 20.0 | 1.309 | 1.216 | 1.618 | 1.478 | 1.961 | 1.775 | 2.334 | 2.090 |
| 22.5 | 1.461 | 1.379 | 1.808 | 1.680 | 2.194 | 2.017 | 2.610 | 2.373 |
| 25.0 | 1.619 | 1.547 | 2.007 | 1.891 | 2.437 | 2.269 | 2.879 | 2.669 |
| 27.5 | 1.783 | 1.728 | 2.213 | 2.111 | 2.689 | 2.531 | 3.196 | 2.976 |
| 30.0 | 1.956 | 1.915 | 2.431 | 2.342 | 2.953 | 2.806 | 3.511 | 3.299 |
| 32.5 | 2.139 | 2.112 | 2.659 | 2.686 | 3.231 | 3.095 | 3.841 | 3.638 |
| 35.0 | 2.331 | 2.321 | 2.901 | 2.841 | 3.524 | 3.400 | 4.191 | 3.998 |
| 37.5 | 2.536 | 2.541 | 3.158 | 3.112 | 3.835 | 3.723 | 4.563 | 4.379 |
| 40.0 | 2.753 | 2.775 | 3.431 | 3.399 | 4.164 | 4.064 | 4.958 | 4.784 |

*d. Stability coefficients m′ and n′ for $c'/\gamma H = 0.05$ and $D = 1.00$*

| | Stability coefficients for earth slopes | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Slope 2:1 | | Slope 3:1 | | Slope 4:1 | | Slope 5:1 | |
| $\phi'$ | $m'$ | $n'$ | $m'$ | $n'$ | $m'$ | $n'$ | $m'$ | $n'$ |
| 10.0 | 0.913 | 0.563 | 1.181 | 0.717 | 1.469 | 0.910 | 1.733 | 1.069 |
| 12.5 | 1.030 | 0.690 | 1.343 | 0.878 | 1.688 | 1.136 | 1.995 | 1.316 |
| 15.0 | 1.145 | 0.816 | 1.506 | 1.043 | 1.904 | 1.353 | 2.256 | 1.567 |
| 17.5 | 1.262 | 0.942 | 1.671 | 1.212 | 2.117 | 1.565 | 2.517 | 1.825 |
| 20.0 | 1.380 | 1.071 | 1.840 | 1.387 | 2.333 | 1.776 | 2.783 | 2.091 |
| 22.5 | 1.500 | 1.202 | 2.014 | 1.568 | 2.551 | 1.989 | 3.055 | 2.365 |
| 25.0 | 1.624 | 1.338 | 2.193 | 1.757 | 2.778 | 2.211 | 3.336 | 2.651 |
| 27.5 | 1.753 | 1.480 | 1.380 | 1.952 | 3.013 | 2.444 | 3.628 | 2.948 |
| 30.0 | 1.888 | 1.630 | 2.574 | 2.157 | 3.261 | 2.693 | 3.934 | 3.259 |
| 32.5 | 2.029 | 1.789 | 2.777 | 2.370 | 3.523 | 2.961 | 4.256 | 3.585 |
| 35.0 | 2.178 | 1.958 | 2.990 | 2.592 | 3.803 | 3.253 | 4.597 | 3.927 |
| 37.5 | 2.336 | 2.138 | 3.215 | 2.826 | 4.103 | 3.574 | 4.959 | 4.288 |
| 40.0 | 2.505 | 2.332 | 3.451 | 3.071 | 4.425 | 3.926 | 5.344 | 4.668 |

## Appendix-III: Long Term Stability Charts Determined Using Bishop's Method

# Factor of safety against slope stability

| | | h = 3.0 m | | | h = 4.0 m | | | h = 5.0 m | | | h = 6 m | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| C' | φ' | 1:1.5 (33.7Deg) | 1:2 (26.6Deg) | 1:2.5 (21.8Deg) | 1:1.5 (33.7Deg) | 1:2 (26.6Deg) | 1:2.5 (21.8Deg) | 1:1.5 (33.7Deg) | 1:2 (26.6Deg) | 1:2.5 (21.8Deg) | 1:1.5 (33.7Deg) | 1:2 (26.6Deg) | 1:2.5 (21.8Deg) |
| 5 | 15 | 0.718 | 0.823 | 0.903 | 0.639 | 0.745 | 0.819 | 0.588 | 0.684 | 0.763 | 0.536 | 0.631 | 0.717 |
| 5 | 20 | 0.820 | 0.957 | 1.084 | 0.727 | 0.865 | 0.982 | 0.680 | 0.814 | 0.918 | 0.622 | 0.758 | 0.869 |
| 5 | 25 | 0.927 | 1.097 | 1.241 | 0.822 | 0.994 | 1.150 | 0.769 | 0.939 | 1.083 | 0.709 | 0.877 | 1.028 |
| 5 | 30 | 1.041 | 1.242 | 1.412 | 0.924 | 1.129 | 1.310 | 0.865 | 1.069 | 1.252 | 0.802 | 1.004 | 1.186 |
| 5 | 35 | 1.169 | 1.403 | 1.598 | 1.034 | 1.276 | 1.486 | 0.970 | 1.214 | 1.426 | 0.901 | 1.145 | 1.362 |
| 10 | 15 | 1.107 | 1.230 | 1.302 | 0.957 | 1.072 | 1.175 | 0.873 | 0.978 | 1.088 | 0.782 | 0.898 | 1.003 |
| 10 | 20 | 1.239 | 1.392 | 1.483 | 1.082 | 1.224 | 1.344 | 0.984 | 1.121 | 1.251 | 0.890 | 1.029 | 1.157 |
| 10 | 25 | 1.356 | 1.549 | 1.675 | 1.207 | 1.387 | 1.520 | 1.100 | 1.267 | 1.414 | 1.004 | 1.168 | 1.322 |
| 10 | 30 | 1.479 | 1.704 | 1.882 | 1.313 | 1.541 | 1.708 | 1.216 | 1.426 | 1.593 | 1.109 | 1.318 | 1.498 |
| 10 | 35 | 1.608 | 1.874 | 2.125 | 1.426 | 1.695 | 1.916 | 1.332 | 1.593 | 1.791 | 1.218 | 1.482 | 1.694 |
| 15 | 10 | 1.303 | 1.417 | 1.519 | 1.138 | 1.246 | 1.353 | 1.019 | 1.126 | 1.224 | 0.921 | 1.029 | 1.121 |
| 15 | 15 | 1.449 | 1.586 | 1.696 | 1.259 | 1.393 | 1.524 | 1.155 | 1.268 | 1.394 | 1.024 | 1.163 | 1.282 |
| 15 | 20 | 1.601 | 1.781 | 1.881 | 1.385 | 1.548 | 1.697 | 1.265 | 1.410 | 1.566 | 1.131 | 1.295 | 1.443 |
| 15 | 25 | 1.767 | 1.954 | 2.074 | 1.519 | 1.710 | 1.876 | 1.383 | 1.563 | 1.741 | 1.245 | 1.434 | 1.608 |
| 15 | 30 | 1.894 | 2.141 | 2.283 | 1.664 | 1.886 | 2.070 | 1.510 | 1.726 | 1.923 | 1.369 | 1.586 | 1.786 |
| 20 | 10 | 1.644 | 1.781 | 1.911 | 1.438 | 1.566 | 1.688 | 1.286 | 1.408 | 1.524 | 1.160 | 1.284 | 1.392 |
| 20 | 15 | 1.790 | 1.950 | 2.088 | 1.561 | 1.714 | 1.869 | 1.398 | 1.554 | 1.693 | 1.266 | 1.420 | 1.553 |
| 20 | 20 | 1.942 | 2.167 | 2.274 | 1.687 | 1.868 | 2.044 | 1.547 | 1.700 | 1.870 | 1.373 | 1.560 | 1.721 |
| 20 | 25 | 2.105 | 2.343 | 2.471 | 1.821 | 2.033 | 2.228 | 1.665 | 1.852 | 2.054 | 1.486 | 1.701 | 1.896 |

Figure 1: Factor of safety against slope stability in worst case (L/h = 1.0) for given shear strength paramaters and slope angles. Slope height, h = 3.0 m

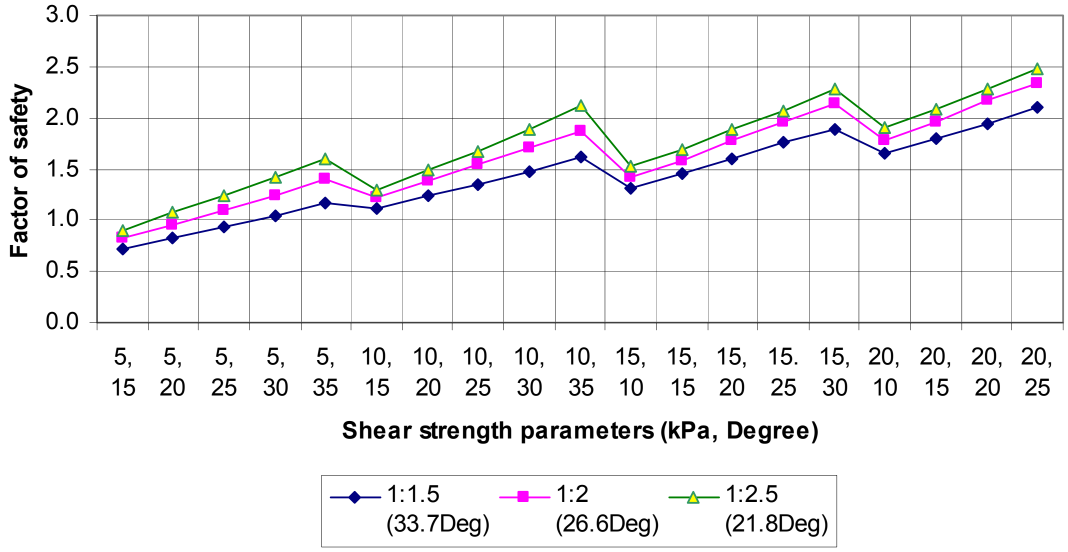
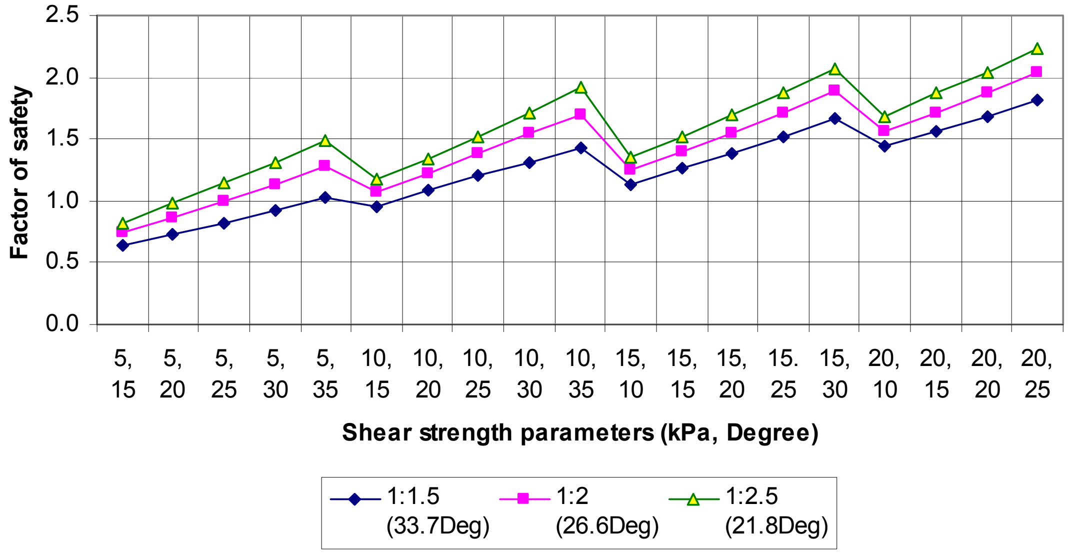



Figure 2: Factor of safety against slope stability in worst case (L/h = 1.0) for given shear strength paramaters and slope angles. Slope height, h = 4.0 m



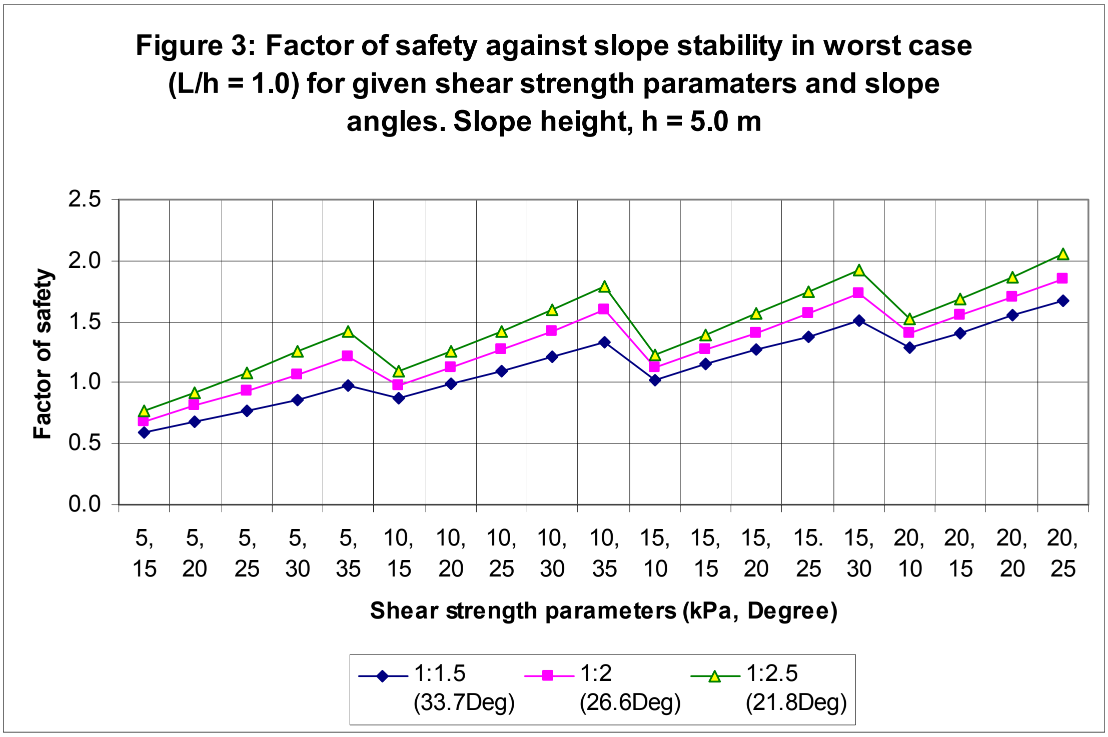


Figure 3: Factor of safety against slope stability in worst case (L/h = 1.0) for given shear strength paramaters and slope angles. Slope height, h = 5.0 m
Factor of safety
2.5
2.0
1.5
1.0
0.5
0.0
5, 15
5, 20
5, 25
5, 30
5, 35
10, 15
10, 20
10, 25
10, 30
10, 35
15, 10
15, 15
15, 20
15. 25
15, 30
20, 10
20, 15
20, 20
20, 25
Shear strength parameters (kPa, Degree)
1:1.5 (33.7Deg)
1:2 (26.6Deg)
1:2.5 (21.8Deg)

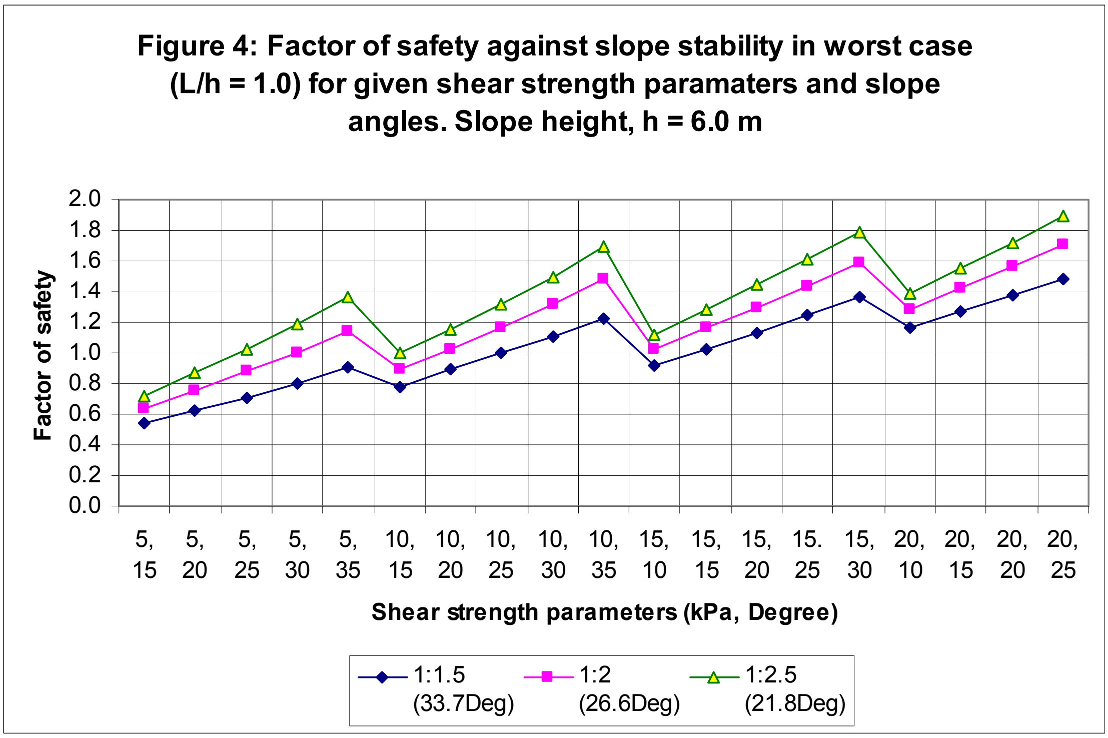
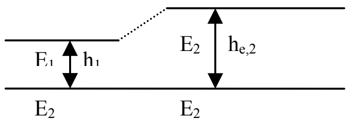


Figure 4: Factor of safety against slope stability in worst case (L/h = 1.0) for given shear strength paramaters and slope angles. Slope height, h = 6.0 m
Factor of safety
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
5, 15
5, 20
5, 25
5, 30
5, 35
10, 15
10, 20
10, 25
10, 30
10, 35
15, 10
15, 15
15, 20
15. 25
15, 30
20, 10
20, 15
20, 20
20, 25
Shear strength parameters (kPa, Degree)
1:1.5 (33.7Deg)
1:2 (26.6Deg)
1:2.5 (21.8Deg)

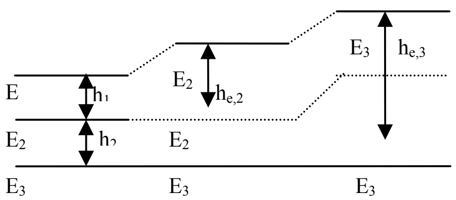
## Appendix IV: Spread Sheet set up for Low's Method

| $h$ | $D$ | $D/H$ | $\beta$ | $\alpha_1$ | $\alpha_2$ | $\lambda$ | $N_m$ | $N_f$ | $N_m+N_f$ | $C_m$ | $\varphi_m$ | $\gamma$ | $C_T$ | $C_D$ | $\Delta C_T$ | $D_c$ | $C_A$ | $(Fs)_D$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 3.5 | 0.50 | 0.14 | 26.5 | 1.246 | 0.545 | 0.251 | 2.781 | 2.766 | 5.547 | 4.5 | 18 | 17.5 | 22.3 | 22.3 | | | 22.3 | 1.44 |
| 3.5 | 1.00 | 0.29 | 26.5 | 1.426 | 0.762 | 0.220 | 2.123 | 3.482 | 5.604 | 4.5 | 18 | 17.5 | 22.3 | 17.1 | | | 18.9 | 1.38 |
| 3.5 | 1.50 | 0.43 | 26.5 | 1.619 | 1.023 | 0.210 | 1.731 | 3.877 | 5.608 | 4.5 | 18 | 17.5 | 22.3 | 28.2 | | | 26.1 | 1.90 |
| 3.5 | 2.00 | 0.57 | 26.5 | 1.820 | 1.327 | 0.205 | 1.467 | 4.135 | 5.602 | 4.5 | 18 | 17.5 | 22.3 | 19.0 | | | 20.1 | 1.57 |
| 3.5 | 2.50 | 0.71 | 26.5 | 2.027 | 1.674 | 0.202 | 1.275 | 4.319 | 5.595 | 4.5 | 18 | 17.5 | 22.3 | 14.3 | | | 17.1 | 1.38 |
| 3.5 | 3.00 | 0.86 | 26.5 | 2.237 | 2.065 | 0.200 | 1.129 | 4.459 | 5.588 | 4.5 | 18 | 17.5 | 22.3 | 34.7 | | | 30.3 | 2.36 |
| 3.5 | 3.50 | 1.00 | 26.5 | 2.449 | 2.499 | 0.199 | 1.014 | 4.568 | 5.582 | 4.5 | 18 | 17.5 | 22.3 | 41.6 | | | 34.8 | 2.74 |
| 3.5 | 4.00 | 1.14 | 26.5 | 2.664 | 2.977 | 0.198 | 0.921 | 4.657 | 5.578 | 4.5 | 18 | 17.5 | 22.3 | 20.4 | | | 21.0 | 1.73 |
| 3.5 | 4.50 | 1.29 | 26.5 | 2.880 | 3.498 | 0.197 | 0.843 | 4.731 | 5.574 | 4.5 | 18 | 17.5 | 22.3 | 10.8 | | | 14.9 | 1.26 |
| 3.5 | 5.00 | 1.43 | 26.5 | 3.097 | 4.063 | 0.196 | 0.778 | 4.793 | 5.571 | 4.5 | 18 | 17.5 | 22.3 | 13.1 | | | 16.3 | 1.38 |
| 3.5 | 5.50 | 1.57 | 26.5 | 3.314 | 4.671 | 0.196 | 0.723 | 4.846 | 5.569 | 4.5 | 18 | 17.5 | 22.3 | 13.3 | | | 16.5 | 1.40 |

## Appendix V: Method of Equivalent Thickness




$$h_{e,2} = h_1 . \left( \frac{E_1}{E_2} \right)^{\frac{1}{3}}$$




$$h_{e,3} = f . \{ h_1 . \left( \frac{E_1}{E_2} \right)^{\frac{1}{3}} + h_2 \} . \left( \frac{E_2}{E_3} \right)^{\frac{1}{3}}$$

$$= f . ( h_{e,2} + h_2 ) . \left( \frac{E_2}{E_3} \right)^{\frac{1}{3}}$$

## Appendix VI: Subgrade Dry Density and Percent Compaction of Tongi-Ashulia Road

| Location | MDD | OMC | FDD | FMC | compaction | CBR | | | Resilient Modulus (MPa) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| KM | kN/m3 | % | kN/m3 | % | % | Un-soaked | soaked | From DCP | Shell | TRRL | Us Army | CSIR |
| 1+300 | 16.971 | 16.11 | 12.449 | 22.45 | 73.35 | 6.72 | 1.04 | 7.30 | 10.40 | 18.05 | 38.31 | 21.20 |
| 1+600 | 16.294 | 16.17 | 13.852 | 22.68 | 85.01 | 3.67 | 2.44 | 5.20 | 24.40 | 31.15 | 70.19 | 36.91 |
| 1+800 | 16.971 | 16.09 | 16.324 | 17.06 | 96.18 | 4.20 | 1.14 | 5.10 | 11.40 | 19.14 | 40.89 | 22.51 |
| 2+200 | 15.637 | 14.23 | 12.253 | 32.23 | 78.36 | 6.29 | 2.00 | 5.80 | 20.00 | 27.43 | 60.95 | 32.43 |
| 2+500 | 15.706 | 14.21 | 14.028 | 21.06 | 89.32 | 4.54 | 1.05 | 5.00 | 10.50 | 18.16 | 38.57 | 21.34 |
| 2+900 | 16.618 | 18.48 | 14.175 | 16.73 | 85.30 | 5.94 | 1.92 | 8.30 | 19.20 | 26.72 | 59.21 | 31.59 |
| 3+200 | 17.374 | 16.26 | 15.225 | 16.69 | 87.63 | 4.71 | 2.36 | 6.00 | 23.60 | 30.49 | 68.55 | 36.12 |
| 3+500 | 15.980 | 18.64 | 13.734 | 23.73 | 85.94 | 6.55 | 1.23 | 5.30 | 12.30 | 20.09 | 43.16 | 23.65 |
| 3+800 | 17.540 | 14.08 | 15.058 | 14.13 | 85.85 | 5.15 | 2.10 | 7.20 | 21.00 | 28.30 | 63.10 | 33.48 |
| 4+200 | 17.884 | 14.44 | 14.460 | 16.25 | 80.86 | 3.84 | 1.75 | 5.30 | 17.50 | 25.18 | 55.44 | 29.74 |
| 4+600 | 17.236 | 18.10 | 13.793 | 23.30 | 80.02 | 6.98 | 2.36 | 9.00 | 23.60 | 30.49 | 68.55 | 36.12 |
| 4+900 | 17.236 | 18.00 | 15.304 | 21.23 | 88.79 | 5.76 | 2.00 | 4.30 | 20.00 | 27.43 | 60.95 | 32.43 |
| 5+300 | 17.168 | 16.11 | 13.597 | 19.69 | 79.20 | 4.02 | 1.92 | 8.20 | 19.20 | 26.72 | 59.21 | 31.59 |
| 5+900 | 15.814 | 14.27 | 13.606 | 24.11 | 86.04 | 5.15 | 1.16 | 6.10 | 11.60 | 19.35 | 41.40 | 22.76 |
| Mean | 16.745 | 16.085 | 14.133 | 20.81 | 84.418 | 5.251 | 1.75 | 6.293 | 17.48 | 24.91 | 54.89 | 29.42 |
| Standard dev | 0.737 | 1.687 | 1.096 | 4.61 | 5.674 | 1.129 | 0.52 | 1.453 | 5.20 | 4.90 | 11.90 | 5.87 |
| Variance | 0.544 | 2.847 | 1.202 | 21.29 | 32.191 | 1.275 | 0.27 | 2.110 | 27.00 | 24.00 | 141.72 | 34.48 |

## Appendix VII: Example of Critical Height ($H_c$) and Bearing Capacity beneath the Embankment due to Embankment loading

**Input Data**

| Parameters | | unit | value |
|---|---|---|---|
| Undrained shear strength at surface | $c_m$ | kN/m$^2$ | 20.00 |
| Shear strength gradient with depth | $\varepsilon$ | kN/m$^3$ | 0.00 |
| Unit weight of fill | $\gamma_f$ | kN/m$^3$ | 17.50 |
| Angle of internal friction | $\varphi_f$ | Degree | 23.00 |
| Embankment slope | $\beta$ | Degree | 26.56 |
| Height of embankment | $H$ | meter | 3.00 |
| | $t$ | meter | 5.50 |
| | $\varepsilon t/c_m$ | 0 | |

$$k \geq \tan^2\left(\frac{\pi}{4} - \frac{\varphi_f}{2}\right)$$

$$\chi = \frac{\bar{\tau}}{c} = \frac{\gamma_f H^2 k}{2bc} \qquad \chi = \frac{\bar{\tau}}{c} = \frac{\gamma_f H^2 k - 2R}{2bc}$$

$$H_c = b\tan\beta\left(1 - \sqrt{1 - \frac{2\bar{q}}{b\gamma_f \tan\beta}}\right)$$

**Estimation of Critical height ($H_c$)**

| $\beta$ | $b$ | $t$ | $b/t$ | $K$ | $H$ | $X$ | $\bar{q}/c_m$ | $\bar{q}$ | $H_c$ | Comment |
|---|---|---|---|---|---|---|---|---|---|---|
| 26.5 | 22.200 | 5 | 4.44 | 0.438092 | 5.00 | 0.22 | 5.80 | 116.0000 | 9.36 | Hc ≠ He |
| 26.5 | 30.920 | 5 | 6.18 | 0.438092 | 9.36 | 0.54 | 4.80 | 96.0000 | 7.75 | Hc ≠ He |
| 26.5 | 27.700 | 5 | 5.54 | 0.438092 | 7.75 | 0.42 | 4.90 | 98.0000 | 7.91 | Hc ≈ He |

**Estimation of Critical height with Reinforcement at the interface of the foundation and embankment**

**Input : Same as above** $R$ = 50 kN/m Tensile strength of reinforcement per unit length

| $\beta$ | $b$ | $t$ | $b/t$ | $K$ | $H$ | $X$ | $\bar{q}/c_m$ | $\bar{q}$ | $H_c$ | Comment |
|---|---|---|---|---|---|---|---|---|---|---|
| 26.5 | 28.200 | 5 | 5.64 | 0.338114 | 8 | 0.25 | 6 | 120.0000 | 9.68 | Hc ≠ He |
| 26.5 | 31.560 | 5 | 6.31 | 0.338114 | 9.68 | 0.36 | 5.8 | 116.0000 | 9.36 | Hc ≠ He |
| 26.5 | 30.600 | 5 | 6.12 | 0.338114 | 9.2 | 0.33 | 5.7 | 114.0000 | 9.20 | Hc ≈ He |

Fully Reinforced, X = 0.0 $R$ = 378 kN/m

| $\beta$ | $b$ | $t$ | $b/t$ | $K$ | $H$ | $X$ | $\bar{q}/c_m$ | $\bar{q}$ | $H_c$ | Comment |
|---|---|---|---|---|---|---|---|---|---|---|
| 26.5 | 30.600 | 5 | 6.12 | 0.338114 | 9.2 | 0.00 | 7 | 140.0000 | 11.30 | Single trial |

**Example: Critical height of embankment** (Detail calculation are shown in above)

| Parameters | Notation | unit | value |
|---|---|---|---|
| Undrained shear strength at surface | $c_m$ | $kN/m^2$ | 20.00 |
| Shear strength gradient with depth | $\varepsilon$ | $kN/m^2/m$ | 0.00 |
| Unit weight of fill | $\gamma_f$ | $kN/m^3$ | 17.50 |
| Angle of internal friction | $\varphi_f$ | Degree | 23.00 |
| Embankment slope | $\beta$ | Degree | 26.56 |
| Height of embankment<br>Thickness of soft layer | $H$<br>$t$ | meter<br>metre | 3.00<br>5.50 |
| | $\varepsilon t/c_m$ | | 0 |

Critical heights of embankment have been determined for unreinforced, partially reinforced and fully reinforced conditions and the summary of the analysis are given as follows.

1. Unreinforced condition

b/t = 5.54, $\chi$ = 0.42, $\varepsilon t/c_m$ = 0.0 from Figure 7.10 $\frac{\bar{q}}{c_m} = 4.90$, $\bar{q} = 98.00$ kPa, $H_c$ = 7.91 m

2. Partially reinforced condition: Horizontal thrust is taken by reinforcement and foundation shear strength

b/t = 6.12, $\chi$ = 0.33, $\varepsilon t/c_m$ = 0.0 from Figure 7.10 $\frac{\bar{q}}{c_m} = 5.7$, $\bar{q} = 114.0$ kPa, $H_c$ = 9.20 m with R = 50 kN/m

3. Fully reinforced: Total horizontal thrust is taken by the reinforcement

b/t = 6.12, $\chi$ = -0.0, $\varepsilon t/c_m$ = 0.0 from Figure 7.10 $\frac{\bar{q}}{c_m} = 7.0$, $\bar{q} = 140.0$ kPa, $H_c$ = 11.30 m with R = 378 kN/m

# Appendix-VIII: Example and Spread Sheet set up for Pressure Berms Design

## Pressure berms: Calculation of height and width

**Height of pressure berms ($h_1$)**

| h | $\rho_m$ | g | $q_{traffic}$ | $S_u$ | F | $\rho_{pb}$ | $h_1$ |
|---|---|---|---|---|---|---|---|
| m | $t/m^3$ | $m/sec^2$ | kPa | kPa | | $t/m^3$ | m |
| 1 | 1.7 | 9.8 | 10 | 15 | 1.5 | 1.7 | -1.71 |
| 2 | 1.7 | 9.8 | 10 | 15 | 1.5 | 1.7 | -0.71 |
| 3 | 1.7 | 9.8 | 10 | 15 | 1.5 | 1.7 | 0.29 |
| 4 | 1.7 | 9.8 | 10 | 15 | 1.5 | 1.7 | 1.29 |
| 5 | 1.7 | 9.8 | 10 | 15 | 1.5 | 1.7 | 2.29 |
| 6 | 1.7 | 9.8 | 10 | 15 | 1.5 | 1.7 | 3.29 |

**Width of pressure berms ($b_2$)**

| h | $\rho_m$ | g | $q_{traffic}$ | $S_u$ | F | q | $q_1=q+q_{trafic}$ | $S_{allow}=S_u/F$ | $S_{allow}/q_1$ | $b_2/D$ Nomograph | D | $b_2$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| m | $t/m^3$ | $m/sec^2$ | kPa | kPa | | kPa | kPa | kPa | | | m | m |
| 1 | 1.7 | 9.8 | 10 | 15 | 1.5 | 16.66 | 26.66 | 10 | 0.38 | | | |
| 2 | 1.7 | 9.8 | 10 | 15 | 1.5 | 33.32 | 43.32 | 10 | 0.23 | | | |
| 3 | 1.7 | 9.8 | 10 | 15 | 1.5 | 49.98 | 59.98 | 10 | 0.17 | 1.62 | 9 | 14.58 |
| 4 | 1.7 | 9.8 | 10 | 15 | 1.5 | 66.64 | 76.64 | 10 | 0.13 | 2.23 | 9 | 20.07 |
| 5 | 1.7 | 9.8 | 10 | 15 | 1.5 | 83.30 | 93.30 | 10 | 0.11 | 3.40 | 9 | 30.60 |
| 6 | 1.7 | 9.8 | 10 | 15 | 1.5 | 99.96 | 109.96 | 10 | 0.09 | 5.20 | 9 | 46.80 |

**Surface slope of the pressure berms**

kfu 0.5

| h | $\rho_m$ | g | $q_{traffic}$ | $\tau_{ofu}$ | F | q | $q_1=q+q_{trafic}$ | k | $\tau_o = T_{ofu}/F$ | $D_1$ | $k.D_1/To$ | $q_1/T_o$ | k/σ From Nomogram | n |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| m | $t/m^3$ | $m/sec^2$ | kPa | kPa | | kPa | kPa | | | m | | | | |
| 1 | 1.7 | 9.8 | 10 | 15 | 1.5 | 16.66 | 26.66 | 0.333 | 10.00 | 9 | 0.3 | 2.67 | | |
| 2 | 1.7 | 9.8 | 10 | 15 | 1.5 | 33.32 | 43.32 | 0.333 | 10.00 | 9 | 0.3 | 4.33 | | |
| 3 | 1.7 | 9.8 | 10 | 15 | 1.5 | 49.98 | 59.98 | 0.333 | 10.00 | 9 | 0.3 | 6.00 | 0.3 | 15 |
| 4 | 1.7 | 9.8 | 10 | 15 | 1.5 | 66.64 | 76.64 | 0.333 | 10.00 | 9 | 0.3 | 7.66 | 0.325 | 16 |
| 5 | 1.7 | 9.8 | 10 | 15 | 1.5 | 83.30 | 93.30 | 0.333 | 10.00 | 9 | 0.3 | 9.33 | 0.325 | 16 |
| 6 | 1.7 | 9.8 | 10 | 15 | 1.5 | 99.96 | 109.96 | 0.333 | 10.00 | 9 | 0.3 | 11.00 | 0.325 | 16 |

## Appendix IX: Photographs of Road Embankments Failures in Bangladesh

**Figure 1: Slope failure of Kaliganj – Ghorashal Road.**

**Figure 2: Slope failure and differential settlement of Tongi-Ashulia Road**


**Figure 3: Slope-failure due to cutting at the toe of embankment of Dhaka bypass**


**Figure 4 Settlement and slope failure of Kaliganj-Ghorashal Road**

## Appendix-X: Example of Traffic Load induced Settlement Prediction by Chai and Miura's (2002) Empirical Method.

Modulus and depth of pavement layers are assumed as in the following table.

| Axle load data | unit | |
|---|---|---|
| $a$ | m | 0.15 |
| $\sigma_o$ | kPa | 552 |

| Layer description | Layers | E (kPa) | ν | Thickness (m) |
|---|---|---|---|---|
| Bituminous layer | 1 | 2000000 | 0.35 | 0.15 |
| Granular layer | 2 | 800000 | 0.35 | 0.60 |
| Subgrade soil + Fill | 3 | 30000 | 0.35 | 2.00 |
| Foundation soil | 4 | 25000 | 0.35 | 2.50 |
| | | | Total | 5.25 |

### Equivalent Thickness by MET and Stress by Boussinesq Equation

| *f* | he,2 | he,3 | he,4 | z | $\sigma_z$ |
|---|---|---|---|---|---|
| 0.769 | 0.20 | 1.85 | 3.60 | 6.10 | 0.50 |

| | |
|---|---|
| $\gamma$ | 20.00 |
| $h$ | 3.05 |
| $\alpha$ | 8.00 |
| $b$ | 0.18 |
| $m$ | 2.00 |
| $n$ | 1.00 |

**Calculation of Traffic Induced Settlement**

| $Z$ | $S_u$ | $C_c$ | $q_d$ | $q_f$ | $q_s$ | $N$ | $a$ | $\varepsilon_p$ | $S_t$ (layers) | $S_t$ (mm) |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.75 | 30 | 0.3 | 5.40 | 60 | 7.5 | 1E+06 | 2.4 | 0.262838 | 286.99 | 351.53 |
| 2.75 | 30 | 0.3 | 1.44 | 60 | 27.5 | 1E+06 | 2.4 | 0.024149 | 17.13 | 64.54 |
| 3.75 | 30 | 0.3 | 0.88 | 60 | 37.5 | 1E+06 | 2.4 | 0.010102 | 19.68 | 47.42 |
| 4.75 | 15 | 0.3 | 0.59 | 30 | 47.5 | 1E+06 | 2.4 | 0.029255 | 19.92 | 27.74 |
| 5.75 | 18 | 0.3 | 0.43 | 36 | 57.5 | 1E+06 | 2.4 | 0.010587 | 7.82 | 7.82 |
| 6.75 | 20 | 0.3 | 0.32 | 40 | 67.5 | 1E+06 | 2.4 | 0.005047 | 0.00 | 0.00 |

## Appendix XI: Example of Slope Stability Analysis by Bishop and Morgenstern

| ***Parameters*** | ***Value*** |
|---|---|
| H | 10.0 m |
| Φ´ | $27.5^0$ |
| c´ | 13.0 kN/m$^2$ |
| γ 20.0 kN/m$^3$ | |
| B | 2:1 |
| $r_u = \sum \frac{u_n}{\gamma h_n}$ | 0.25 (weighted average value) |

Solution:

For a given slope 2:1, φ´ = $27.5^0$, and $r_u = 0.25$, we find

$$\frac{c'}{\gamma H} = \frac{13}{(20)(10.0)} = 0.05$$

From table we can find the value m´ and n´ and complete the calculation as the following table.

| D | m´ | n´ | $F = m' - n'r_u$ |
|---|---|---|---|
| 1.00 | **1.753** | **1.48** | 1.383 |
| 1.25 | **1.988** | **1.769** | 1.545 |
| **1.50** | 2.350 | 2.117 | 1.82 |

So, minimum factor of safety found as 1.383 ≈ 1.40.

## Appendix XII: Probabilistic Stability Analysis

To take into account this soil variability, literature was investigated to find out such reliability factor. Reliability index ($\beta$) is one of those factors which described in the following sections. To understand clearly the concept of reliability index, basic concepts of probability analysis relating to it are described as follows.

**Basic Concept**

**Mean:** The mean value is the expected value for an input parameter. The expected value, *E{x}* can be defined as

$$E\{x\} = \frac{\sum x_i}{n} \qquad \text{A1}$$

**Variance:** Variance is a measure of how the data are dispersed from its mean or average value. If variance is high means the value of data are more scattered, if variance is low means the data values are closer to each other. The variance of the parameters, *V{x}* is given by

$$V\{x\} = \frac{\sum (x_i - E\{x\})^2}{n-1} \qquad \text{A2}$$

**Standard deviation:** Standard deviation is also a measure of dispersion of parameters from the mean of all the available value for a particular parameter. The standard deviation of a parameter is given by

$$\sigma\{x\} = \sqrt{V\{x\}} \qquad \text{A3}$$

**Coefficient of variation**: The coefficient of variation is the ratio of standard deviation and mean.

$$Cov\{x\} = \frac{\sigma\{x\}}{E\{x\}} \qquad \text{A4}$$

**Covariance:** The covariance of two variables C{x, y} is given by

$$C\{x,y\} = \frac{\sum (x - E\{x\}).(y - E\{y\})}{n-1}$$ A5

The covariance of a single variable, $C_x\{r\}$ is given

$$C_x\{r\} = \frac{\sum (x_i - E\{x\})(x_{i+r} - E\{x\})}{n-1}$$ A6

Autocorrelation coefficient: The autocorrelation coefficient, $R_x\{r\}$ can be defined as

$$R_x\{r\} = \frac{C_x\{r\}}{V\{x\}}$$ A7

**Reliability index:** The reliability index is used to define the uncertainty in stability of embankment in the probabilistic approach of stability analysis (Christian, 1994) and it can be expressed as

$$\beta = \frac{E\{F\} - 1.0}{\sigma\{F\}}$$ A8

OR

$$\beta = \frac{E\{F\} - 1.0}{E\{F\}.Cov\{F\}}$$ A9

Where

*E{F}* = Mean value of computed factor of safety and is determined from the mean value of soil parameters

*F* = Computed factor of safety determined on the basis of mean value of all parameters by using a suitable method.

$$\sigma\{F\} = \sqrt{V\{F\}}$$ A10

The value of *V{F}* can be determined from the derivative of predictive model for factor of safety and it can be given by

$$V\{F\} = \sum_{i=1}^{k} \left( \frac{\Delta F}{\Delta x_i} \right)^2 V(x_i) + V(e))$$ A11

$$\frac{\Delta F}{\Delta X_i} = \frac{F^+ - F^-}{2m\sigma\{x_i\}} \qquad \text{A12}$$

(Bhattacharya et al, 2003)

Where, $\Delta F$ = Change in factor of safety due to change in parameter $x_i$, $F+$ and $F-$ = values of factor of safety due to increment and decrease of value of parameter by $m\sigma\{x\}$ respectively, $m$ = is an arbitrary constant, $V(x)$ = variance of soil parameter, $V(e)$ = variance due to model error and it is not related to the uncertainties of the model parameters.

The reliability index can also be expressed by *Hasofer-Lind index* (Xu and Low, 2006) as following relation

$$\beta = \min_{X=F} \sqrt{\left(\frac{x_i - m_i}{\sigma_i}\right)^T (R)^{-1} \left(\frac{x_i - m_i}{\sigma_i}\right)} \qquad \text{A13}$$

Where, $(R)^{-1}$ = inverse of correlation matrix, $\sigma_i$ = standard deviation of variable $x_i$, $T$ = stands for transpose of matrix, $x_i$ and $m_i$ = any value and mean value of $i$ soil parameter respectively, $\sigma_i$ = standard deviation of each parameters.

The correlation matrix of parameter $x_1, x_2, x_3, x_4$ is given by

$$R = \begin{bmatrix} x_{11} & x_{12} & x_{13} & x_{14} \\ x_{21} & x_{22} & x_{23} & x_{24} \\ x_{31} & x_{32} & x_{33} & x_{34} \\ x_{41} & x_{42} & x_{43} & x_{44} \end{bmatrix} \qquad \text{A14}$$

Determination of the reliability by the above method needs to determine the performance function of slope stability and relating it to reliability index. The value of reliability index will be selected for performance function $G(X) = 0$ where $G(X) = F(X) - 1$ and $F(X)$ is the factory of safety function.

There is a common approach to assume that the factor of safety ($F$) is log-normally distributed and the reliability index is given by the following relation (US Army, cited in Bhattacharya et al., 2003).

$$\beta = \frac{E[\ln\{F\}]}{\sigma_{\ln F}} \qquad \text{A15}$$

where

$$E[\ln\{F\}] = \ln(E\{F\}) - \frac{\sigma_{\ln F}^2}{2} \qquad \text{A16}$$

and

$$\sigma_{\ln F} = \sqrt{\ln\left(1 + \left(\frac{\sigma_F}{E\{F\}}\right)^2\right)} \qquad \text{A17}$$

The lognormal reliability index has also been given by Wolf and Wang (1992, cited in Liang et al., 1998) as follows.

$$\beta = \frac{\ln\left(F_{av} / \sqrt{1 + COV_F^2}\right)}{\sqrt{\ln\left(1 + COV_F^2\right)}} \qquad \text{A18}$$

Where $COV_F$ is the covariance of factor of safety which is given as

$$COV_F = \frac{\sigma_F}{F_{av}} \qquad \text{A19}$$

Where $F_{av}$ is the factor of safety found for mean value of all parameters and $\sigma_F$ can be found by numerical method of Taylor series expansion (Wolf, 1994; US Army, 1998; cited in Duncan and Wright, 2005)

$$\sigma_F = \sqrt{\left(\frac{\Delta F_1}{2}\right)^2 + \left(\frac{\Delta F_2}{2}\right)^2 + \ldots\ldots\ldots\ldots\left(\frac{\Delta F_n}{2}\right)^2} \qquad \text{A20}$$

Where

$$\Delta F_1 = F_1^+ - F_1^-$$

Where $F_1^+$ = Factor of safety for value $\overline{X} + \sigma$ of parameter 1, $F_1^-$ = Factor of safety for value $\overline{X} - \sigma$ of parameter 1.

**Probability of Failure**

The probability of failure can be defined as

$$P_f = 1 - \Phi(\beta) \qquad \text{A21}$$

Where $\beta$ is the reliability index and $\Phi(\beta)$ is the cumulative distribution function (CDF) of standard normal variable ($\beta$). Standard normal cumulative distribution function can be defined as

$$f(Z) = \frac{1}{\sqrt{2\pi}} e^{-\frac{z^2}{2}} \qquad \text{A22}$$

which is a standardized form of the normal cumulative distribution function found by putting mean ($\mu$) and standard deviation $\sigma$ equal to 0 (zero) and 1 (one) respectively. The value of CDF can also be found directly from Excel 2007 spreadsheet by inserting the function NORMSDIST.

**Methods of analysis**

The reliability index and probability of failure of stability of an embankment slope can be determined by the following procedure.

1. Estimate the standard deviation of the variables (parameters) involved in the stability analysis. The variables are $s_u$, $\gamma$, u, water level outside the slope.
2. Calculate factor of safety using mean, mean ± $\sigma$ for each variable keeping other variables in mean values.
3. Using numerical method of Taylor series expansion calculate the standard deviation of factor of safety by Equation A20 and determine the covariance of factor of safety by the following formula.

   $$COV_F = \frac{\sigma_F}{F_{av}}$$

4. Calculate lognormal reliability index ($\beta$) with the Equation A20
5. Find probability of failure by the standard normal distribution function stated in Equation A22. The reliability ($R$) can be directly determined from the Excel 2007

spread sheet statistical function NORMSDIST and then failure probability can be found by the following equation.

$$P_f = 1 - R$$

The probability of failure can also be found directly from the following graph (Figure A1).

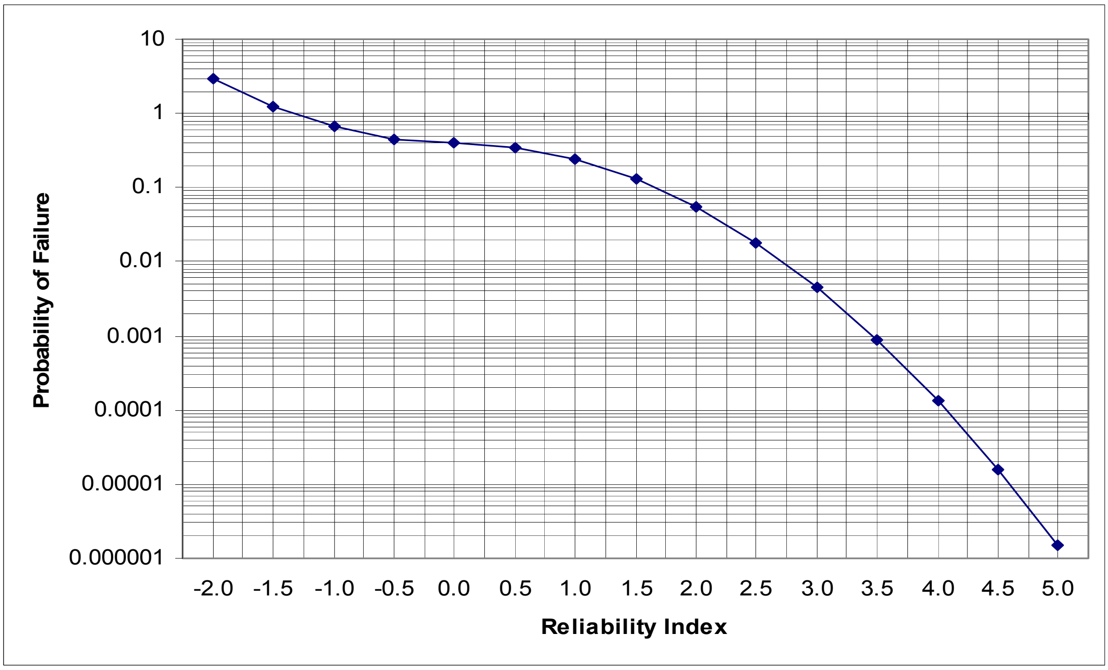



Figure A.1 Probability of Failure with Reliability Index

## Example of reliability analysis for stability of embankment in various methods

An example of reliability analysis with detail calculation has been shown below and the reliability index and probability of failure found in different methods are given in Table A.1.

Soil parameters obtained from Tongi-Ashulia (N302) road are used for the reliability analysis. Three sets of data were available for three parameters; shear strength parameters (c´, φ') and unit weight soil. Reliability analysis has been carried out based on several methods and the results are given as follows.

**Method: Wolf and Wang (1992)**

| Soil parameters | *Fav* | *F+* | *F-* | *ΔF* | *(ΔF /2)2* | $\sigma_F$ | $COV_F$ | *β* | *φ(β)* | *Pf* |
|---|---|---|---|---|---|---|---|---|---|---|
| Unit weight, *γ* | 0.815 | 0.806 | 0.827 | -0.021 | 0.0001 | 0.14 | 0.17 | -1.32 | 0.094 | 0.906 |
| Cohesion, *C′* | 0.815 | 0.915 | 0.735 | 0.180 | 0.0081 | | | | | |
| Frictional angle, *φ′* | 0.815 | 0.919 | 0.716 | 0.203 | 0.0103 | | | | | |

**Method: US Army**

| Soil parameters | *Fav* | *F+* | *F-* | *ΔF* | *(ΔF /2)2* | $\sigma_F$ | *σlnF* | *E{ln{F}]* | *β* | *φ(β)* | *Pf* |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Unit weight, *γ* | 0.815 | 0.806 | 0.827 | -0.021 | 0.0001 | 0.14 | 0.17 | -0.22 | -<br>1..32 | 0.094 | 0.906 |
| Cohesion, *C′* | 0.815 | 0.915 | 0.735 | 0.180 | 0.0081 | | | | | | |
| Frictional angle, *φ′* | 0.815 | 0.919 | 0.716 | 0.203 | 0.0103 | | | | | | |

**Method: Christian (1994)**

| Soil parameters | $F_{av}$ | $F^+$ | $F^-$ | *ΔF* | $(\Delta F/2)^2$ | $\sigma_F$ | $COV_F$ | *β* | *φ(β)* | $P_f$ |
|---|---|---|---|---|---|---|---|---|---|---|
| Unit weight, γ | 0.815 | 0.806 | 0.827 | -0.021 | 0.0001 | 0.14 | 0.17 | -1.36 | 0.09 | 0.913 |
| Cohesion, C' | 0.815 | 0.915 | 0.735 | 0.180 | 0.0081 | | | | | |
| Frictional angle, φ' | 0.815 | 0.919 | 0.716 | 0.203 | 0.0103 | | | | | |

Table A.1 Reliability index and probability of failure in different methods

| Methods | Reliability index | Probability of failure (%) |
|---|---|---|
| Wolf and Wang, 1992 | -1.32 | 90.6 |
| US Army | -1.32 | 90.6 |
| Christian, 1994 | -1.36 | 91.3 |

**Appendix XIII: Example for the Selection of Reinforcement and Slope for overall Stability of Embankment Following Kanairaj (1988) Method**.

Input parameters

| | |
|---|---|
| **Embankment** | |
| *H (m)* | 4.0 |
| $\gamma_f$ *(kN/m3)* | 16.0 |
| $\varphi'$ *(radian)* | 0.524 (30 degree) |
| *b (m)* | 12.2 |
| ***Foundation*** | |
| $H_s$ *(m)* | 5.5 |
| $\gamma_s$ *(kN/m3)* | 15.5 |
| $S_u$ *(kPa)* | 14.0 |
| ***Reinforcement details*** | |
| *X* | 1 (placed at the interface of embankment and foundation) |

Steps to be followed to determine reinforcement for overall stability are as follows:

Step 1: Find the minimum value of n ($n_1$) for bearing capacity

$n_1$ = -0.88

Step 2: Find minimum value of n ($n_2$) to protect sliding

$n_2$ = 2

Step 3: Find minimum value of n ($n_3$) to prevent squeezing out of foundation soil

$n_3$ = 0.393

Step 4: Find maximum value of n among step 1 to 3

*n* = 2

Step 5: Analyse slope stability by Bishop's method (OASYSYS SLOPE)

*n* = 2 with no reinforcement for factor of safety 1.2

Step 6: Maximum reinforcement force required, $P_R$

$P_R > 64.68$ kN/m

**Results:**

Slope 1:2 and a 64.68 kN/m reinforcement force are required to get over all stability against sliding and squeezing failure although no reinforcement is required to achieve factor of safety 1.2 against slope stability.

## Appendix XIV: Charts for Settlements

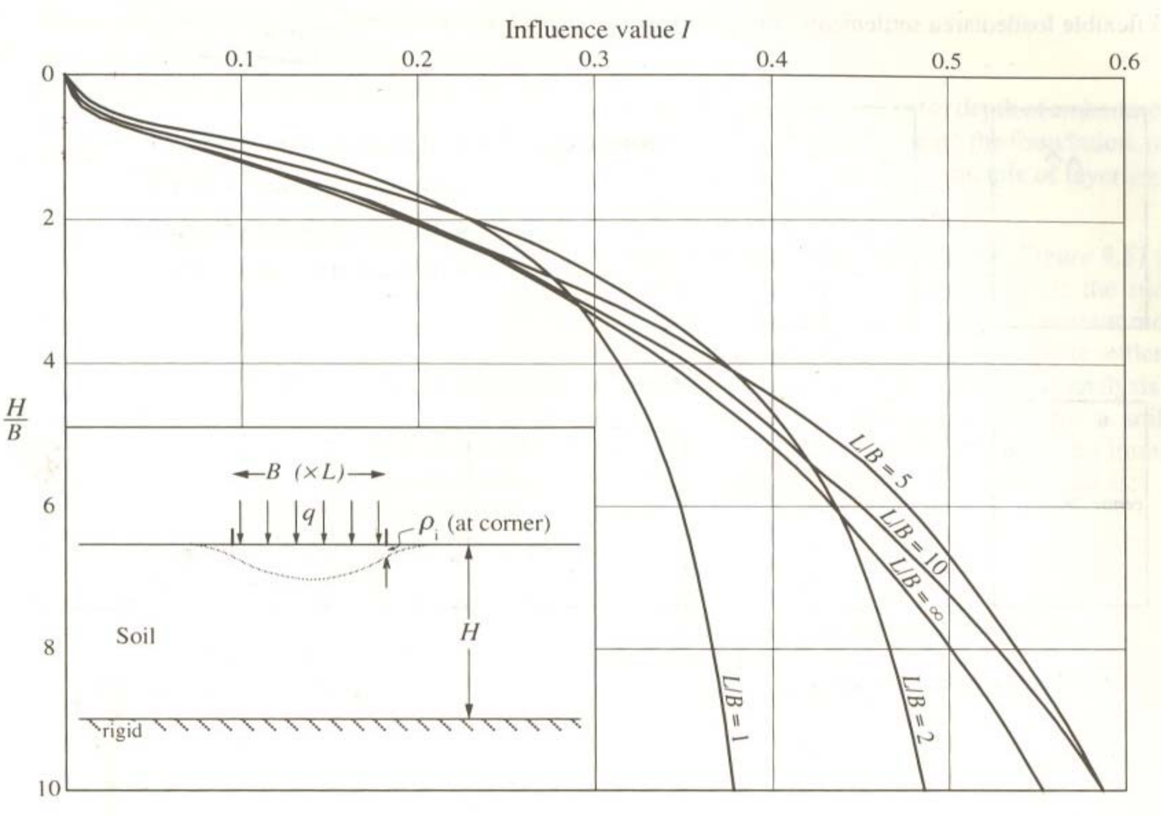



**Figure.1 Influence values for immediate settlement (after Ueshita and Meyerhof, 1968)**

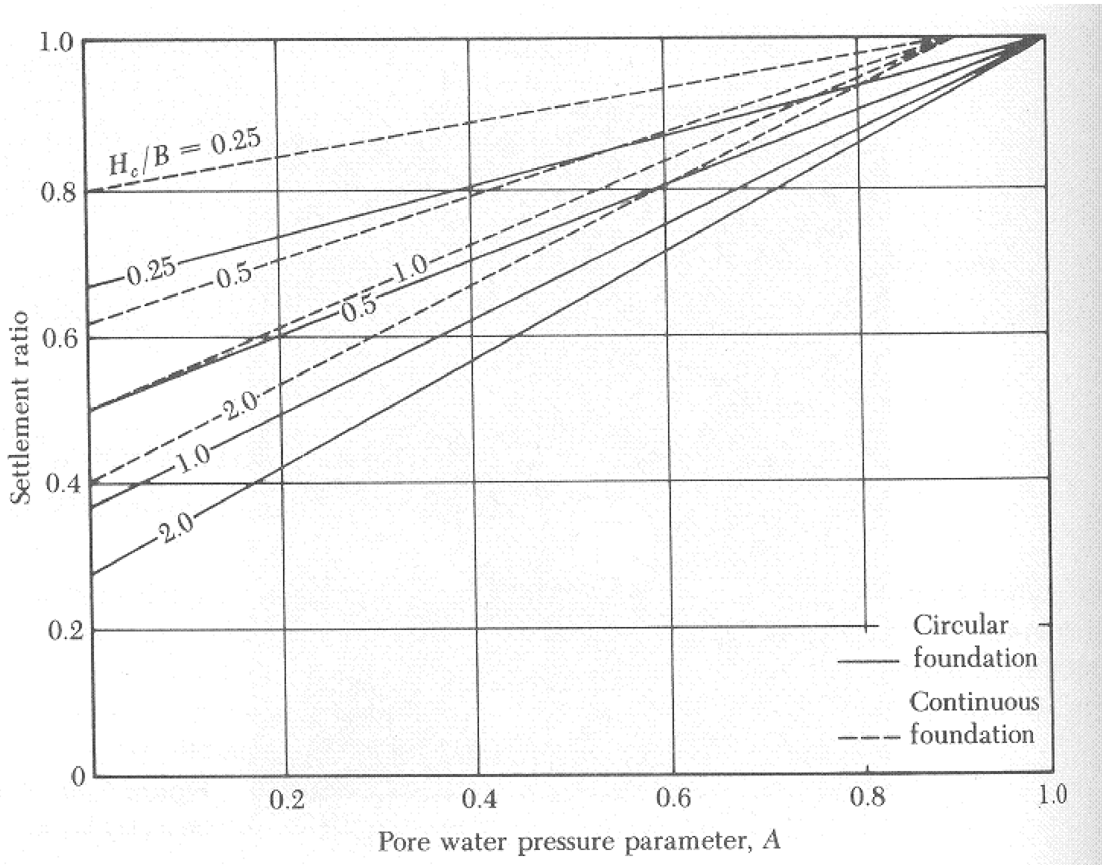



**Figure.2 Settlement Ratios for Circular and Continuous foundations (after Skempton-Bjerrum, 1957)**

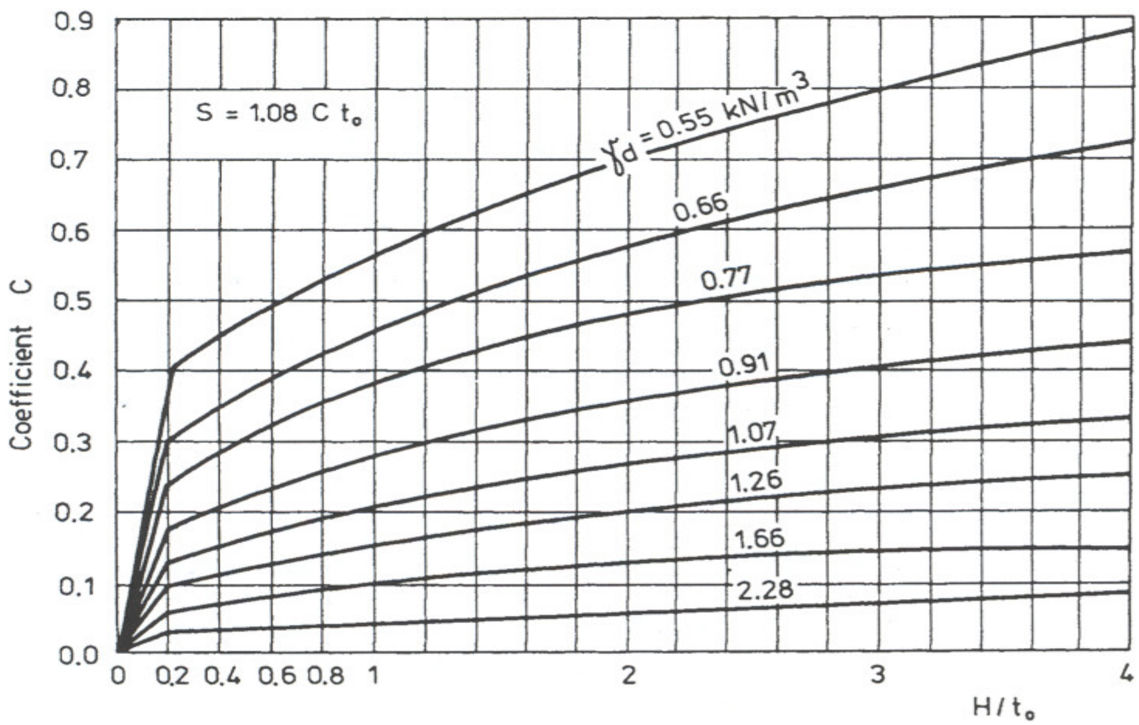



**Figure.3 The coefficient C for (after Ostromecki, 1956, cited in Szymanski et al, 1996)**

## Appendix XV: Solution of Olson's Equation for Degree of Drainage in Ramp Loading

$$\sum_{m=0}^{m=\infty} \frac{1}{M^{4}}\left\{\exp\left(M^{2}T_{c}\right)-1\right\}\exp(-M^{2}T)$$

| $H_{dr}$ | $t_c$ | $T_c$ | $t$ | $T_v$ | m | 0.000 | 1 | 2 | 3 | 4 | 5 | So | S1 | S2 | S3 | S4 | S5 | Sum (So..S5) | Uvi |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 6 | 40 | 0.023 | 50 | 0.03 | M | 1.57 | 4.71 | 7.85 | 11.00 | 14.14 | 17.28 | 0.009023 | 0.000719 | 0.000140 | 0.000032 | 0.000008 | 0.000002 | 0.009923 | 0.146 |
| 6 | 100 | 0.058 | 100 | 0.06 | M | 1.57 | 4.71 | 7.85 | 11.00 | 14.14 | 17.28 | 0.021936 | 0.001470 | 0.000256 | 0.000068 | 0.000025 | 0.000011 | 0.023766 | 0.182 |
| 6 | 125 | 0.073 | 125 | 0.07 | M | 1.57 | 4.71 | 7.85 | 11.00 | 14.14 | 17.28 | 0.026946 | 0.001624 | 0.000260 | 0.000068 | 0.000025 | 0.000011 | 0.028934 | 0.203 |
| 6 | 150 | 0.087 | 150 | 0.09 | M | 1.57 | 4.71 | 7.85 | 11.00 | 14.14 | 17.28 | 0.031780 | 0.001735 | 0.000262 | 0.000068 | 0.000025 | 0.000011 | 0.033881 | 0.222 |
| 6 | 200 | 0.116 | 200 | 0.12 | M | 1.57 | 4.71 | 7.85 | 11.00 | 14.14 | 17.28 | 0.040943 | 0.001874 | 0.000263 | 0.000068 | 0.000025 | 0.000011 | 0.043184 | 0.257 |
| 6 | 250 | 0.145 | 250 | 0.15 | M | 1.57 | 4.71 | 7.85 | 11.00 | 14.14 | 17.28 | 0.049472 | 0.001947 | 0.000263 | 0.000068 | 0.000025 | 0.000011 | 0.051787 | 0.287 |
| 6 | 300 | 0.174 | 300 | 0.17 | M | 1.57 | 4.71 | 7.85 | 11.00 | 14.14 | 17.28 | 0.057411 | 0.001986 | 0.000263 | 0.000068 | 0.000025 | 0.000011 | 0.059764 | 0.314 |
| 6 | 300 | 0.174 | 350 | 0.20 | M | 1.57 | 4.71 | 7.85 | 11.00 | 14.14 | 17.28 | 0.053440 | 0.001042 | 0.000044 | 0.000002 | 0.000000 | 0.000000 | 0.054528 | 0.374 |
| 6 | 300 | 0.174 | 400 | 0.23 | M | 1.57 | 4.71 | 7.85 | 11.00 | 14.14 | 17.28 | 0.049744 | 0.000546 | 0.000007 | 0.000000 | 0.000000 | 0.000000 | 0.050298 | 0.423 |
| 6 | 300 | 0.174 | 450 | 0.26 | M | 1.57 | 4.71 | 7.85 | 11.00 | 14.14 | 17.28 | 0.046303 | 0.000287 | 0.000001 | 0.000000 | 0.000000 | 0.000000 | 0.046591 | 0.465 |

## Appendix XVI: Spreadsheet set up for Osterberg's Influence Factor and Stress Increase for Embankment Loading

height of embankment, h = 3.5 Unit weight of soil = 16.5

| Z | B1 | B2 | $\alpha_1$ | $\alpha_2$ | I | $q_o$ | $\Delta\sigma_{av} = 2Iq_0$ |
|---|---|---|---|---|---|---|---|
| 0.0 | 6.1 | 7.00 | | | | | |
| 1.0 | 6.1 | 7.00 | 0.086 | 1.408 | 0.500 | 57.750 | 57.714 |
| 2.0 | 6.1 | 7.00 | 0.165 | 1.254 | 0.498 | 57.750 | 57.477 |
| 3.0 | 6.1 | 7.00 | 0.232 | 1.114 | 0.493 | 57.750 | 56.904 |
| 4.0 | 6.1 | 7.00 | 0.284 | 0.990 | 0.484 | 57.750 | 55.955 |
| 5.0 | 6.1 | 7.00 | 0.322 | 0.884 | 0.473 | 57.750 | 54.662 |
| 6.0 | 6.1 | 7.00 | 0.348 | 0.794 | 0.460 | 57.750 | 53.097 |
| 7.0 | 6.1 | 7.00 | 0.363 | 0.717 | 0.445 | 57.750 | 51.345 |
| 8.0 | 6.1 | 7.00 | 0.371 | 0.651 | 0.428 | 57.750 | 49.483 |
| 9.0 | 6.1 | 7.00 | 0.373 | 0.596 | 0.412 | 57.750 | 47.575 |