STABILITY OF EMBANKMENT SLOPES SUBJECTED TO SEEPAGE

BY

SHAIKH ABUL KALAM

A thesis submitted to the Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, in partial fulfilment of the requirements for the degree

of

MASTER OF SCIENCE IN CIVIL ENGINEERING

DECEMBER, 2001

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SHAIKH ABUL KALAM

Approved as to style and content by:

(Dr. Tahmeed M. Al-Hussaini)
Associate Professor,
Dept. of Civil Engineering,
BUET, Dhaka.

Chairman
(Supervisor)

(Dr. Md. Abdur Rouf)
Professor and Head,
Dept. of Civil Engineering,
BUET, Dhaka.

Member

(Dr. Syed Fakhrul Ameen)
Professor,
Dept. of Civil Engineering,
BUET, Dhaka.

Member

(Dr. Md. Keramat Ali Molla)
Director and Professor,
BIT, Rajshahi..

Member
(External)

DECEMBER, 2001

DECLARATION

I hereby declare that the research work reported in this thesis has been performed by me and this work has not been submitted elsewhere for any other purpose, except for publications.

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DECEMBER, 2001

(Shaikh Abul Kalam)

ACKNOWLEDGEMENT

The author wishes to express his sincere and profound deepest gratitude to Dr. Tahmeed M. Al-Hussaini, Associate Professor, Department of Civil Engineering, BUET, for his guidance, invaluable suggestions, and affectionate encouragement at every stage of this study.

The author is grateful to Dr. Md. Abdur Rouf, Professor and Head, Department of Civil Engineering, BUET, for his co-operation and valuable suggestions.

Thanks are also due to Dr. Syed Fakhrul Ameen, Professor, Department of Civil Engineering, BUET for his valuable remarks.

The author is grateful to Dr. Md. Keramat Ali Molla, Professor and Director, BIT, Rajshahi for acting as external member of the thesis committee.

The author would like to express his gratitude and appreciation to all who have helped in so many different ways during this investigation.

ABSTRACT

Bangladesh is badly affected by floods every year due to the sudden onrush of rain waters on its extensive network of rivers during the monsoons, most of it coming from neighbouring countries. Many highway and flood control embankments of the country are subjected to high water tables on one-side. The resulting seepage through the embankment may result in significant reduction in the stability of the embankment slope on the country side. These embankments are generally constructed with locally available cohesive soil. Due to improper compaction and lack of quality control in embankment construction practice in Bangladesh, many of these embankments are likely to have low shear strengths. Moreover, steep slopes are not uncommon. It is therefore of concern if the slopes of these embankments will be stable under adverse conditions of floods and earthquakes.

An extensive numerical analysis is carried out on the slope stability of earthen embankments subjected to high flood level on one side using the computer program PC-STABL. The program calculates the factor of safety against the instability of a slope by the method of slices, based on two-dimensional limiting equilibrium. Simplified Janbu's method with correction factor and Simplified Bishop's method have been used to obtain the minimum factor of safety on circular slip surfaces, commonly observed during slope failures in cohesive soils. The embankment is assumed to be homogeneous, isotropic and without any drainage filters which is quite compatible with common practice of embankment construction in Bangladesh. The shape of the phreatic surface is assumed to be that given Casagrande's method. Effective shear strength parameters (c', φ') are considered for the stability analysis. The influence of various parameters such as embankment height, slope angle, soil strength parameters, unit weight on the slope stability for conditions of high flood level is studied. Embankment height is varied from 3m to 12m. Slope varies from 1H:1V to 3H:1V. The range considered for shear strength parameter c' is 10 kPa to 30 kPa while that for φ' is 10^0 to 40^0. Extreme condition of flood related seepage is studied by considering the water level on the river side to be 0.5m below the crest level. Some slope stability analyses are also done for embankment on softer foundation.

Pseudo-static limiting equilibrium analysis is performed to study the effect of horizontal ground motion generated by earthquakes on an embankment already affected by seepage. The cases of 0.15g and 0.25g peak ground acceleration are considered.

Finally, design charts have been developed, similar to those of Bishop and Morgenstern (1960). These charts are based on dimensionless parameters $c'/\gamma H$, slope and φ'. They give the minimum factor of safety for the conditions of extreme seepage, no seepage and seepage with earthquake. As further design aids, minimum slope required for different embankment heights and soil properties are presented for several cases. These charts are expected to be useful for rapid preliminary design or stability reassessment of flood control embankments in Bangladesh.

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NOTATION

c'	effective cohesion intercept
φ'	effective angle of internal friction
σ	normal stress.
θ	inclinations of the resultant inter-slice forces
φ'_m	mobilized friction angle
α	slope angle
$\lambda_{c\theta}$	Cousins' dimensionless number
γ_{sat}	saturated unit weight
γ_{moist}	moist unit weight
a_L, a_R	perpendicular distance from the resultant external water force to the centre of rotation.
A_L, AR_L	resultant external water forces
A_a	area of the slice 'a'
AW	horizontal inertia force
b_n	width of the nth slice.
D	depth factor
E	horizontal inter-slice normal forces.
F	Fetch in km.
F_B	freeboard
F_f	factor of safety from horizontal force equilibrium equation
F_m	factor of safety with respect to moment
F_s	factor of safety
H_c	critical height of slope
h_w	wave height in metre
l	length of failure surface at the base of each slice
ΔL_n	length of slip surface at base on nth slice
N_s	Taylor's stability factor
m	factor of safety with respect to total stresses
n	stability coefficients representing effect of pore pressure
N_F	the stability number

N_r	normal component of reaction
P_n and P_{n+1}	normal forces on sides of nth slice
R	radius or the moment arm associated with S_m
r_u	pore pressure ratio
r_{ua}	average value of r_u in slice 'a'
T_n and T_{n+1}.	shearing forces on sides of nth slice
T_r	tangential component of reaction
u	pore water pressure
V	maximum wind velocity in km per hour
W_n	weight of nth slice
x	horizontal distance from the centroid of each slice to the centre of rotation
X	vertical inter-slice shear forces.
z	depth of the point in the soil mass below the soil surface

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Earthen embankments are the most ancient type of embankments, as they can be built with natural materials with a minimum of processing and with primitive equipment. Embankments are constructed for many different purposes including highways, railroads, dams, levees and flood control. In Bangladesh, a land with half of the area situated only about 20 ft above mean sea level, embankments have an important role to play. Construction of earthen embankment is an established practice in Bangladesh for protecting crops, and other properties against flood damages. Very little published information is available on the construction records and performance of flood and road embankments in Bangladesh. There have been several embankment failures and most of these failures occurred at a time when the river water had a high stage flowing very near to the embankment top (Safiullah, 1977, 1988). Several slope failures occurred during the recent 1998 flood. Most of these failures have not been properly studied and analyzed.

Earthen embankments are used to protect the land from high water level and for use as roads. High water level on one side (river side) of an embankment causes seepage flow through the embankment which may intersect the slope on the other side (country side). The movement of water from a high to a lower elevation is a natural occurrence, therefore, seepage of water is to be expected through the earthen embankment. The seepage of water can appreciably affect the stability of a slope by affecting inter-granular pressures and also by piping action. Properly designed and compacted embankments with drainage filters are necessary. However, the construction practice in Bangladesh is still primitive and proper compaction may not be achieved. Moreover drainage filters may not be present. In many cases, proper slope is not maintained. As a result poorly compacted embankments with low shear strength, steep slopes and no drainage filters is a virtual reality in Bangladesh. These embankments may have a low factor of safety under the action of seepage. Bangladesh being a highly flood prone country, this situation is quite common for the many embankments of Bangladesh during the monsoons. Consequences of failure of earthen embankment can be disastrous leading to flooding of new lands. Also breaches i.e., imminent failure of important embankments may require large costs for preventing outright failure of embankment.

Bangladesh is located in a region of significant seismic activity. Several earthquakes of large magnitude (Richter magnitude 7.0 or higher) with epicentres within Bangladesh and India close to Indo-Bangladesh border have affected Bangladesh. Earthquake may cause significant failures and movements of natural slopes, embankments and earth dams. Canal banks in particular have a long history of slope failures during earthquakes. During the 1940 El Centro Earthquake (Wiegel, 1970), the banks of All America Canal failed and bank disruption with associated flooding along a length of the Solfatara Canal occurred.

Development of the subject of soil mechanics, techniques of determination of soil properties and their control during placement as well as rational methods of stability analysis have been developed so that an earth dam is an engineering structure whose safety can be predicted with almost the same degree of accuracy as that for other types of embankment.

A huge sum of money has been spent for embankment construction. Although the embankments are probably the cheapest form of flood protection measure, such construction without proper attention to the properties of the construction materials and method of construction may incur huge extra cost through over conservative design or remedial measures when breaches occur or due to failure. It is hoped that this study will be helpful for rapid assessment of slope stability for the flood control embankments of Bangladesh.

1.2 OBJECTIVES OF THE RESEARCH

The principle objectives of the present study are:

- To conduct a thorough literature survey on available information on slope stability problems related to seepage and earthquakes.
- To study the effect of seepage on the country side slope stability of earthen embankments for a variety of embankment dimensions and soil properties.
- To study the effect of horizontal forces caused by earthquake on slope stability of embankments.
- To prepare design charts/design aid for convenient preliminary design or rapid assessment of flood protection embankments.

1.3 SCOPE OF RESEARCH

Poor compaction and absence of drainage filters in a majority of earthen embankments in Bangladesh make them vulnerable to floods and earthquakes. Earthen embankments, subjected to high flood level on one side, are studied for stability against general slope failure. These embankments are assumed to be homogeneous, isotropic and without drainage filters. The computer program PC-STABL is used to calculate the factor of safety against slope failure by the method of slices based on two-dimensional limiting equilibrium method commonly used in slope stability analysis. The embankment material is assumed to be cohesive, as commonly used in Bangladesh. Effective shear strength parameters are considered. Three cases have been considered for various embankment geometries and soil parameters. These are (a) no seepage (b) seepage and (c) seepage with earthquake. Some analyses have also been performed for the case of embankment on soft soil. Slip-circle analysis is conducted to obtain minimum factor of safety by both simplified Bishop's and simplified Janbu's method (with correction factor) for specified phreatic surface or pore pressure ratio. The phreatic line due to seepage is constructed using Casagrande's method. Pseudo-static limit equilibrium procedure is used to assess the effect of earthquake induced horizontal forces.

1.4 OUTLINE OF THESIS

The results of this study have been divided into several topics and presented in five chapters.

A brief introduction to the general problem of slope stability of earthen embankments constructed in Bangladesh is presented in the first chapter. The major objectives and scope of the work are also outlined in this chapter.

Chapter 2 presents general design principles and construction practice of earthen embankments in Bangladesh. Failure of slopes of such embankments and probable causes are discussed.

Chapter 3 deals elaborately with topics related to the various methods for slope stability analysis. Methods for including the effect of seepage and earthquakes are discussed. Existing design charts for slope stability analysis are briefly presented. The main features of the computer program used is also presented.

Chapter 4 presents the results from an extensive numerical study of slope stability for a wide range of parameters. Finally designs charts are prepared which may be used for rapid assessment of embankment slopes during high flood level and during earthquake induced shaking.

The conclusions of the study and some recommendations for further research are presented in Chapter 5.

CHAPTER 2

EMBANKMENT DESIGN AND CONSTRUCTION IN BANGLADESH

2.1 INTRODUCTION

Embankments built with locally available soil have been built all over Bangladesh for protection against floods and for use as roads. Fig. 2.1 shows the different components of an embankment which is a raised earthen structure with a flat top called the crest and slopes on both sides. The slope is necessary to provide stability of the embankment built. High water table due to flooding on one side of the embankment causes seepage through the embankment. The top flow line of this seepage, also known as the phreatic line is shown in Fig. 2.1. The embankment material is excavated from close by locations known as the borrow pit. The intersection of the slope with the foundation is called toe. Sometimes a raised land called berm is built at the base of the embankment which provides additional slope stability. The side of the embankment facing the river or high flood level is called the river side, while the other side facing the lands to be protected is known as the country side.

This chapter deals, with the design, construction and performance of earthen embankments with particular reference to Bangladeshi practice.

2.2 DESIGN OF EARTHEN EMBANKMENTS

2.2.1 GUIDELINES FOR EMBANKMENT GEOMETRY

Common practice in selecting embankment height, width and side slopes depend on the following considerations:

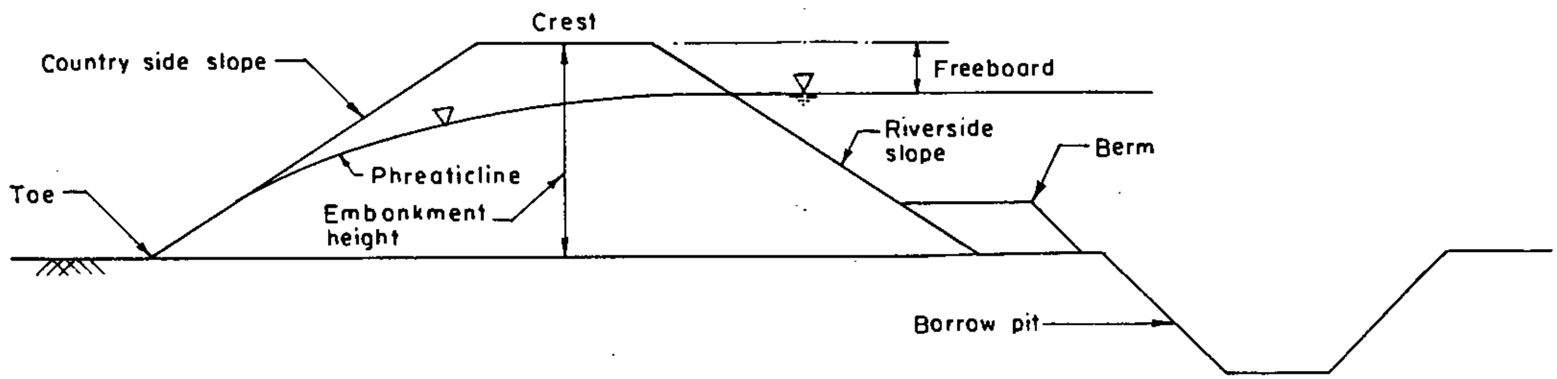


Fig. 2.1 Different component related to embankment.

Height

The height of embankment should be such that the top is above the highest flood level (HFL). Freeboard is the vertical distance between the crest level of the embankment and HFL. The freeboard must meet the requirements for long-time condition. It must be sufficient to prevent seepage through the top portion of core which may be loosened or cracked due to drying action. Freeboard is provided to prevent overtopping of the embankment by wind-induced wave action which may coincide with the occurrence of the high flood. The rational determination of freeboard would require a determination of the wave height. Various empirical formula depending on wind velocity and reservoir fetch have been suggested for computing wave heights. For determination of maximum wave height, Stevension's formula as modified by Molitor to include the effect of wind velocity are normally used (Islam, 1991) which are as follows:

$$h_w = 0.032\sqrt{FV} + 0.763 - 0.271\sqrt[4]{F} \tag{2.1}$$

where F < 32 km

$$h_w = 0.032\sqrt{FV} \tag{2.2}$$

where F> 32 km

h_w= wave height in metres

V = maximum wind velocity in km per hour

F = Fetch in km.

United States Bureau of Reclamation (U.S.B.R.) recommended the freeboard to be 2-3 m over the maximum flood level for any height of embankments when the spillway is free (Lambe, 1951). Table 2.1presents freeboards (Punmia, 1981) recommended by U. S. B. R.

Table 2.1 U. S. B. R. recommended values of freeboard (after Punmia, 1981)

Fetch (km)	Normal freeboard (m)	Minimum freeboard (m)
Less than 1.5	1.25	1.00
1.5	1.50	1.25
4.0	1.80	1.50
8.0	2.50	1.80
15.0	3.00	2.20

For normal conditions (fetch=2 km, wind speed = 150 km/h) in Bangladesh the freeboard is only 0.6 m, when the minimum computed freeboard, coming from a realistic design criteria is 1.40 m (Peck, et al, 1974).

Freeboard is also provided for safety factor against many contingencies such as settlement of the embankment, over rising of the water level as a result of malfunction of controlled sluice gates etc. Bangladesh Water Development Board (BWDB) recommends the freeboard to be 10% of the embankment height as allowance for shrinkage (settlement) of the embankment and another 5% for possible errors during surveys and construction. However it was recommended by BWDB to adopt a total minimum freeboard of 0.9m (3 ft) for the embankment along the Atrai as well as along the Jamuna river (BWDB 1984). The freeboard recommended by BWDB for DFC-III project was one quarter of the water depth plus 0.30 m (1 ft) with a maximum limit of 2.0 m (6.5 ft).

While deciding the height of the embankment, settlement allowance should be taken into consideration seriously as settlement of an embankment may be caused by consolidation in the foundation and in the fill over a period of many years. In some areas of Bangladesh a practice of 20% shrinkage (Islam, 1991) on hand placed embankment are made (i.e. embankment height is built 20% higher than design height). The consolidation settlement however, may be estimated using Terzaghi's equation (Safiullah, 1988). BRTS (1978) suggests settlement allowances based on experience, which are presented in Table 2.2.

Table 2.2 Settlement allowance to be made on embankment height due to consolidation of subsoil (after BRTS, 1978)

Location	Percentage of embankment height
a) Shallow ridges and basins of the flood plains valleys of the uplifted terraces	10
b) Deep basins, beels, peat deposits of flood plains	20
c) High land areas of the uplifted terraces	5
d) Hills	0

From a series of reports and earth work manuals (BWDB, 1969, 1982, 1984) and observations, it is seen that for ordinary embankments a minimum freeboard of 0.8 m (2.6 ft) to 1.7 m (5.6 ft) is normally used in Bangladesh.

Crest width

The crest width adopted previously in small or medium schemes was 3 m (UNDP 1988). The crest width of embankments is usually determined by the use to which they are to be put, with a minimum width of about 3.5-4 m to permit movement of maintenance equipments. The crest width may be determined by the following empirical expressions (Punmia, 1981; Garg, 1987):

$$b = \frac{H}{5} + 3 \tag{2.3}$$

$$b = 0.55\sqrt{H} + 0.2H \tag{2.4}$$

$$b = 1.65\sqrt[3]{(H + 1.5)} \tag{2.5}$$

where b = crest width (m)

H = height of embankment (m)

Equation (2.3) is applicable for very low embankments. Equation (2.4) is applicable for embankments lower than 20 m and Equation (2.5) given by U.S.B.R. is applicable for embankments higher than 30 m. For people's shelter during high flood or in case of embankment failure additional 1-2 m should be added to the crest width calculated by the above formula.

Side slopes

The evaluation of slope stability may be complicated due to the fact that embankment may contain heterogeneous soil due to non-uniform compaction and non-uniformity in borrow material. In many situations the variables that affect the shear strength in the field are only approximately known. Hence, for small projects and for embankments of low height, it may be adequate to rely for slope selection on the available experience for a zone. Although embankments are being constructed in Bangladesh for a considerable time, none such experience is on record (Safiullah, 1988).

The slopes of the embankments vary widely depending on the character of the materials available, foundation conditions and the height of the structure. The slope also depend upon the type of embankments (i.e. homogeneous, zoned embankment type etc.) and on the nature of the construction materials and other geotechnical characteristics. Table 2.3 gives the side slopes for preliminary design of embankments according to Terzaghi and Peck (1967).

Table 2.3 Side slopes for earth embankment (after Terzaghi and Peck, 1967).

SL. No.	Type of material	River side slope (horizontal :vertical)	Country side slope (horizontal: vertical)
1	Homogeneous well graded	2.5:1	2:1
2	Homogeneous coarse silt	3:1	2.5:1
3	Homogeneous silty clay (i) Height less than 15 m (ii) Height more than 15 m	 2.5:1 3:1	 2:1 2.5:1
4	Sand or sand and gravel with a central clay core	3:1	2.5:1
5	Sand or sand and gravel with reinforced concrete diaphragm	2.5:1	2:1

Ministry of Local Government Rural Development and Co-operatives of Bangladesh (LGRD) recommended the side slopes for flood embankments as shown in Table 2.4.

Table 2.4 Side slopes both for river and country sides (after LGRD)

Type of soil	Permissible side slopes (horizontal: vertical)
1) Normal soil (silt or silty clay)	2:1 to 3:1
2) Loose sandy soil	3:1 to 5:1

BWDB (MPO, 1985) recommends side slopes for embankments to be 2:1 on the country side and 3:1 on the river side. These are normally adopted for design in Bangladesh since it appears to provide sufficient safety against slope instabilities (NEDECO, 1984). Riverside slope may vary from 2:1 to as flat as 4:1 for stability because of the relatively poor construction materials (Punmia, 1981). UNDP (1988) recommended slopes for small homogeneous earth fill embankments, without rapid drawdown as a design condition and given soil conditions of Bangladesh to be 3:1 for riverside slope and 2.5:1 for countryside slope.

There are, however, embankments in the country which have slopes much steeper than these recommended values. Islam (1991) based on survey of several flood-control embankments such as Tyebpur-Kashimpur embankment, Savar, Chalan Beel embankment, Naogaon, Teesta embankment, Lalmonirhat, Brahmaputra embankment, Serajgonj reports slopes to be in the range of 1:1 to 1.7:1.

Berm and borrow pits

A shelf of land called berm is left between the bottom edge of the embankment and the top of the borrow pit. To make use of the least valuable land and to encourage siltation in the pits, it is proposed to locate the borrow pits at the river side (MLGRDC). They should be located in such a way that a berm of approximately 3.5 m (10-15 ft.) width is left between the toe of the embankment and the edge of borrow pit. The excavation depth should not exceed 2.0 m (7 ft) (BWDB, 1984). To prevent the development of flow concentration during high river stages cross berms perpendicular to the embankment should be left in the borrow pits every 30 m (100 ft) measured along the embankment (NEDECO, 1984). The borrow pits should be rectangular and the depth of cutting should not exceed 1.2 m (4 ft) on the river side and 0.9 m (3 ft) on the country side. In most areas, soils at greater depths are more moist than required for proper compaction. Besides, deeper borrow pits will increase the cost of excavation. Shallow borrow pits (approximately 0.6 m or 3 ft deep) can be used for cultivation in some places (NEDECO, 1984).

2.2.2 GENERAL CRITERIA FOR DESIGN

An embankment should be so designed that it is safe against overtopping, wave action, seepage effects (piping or sloughing), sliding, damage to slope paving, base displacement, river transgression etc. Based on the experience of failure, as discussed later in Art. 2.5, the following general criteria can be laid down for the safe design of earthen embankments.

Analysis is required to ensure stability of the embankment for the following cases:

- The country side slope should remain stable during steady seepage at design high flood level.
- The river side slope must be stable during rapid drawdown conditions if they prevail.
- The river side slope and country side slope have to be checked for end of construction condition when rapid mechanised construction is carried out, which generates large undissipated pore pressures in the compacted layers. Instability may also arise from the presence of thin pervious seams in clay foundation, which may transmit high consolidation pore pressure generated under the embankment by its load to lightly loaded areas beyond the toe of the embankment and thus cause failure.
- The resistance of the foundation should be sufficient to prevent sliding of the embankment due to lateral forces exerted by high water level, wave action or seismic forces.
- In seismic zones, pore pressure condition due to seepage, rapid construction or rapid draw down may have to be combined with seismic effects. Earthquakes generate horizontal forces within the embankment soil mass thereby lowering the factor of safety against slope failure.
- The foundation shear strength should be sufficient to provide a suitable margin of safety against bearing failure of the embankment.

In addition, the following safety measures should be taken:

- The embankment must not be overtopped during the passage of the design flood. It should have sufficient freeboard for wind induced wave action and allowance for embankment settlement.
- A fill of sufficiently low permeability should be provided out of the available materials, so as to serve the intended purpose with minimum cost. Borrow pit should be as close to dam site as possible, so as to reduce the carrying cost.
- Piping action and sloughing of country side face (see Art. 2.5.2) should be prevented through the use of proper soil, provision of drainage, preventing cracks and openings

in the embankment. There should be no opportunity for free flow of water from river side to country side face. Free flow may occur through internal cracks, along conduits or after erosion caused by leaks from pressure conduits, through layers left loosely compacted, through holes made by aquatic animals or those left by rotten roots of dead trees etc. Once a concentrated leak starts, it is almost impossible to avoid failure. Precautions have to be taken against all these eventualities.

- Water passing through or under the embankment should not be allowed to remove material of the embankment or its foundation. The criterion is meant for protection against piping failures and involves provision of a minimum core thickness in the embankment section and seepage control measures for foundation.
- The riverside slope must be protected against wave action and the crest and country side slope must be protected against erosion by wind and rain. Use of revetment structure for protection of river side slope from severe wave action would require analysis for its design.

2.3 CONSTRUCTION OF EARTHEN EMBANKMENT

2.3.1 Materials for Earthen Embankments

According to Indian standard 8826-1978 the suitability of construction of earthen embankments are shown in Table 2.5.

Table 2.5: Suitability of soils for construction of earthen embankment (after Islam, 1991).

Relative suitability	Homogeneous sections
1. Very suitable	Clayey gravels (GC)
2. Suitable	Gravely clay (GL), clay of intermediate plasticity (CI)
3. Fairly suitable	Poorly graded sand (SP), silty sand (SM), Inorganic clay of high plasticity (CH).

As earthen embankments require very large quantities of materials, it is economical to utilise whatever is available near the site. In general, an embankment can be designed to fulfil its functions satisfactorily with any type of materials available. Thus an embankment section can be designed entirely of highly pervious non-cohesive material like sand and gravel or entirely of impervious cohesive material like silts and clays. Such sections designed of only one type of material are called 'homogeneous sections'. It may be desirable to have two types of materials available, one sandy to provide stability and good drainage and the other clayey to cut off seepage. Sections designed with materials of two or more types are called 'non-homogeneous' sections.

While earthen embankments can be designed with any type of material as stated above, more economical designs will be possible at locations where the materials possess desirable properties. For sandy material, desirable properties are good grading to achieve high compacted density and high angle of internal friction, and also good drainage. For clayey soils, the requirements are moderate plasticity index, high compacted density and shear strength, and low permeability.

2.3. 2 Method of Construction and Maintenance

There are mainly two methods of earth embankment construction followed in most countries: i) Rolled fill method and ii) Hydraulic fill method

In rolled fill method, the embankment is constructed in successive, mechanically compacted layers. The material from borrow pits and that suitable from required excavations is delivered to the embankment site. It is spread by bulldozers after moisture adjustment, if necessary, to form layers of limited thickness having the proper moisture content, which are then thoroughly compacted. Rolled fill construction accounts for practically all dams constructed in recent years.

In hydraulic fill method, construction materials are excavated, transported and placed by hydraulic methods. The materials is washed or pumped from the borrow pits into flumes or sluices extending along the outer edges of the embankment are provided with outlets at intervals along their length and the discharge form these outlets flows inward to a central pool. The coarse material is automatically deposited on the outer edges of the embankment, the finer moves towards the centre, and the finest and most impervious is deposited in the pool to form the central or impervious central core supported by relatively pervious and more stable outer zones grading in particle size from the fine to coarse towards the outer slopes.

After construction, the embankment should be kept free from all traffic for at least one monsoon so that it is properly stabilized. After stabilization, the top and sides of the embankment to the specified height and slopes should be dressed with the help of hoe and tamper, if needed fresh earth should be added and compacted properly. To protect the side slopes against rainfall, erosion and wave attack, turfing of the slopes and the berms (if any) using locally known grass species 'Durba' is recommended. Turfed side slope will also increase the stability of the side slopes.

Regular inspection of the embankment should be performed. At least once a year in the early rainy season, a thorough inspection is required by the responsible engineer. Any damage or defects on the embankment should be repaired immediately. Damage of the embankments includes damage to the turfed surface, cracks in the embankment, erosion due to river action and rainfall, erosion due to seepage, human action etc. Inspection of the embankment during high water stages should not be limited to the river side slope and the crest but also the country side slope, especially the toe of the embankment (internal erosion or piping due to seepage).

2.3.3 Construction Practice in Bangladesh

In Bangladesh borrow materials are usually collected from area close to the embankment on the river side. These materials include silts and clays of low to high plasticity with fine sand depending upon the land system to which these belong. Construction of embankments require the use of a large labour force for excavation, transportation, placement and compaction of the fill. All of these activities are done manually. Much of the embankment stability and seepage conditions depend on the way these acts are performed. In many projects no compaction is specified. In some projects specification for fill is only limited to density that would be attained at the end of construction (usually a percentage of a standard laboratory compaction). Due to poor quality control during field compaction, such specification may not be satisfied. Quality of the soil being excavated can vary from day to day as a result of changes in ground conditions, in soil type or in weather. With heterogeneous soils involving mixture of wet and dry lumps of clay, often the strength of the weaker lumps of clay control the overall behaviour of the samples. The control of quality of the material should logically be made at the excavation location. Allowances have to be made for the effect of weather conditions. Specifications based on insitu (plate bearing and field vane) and undisturbed shear strength tests would be more appropriate at fill locations. While placement of fill in a very wet condition may lead to low strength of the fill, compaction at low water content may create macro pores. Piping may initiate through these macro pores between soil chunks. It is clear from experience in the United States that it is dangerous to construct embankments at water contents much below Standard AASHTO optimum. A small percentage difference in water content can have a large influence on the susceptibility to cracking (Sherard et al, 1963). This emphasises the importance of a knowledge of field moisture content.

Some adverse effects of the way embankments are used by people for cattle grazing and planting trees should be studied. Very often when an embankment fail, indigenous methods such as bamboo piling, gunny bag placing are used for correction. These techniques can be significantly improved by application of the principles of soil mechanics.

In Bangladesh the flood embankments may be divided into two categories:

i) Embankment constructed by Bangladesh Water Development Board (BWDB);

ii) Embankments constructed under Food for Works Programme (FWP) by BWDB, LGRD, Local administration or others.

In the first category, the BWDB has design offices that are capable of designing embankments and office staffs that are capable of their proper construction. In the second category, the embankments are constructed under Food for Works Programme, where proper design and construction procedure are generally ignored during the construction. In Bangladesh, skilled and scientific methods are usually not followed in the construction of flood embankments, dikes etc. Embankments are commonly constructed by basket-head method. In this method construction materials are excavated from borrow pits parallel to the embankment and are carried in a bamboo made basket on head by unskilled labourers to the site of construction. Compaction of embankments are quite unusual in Bangladesh. Protective measures (such as turfing, mattressing, grassing etc.) are generally very rarely undertaken for protection of the embankment surface. Drainage facilities are defective or missing for which in many instances significant pore water pressure or seepage force exists inside the embankments during steady seepage (Safiullah, 1988) conditions. The various aspects of embankment design and construction described above illustrate that although embankment construction in Bangladesh is a very old practice, the fruits of experience are yet to crystalize in a Code of Practice to meet the challenge the country now faces. Such a Code of Practice can be developed through systematic analysis of performance records, failure incidences and continued research.

2.4 ENVIRONMENTAL FORCES AFFECTING EMBANKMENT STABILITY

2.4.1 Flood Situation

During the monsoons, the river network of the country is overloaded with rainwater from vast catchment areas in Bangladesh and neighbouring countries. This results in flooding in vast areas of the country. The flat topography of the country is responsible for flooding in about one third of the land annually with flood water varying in height from a few feet to as much as 30 ft for several months (Safiullah, 1977). The method of flood control by constructing marginal embankments along the bank of the river for checking the flow is now accepted all over the world. Most experts on flood control consider embankments as one of the most practical methods of flood control. Flood protection works in Bangladesh so far consist mostly in constructing marginal embankments along the major rivers and tributaries to protect the land against upland flood discharge and tidal inundation.

2.4.2 Earthquake Scenario

Several earthquakes of large magnitude (Richter magnitude 7.0 or higher) with epicenters within Bangladesh and in India close to Indo-Bangladesh border have occurred (Ali and Choudhury, 1994). Table 2.6 provides a list of these major earthquakes that have affected Bangladesh. Moreover, there are faults within Bangladesh and neighbouring India and Burma that may be sources of earthquakes affecting Bangladesh. Table 2.7 (Ali and Choudhury, 1992) shows the probable magnitudes of operational basis earthquakes and maximum credible earthquakes, along with depth of focus in these fault zones. According to the Bangladesh National Building Code (HBRI, 1993) the country is divided into three zones namely zones 1, 2 and 3, with zone 3 and zone 1 being the most and least severe respectively (Fig. 2.2). The zone coefficients (z) for zones 1, 2, 3 are 0.075, 0.15 and 0.25 respectively which represent the maximum ground acceleration in 'g' (acceleration of gravity). This information clearly signifies that the probability of occurrence of earthquakes of large magnitudes is considerable in this country.

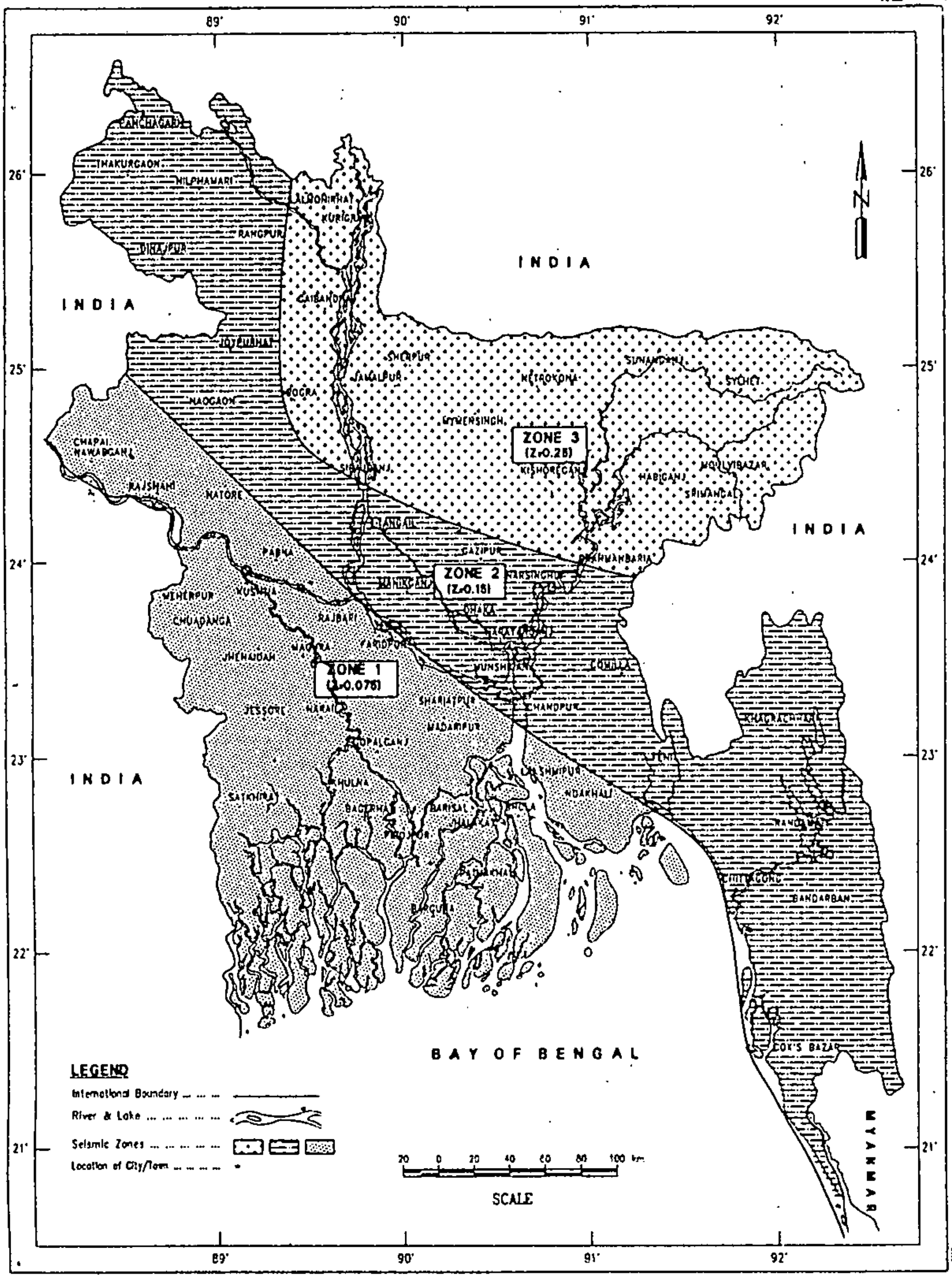


Fig. 2.2 Seismic zoning Map of Bangladesh.

Table 2.6 List of Major Earthquakes Affecting Bangladesh (after Ali and Choudhury, 1994)

Date	Name of Earthquake	Magnitude (Richter)	Epicentral Distance from Dhaka (km)
10 January, 1869	Cachar Earthquake	7.5	250
14 July, 1885	Bengal Earthquake	7.0	170
12 June 1897	Great Indian Earthquake	8.7	230
8 July, 1918	Srimongal Earthquake	7.6	150
3 July, 1930	Dhubri Earthquake	7.1	250
15 January, 1934	Bihar-Nepal Earthquake	8.3	510
15 August, 1950	Assam Earthquake	8.5	780

Table 2.7 Tectonic provinces and their Earthquake Potential (after Ali and Choudhury, 1992)

Location	Operating Basis Magnitude(Richter)	Maximum Credible Magnitude(Richter)	Depth of focus (km)
Assam fault zone	8.0	8.7	0-70
Tripura fault zone	7.0	8.0	0-70
Sub-Dauki fault zone	7.3	7.5	0-70
Bogra fault zone	7.0	7.5	0-70

2.5 FAILURE OF EARTHEN EMBANKMENT

Earthen embankments may fail like other engineering structures due to improper design, faulty construction, lack of maintenance etc. The various modes of failure of embankment slopes are presented in the following sections:

2.5.1 General Slope Failure

When the embankment slopes are too steep for the strength of soil, the entire slope may slide down along a plane surface or curved surface causing an outright failure of the embankment. Fig. 2.3 shows curved surfaces along which slope failure may take place. Excess pore pressures that may be generated in an embankment due to fast construction, seepage action or rapid drawdown cause reduction in effective stress resulting in lowering of the factor of safety against such general slope failure.

The most critical condition of the slide of the riverside slope is the sudden draw-down of the reservoir and country side slope is most likely to slide, when the reservoir is full Pore pressures developed from steady seepage through an embankment due to high flood level reduces the factor of safety against slope instability and may lead to overall slope failure on the country side. Also if rapid draw-down occurs, the river side slope could be susceptible to similar danger due to the pore pressures within the embankment which could not be dissipated so fast and the advance of the high water level on the river side. The river side slope failures seldom lead to catastrophic failures, but the country side slope failures are very serious (Garg, 1983).

2.5.2 Piping Failure

Uncontrolled or concentrated seepage through the dam body or through the foundations may lead to piping or sloughing locally which may lead to subsequent failure of the dam. Piping is the progressive erosion and subsequent removal of the soil grains from within the body of the dam or the foundations of the dam. Sloughing is the progressive removal of soil from the wet downstream face. Seeping water generates a viscous drag which tend to pull the soil particles in its travel through the embankment. If the resisting forces in the soil is less than the drug forces, the soil particles are washed away and piping commences. When the concentrated flow channels get developed in the body of the embankment soil

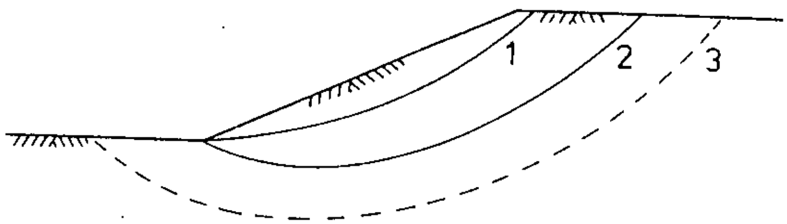


Fig.2.3 General slope failure.

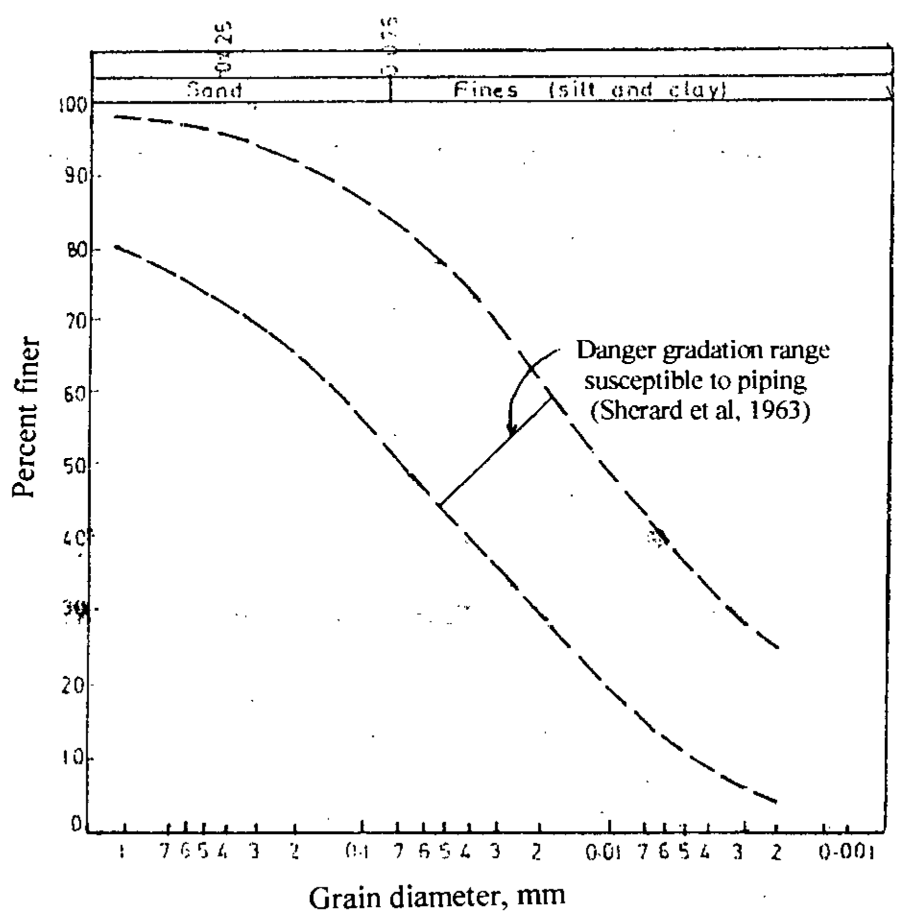


Fig.2.4 Gradation range for soils susceptible to piping (after Sherard et al, 1963)

may be removed leading to the formation of hollows in the embankment body, and subsequent subsiding of the embankment.

High seepage pressure may exist where the phreatic line intersects the exposed country side slope near the toe of the embankment. This force depends on the hydraulic gradient at the exit point. Naturally, this force can be resisted only by soil cohesion as the friction between grains, cannot develop due to lack of confinement. In soils with very low plasticity, little or no amount of resistance can develop and soil erosion is bound to occur at the spot where free water line touches the slope. Sherard et al. (1963) provides a gradation range of soils susceptible to piping (Fig. 2.4). It is recommended to provide drainage filters so that the phreatic line does not touch the exposed country side slope, or to use soils that are not susceptible to piping.

2.5.3 Erosion Failure

The waves developed near the top water surface due to the winds try to notch out the soil from the river side face and may even, sometimes, cause the slip of the river side slope. River side stone pitching or riprap may be needed to avoid such failures. Heavy rains falling directly over the countryside slope and the erosive action of the moving water, may lead to the formation of gullies on the country side face, which may lead to the slope failure due to steepening of slope This may be avoided by proper maintenance and turfing of the side slopes. Also the country side toe of the earthen embankment may get eroded due to tail water coming from seepage which may ultimately lead to further slides. Such damage may be prevented using drainage filters near the toe, strengthening of toe or using berms.

2.5.4 Earthquake Induced Failure

Earthquake may cause significant failures and movements of natural slopes and earthen embankments. Earthquakes can generate both horizontal and vertical motion. Usually the horizontal motion is predominant. Horizontal inertia forces have a much greater influence on embankment stability than vertical inertia forces. Failure may result from increased shear stress within a soil mass or from decrease or loss of strength during dynamic loading conditions imposed by an earthquake. An important reason for decrease or loss of strength is the development of high excess pore water pressure in saturated soils during dynamic loading. Many slope failures may result predominantly from increased shear stress and only to a minor extent from decrease of strength due to cyclic loading, and increased pore water pressure. Failure of an earthen embankment during high flood level may be cased by general slope failure induced by ground motions or piping failure due to cracks induced by ground motions.

It is regarded as fortunate that few major earth embankments have been subjected to very severe shaking during earthquakes. Only a small number of earth embankments have failed completely. However, a large number of earth embankments have suffered significant damage during earthquakes (Choudhury, 1992).

2.6 CASE STUDIES OF EMBANKMENT FAILURE

Occurrences of failures or breaches at portions of flood control embankments is quite common in Bangladesh. Corrective measures are taken to halt progression of many of such breaches, which have some times turned to be very costly. Islam (1991) reports analysis of some cases of failures, some of which are briefly reported below:

(i) Chalan Beel Flood Control Embankment

The Chalan Beel Flood Control Embankment failed at village Malipukur and village Dangapara, Atrai, Nagaon. At the failed sections the height, crest width, country and river side slopes at Malipukur location were 3.0 m, 4.0 m, 1.35:1 and 1.40:1 respectively and at Dangagapara location were 3.0 m, 3.0 m, 1.2:1 and 1.3:1 respectively. HFL at these two locations were 0.5 m below crest level in August 1985 when breaches occurred. The phreatic lines for these cases obtained by Casagrande's method touches the country side slope. At

Malipukur the soil of the failed section consisted of sandy clay. At Dangapara, the soil consisted of inorganic clay of medium plasticity, where piping related failure is unlikely. A general slope failure is the likely cause and Islam's (1991) analysis supports that.

(ii) The Naogaon-Atrai Flood Control Embankment

The Naogaon-Atrai Flood Control Embankment failed at village Nandaibari, Raninagar, Naogaon. At the failed sections, the height, crest width, country and river side slopes were 3.0 m, 4.0 m, 1.15:1 and 1.2:1 respectively. Both the river and country side slopes of the Naogaon-Atrai Embankment were steep. HFL at this location was 0.5 m below crest level when breach occurred. The soil of the failed section consisted of sandy clay. Islam (1991) computed the factor of safety with seepage to be below 1.0.

(iii) The Dharala Right Bank Embankment

The Dharala Right Bank Embankment failed at village Palashbari, Kurigram. At the failed sections, the height, crest width, the country and river side slopes, at Palashbari location were 3.3 m, 3.4 m, 1.2:1 and 1.25:1 respectively. HFL at this location were 0.8m below crest level when breach occurred. The phreatic lines for these cases obtained by Casagrande's method touches the country side slope. The soil at the breached location of the embankment consisted of clayey sand or silty sand. The breached section was partially susceptible to piping as the soil was almost non-cohesive. Islam (1991) reports factor of safety against general slope failure to be below 1.0 for seepage condition.

(iv) Dhaka- Narayangonj-Demra (DND) Embankment

Dhaka- Narayangonj-Demra (DND) embankment protects a vast suburban area near Dhaka against flood. Breaches occurred at several section of this very important embankment during the 1998 flood, which was unprecedented both in magnitude and duration. The embankment suffered slope failures due to seepage, piping, sliding and partial overtopping. The adverse situation are mitigated by adopting emergency measures by BWDB such as bamboo and bullah piling and placing sand-filled gunny bags on the river side and country side slopes.

2.7 FACTOR OF SAFETY

In general, a reasonable margin of safety should be provided against failure from any cause that can be anticipated.

Although some questions remain regarding the accuracy of the mechanics of slope stability analysis, in practical situations the greatest uncertainties lie in the estimation of the pore pressures and especially in the selection of strength parameters. A safety factor, as defined in chapter three, indicates the degree to which the expected strength parameters can be reduced before failure would occur, and hence essentially is a safety factor against an error in the estimation of these parameters. For intact homogeneous soils, when the strength parameters have been chosen on the basis of good laboratory tests and a careful estimate of pore pressure has been made, a safety factor of at least 1.5 is commonly employed. With fissured clays and for non homogeneous soils larger uncertainties will generally exist and more caution is necessary.

According to Varshney et al (1979) the minimum required factor of safety in earthen embankments never exceeds 1.5. According to U.S.B.R. practice a factor of safety of 1.5 is adopted for all conditions. The factor of safety recommended by U.S.B.R. are on higher side. High embankments have recently been designed with lower factor of safety up to 1.25 for reservoir drawdown and end of construction conditions. When under earthquake conditions, factor of safety of unity is considered adequate.

Singh and Prakash (1976) states that for sustained or long term conditions, e.g. steady seepage, a factor of safety of 1.5 is usually accepted for embankment design. For transient conditions like sudden drawdown, or earthquake, factor of safety of 1.1 to 1.3 are adopted.

CHAPTER 3

METHODOLOGY FOR SLOPE STABILITY ANALYSIS OF EARTHEN EMBANKMENTS

3.1 GENERAL

Gravitational and seepage forces tend to cause instability in natural and man made slopes. The failure of a mass of soil in a drawdown and outward movement of a slope may be called slope failure. The most important types of slides (Chowdhury, 1978) occurring in slope of cohesive soil are illustrated in Fig. 3.1. In rotational slip the shape of the slip surface in section may be a circular or a non-circular curve. In general, circular slips are associated with homogeneous soil conditions and non circular slips with non-homogeneous conditions. Translational and compound slips occur where the form of the failure surface is influenced by the presence of an adjacent stratum of significantly different strength. Translational slips tend to occur where the adjacent stratum is at a relatively shallow depth below the surface of the slope, the failure surface tends to be plane and roughly parallel to the slope. Compound slips usually occur where the adjacent stratum is at greater depth, the failure surface consisting of curved and plane sections.

In practice, limiting equilibrium methods are used in the analyses of slope stability. It is considered that failure is on the point of occurring along an assumed or a known failure surface. The shear strength required to maintain a condition of limiting equilibrium is compared with the available shear strength of the soil, giving the average factor of safety along failure surface.

In the following sections, various methods of analyses of slope stability for general slope failure incorporating seepage and seismic effects are briefly described.

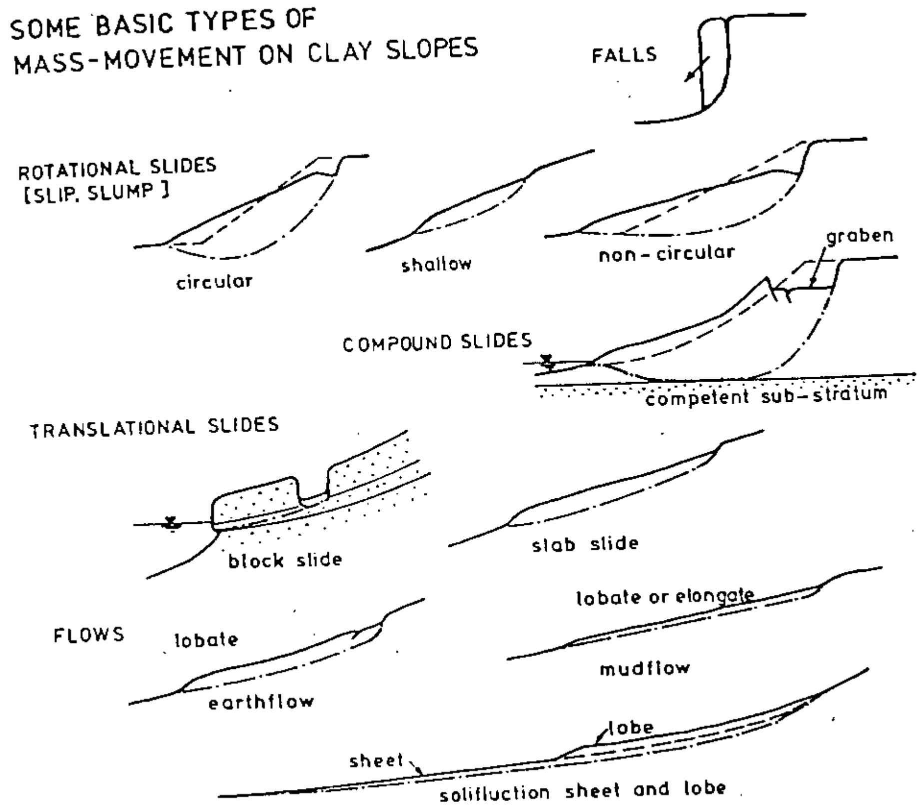


Fig. 3.1 Different types of slides in clay slopes(after Chowdhury, 1978)

3.2 SLOPE STABILITY ANALYSIS FOR GENERAL SLOPE FAILURE

3.2.1 Principles of Limiting Equilibrium Analysis

Limiting equilibrium analysis has been widely used in practice for estimating the stability of slopes. It should be noted that the following assumptions are made in this procedure:

(i) The shape and location of the failure surface is assumed rather than determining it from analysis.

(ii) Three-dimensional effects of slope failure is neglected and plain-strain deformation is assumed. This assumption gives conservative results.

(iii) The sliding mass is assumed to move as a rigid block with the movement taking place only along the failure surface.

(iv) Shear stresses are assumed to be uniformly mobilized along the whole length of the failure surface i.e., progressive failure is not considered.

A major advantage of this approach is that complex soil profiles, seepage and a variety of loading conditions can be easily dealt with. It has been the most popular method for slope stability calculations.

Because of the approximate and somewhat arbitrary nature of limit equilibrium analysis, concern is often voiced about how accurate these solutions are. There are indeed no exact solutions against which these results can be checked. The results represent neither upper bounds nor lower bounds of the failure. An alternative and rigorous method is to use finite element based limit analysis procedure to obtain lower and upper bound solutions for the stability of slopes. Such limit analysis models the soil as a perfectly plastic material obeying an associated flow rule. Yu et al. (1998) concluded that the limit equilibrium method of Bishop gave reasonable solutions for homogeneous slopes, based on comparison with rigorous limit analysis.

3.2.2 Methods of Limiting Equilibrium Analysis

3.2.2.1 Ordinary Method of Slices

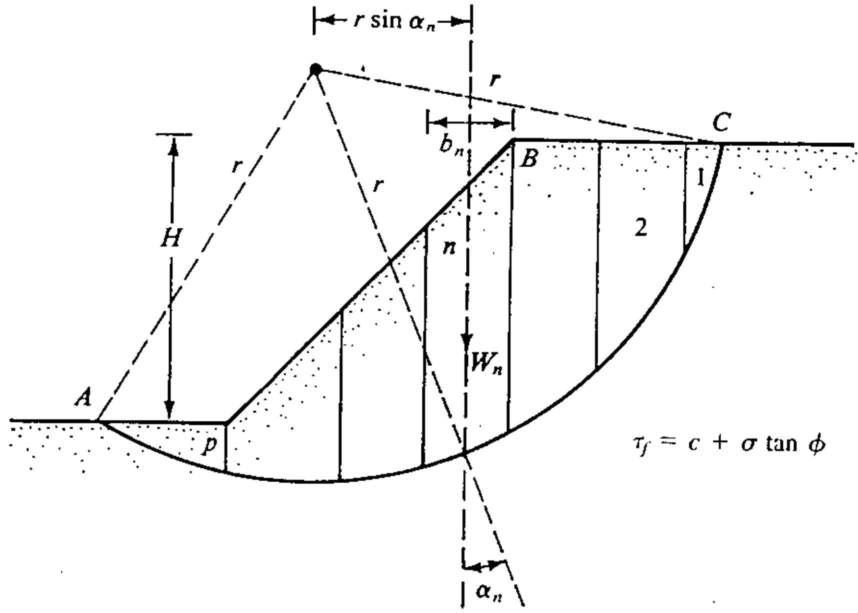
Slope stability problems in engineering works are usually analyzed using limit equilibrium methods. Many such methods are available in practice and the most common one calls on the principle of slices. In this method the sliding soil mass is broken up into a series of vertical slices and the equilibrium of each of these slices is considered. This procedure allows both complex geometry and variable soil and pore pressure conditions of a given problem to be considered.

Stability analysis by using the method of slices can be explained with the use of Fig. 3.2a in which AC is an arc of a circle representing the trial failure surface. The soil above the trial failure surface is divided into several vertical slices. Considering unit thickness perpendicular to the cross-section shown, the forces acting on a typical slice (nth slice) are shown in Fig. 3.2b. W_n is the weight of the slice. The forces N_r and T_r are the normal and tangential components of the reaction R. P_n and P_{n+1} are the normal forces acting of the sides of the slice. Similarly, the shearing forces acting on the sides of the slice are T_n and T_{n+1}. For simplicity, the pore water pressure is assumed to be zero here. This is a statically indeterminate problem and necessary assumptions are needed to solve this problem.

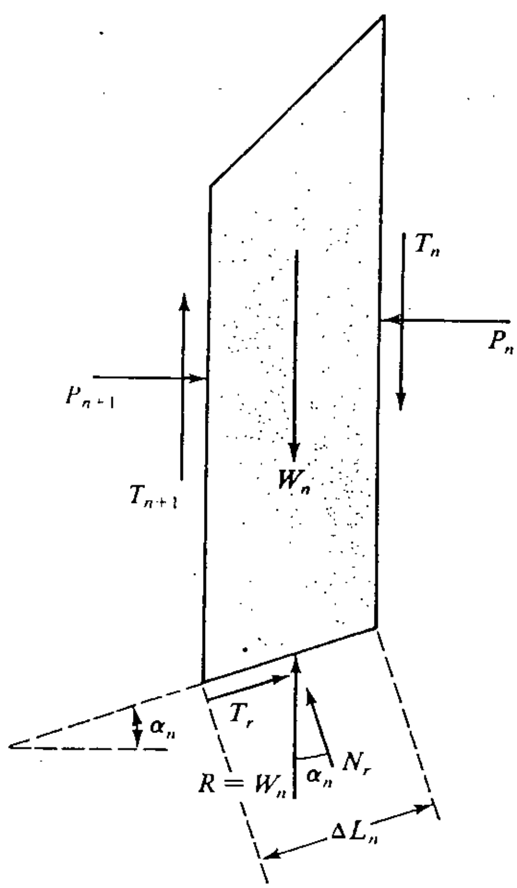
In the ordinary method of slices, it is assumed that the resultants of P_n and T_n are equal in magnitude to the resultants of P_{n+1} and T_{n+1} and also that their line of action coincide. In other words, the forces working between slices i.e., interslice forces are ignored.

For equilibrium consideration

$$N_r = W_n \cos\alpha_n$$



(a)



(b)

Fig. 3.2 Stability analysis by ordinary method of slices: (after Das, 1983)

(a) trial failure surface; (b) forces acting on nth slice

The resisting shear force along the slip surface of length ΔL_n, assuming a factor of safety of F_s can be expressed as:

$$T_r = \tau_d(\Delta L_n) = \frac{\tau_f(\Delta L_n)}{F_s} = \frac{1}{F_s}[c + \sigma \tan\varphi]\Delta L_n \tag{3.1}$$

where the soil shear strength parameters are c and φ, and σ is the normal stress.

The normal stress in the preceding Eq. (3.1) is equal to

$$\sigma = \frac{N_r}{\Delta L_n} = \frac{W_n \cos\alpha_n}{\Delta L_n}$$

For equilibrium of the trial wedge ABC, the moment of the driving force about O equals the moment of the resisting force about O, or

$$\sum_{n=1}^{n=p} W_n r \sin\alpha_n = \sum_{n=1}^{n=p} \frac{1}{F_s}\left(c + \frac{W_n \cos\alpha_n}{\Delta L_n}\tan\varphi\right)(\Delta L_n)(r)$$

The factor of safety can thus be found as:

$$F_s = \frac{\sum_{n=1}^{n=p}(c\Delta L_n + W_n \cos\alpha_n \tan\varphi)}{\sum_{n=1}^{n=p} W_n \sin\alpha_n} \tag{3.2}$$

ΔL_n is approximately equal to $(b_n/\cos\alpha_n)$ where b_n is the width of the nth slice. The value of α_n may be either positive or negative. The value of α_n is positive when the slope of the arc is in the same quadrant as the ground slope.

Note that the factor of safety is expressed as a ratio of the resisting forces and driving forces in Eq. (3.2) for the given failure surface. The driving forces are simply due to the gravity forces i.e., the weight while the resisting forces are due to the shear strength, which is again influenced by the normal stress. To find the minimum factor of safety – that is, the factor of safety for the critical circle- several trials are to be made by changing the centre and radius of the trial circle.

3.2.2.2 Simplified Bishop's Method

In 1955, Bishop proposed a more refined solution to the ordinary method of slices. In this method, the effect of forces on the sides of each slice are accounted for to some degree. The forces acting on the nth slice shown in Fig. 3.2b, have been redrawn in Fig. 3.3a. Let $P_n - P_{n+1} = \Delta P$; $T_n - T_{n+1} = \Delta T$. Also, it can be written that

$$T_r = N_r(\tan\varphi_d + c_d \Delta L_n) = N_r\left(\frac{\tan\varphi}{F_s}\right) + \frac{c\Delta L}{F_s} \tag{3.3}$$

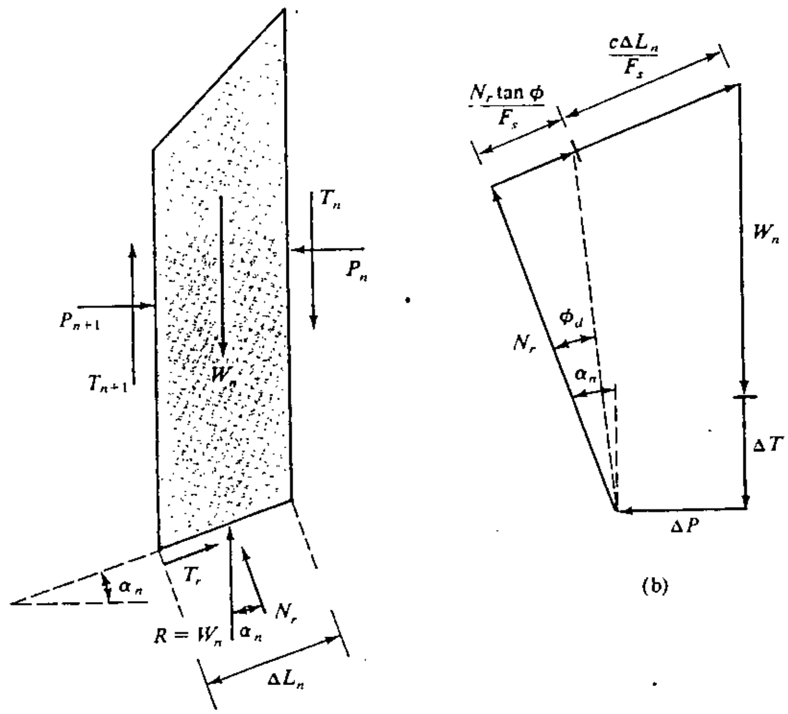
c_d and φ_d are respectively, the cohesion and the angle of friction that develop along the potential failure surface. Fig. 3.3b shows the force polygon for equilibrium of the nth slice. Summing the forces in the vertical direction

$$W_n + \Delta T = N_r \cos\alpha_n + \left[\frac{N_r \tan\varphi}{F_s} + \frac{c\Delta L_n}{F_s}\right]\sin\alpha_n$$

or

$$N_r = \frac{W_n + \Delta T - \frac{c\Delta L_n}{F_s}\sin\alpha_n}{\cos\alpha + \frac{N_r \tan\varphi \sin\alpha_n}{F_s}} \tag{3.4}$$

For equilibrium of the wedge ABC (Fig. 3.2a), taking the moment about O



(a)

Fig. 3.3 Simplified Bishop method of slices: (after Das, 1983)

(a) forces acting on nth slice ; (b)trial failure surface.

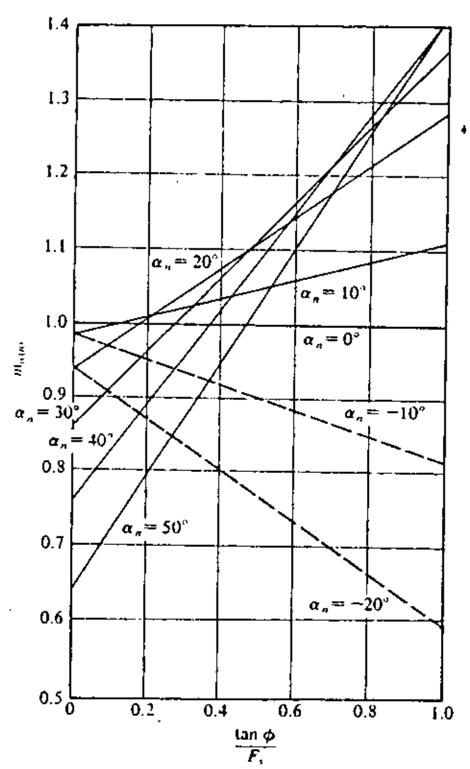


Fig. 3.4 Variation of $m_{\alpha(n)}$ with $(\tan\varphi)/F_s$ and α_n (after Das, 1983).

$$\sum_{n=1}^{n=p} W_n r \sin\alpha_n = \sum_{n=1}^{n=p} T_r r \tag{3.5}$$

where

$$\begin{aligned} T_r &= \frac{1}{F_s}[c + \sigma \tan\varphi]\Delta L_n \\ &= \frac{1}{F_s}[c\Delta L + N_r \tan\varphi] \end{aligned} \tag{3.6}$$

Substitution of Eqs. (3.4) and (3.6) in Eq. (3.5) and for simplicity assuming $\Delta T = 0$, Eq. (3.7) is obtained.

$$F_s = \frac{\sum_{n=1}^{n=p} \{cb_n + (W_n \tan\varphi\} \; {}^{1}\!/_{m_{\alpha(n)}}}{\sum_{n=1}^{n=p} W_n \sin\alpha_n} \tag{3.7}$$

where,

$$m_\alpha = (1 + \frac{\tan\alpha_n \tan\phi}{F_s})\cos\alpha_n \tag{3.8}$$

Note that the term F_s is present on both sides of Eq. (3.7). Hence, a trial and error procedure needs to be adopted to find the value of F_s. Fig. 3.4 shows the variation of m_α with $(\tan\varphi)/F_s$ for various values of α_n. As in the case of the method of ordinary slices, a number of failure of surfaces have to be investigated to find the critical surface that provides the minimum factor of safety.

Bishop's simplified method is probably one of the most widely used methods. When incorporated into computer programs it yields satisfactory results in most cases (Das, 1983).

3.2.2.3 Simplified Janbu's Method

The simplified Janbu method as described by Fredlund et al. (1981), is briefly reported here. Moment equilibrium is not satisfied. The normal force P on the base of a slice (Fig. 3.5) is given by:

$$P = \frac{W - (X_R - X_L) - \frac{c'l\sin\alpha}{F} + \frac{ul\tan\varphi'\sin\alpha}{F}}{m_\alpha} \tag{3.9a}$$

where $$m_\alpha = (\cos\alpha + \frac{\sin\alpha\tan\phi'}{F_s})$$

The shear force mobilized at the base of a slice is given as:

$$S_m = \frac{l}{F_s}[c' + (\sigma_n - u)\tan\varphi'] \tag{3.9b}$$

W= Total vertical force due to the mass of a slice of width 'b' and height 'h'.

E = Horizontal interslice normal forces.

X= Vertical interslice shear forces.

R= Radius or the moment arm associated with S_m

x= Horizontal distance from the centroid of each slice to the centre of rotation.

a= Perpendicular distance from the resultant external water force to the centre of rotation.

b = Width of nth slice.

A_L, A_R = Resultant external water forces.

α = Angle between tangent to the centre of the base of each slice and horizontal.

c′ = Effective cohesion intercept.

φ′ = Effective angle of internal friction.

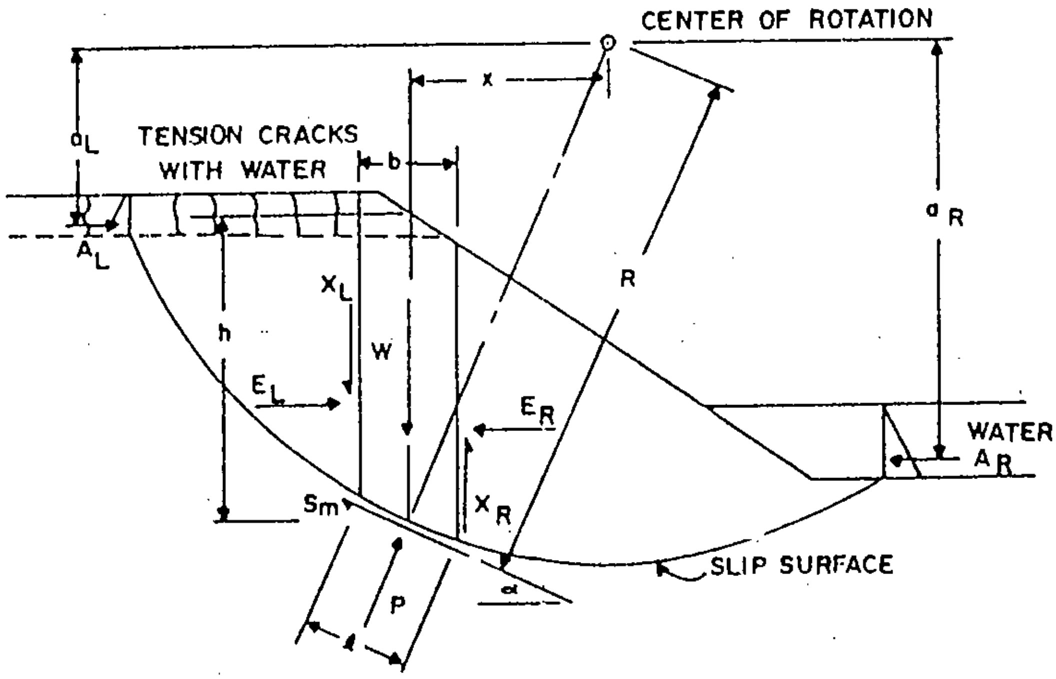


Fig. 3.5 Forces acting on a slices (Circular slip Surface)
(after Fredlund et al, 1981)

l = Length of failure surface at the base of each slice.

F_s = Factor of safety.

$\sigma_n = p/l$ u = pore water pressure.

The 'L' and 'R' subscripts on the 'E', 'X', 'a' and 'A' variables designate the left and right sides, respectively.

In the simplified Janbu method, the interslice shear forces are assumed to be zero (Janbu et al, 1956). The factor of safety, F_f is computed from the following horizontal force equilibrium equation

$$F_f = \frac{\sum [c'l + (P - ul)\tan\phi']\cos\alpha}{\sum p\sin\alpha \pm A} \tag{3.10}$$

Then an empirical correction factor is multiplied by the computed factor of safety in an attempt to account for the effect of the interslice shear forces. The empirical correction factor f_o, as shown in Fig. 3.6 , is related to the shear strength properties and the shape of the slip surface. The empirical correction factor generally increases the factor of safety by up to approximately 10 percent.

3.2.2.4 Janbu's Generalised Procedure of Slice

The generalized Janbu's method includes the effect of interslice forces by making an assumption regarding the point at which the interslice forces act (i.e., the line of thrust; Janbu, 1954; Janbu et al, 1956). The normal force equation is derived from the summation of vertical forces Eq. (3.9).

The factor of safety equation is derived from the horizontal force equilibrium equation (Eq. 3.10). In order to solve for the factor of safety, the interslice forces are computed from the summation of the moments about the center of the base of each slice.

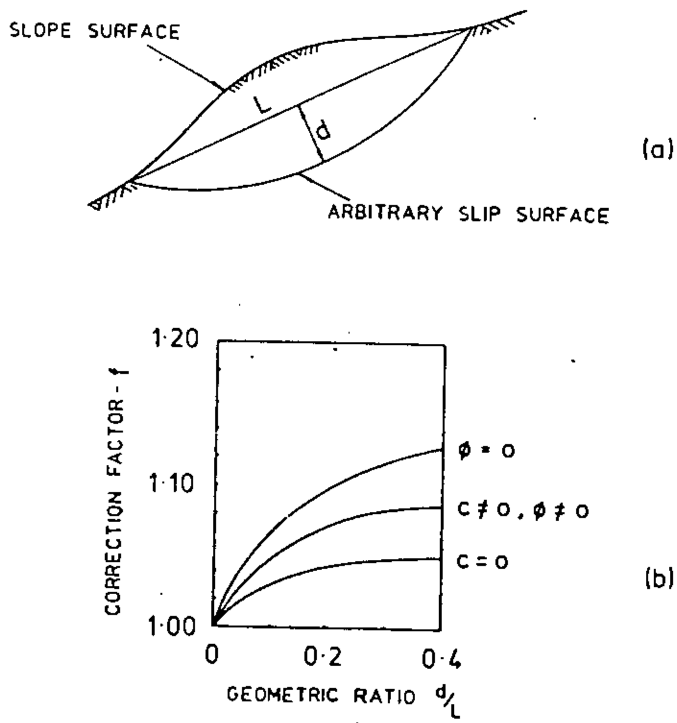


Fig. 3.6 Correction factor f_o for use in Simplified calculations after Janbu et al (1956) shown in (b) corresponding to geometric ratio d/L shown in (a), (after Chowdhury, 1978).

$$X_R = E_R \tan\alpha_t - (E_R - E_L)t_R / b \qquad (3.11)$$

where α_t = Angle between the line of thrust on the right side of a slice and the horizontal.

t_R = Vertical distance from the base of the slice to the line of thrust on the right side of the slice.

The horizontal interslice forces required for Eq. (3.11), are obtained by summing forces in the horizontal direction on each slice.

$$E_L - E_R = S_m \cos\alpha - P\sin\alpha \qquad (3.12)$$

Once Eq. (3.10) has been solved, it is possible to plot the computed interslice shear and normal forces and determine a corresponding side force function. Note that the interslice forces depend on the factor of safety, which is itself to be determined from the calculations. An iterative solution must be adopted. Computation continues until consistent values of F_f and thrust line position is found.

3.2.2.5 Spencer's Method

Spencer (1967) used two factor of safety equations; one satisfying force equilibrium, the other satisfying moment equilibrium. This method assumes that the inclinations θ of the resultant interslice forces Q are constant; that is:

$$\tan\theta = X_L / E_L = X_R / E_R = \text{constant for all slices} \qquad (3.13)$$

and the (X_L- X_R) term in Eq. (3.9) for can be rewritten using :

$$(X_L - X_R) = (E_L - E_R)\tan\theta \qquad (3.14)$$

The factor of safety with respect to moment F_m are given in Eq. (3.15)

$$F_m = \frac{\sum[c'l + (P - ul)\tan\phi']R}{\sum Wx \pm Aa} \tag{3.15}$$

Indeterminacy implies that the value of θ is not known at the beginning of computation. Spencer's method solves Eq. (3.10) for F_f and Eq. (3.15) for F_m by iteration for several assumed values of θ. The final answer is taken to be at the value of θ when $F_f = F_m$.

3.2.2.5 Main Features of Different Methods

Features of different slope stability methods as presented by Chowdhury (1978) are briefly described below:

Method	Features
Fellenious	Inter-slice forces ignored. Normal force on base of slice obtained by resolving total forces normal to base. Underestimates factor of safety. Errors (on the safe side) large for deep failure masses with high pore pressures. Effective normal stresses on the bases of some slices can become negative. Factor of safety is defined as ratio of resisting to disturbing moments or forces. Strictly only applicable to circular failure surfaces. Adequate for total stress analyses of circular failure surfaces but not always suitable for effective stress analyses.
Simplified Bishop	Inter-slice forces ignored. Normal force on base of slice obtained by resolving forces on slice vertically. Gives fairly accurate results but is restricted to slip surfaces of circular shape. Iterative procedure required for solution but convergence rapid. Useful for hand calculations. Errors possible where portion of slip surface has steep negative slope near toe. Calculation of normal forces on slip surface possible. This should be done

in addition to determining F. Suitable for both total and effective stresses analyses of circular failure surfaces in soil and soft rock.

Janbu	Requires assumption of inter-slice forces. Iterations made with successive sets of inter-slice forces till convergence reached. Suitable for slip surfaces of arbitrary shape. Computer desirable but not essential. Convergence generally rapid but sometimes slow due to large changes in inter-slice forces between iterations. Necessary to check acceptability of solution in terms of position of line of thrust, any implied tension or violation of failure criterion if solution to be regarded as rigorous. Suitable for both total and effective stresses analyses of soil and rock slopes. The Janbu method is also useful in analyzing the influence of partial submergence and drawdown conditions and the effects of tension cracks and surcharge.
Simplified Janbu	Use of correction factors necessary to applied to the F. S. to reduce the conservatism produced by the assumption of no inter-slice forces. Suitable for slip surfaces of arbitrary shape in soil and rock. Fairly reliable. No need to account for inter-slice forces. Suitable for both total and effective stresses analyses.
Spencer	Spencer's method (1967) is based on the work of Fellenius (1927) and Bishop (1955). Originally devised for circular failure surfaces, but adapted for non-circular failure surfaces. Assumes inter-slice forces to be parallel. Accuracy acceptable. Satisfies both force and moment equilibrium. Use of computer desirable. Specially devised in relation to embankment stability problems, but may be used all types of problems.

3.2.3 Total Stress and Effective stress Methods

For use in the limit equilibrium procedure, there are two approaches of accounting for the shear strength of soil, based on 'total stress' and 'effective stress' In the total stress approach, shear strength parameters based on undrained tests are used and pore water pressures are ignored. In the effective stress approach, effective shear strength parameters (c',φ') based on drained tests (or undrained tests with pore pressure measurements) are used. In addition, a knowledge of pore pressures in the field is necessary. The estimation of these pore water pressures in the field in advance of construction is often difficult. Skempton's well known pore pressure equation in terms of parameters A and B can be used, but A and B must be determined from laboratory tests or selected from past experience. One advantage of the 'effective stress' approach is that when actual pore pressures from piezometers installed in the field become available, the analyses can be checked. In principle both methods of analysis should lead to the same result (i.e., in this case the same factor of safety) whether short-term (end of construction) or long-term stability of a slope is being analysed. However, experience has shown that each method has advantages in particular situations.

Usually total stress analysis requires less work than effective stress analysis. However, the later approach is more logical and straightforward because, in reality, strength is controlled by effective stresses. While a total stress analysis is simple in itself, shear strength parameters have to be measured and selected with great care. The test conditions must correspond to the conditions of consolidation (isotropic or anisotropic) that exist in the field followed by shear under conditions of drainage that may be applicable. These conditions are not always easy to select and set up. Consideration must also be given to the requirement of undisturbed samples for testing especially in natural soils. Undrained strength required for use in a total stress analysis is usually far more sensitive to sample disturbance than are drained strength parameters. On the other hand tests to determine effective stress parameters from drained tests are often time consuming. Also the accuracy of estimated field pore pressures required for effective stress analyses is often in doubt.

Considering these features, merits of using each method in particular situations have been recognised and the current trend is to use both types of analysis where time, resources and facilities permit. Terzaghi and Peck (1967) state that it is not possible to say that effective stress analyses are always superior to total stress analyses or vice versa.

For analysis of steady seepage conditions for earthen embankments, effective stress analysis should be performed using pore pressures obtained from flow net construction.

3.2.4 Situations Critical for Embankment slope Stability

The stability of slopes of flood control earthen embankments needs to be investigated for critical conditions during its construction and later during its operation when it is subjected to different water levels on the river side slope. These are: (a) Stability during and at end of construction (b) Stability during steady seepage (c) Stability during rapid drawdown

3.2.4.1 Stability during and at End of Construction

When a dam is built of relatively impervious soil, e.g., clayey soils, excess pore pressures develop in the air and water entrapped in the pore space. This is because the soil mass undergoes a change in volume due to compaction carried out during construction or due to consolidation under its own weight. The pore pressures developed depend upon the placement water content, compaction, state of stress resulting from the weight of superimposed layers, and the rate of dissipation of pore pressure during construction. In the absence of proper drainage and when the placement water content is more than a few per cent in excess of the optimum, an initial pore pressure at any point of up to almost hundred percent of the weight of overlying fill may be reached. Estimation of pore pressures may be based on laboratory tests or on actual field measurements or past experience with similar projects. Bishop's (1957) method based on triaxial test results or Hilf's (1948) method based on consolidation test results may be used to predict pore pressures. However, field measurements are the most reliable. These pore pressures affect the shear strength of soil. In the method of slices, the pore pressure acts normal to the slip surface at the base of the slice. As the construction pore pressures gradually dissipate with time, the stability of the slope increases.

3.2.4.2 Stability during Steady Seepage

Percolating water sets up seepage pressure in an earth embankment in the zone below the phreatic line. These pressures are maximum when the water level on the river side is maximum and percolation is at its maximum rate. Relative to the river side slope, the seepage forces are directed inwards and hence tend to increase the stability of the slope, whereas, for the country side slope, the directions of the seepage forces tend to decrease stability. The steady seepage condition is considered critical for the country side slope of an earth embankment.

The resultant body force of a slice in the method of slices can be obtained by considering either a combination of the submerged weight of soil and the seepage force acting over the slice, or a combination of the saturated weight of soil and the boundary pore pressures acting over the slice. Boundary pore pressures on a slice are obtained from a flow net which is drawn for the embankment cross section under steady seepage condition. The later approach is convenient, as evaluation of seepage pressure is more cumbersome. Pore pressure acts normally at the vertical sides of a slice and at the base of the slice.

When a slope is partly submerged with free water standing against the slope and seepage is also occurring through the slope, the pore pressure on the base of slice should be expressed as an excess over the hydrostatic pressure corresponding to the water level outside that. For calculating weight of the slice, submerged density should be used for the part of the slice lying below the level of the external free water surface. For the upper part, bulk and saturated densities are used.

3.2.4.3 Stability during Rapid Drawdown

The critical condition for a river side slope develops when a rapid drawdown occurs following a long period of high water level. The severity of the drawdown depends upon the rate of drawdown and the type of soil in the slope. If the slope material is of low permeability compared with rate of drawdown, no appreciable change in the water level within the saturated soil of the slope may take place as the river side water level goes down . The most critical condition is when the drawdown is assumed to be sudden and complete without allowing any appreciable change in the water content of the saturated slope. When a sudden and complete drawdown occurs, the water pressure acting on the river side slope at the time of high water table is removed. The weight of water which is still present in the soil helps to cause a sliding failure, without the external water pressure on the slope to counteract it. In other words, while the driving force is the tangential component of the saturated weight, the shear resistance is considerably reduced due to the existence of pore pressures on a likely slip surface. The effect of drawdown on slope stabilities varies appreciably with the drainage pattern set up at the time of drawdown. With an impervious base, the flow lines tend to be horizontal and directed outwards towards the slope. Such is a quite unfavourable condition with respect to stability. With passage of time, the pore pressures get gradually dissipated and the stability of the slope increases.

The distribution of pore pressure along a trial slip surface is estimated from a flow net corresponding to the instant of drawdown. The stability is calculated similar to the steady seepage case.

Another approximate method is to consider the bulk or saturated unit weight for calculating the driving forces and the submerged (buoyant) unit weight for calculating the resisting forces. Below the draw down level, only the submerged density is used both for the evaluation of driving and resisting forces.

3.2.4.4 Stability during Earthquakes

An earthquake produces cyclic ground motions which in turn induce large inertia forces in slopes and embankments. The inertia forces are of short duration and of cyclic nature, alternating in direction many times. Earthquakes can generate both horizontal and vertical ground motion. Usually the horizontal motion is predominant. Horizontal inertia forces have a much grater influence on embankment stability than vertical inertia forces. Methods of analysis for studying the stability of slopes may be grouped into two broad classes: (i) Simplified Pseudo-static analysis and (ii) Rigorous Dynamic analysis.

Pseudo-static analysis:

Pseudo-static analysis methods involves methods that use equivalent static forces to approximate the effect of dynamic forces. The Pseudo-static limit equilibrium procedure belongs to this category.

In the Pseudo-static limit equilibrium procedure, which has been used in this study, the effect of an earthquake can be replaced by an acceleration equal to Ag, where A is a seismic coefficient and g is acceleration due to gravity. This acceleration is usually assumed to act in a horizontal direction inducing a constant inertia force A times W (AW) in the slope in which W is the weight of the potential sliding mass (Fig. 3.7). This inertia force AW is considered as a static force and not a dynamic force of short duration. The effect of pore pressure increase due to ground shaking is not taken into account in this procedure.

In spite of several limitations of this approach, it is still used widely by engineers (Chowdhury, 1978), possibly because of its simplicity and convenient integration with conventional limit equilibrium analysis.

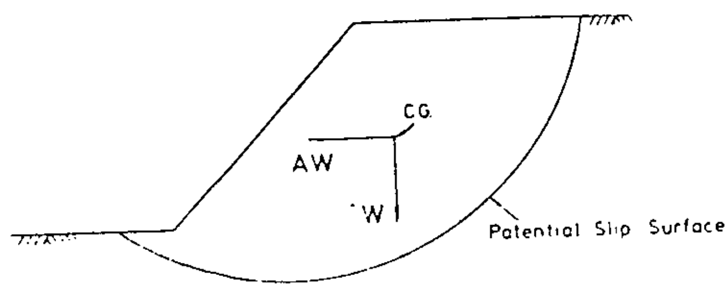


Fig. 3.7 Pseudo-static approach for considering earthquake induced forces. (after Chowdhury, 1978).

Dynamic Analysis:

Even though the factor of safety during any pulse of ground motion falls below unity only limited deformations may occur because of the short duration of the load. Therefore, it is necessary to consider the complete time history of ground motion to study the variation of stresses and deformations in a slope. The pulsating stresses due to an earthquake must, of course, be superimposed on the initial stresses which are determined not only by gravitational loads but also by the previous stress history of natural slopes and by the method of construction in the case of embankments. Significant progress has been achieved in using the finite element method for time dependent dynamic analysis of embankments.

3.2.5 Determination of Phreatic line

For construction of flow nets for seepage through earth embankments, the phreatic line needs to be established first. This is usually done by the method proposed by A. Casagrande (1937). AIK in Fig. 3.8 is the actual phreatic line. The base of the embankment NF is assumed to be impermeable. The curve AIJG is a parabola with its focus at F. The phreatic line coincides with this parabola, but with some deviations at the upstream and the downstream faces. It is assumed that the horizontal projection of the upstream face of the embankment NA as shown in Fig. 3.8, above water level be BA. On the water surface, a distance, CA = 0.3BA is considered, the point C is the starting point of the base parabola. The parabola AILJG can be constructed as follows:

Let the distance FD be equal to s. Now, referring to Fig. 3.8b, PF=PM (based on the properties of a parabola) and PM = s+x. Thus,

$$PF = \sqrt{\left(x^2 + y^2\right)}$$

$$\sqrt{\left(x^2 + y^2\right)} = s + x \tag{3.16}$$

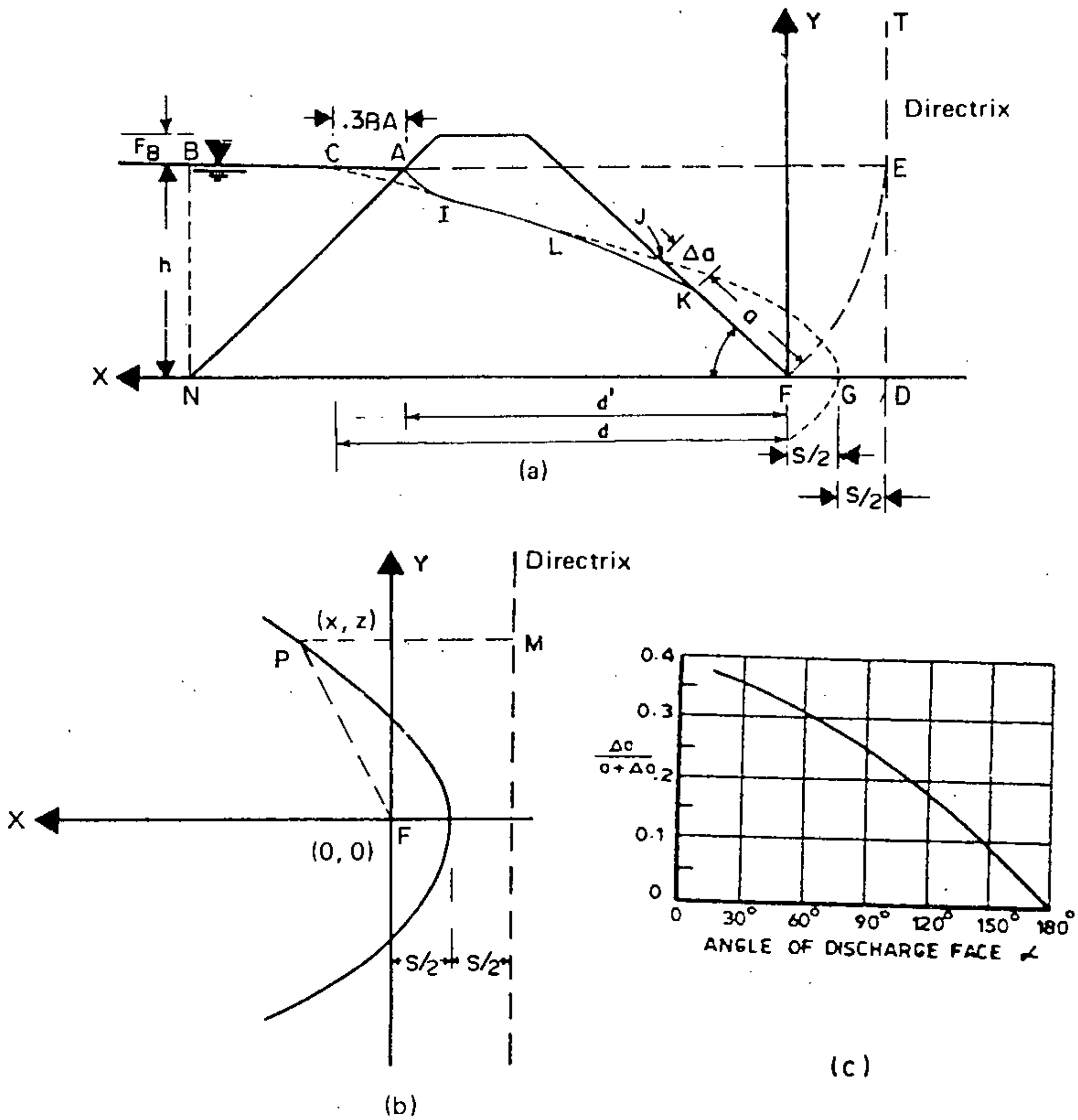


Fig. 3.8 Determination of phreatic line for seepage through an earthen embankment.

At x = d, z = h. Substituting this condition into Eq. (3.16) and rearranging,

$$s = \left(\sqrt{d^2 + h^2} - d\right) \tag{3.17}$$

Since d and h are known, the value of s can be calculated. The vertex G of the base parabola shall be situated at a distance equal to s/2 from F, beyond the country side toe of the dam. A few more co-ordinates of the base parabola at known distances (x) are worked out, From Eq.(3.16),

$$x^2 + y^2 = s^2 + x^2 + 2sx$$
$$x = \left(\frac{y^2 - s^2}{2s}\right) \tag{3.18}$$

With s known, the values of x for various values of y can be calculated from Eq. (3.18) and the parabola can be constructed.

The directrix of the parabola is drawn taking the point C as centre and with a radius CF to cut the horizontal line through CA at E. A vertical tangent is drawn to the curve FE at E. The vertical line ET is the Directrix.

The parabola has now to be corrected at the entry and exit points. The phreatic line must be from A and not from C. The phreatic line is a flow line and must start perpendicular to the upstream face NA which is an equipotential line. Hence a portion of the phreatic line at A is sketched free hand as a reverse curvature AI in such a way that it starts perpendicularly to NA.

At the exit the base parabola will cut the downstream slope at J, it is extended beyond the limits of the embankments as shown by the broken line in Fig. 3.8. But according to exit condition, the phreatic line must emerge at some point K, meeting the downstream face tangentially there. The portion KF (dimension a) is known as the discharge face and always remains wet. The correction a, by which the parabola is to be shifted downward, can be determined easily by using the equation given by

$$a = (a + \Delta a)(180 - \alpha)/400 \tag{3.19}$$

where a, Δa and α (in degrees) are as shown in Fig. 3.8c Having computed Δa, the point K is plotted and the phreatic line AIK coploted.

A general analytical method for computation of 'a' is as follows:
d is obtained using Eq (3.20)

$$d = (H_C / V_C)h + b + (H_C / V_C + H_R / V_R)F_B + AC \tag{3.20}$$

where H_R: V_R =River side slope (Horiz : Vert), BA= $(H_R/V_R)h$, AC=0.3BA
b = crest width, freeboard = F_B, $d' =$ d-0.3BA

To complete the phreatic line, the portion AI has to be approximated and drawn by hand.

$$a = d'/(\cos\alpha) - \left(d'^2/\cos^2\alpha - h^2/\sin^2\alpha\right)^{1/2} \tag{3.21}$$

When $\alpha<30^0$, the value of a can be calculated from Eq (3.36) as
A=KF in Fig. 3.8. Once point K has been located, the curve LK can approximately be drawn by hand. When α lies between 30^0 and 60^0.

$$a = \sqrt{d^2 + h^2} - \sqrt{d^2 - h^2\cot^2\alpha} \tag{3.22}$$

3.2.6 Determination of Pore Pressure Ratio

A pore pressure ratio may be defined as a dimensionless number that indicates the fraction of total stress increment that shows up an excess pore pressure for the condition of no drainage, that is constant mass. For purposes of analysis and tabulation, it is most convenient to express the pore-pressure u at any point in terms of the pore pressure ratio r_u defined by the Eq. (3.23)

$$r_u = \frac{u}{\gamma z} \tag{3.23}$$

where z is the depth of the point in the soil mass below the soil surface, γ is the bulk density of the soil .

Water may affect the stability of an unretained slope in a number of different ways. The stability of a slope and the computed factor of safety, may be highly dependent upon the magnitude and distribution of pore pressures within the slope.

The pore pressure ratio r_u in a section is not constant in an embankment and an average value of r_u must be used in calculating the factor of safety. An averaging method that has proved to be successful in giving values of the factor of safety that correspond closely to those obtained by direct calculation for cases in which the pore-pressure distribution in terms of the pore-pressure ratio r_u varied considerably throughout the section is presented below.

Generally r_u will not be constant over the cross section of an embankment and the following procedure can be used to determine an average value. In Fig. 3.9 the stability of the downstream slope is to be determined. From the centre line of the cross section divide the base of the dam into a suitable number of vertical slices, and on the centre line of each

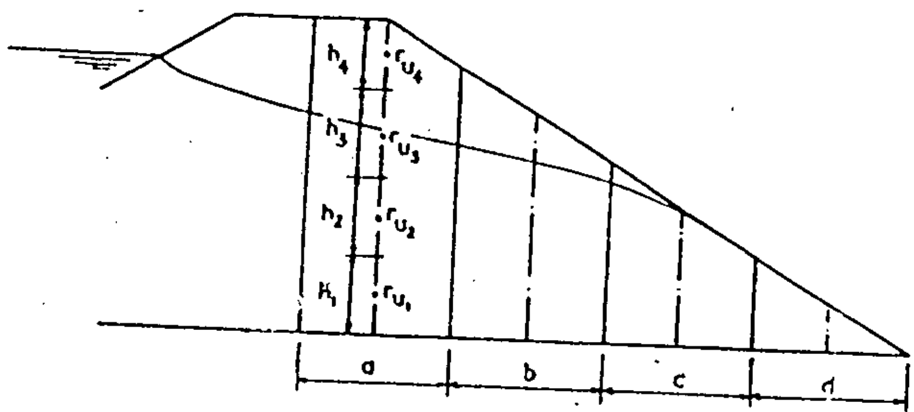


Fig. 3.9 Determination of average r_u value (after Smith, 1968)

slice determine r_u values for a series of points as shown (Fig. 3.9). Then the average pore pressure ratio on the centre line of a particular slice,

$$\text{Average } r_u = \frac{h_1 r_{u1} + h_2 r_{u2} + h_3 r_{u3} + \dots\dots\dots}{h_1 + h_2 + h_3 + \dots\dots\dots\dots\dots} \tag{3.24}$$

The average r_u for whole cross section

$$= \frac{A_a r_{ua} + A_b r_{ub} + A_c r_{uc} + \dots\dots}{A_a + A_b + A_c + \dots\dots} \tag{3.25}$$

where A_a =area of the slice 'a' and r_{ua} =average value of r_u in slice 'a'.

3.2.7 DESIGN CHARTS

Charts for investigating the stability of simple homogeneous earth slopes for soils with cohesion and friction have been available for many years. There are several stability charts. Probably the best known of these are Taylor's (1937, 1948), Bishop and Morgenstern's (1960), Spencer's (1967), Janbu's (1967) and Cousin's (1977).

The first slope stability charts were devised by Taylor (1937, 1948). The analysis is based on total stresses and assumes that the cohesion c is constant with depth. For a given value of angle of internal friction φ, the critical height of a slope is given by the equation $H_c = cN_s/\gamma$, where c= cohesion, γ = unit weight of soil, and N_s =stability factor. The stability factor N_s is a pure number, depending only on the slope angle β and friction angle φ. The relationships between N_s, β and φ are shown in Fig. 3.10. In the chart β value varies from 0^0 to 90^0 with φ varying from 0^0 to 25^0. Fig. 3.11 presents the case for φ =0. The depth factor D, shown in Fig. 3.10 is defined as the depth to the hard stratum divided by the height of the slope. In the chart the value of D varies from 1.0 to ∝.

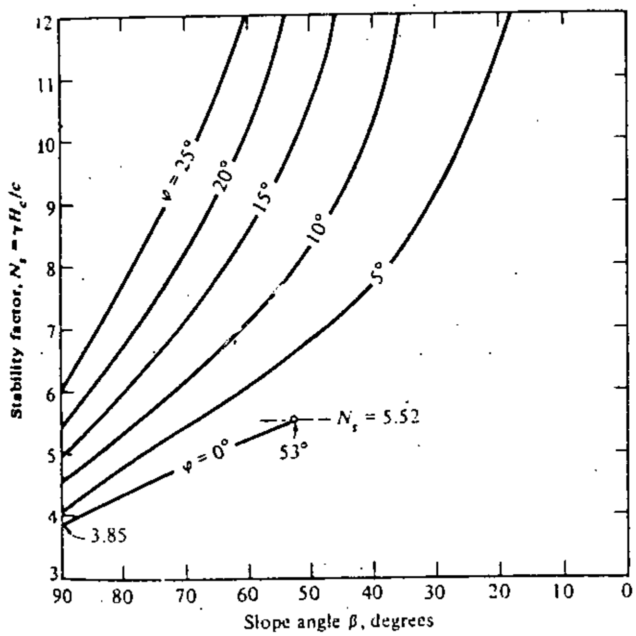


Fig. 3.10 Taylor method stability factor vs. slope angle with various friction angles φ (after Taylor, 1937; and Terzaghi and Peck, 1967).

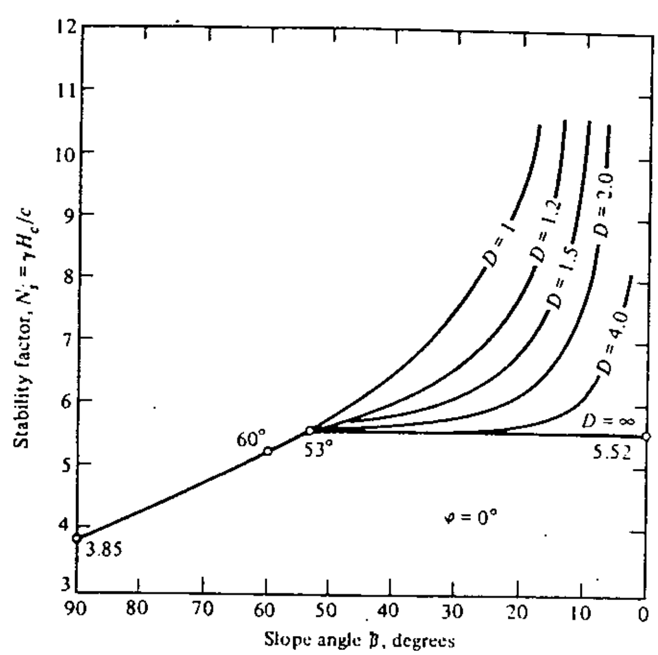


Fig. 3.11 Stability factor vs. slope angle for various depth factors D for $\varphi'=0^0$. (after Taylor, 1937; and Terzaghi and Peck, 1967).

Bishop and Morgenstern (1960) found a linear relationship between factor of safety of a homogeneous slope and pore pressure ratio r_u in the range of 0.0 to 0.7. The pore pressure ratio r_u is assumed to be constant throughout the embankment cross section. The factor of safety F_s is given as

$$F_s = m - nr_u \tag{3.26}$$

where, m is the factor of safety with respect to total stresses (i.e., when no pore pressures are assumed) and n is the coefficient which represents the effect of the pore pressures on the factor of safety. These terms m and n are known as stability coefficients and are determined from charts. They presented charts for (stability number) $c'/\gamma H$=0.0, 0.025, 0.05 and D (depth factor)=1.0, 1.25, 1.5. D is the depth from embankment top to a hard stratum divided by embankment height. The charts cover slopes of 2:1 to 5:1 and $\varphi'=10^0$ to 40^0. Fig. 3.12 presents charts for $c'/\gamma H$= 0.05, and D (depth factor)=1.5.

Spencer's (1967) analysis is in terms of effective stress and satisfies two equations of equlibrium, the first with respect to forces and second with respect to moments. Spencer provides chart for range of stability factors N_s from 0 to 0.12 with mobilized friction angle φ'_m varying from 10^0 to 40^0 and slope angle β up to 34^0. Three pore pressure ratios r_u with values of 0, 0.25 and 0.5 are provided. This chart is shown in Fig. 3.13. Values of r_u falling between the charts can be obtained sufficiently accurate for practical purposes by linear interpolation.

Cousins (1977) used Taylor's (1937, 1948) friction circle method to devise charts in terms of effective stresses. The stability charts are given in Figs. 3.14a, 3.14b, 3.14c and 3.14d. The first (Figs. 3.14a, and 3.14b) give the stability number, N_F, and thus the safety factor, F. The second group (Figs. 3.14c –3.14d) give comprehensive details of the critical slip circles. Generally, the charts have been drawn for values of the pore pressure ratio r_u equal to 0.0, 0.025 and 0.5. In all cases the slope angle α, is drawn as the abscissa and the dimensionless number, λ_{c0} = $(\gamma H\tan\varphi)/c$ is given in parametric form.

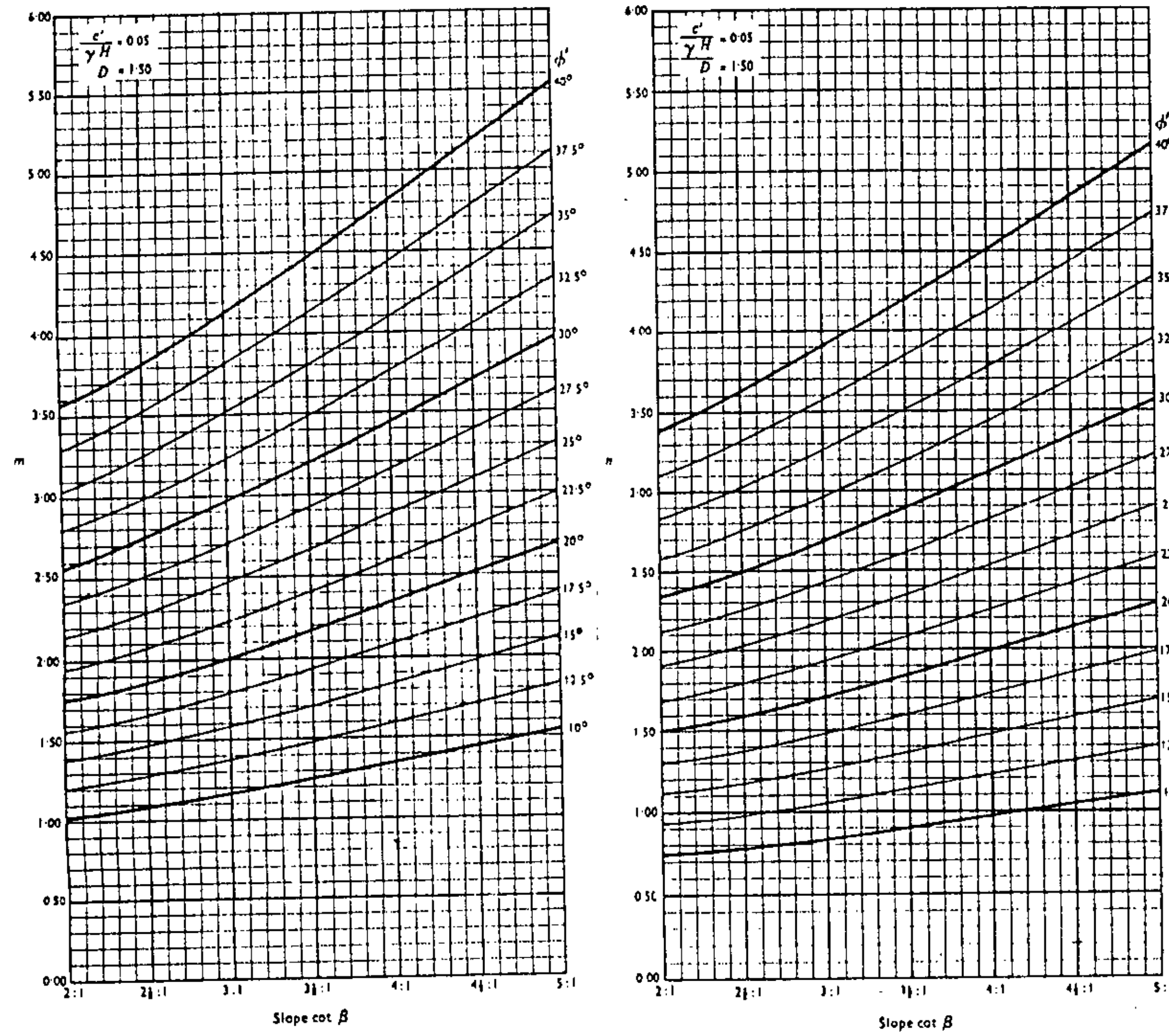


Fig. 3.12 Stability coefficients m and n for c'/γH=0.05 and D= 1.0

(after Bishop and Morgenstern, 1960)

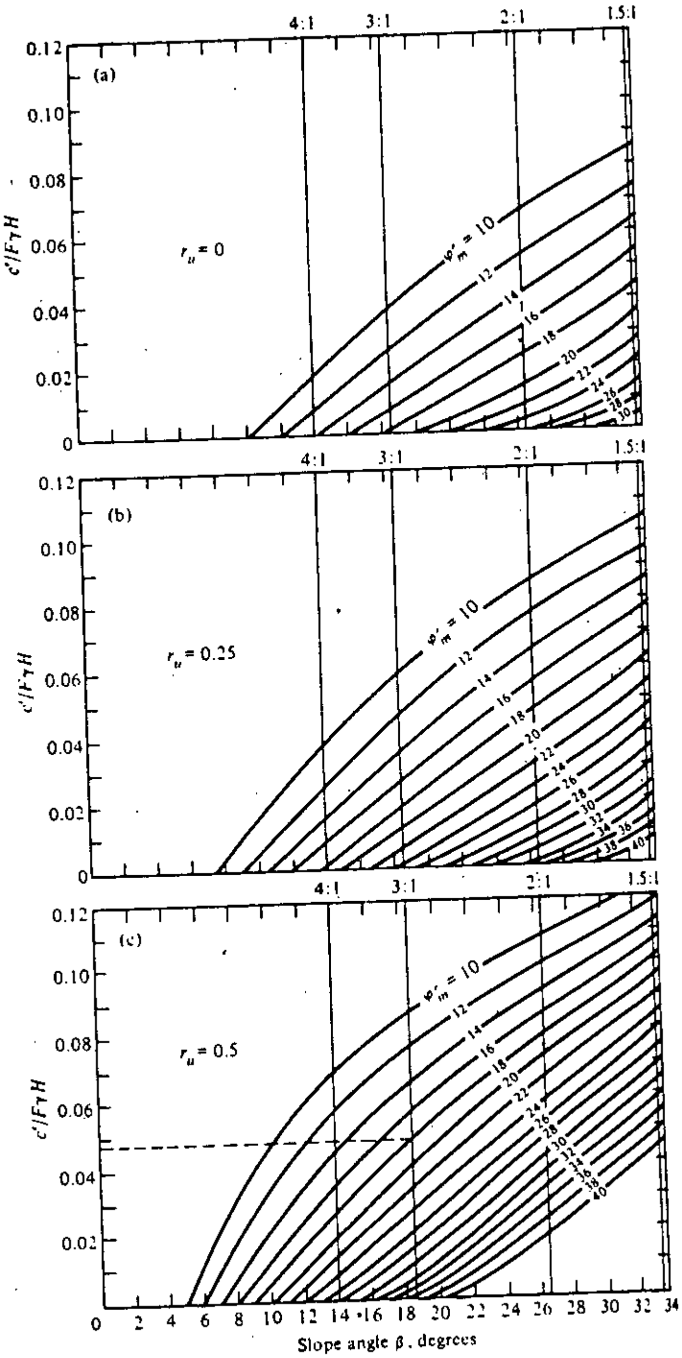


Fig. 3.13 Stability charts, (after Spencer, 1967)

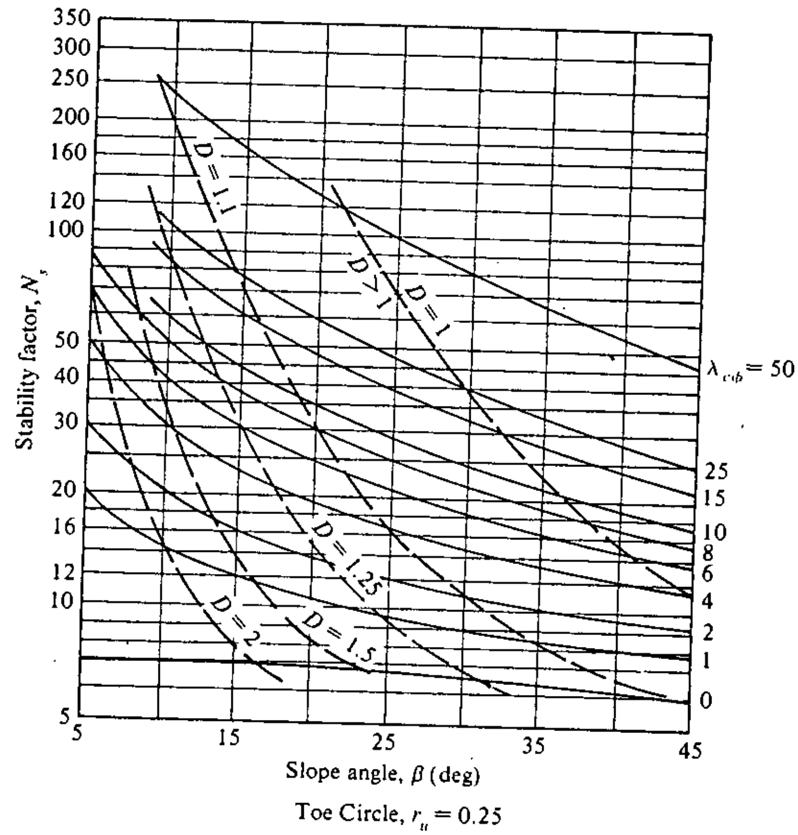


Fig. 3.14a Stability Numbers for Toe Circles and r_u =0.25 (after Cousin, 1978)

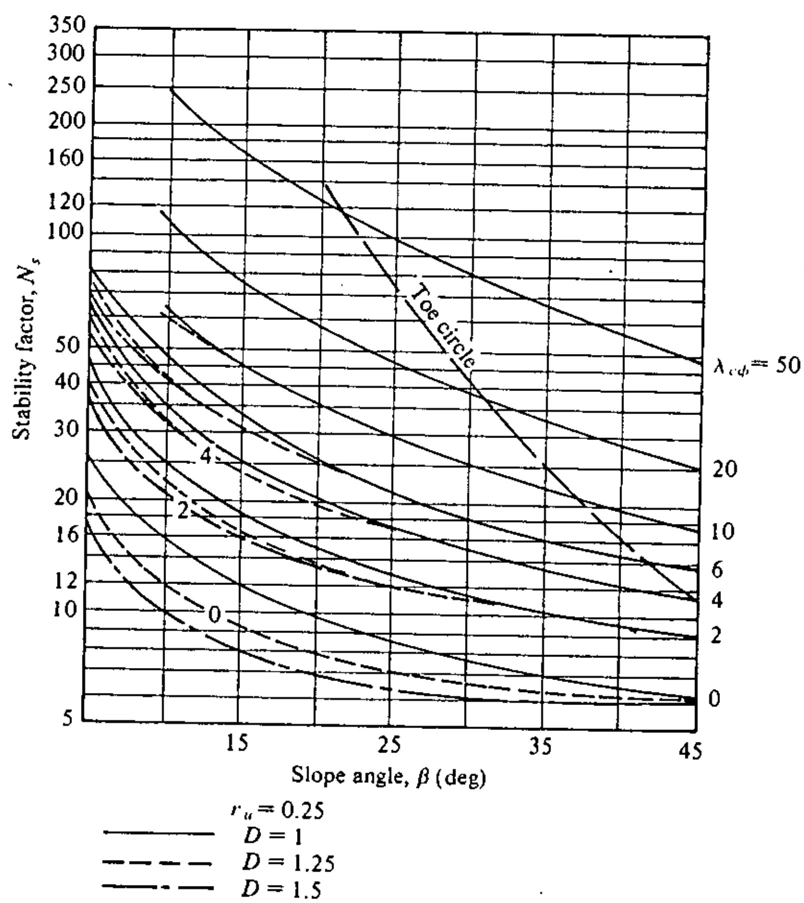


Fig. 3.14b Stability Numbers for depth factor D =1.25, r_u =0.25 (after Cousin, 1978)

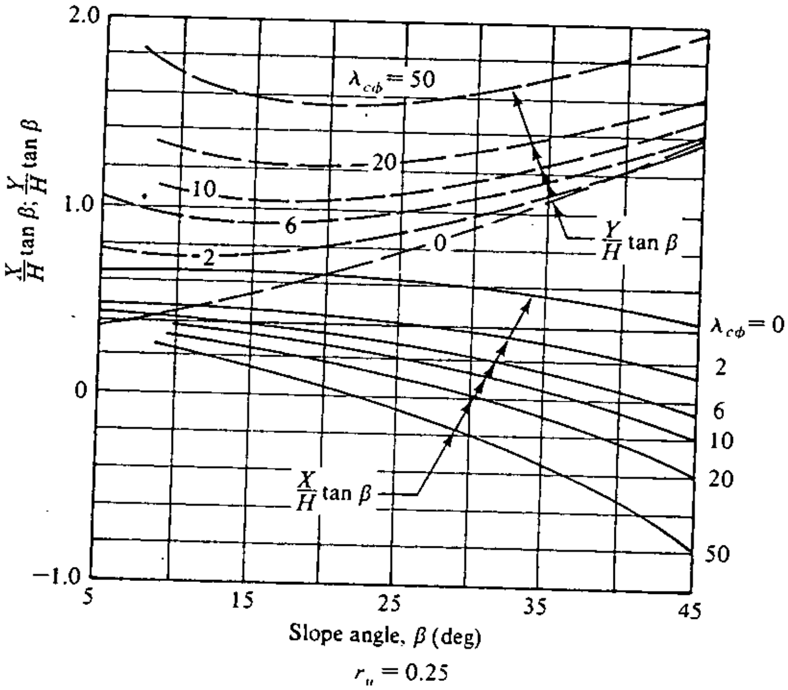


Fig. 3.14c Coordinates of critical slip circles for Toe circles and r_u =0.25 (after Cousin, 1978)

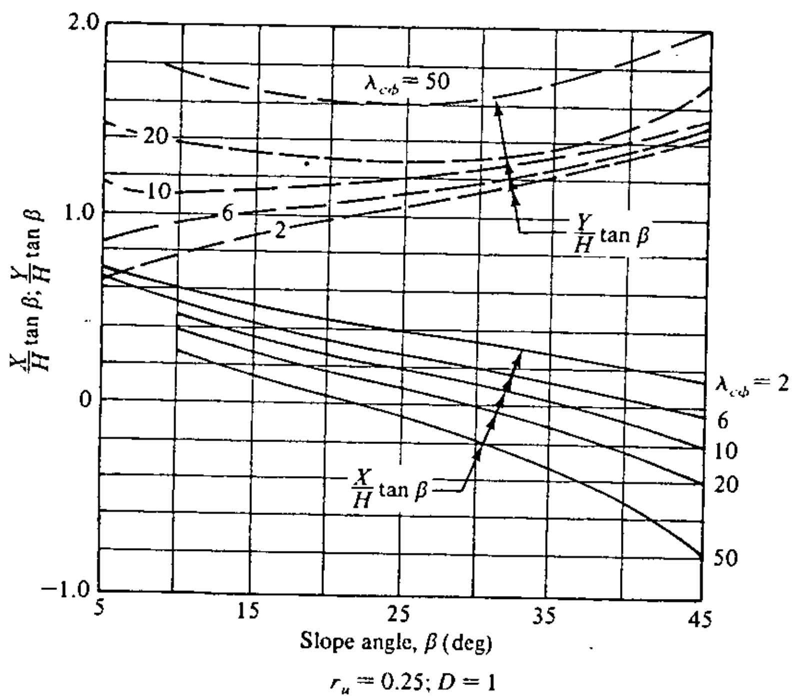


Fig. 3.14d Coordinates of critical slip circles for Depth factor D=1 and r_u =0.25 (after Cousin, 1978)

The first group of charts can be further subdivided in to those giving the stability number, N_F, for toe circles (Fig. 3.14a, r_u=0.25) and those giving the stability number, N_F, for circles with a given depth factor, D (Fig. 3.14b, r_u=0.25). The depth factor, D, can also be estimated for toe circles in Fig. 3.14a. In Fig. 3.14b the stability number, N_F, for the depth factor D= 1.25 has only been given where it is lower than the stability number, N_F, for the depth factor D= 1. Figs. 3.14c (r_u=0.25)and 3.14d (r_u=0.25)give the co-ordinates of the centre of rotation of the critical slip circles. All the charts are in non dimensional form. However, the values of X/H and Y/H have been multiplied by $\tan\alpha$ in order to make the charts easier to use. It is considered that this method of locating the critical slip circles is superior to specifying setting out angles as done by Spencer because it is more direct. Figs. 3.14c and 3.14d give the co-ordinates of the slip circles for toe circle and for circles with depth factor D=1 for values of the pore pressure ratio r_u = 0.25

All these charts have limitations or drawbacks that restrict their use (chowdhury 1978; Cousins, 1978). Taylor charts are the most well known charts and are strictly applicable for analysis in terms of 'total stress' approach only (Note that pore pressure u or pore pressure ratio r_u are not considered in these charts). Taylor used the friction circle method. In addition, an iterative procedure is required to determine the factor of safety for a given slope. Bishop and Morgenstern's (1960) charts are based on effective stresses. The charts are for a limited slope angle range (11^0-27^0) and a considerable amount of interpolation and extrapolation is required to determine the factor of safety. Also no information is given on the location of the critical slip circles. Spencer's (1967) charts are also based on effective stresses but they cover a wider slope angle range (up to 34^0) than Bishop and Morgenstern's charts. However, the charts are for toe circles only, and an iterative procedure is required to determine the factor of safety for a given slope. Janbu's (1967) charts are also for toe circles only, but because the number of charts has been considerably condensed the need for so much interpolation and extrapolation has been removed. One good feature of his charts is that all the information has been packed in a relatively small number of charts. Cousin's charts give results which are in good agreement with the results obtained by methods used by Bishop and Morgenstern's (1960), Spencer (1967) and Taylor (1937, 1948) for the range of values of parameters for which respective charts can be used. The Cousin's charts Fig. 3.14a-d have the advantage of not being overly congested and at the same time allow easy determination of the safety factor F. Also the charts allow easy determination of the slope height or angle for a required safety factor.

3.3 PROGRAM STABL

The two-dimensional limit equilibrium slope stability program PC-STABL (version 5M) was used to conduct this numerical study. This program was originally developed by Siegel (1975) and later modified by Boutrup et al (1979), Carpenter (1965, 1986), Verduin (1987). The program offers the user a choice of the following methods of slope stability analysis:

(a) Simplified Bishop method applicable to circular shaped failure surfaces
(b) Simplified Janbu method applicable to failure surfaces of general shape and the option of using Janbu's correction factor
(c) Spencer method applicable to any type of surface.

Complications which STABL is programmed to handle include the following: heterogeneous soil systems, anisotropic soil strength properties, excess pore water pressure due to shear, static groundwater and surface water, pseudo-static earthquake loading, surcharge boundary loading and tieback loading.

Plotting facilities are provided as a visual aid to confirm the correctness of problem input data, and to view the most critical failure surfaces obtained from the analysis.

STABL can generate any specified number (few hundreds) of trial failure surfaces in random fashion. Usually hundred surfaces are adequate. Each surface must meet the specified requirements given by the user. The option of surface generation include circular shapes, wedge shape (sliding block failure) and irregular surface of random shape. As each acceptable surface is generated, the corresponding factor of safety is calculated. The ten most critical surfaces having the lowest factor of safety are shown in the output and plotted so that the pattern may be studied. If the ten most critical surfaces form a thin zone and if the range in the value of the factor of safety for these ten surfaces is small, an additional refined search may not be necessary.

Trial circular and irregular shaped failure surfaces are approximated by series of straight line segments of equal length. Fig. 3.15 shows generation of a circular failure surface, which is started from the initiation point at an angle within the range specified by the user. The default range of angle for the initiation of the first line segment is such that an angle of 5^0 less than the inclination of ground surface would be one limit, while an angle of 45^0 downward from the horizontal would be another limit Fig. 3.16.

Seepage induced pore pressures may be incorporated either by specifying a piezometric surface coinciding with the phreatic surface or by specifying a pore pressure parameter. If the piezometric surface is specified, the following procedure is used in the current version of PCSTABL. Fig. 3.17 presents the seepage problem specified by a phreatic surface. The resulting pore pressures are computed as follows. The old method (used in earlier version 5 of STABL5) computes pore pressure based on hydrostatic pressure, i.e., the head is the vertical distance from the base of the slice to the phreatic surface immediately above Fig. 3.18. This pressure head can be as much as 30% higher than the actual head when the piezometric surface is dipping at 35^0. The perpendicular method approximates the equipotential line as a straight line from the base of the slice perpendicular to the line through the piezometric surface bounding the top of that slice. The pressure head can be as much as 10% lower than the actual head when the piezometric surface is dipping at 35^0. Since the old method is increasing in conservatism with steeper phreatic surface and the perpendicular method is increasing in nonconservatism, the average value of the two would tend to control the degree of conservatism.

Each soil type is described by the following set of isotropic parameters: moist unit weight, saturated unit weight, Mohr-Coulomb cohesion intercept, Mohr-Coulomb friction angle,

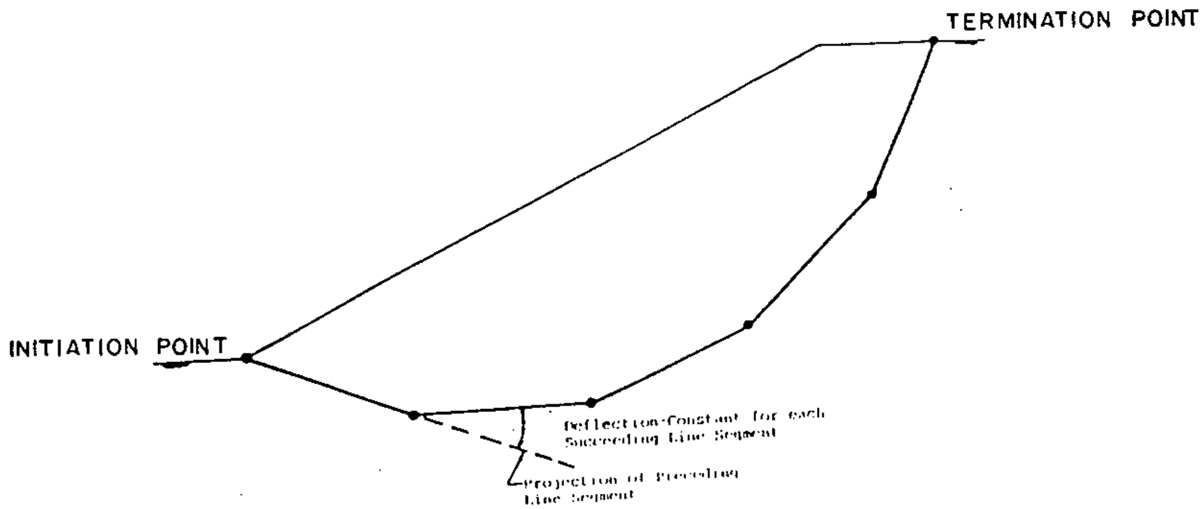


Fig. 3.15 Circular surface generation

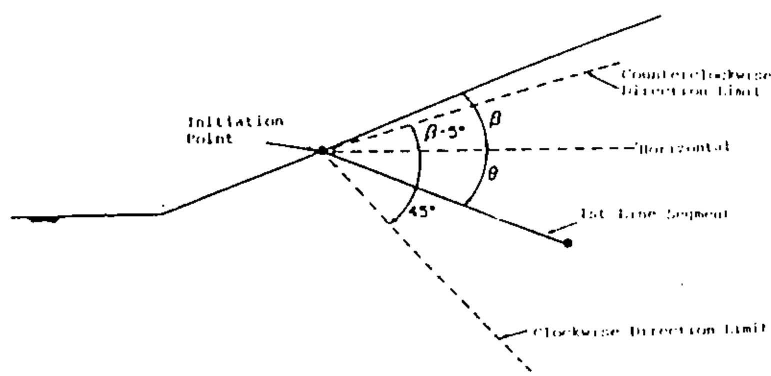


Fig. 3.16 Generation of first line segment

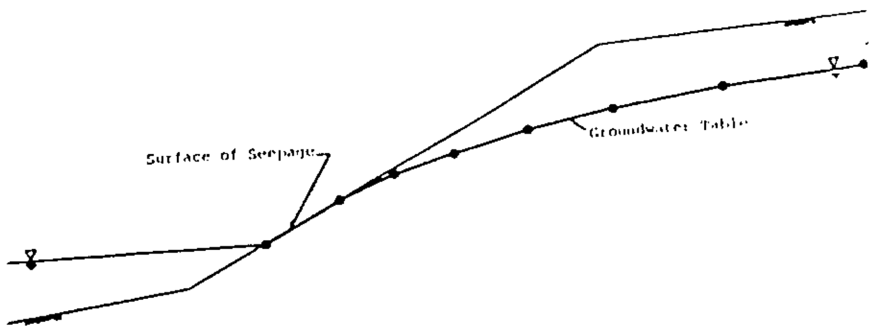


Fig. 3.17 Water surface defined across entire extent of defined problem

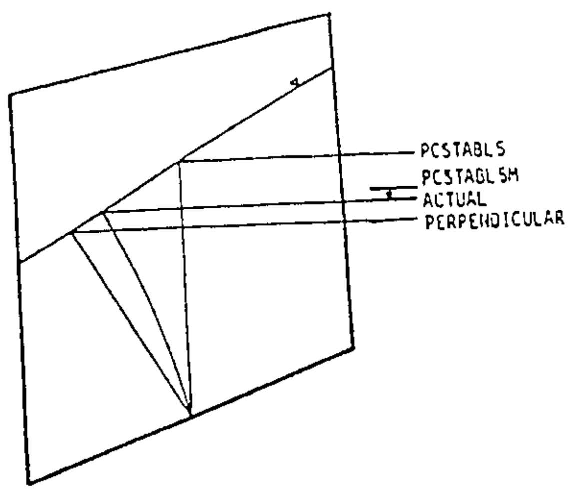


Fig. 3.18 Methods of Pore Pressure Determination

pore pressure parameter, pore pressure constant, piezometric surface. Either an effective stress analysis (c', φ') or total stress analysis (c, $\varphi=0$) may be performed by using the appropriate values for the Mohr-Coulomb strength parameters.

The pore pressure constant u_c of a soil type defines a constant pore pressure for any point within the soil. Another option is that pore water pressure may be assumed to be related to the overburden pressure and expressed by the pore pressure ratio r_u.

The use of earthquake coefficients allows for a pseudo-static representation of earthquake effects within the limiting equilibrium model. A direct relationship is assumed to exist between the pseudo-static earthquake force acting on the sliding mass and the weight of the sliding mass. Specified horizontal and vertical coefficients can be used to scale the horizontal and vertical components of the earthquake force relative to the weight of the sliding mass. Positive horizontal and vertical earthquake coefficients indicate that the horizontal and vertical components of the earthquake force are directed leftward and upward respectively where the slope is to the left negative coefficients are allowed. These inertial forces due to the seismic coefficients act at the centre of gravity of each slice. It is assumed that these forces do not change the pre-earthquake pore pressures in the slope.

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1GENERAL

Embankments constructed in the country for the purpose of roads, highways and flood control are usually built with locally available cohesive (silty clay, clayey silt) soil, which may also contain fine sand. Many of these embankments may be poorly compacted during construction and may thus have low shear strength parameters. Slope stability of these embankments may be a matter of critical importance under adverse environmental conditions.

During the monsoons many of these earthen embankments of the country are faced with high water table on one side. Resulting seepage flow through the embankment induces pore pressures which may result in a significant reduction in the factor of safety for the stability of the country-side slope. Occurrence of a major earthquake will further lower the factor of safety. This chapter presents results from an extensive numerical study of this problem.

4.2 NUMERICAL MODEL OF THE PROBLEM

Slope stability analysis is carried out using the computer programme PC-STABL, to calculate the factor of safety against the instability of a slope by the method of slices based on two-dimensional limiting equilibrium (approach is that Mohr-Coulomb's failure criterion is satisfied along the assumed failure surface which may be a straight line, circular arc, logarithmic spiral or other irregular surface) method commonly used in slope-stability analysis. The minimum factor of safety is obtained by both simplified Bishop's and simplified Janbu's method (with correction factor) for specified phreatic surface or pore pressure ratio and assumed circular slip surfaces. The actual shape of a slip surface, though curvilinear, is quite variable and the most commonly assumed shape is circular, as it simplifies the stability analysis and also it approximately coincides with the real shape of failure surface observed in nature. To incorporate the effect of seepage on the overall slope stability of the country-side slope, phreatic line is considered. Effective stress analysis is performed to account for the effect of seepage induced pore pressures and effective shear strength parameters (c', φ') are used. The Embankment is assumed to be homogeneous, isotropic and without any drainage filters. The phreatic line is drawn using Casagrande's (1937) method, which assumes homogeneous isotropic soil and impermeable boundary at the base of the embankment. In addition, Pseudo-static limit equilibrium procedure is used to assess the effect of earthquake induced horizontal forces.

Fig. 4.1 shows a schematic diagram of the problem studied, where an embankment is subjected to seepage flow due to high flood level (HFL) on the river side. The embankment height is H, crest width is b, and side slopes are s:1 (horizontal : vertical). F_B represents the freeboard. The factor of safety (F.S.) of the country side slope is determined assuming trial circular slip surfaces, similar to the one shown in figure. A rigorous trial procedure is adopted where hundreds of circular slip surfaces are generated in random fashion and factor of safety computed for each surface. The lowest F. S. thus obtained is taken as the F. S. for the country side slope.

4.2.1 Embankment Geometry

The height of embankments are dictated by the maximum flood level. Heights considered here are 3 m, 5 m, 8 m, 10 m and 12 m. As discussed earlier, a minimum freeboard of 0.8 m (2.6 ft) to 1.7 m (5.6 ft) is normally used in Bangladesh. Smaller the freeboard, more critical is the action of seepage on the stability of the country side slope. In this analysis two freeboards have been considered, 0.5 m (most critical condition) and 1.25 m. Considering possible critical embankment slopes in Bangladesh side slopes (HORIZONTAL : VERTICAL) are taken as 1:1, 1.5:1, 2:1, 2.5:1 and 3:1. For lower height (3 m, 5 m) embankments five side slopes are considered, where as for greater height embankments (8 m, 10 m, 12 m) four side slopes (1.5:1, 2:1, 2.5:1 and

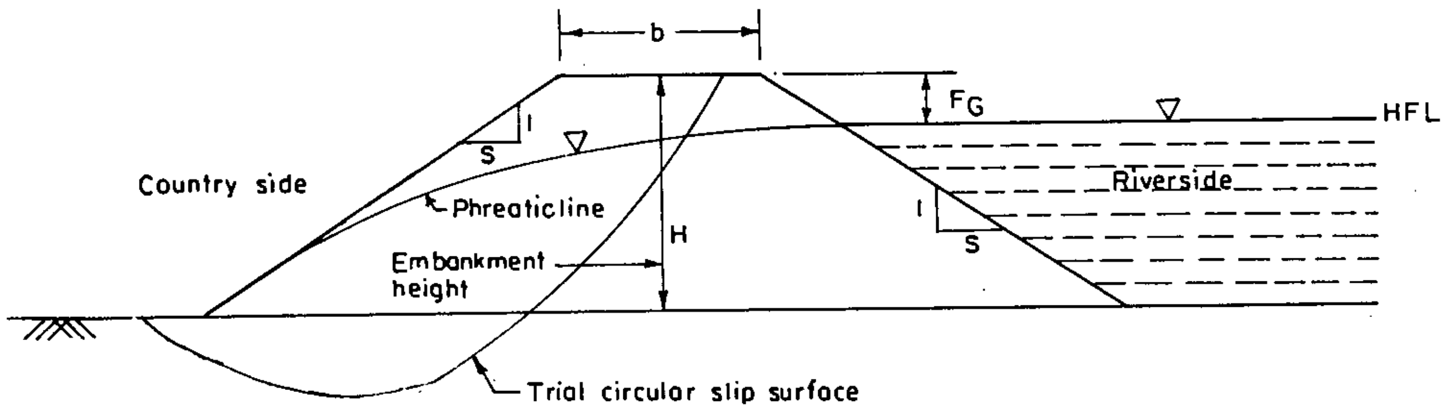


Fig. 4.1 Schematic diagram of the problem : Slope Stability Analysis of Embankment Subjected to steady seepage flow.

3:1) are considered. Based on guidelines presented in Chapter 2, the crest widths have been chosen.

Table 4.1 gives values of the different geometrical parameters of the embankment used in the study.

Table 4.1 Embankment Geometry

Sl. No.	Height H (m)	Slope s:1 (Hor : Vert)					Crest width b (m)
1	3	1:1	1.5:1	2:1	2.5:1	3:1	4
2	5	1:1	1.5:1	2:1	2.5:1	3:1	5
3	8	-	1.5:1	2:1	2.5:1	3:1	6
4	10	-	1.5:1	2:1	2.5:1	3:1	6
5	12	-	1.5:1	2:1	2.5:1	3:1	6

4.2.2 Phreatic Line

For study of the embankment under seepage (Fig. 4.1) the phreatic line needs to be constructed which depends on the embankment geometry and the location of the flood water level on the river side. The location of the phreatic line has been estimated and drawn using Casagrande's (1937) method. Twenty two different embankment geometries (Table 4.1) along with two different freeboards give rise to forty four cases of phreatic line construction. Fig. 4.2 to Fig. 4.6 show phreatic lines for embankment height of 3m, 5m, 8m, 10m and 12m corresponding to a freeboard of 0.5 m. Fig 4.7 to Fig. 4.11 represent the same but for a freeboard of 1.25 m. Table 4.2 presents values of the discharge face length 'a' for the different cases.

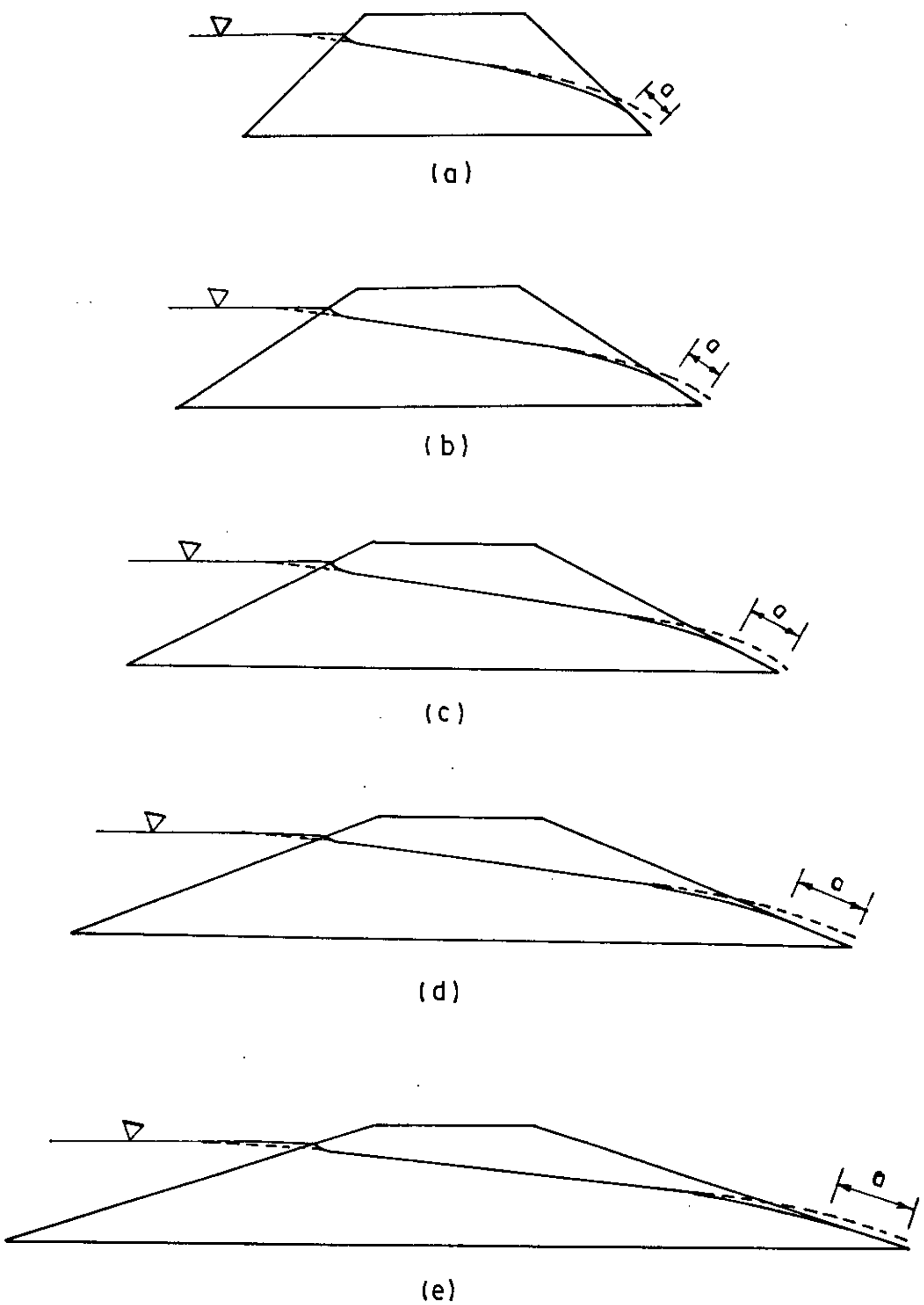


Fig. 4.2 Phreatic line for 3m high embankment with various slopes(F_B=0.5m)
(a) 1:1, (b) 1.5:1, (c) 2:1, (d) 2.5:1, (e) 3:1.

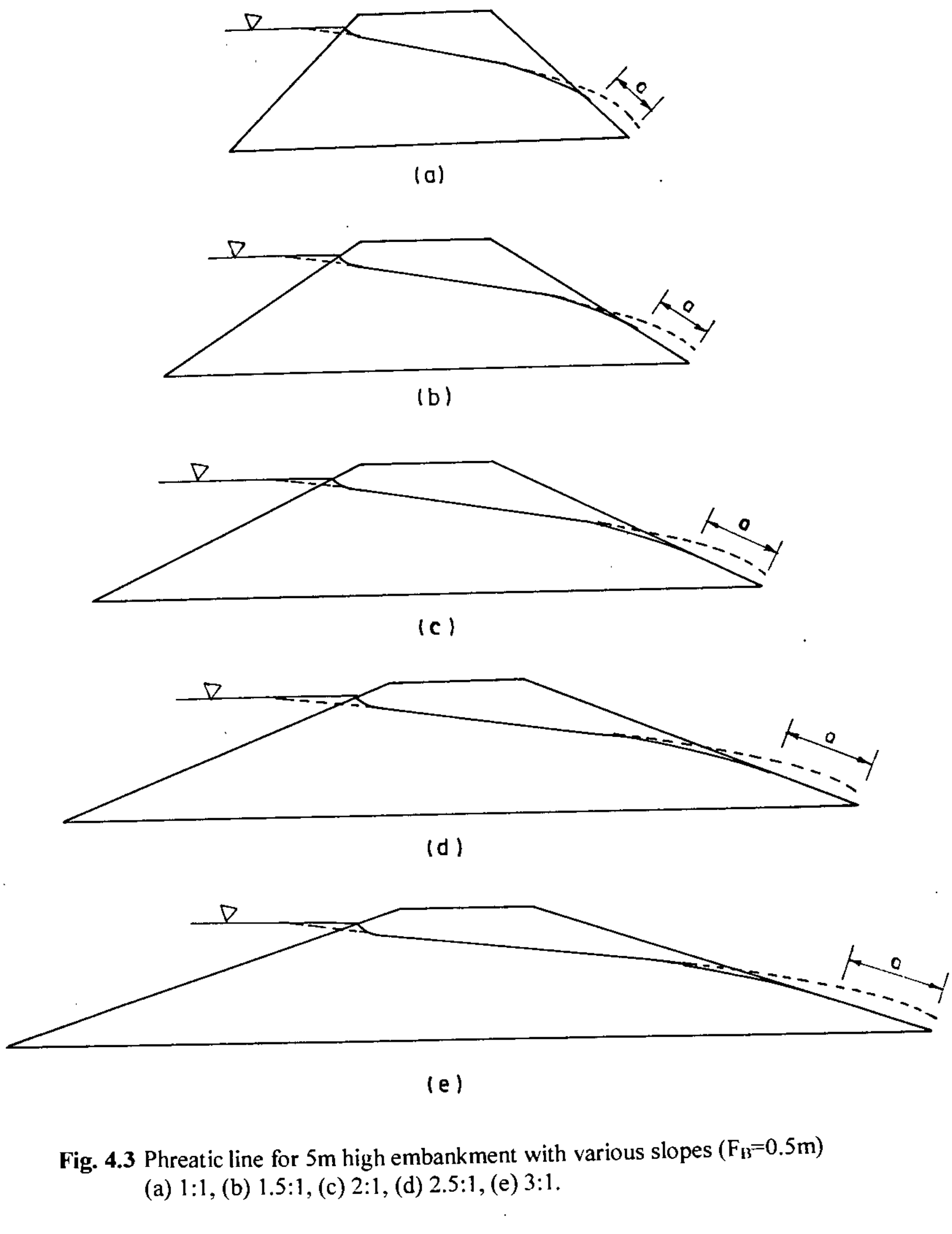


Fig. 4.3 Phreatic line for 5m high embankment with various slopes (F_B=0.5m)
(a) 1:1, (b) 1.5:1, (c) 2:1, (d) 2.5:1, (e) 3:1.

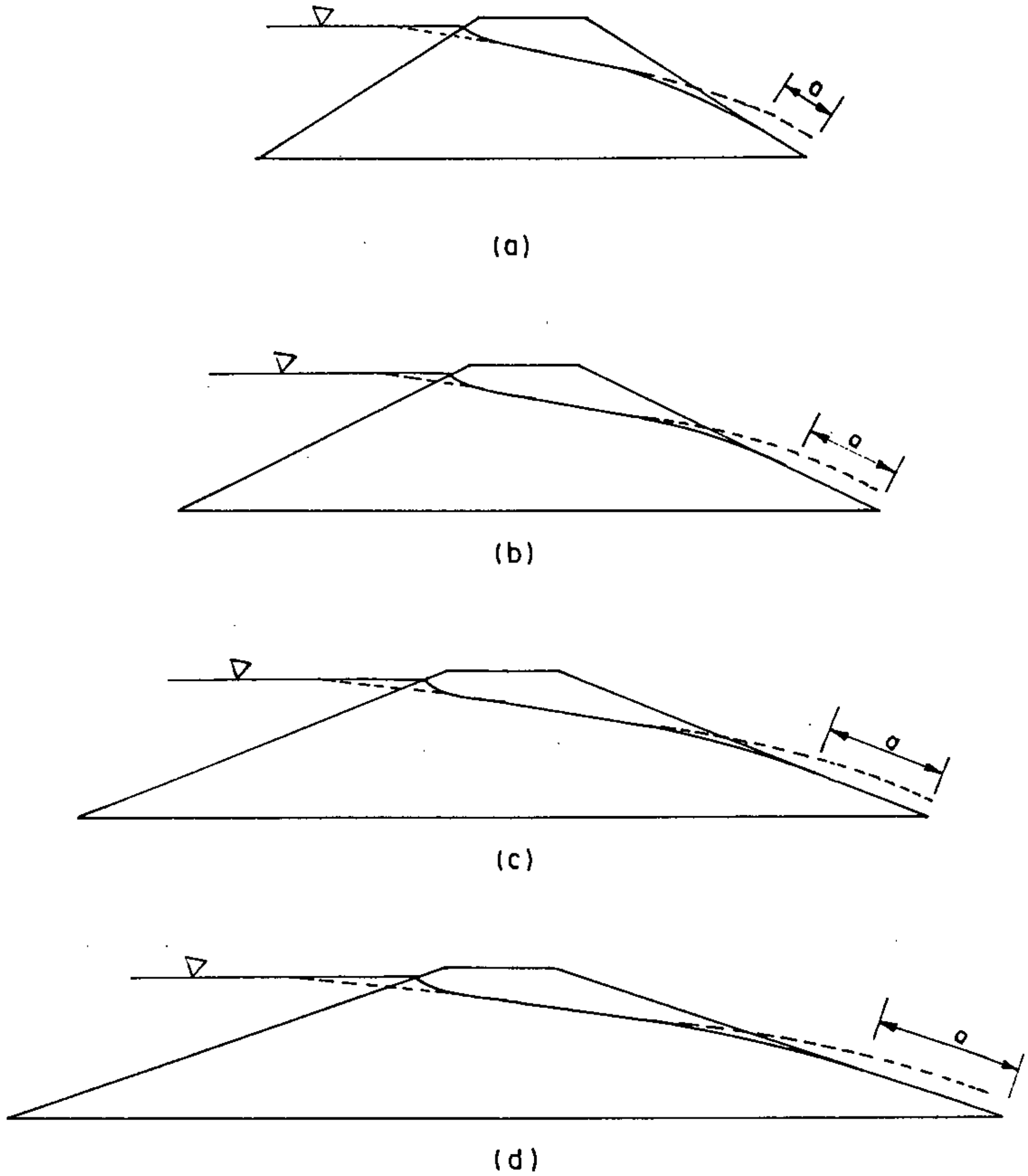


Fig. 4.4 Phreatic line for 8m high embankment with various slopes (F_B=0.5m) (a) 1.5:1, (b) 2:1, (c) 2.5:1, (d) 3:1.

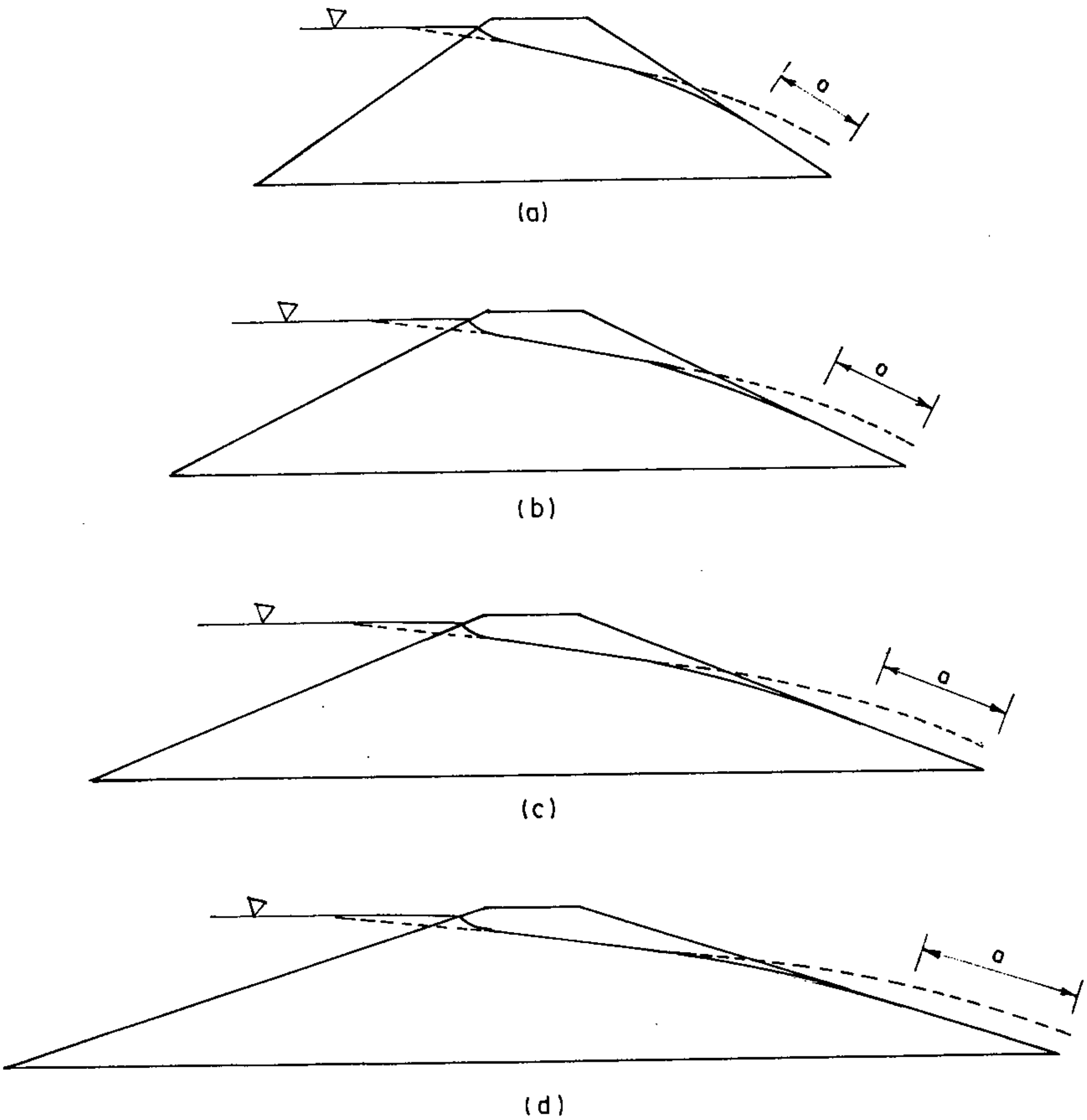


Fig. 4.5 Phreaticline for 10m high embankment with various slopes (F_B=0.5m)
(a) 1.5:1, (b) 2:1, (c) 2.5:1, (d) 3:1.

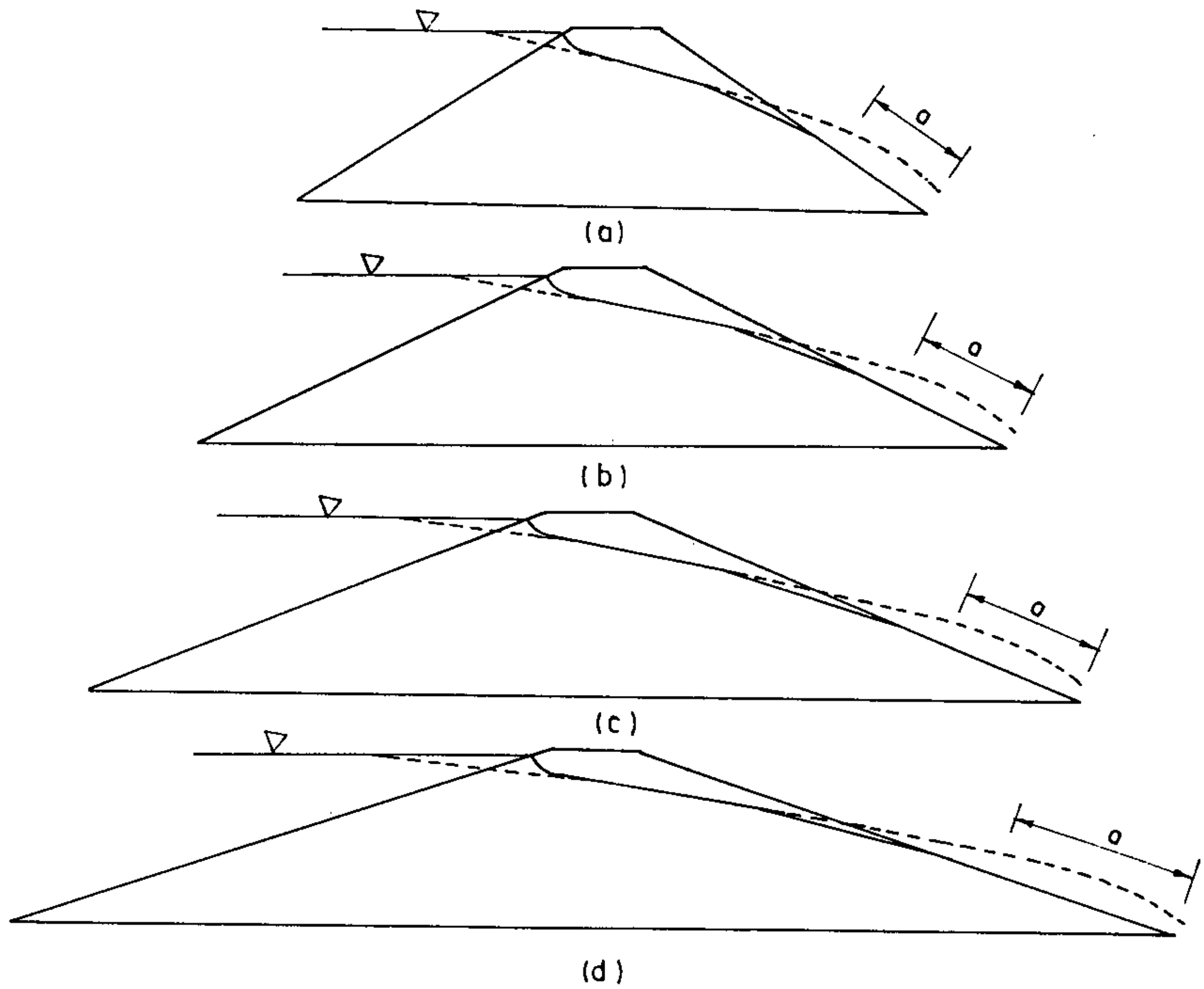


Fig. 4.6 Phreatic line for 12m high embankment with various slopes (F_B=0.5m) (a) 1.5:1, (b) 2:1, (c) 2.5:1, (d) 3:1.

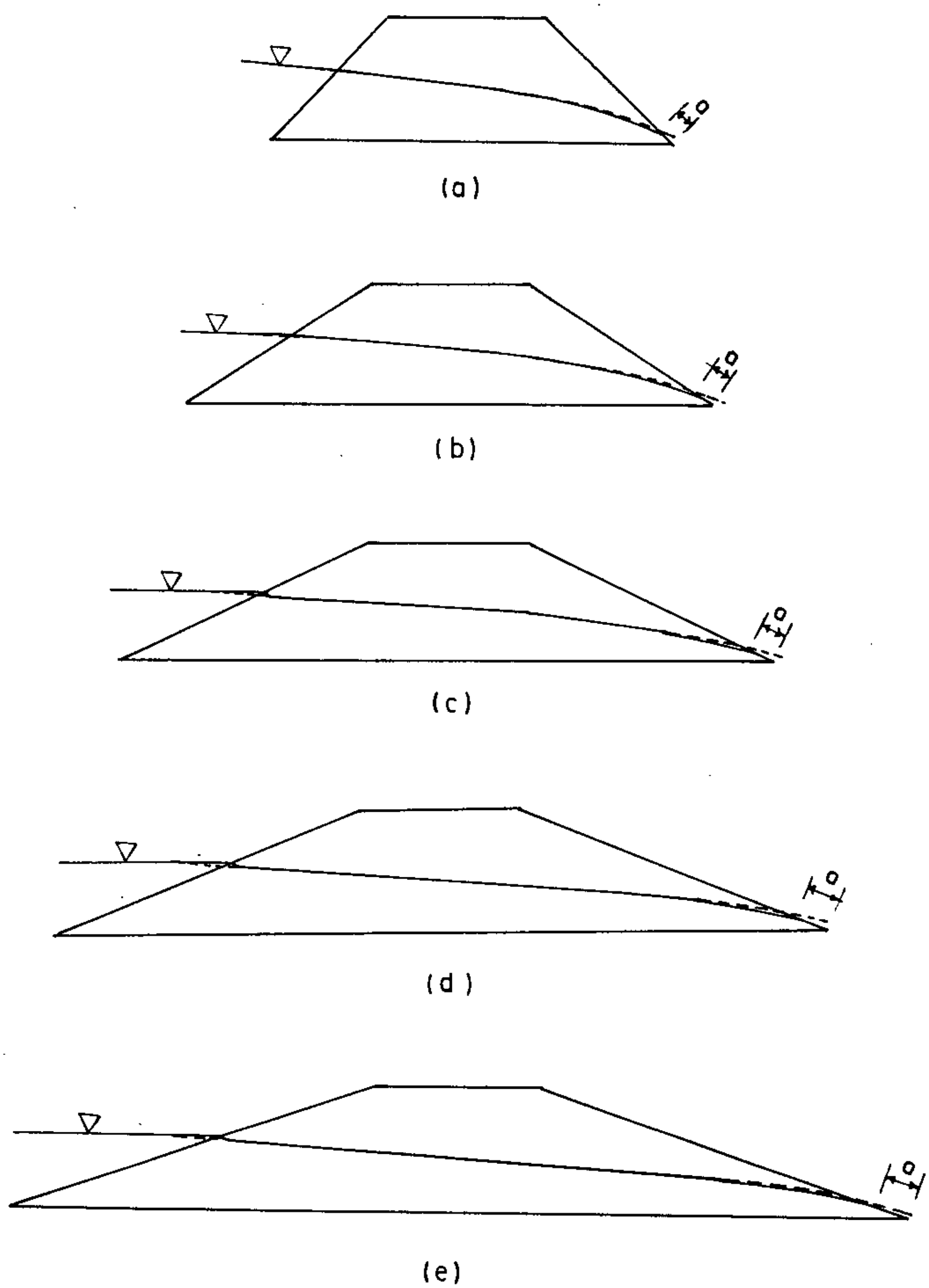


Fig. 4.7 Phreatic line for 3m high embankment with various slopes (F_B=1.25m) (a) 1:1, (b) 1.5:1, (c) 2:1, (d) 2.5:1, (e) 3:1.

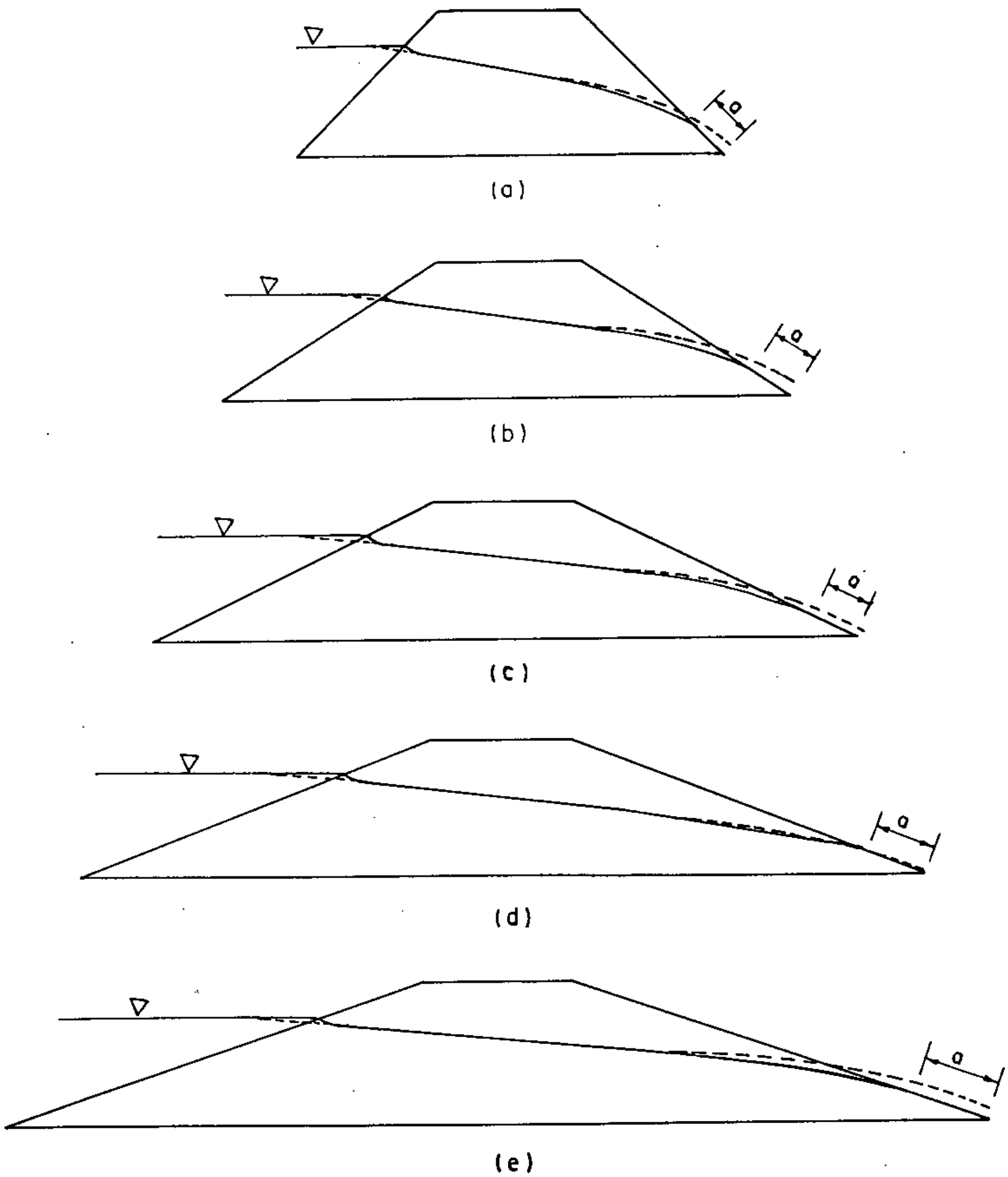


Fig. 4.8 Phreatic line for 5m high embankment with various slopes (F_B=1.25m)
(a) 1:1, (b) 1.5:1, (c) 2:1, (d) 2.5:1, (e) 3:1. 1

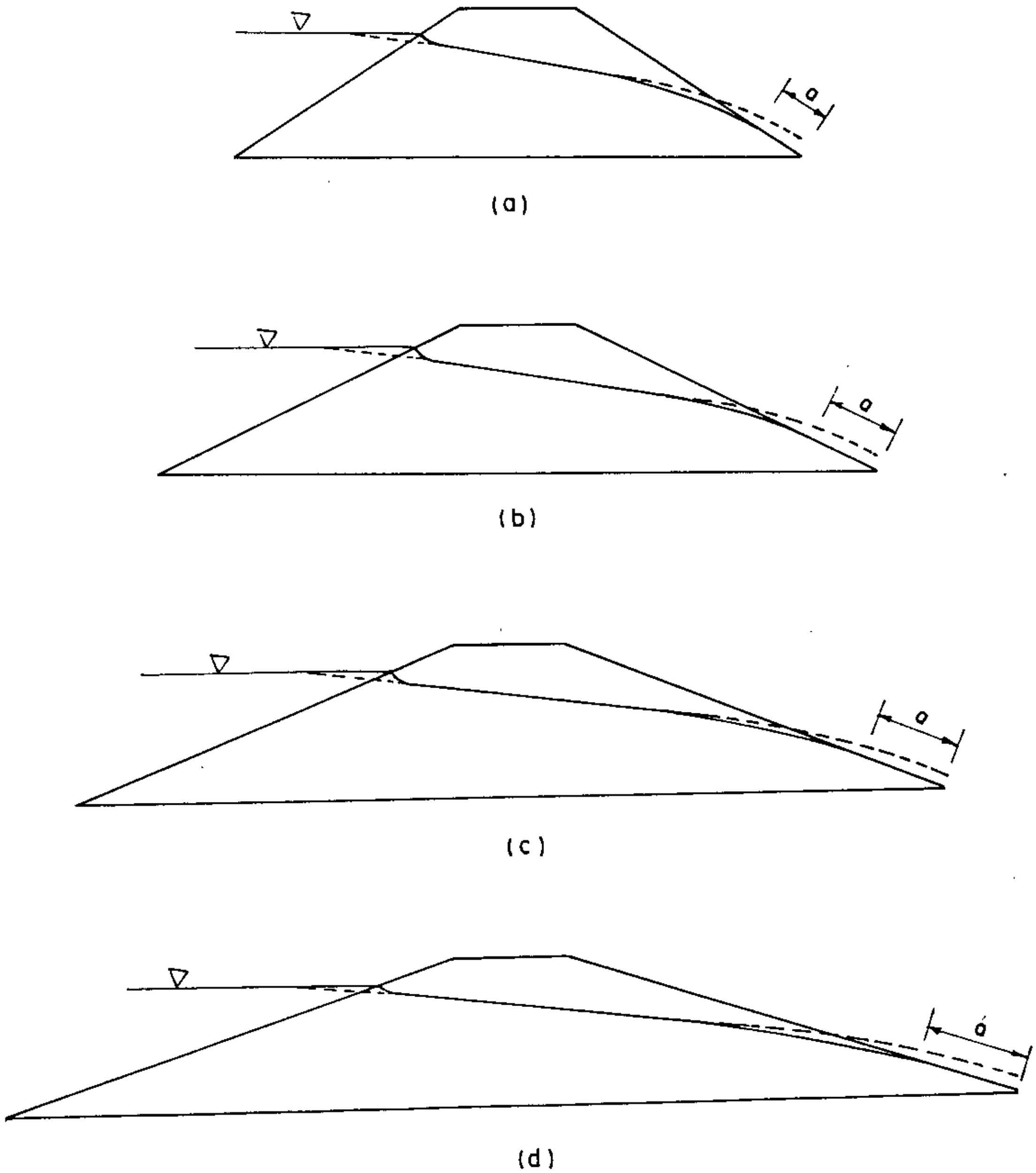
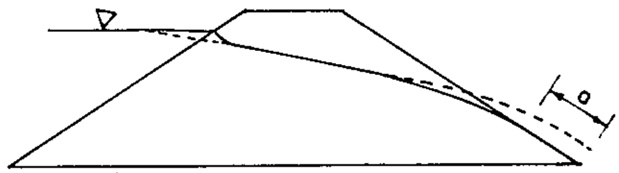
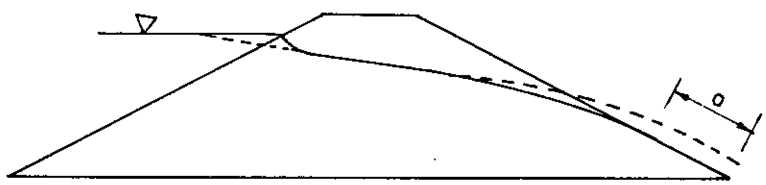


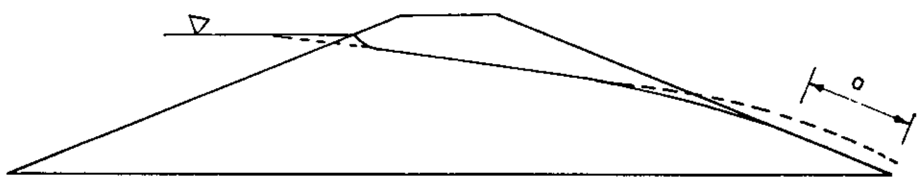
Fig. 4.9 Phreatic line for 8m high embankment with various slopes (F_B=1.25m)
(a) 1.5:1, (b) 2:1, (c) 2.5:1, (d) 3:1.



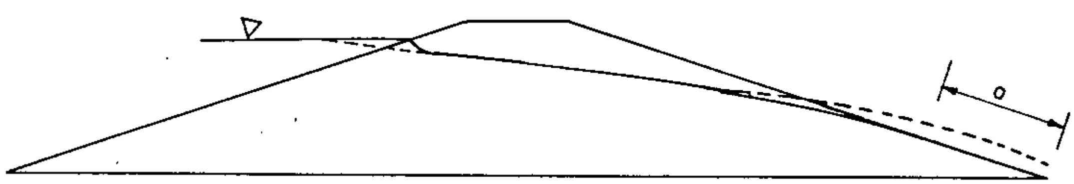
(a)



(b)



(c)



(d)

Fig. 4.10 Phreatic line for 10m high embankment with various slopes (F_B=1.25m) (a) 1.5:1, (b) 2:1, (c) 2.5:1, (d) 3:1.

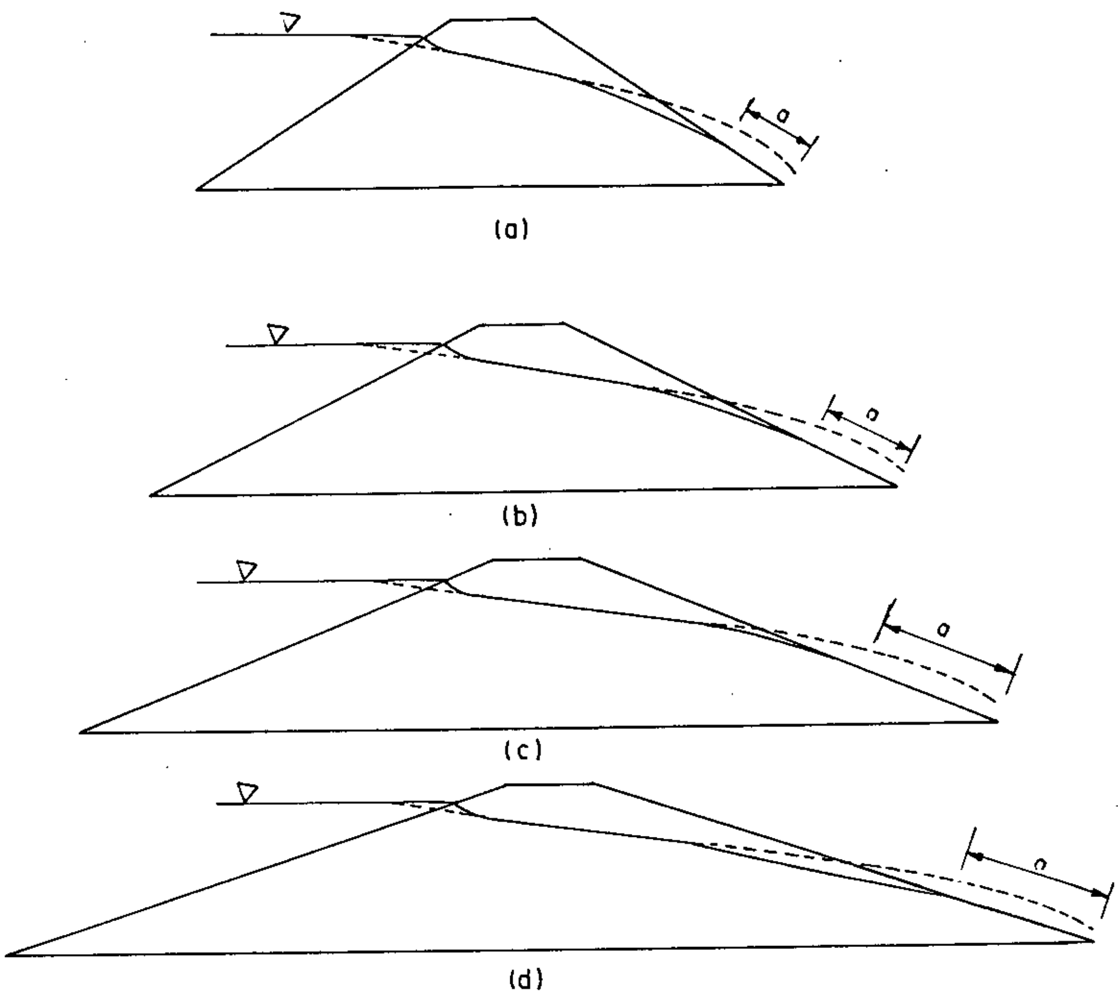


Fig. 4.11 Phreatic line for 12m high embankment with various slopes (F_B=1.25m)
(a) 1.5:1, (b) 2:1, (c) 2.5:1, (d) 3:1.

Table 4.2 Discharge face length 'a' (m) for different cases.

	Embankment Height, H									
	3 m		5 m		8 m		10 m		12 m	
Slope (Hor:Vert)	F_B=0.5m	F_B=1.25m	F_B=0.5m	F_B=1.25m	F_B=0.5m	F_B=1.25m	F_B=0.5m	F_B=1.25m	F_B=0.5m	F_B=1.25m
1:1	0.76	0.35	1.71	1.14	-	-	-	-	-	-
1.5:1	0.84	0.38	1.89	1.22	3.70	2.84	5.11	4.13	6.58	5.50
2:1	1.17	0.51	2.58	1.65	4.98	3.79	6.81	5.47	8.70	7.24
2.5:1	1.51	0.66	3.31	2.11	6.32	4.80	8.58	6.87	10.91	9.05
3:1	1.87	0.81	4.07	2.58	7.72	5.84	10.43	8.32	13.20	10.92

4.2.3 Soil Parameters

The Embankment is assumed to be homogeneous, isotropic and without any drainage filters. The Embankment material is cohesive and different values of effective shear strength parameters (c', φ') are considered. Values of shear strength parameter c' (cohesion intercept) considered are respectively 5 kPa, 10 kPa and 30 kPa while that for φ' (internal friction angle) are 10^0, 20^0, 30^0 and 40^0. The moist unit weight γ_{moist} (above phreatic surface) and saturated unit weight γ_{sat} (below phreatic surface) of soil are taken as 18 KN/m^3 and 19 KN/m^3 respectively. In addition, some analyses have been performed taking γ_{moist}=15 KN/m^3, γ_{sat} =16 KN/m^3 and γ_{moist} =21 KN/m^3, γ_{sat} =22 KN/m^3.

4.2.4 Cases Considered

Although the main emphasis is on the effect of seepage on stability of embankment slopes, the effect of earthquake is also studied in a simplified manner. Three cases have been considered. These are (a) no seepage (b) seepage and (c) seepage with earthquake.

(a) No seepage:- Slope stability analysis is performed to determine the factor of safety considering dry or moist condition without any ground water flow. In this case, the factor of safety is higher than the other two cases. This case needs to be studied to see the effect of seepage.

(b) Seepage:- As discussed in Art. 4.2, high water table on the river side of embankment during floods is considered as a critical seepage condition. The water level on the river side is considered to be 0.5 m and 1.25 m below the top crest level of the embankment. Freeboard of 0.5 m represents an extreme condition of flood.

(c) Seepage with earthquake:- Occurrence of earthquakes during flood can be considered a quite likely event. Simultaneous action of earthquake and seepage will further lower the factor of safety of flood control embankments. Equivalent static horizontal seismic coefficients of 0.15g and 0.25g are considered based on zones 2 and 3 of Bangladesh. The freeboard is assumed to be 0.5 m.

Some analyses have also been performed for the case of embankment on soft soil. The parameter $c^{/}$ is taken as half as that of the embankment soil. Pore pressure coefficient is used to represent seepage effects in this case.

4.2.5 Input Data

As mentioned in Art. 3.3, input data required for slope stability analysis using PCSTABL includes the following: problem geometry, soil parameters, location of phreatic surface, method of analysis, and specifications for trial failure surface generation. After several trials, the optimum specifications for trial failure surface generation were determined with the objective that the minimum factor of safety is obtained and no further trials are needed. Such specifications include location of initiation point, location of termination point, angle of failure surface at initiation point and segment length. Input for one of the problems studied is presented in Appendix A-1. The number of failure surfaces generated for this input file is 350. Also output data for the same problem is presented. Graphical plots obtained for the failure surfaces generated and for the ten most critical failure surfaces giving the lowest factor of safety are also presented.

4.3 RESULTS OF ANALYSIS

Extensive parametric studies have been conducted varying the different parameters as mentioned in Art 4.2. Results of this analysis are presented here. Unless otherwise specified, free board F_B=0.5 m, unit weights γ_{moist} =18 KN/m^3, γ_{sat} =19 KN/m^3. Also, plots and analysis are based on simplified Janbu's method of slope stability analysis.

4.3.1 Comparison of Bishop and Janbu Methods of Analysis

Both simplified Bishop method and simplified Janbu method with correction factor was used to analyse the slope stability of different embankment geometries for a freeboard of 0.5 m. The two methods have been used for a comparative study of three cases of no seepage, seepage and seepage with earthquake. Tables 4.3, 4.4, 4.5, 4.6 present results for comparison for the four conditions of no seepage, seepage, seepage with 0.15g and 0.25g earthquake. For the majority of cases, Bishop's method yields slightly higher factor of safety. In some cases with lower height embankments and steep slopes, the Janbu method gives slightly higher F. S. In general, the two methods agree reasonably well. The variation of factor of safety for most cases is 0 to 7%. Both methods are quite reliable for circular failure surfaces and effective stress analyses. It was thus decided to carry out further analysis with results obtained with the Janbu method. This method incorporates a correction factor given by Janbu and there is no need for considering inter-slice forces in this procedure.

4.3.2 Effect of Embankment Geometry

Figs. 4.12 to 4.15 show the effect of embankment height and side slope on the factor of safety for various cases. Figs. 4.12(a) to Figs 4.12(l) represent the case of different soil parameters c', ϕ' for seepage and no seepage conditions. Figs. 4.13(a) to Figs

Table 4.3 Comparison of F. S. obtained by Janbu and Bishop Method for no seepage condition (F_B= 0.5m)

c' (kPa)	φ' (deg)	Method	Factor of Safety (F. S.)																					
			H= 3m					H= 5m					H= 8m				H= 10m				H= 12m			
			1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V
5	10	Janbu	0.947	1.079	1.200	1.319	1.429	0.682	0.801	0.917	1.028	1.141	0.639	0.748	0.856	0.962	0.580	0.687	0.794	0.900	0.542	0.646	0.753	0.856
		Bishop	0.905	1.067	1.215	1.346	1.473	0.662	0.803	0.931	1.051	1.165	0.642	0.758	0.872	0.980	0.586	0.696	0.806	0.911	0.545	0.655	0.761	0.864
	20	Janbu	1.253	1.484	1.739	1.950	2.172	0.973	1.187	1.409	1.627	1.849	1.006	1.212	1.423	1.634	0.940	1.144	1.352	1.560	0.891	1.096	1.302	1.508
		Bishop	1.230	1.490	1.749	1.983	2.221	0.968	1.199	1.425	1.644	1.861	1.008	1.222	1.434	1.642	0.952	1.161	1.357	1.561	0.893	1.102	1.305	1.506
	30	Janbu	1.564	1.912	2.270	2.640	2.986	1.289	1.590	1.926	2.269	2.604	1.401	1.720	2.044	2.368	1.342	1.648	1.963	2.284	1.270	1.548	1.907	2.226
		Bishop	1.546	1.942	2.297	2.664	3.009	1.275	1.618	1.951	2.282	2.611	1.407	1.731	2.050	2.368	1.338	1.663	1.969	2.283	1.277	1.590	1.909	2.222
	40	Janbu	1.935	2.432	2.921	3.457	3.938	1.631	2.083	2.560	3.039	3.511	1.865	2.333	2.798	3.259	1.808	2.247	2.710	3.167	1.719	2.178	2.647	3.100
		Bishop	1.934	2.489	2.968	3.490	3.943	1.628	2.128	2.594	3.060	3.523	1.865	2.352	2.807	3.258	1.794	2.265	2.694	3.143	1.719	2.189	2.618	3.062
10	10	Janbu	1.569	1.738	1.863	1.983	2.098	1.076	1.211	1.335	1.453	1.571	0.907	1.024	1.137	1.250	0.800	0.915	1.028	1.139	0.729	0.841	0.952	1.062
		Bishop	1.478	1.693	1.863	2.018	2.160	1.018	1.191	1.344	1.481	1.613	0.904	1.037	1.162	1.282	0.801	0.930	1.049	1.165	0.732	0.855	0.971	1.085
	20	Janbu	1.912	2.182	2.432	2.679	2.905	1.381	1.625	1.863	2.092	2.326	1.299	1.525	1.746	1.966	1.181	1.403	1.623	1.840	1.104	1.320	1.539	1.751
		Bishop	1.829	2.162	2.465	2.732	2.991	1.343	1.630	1.893	2.139	2.375	1.306	1.544	1.780	2.002	1.195	1.420	1.646	1.863	1.112	1.335	1.556	1.769
	30	Janbu	2.259	2.659	3.049	3.400	3.765	1.708	2.072	2.439	2.790	3.132	1.715	2.058	2.400	2.742	1.591	1.927	2.262	2.597	1.504	1.834	2.169	2.499
		Bishop	2.197	2.653	3.105	3.487	3.857	1.689	2.096	2.473	2.823	3.181	1.734	2.083	2.428	2.766	1.612	1.952	2.281	2.613	1.514	1.858	2.179	2.505
	40	Janbu	2.677	3.196	3.756	4.269	4.769	2.115	2.582	3.086	3.590	4.091	2.220	2.689	3.174	3.655	2.089	2.550	3.024	3.499	1.977	2.451	2.920	3.390
		Bishop	2.627	3.220	3.784	4.321	4.853	2.115	2.615	3.127	3.624	4.122	2.222	2.709	3.188	3.663	2.112	2.589	3.033	3.498	1.985	2.457	2.927	3.388
30	10	Janbu	4.043	4.174	4.286	4.420	4.557	2.541	2.748	2.869	2.995	3.116	1.920	2.043	2.157	2.278	1.633	1.754	1.867	1.987	1.448	1.588	1.670	1.787
		Bishop	3.685	4.020	4.219	4.404	4.585	2.359	2.641	2.828	3.003	3.173	1.847	2.028	2.186	2.336	1.574	1.750	1.904	2.043	1.401	1.562	1.708	1.845
	20	Janbu	4.404	4.785	5.014	5.274	5.530	2.908	3.248	3.496	3.741	3.993	2.344	2.600	2.839	3.078	2.045	2.289	2.526	2.762	1.838	2.077	2.311	2.546
		Bishop	4.094	4.619	4.985	5.332	5.643	2.737	3.163	3.511	3.802	4.087	2.313	2.621	2.900	3.162	2.026	2.317	2.585	2.840	1.830	2.112	2.368	2.617
	30	Janbu	4.797	5.338	5.756	6.141	6.516	3.313	3.743	4.146	4.535	4.914	2.824	3.204	3.572	3.946	2.502	2.874	3.241	3.603	2.286	2.652	3.012	3.370
		Bishop	4.531	5.211	5.752	6.250	6.701	3.140	3.692	4.178	4.620	5.043	2.819	3.255	3.651	4.039	2.508	2.926	3.306	3.681	2.296	2.691	3.068	3.436
	40	Janbu	5.280	5.969	6.556	7.113	7.659	3.745	4.345	4.898	5.444	5.976	3.376	3.904	4.433	4.950	3.047	3.563	4.083	4.592	2.818	3.333	3.839	4.342
		Bishop	5.016	5.858	6.607	7.268	7.882	3.606	4.321	4.962	5.557	6.123	3.404	3.979	4.532	5.059	3.066	3.614	4.149	4.673	2.839	3.372	3.889	4.398

Table 4.4 Comparison of F. S. obtained by Janbu and Bishop Method for seepage condition (F_B= 0.5m)

C'. (kPa)	φ' (deg)	Method	Factor of Safety (F. S.)																					
			H= 3m					H= 5m					H= 8m				H= 10m				H= 12m			
			1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V
5	10	Janbu	0.859	0.925	0.979	1.037	1.092	0.584	0.646	0.705	0.758	0.814	0.472	0.533	0.583	0.655	0.417	0.466	0.524	0.591	0.392	0.431	0.499	0.539
		Bishop	0.808	0.908	0.988	1.061	1.125	0.558	0.641	0.717	0.779	0.842	0.473	0.544	0.600	0.675	0.421	0.478	0.540	0.609	0.397	0.444	0.514	0.554
	20	Janbu	1.110	1.233	1.342	1.462	1.567	0.797	0.909	1.026	1.133	1.246	0.695	0.816	0.915	1.063	0.633	0.731	0.846	0.979	0.615	0.694	0.830	0.906
		Bishop	1.066	1.226	1.366	1.499	1.618	0.769	0.911	1.053	1.163	1.280	0.704	0.832	0.936	1.086	0.644	0.747	0.864	0.999	0.622	0.710	0.847	0.921
	30	Janbu	1.380	1.556	1.720	1.907	2.076	1.009	1.187	1.375	1.533	1.711	0.939	1.119	1.281	1.511	0.864	1.019	1.200	1.408	0.864	0.981	1.195	1.312
		Bishop	1.349	1.547	1.774	1.960	2.131	0.992	1.196	1.407	1.574	1.752	0.953	1.133	1.297	1.532	0.879	1.031	1.214	1.425	0.871	0.995	1.210	1.321
	40	Janbu	1.691	1.910	2.177	2.439	2.675	1.262	1.515	1.783	2.021	2.270	1.212	1.480	1.712	2.052	1.140	1.357	1.620	1.923	1.149	1.321	1.633	1.798
		Bishop	1.656	1.916	2.245	2.521	2.754	1.245	1.519	1.814	2.068	2.325	1.238	1.496	1.721	2.068	1.152	1.371	1.629	1.936	1.164	1.333	1.643	1.799
10	10	Janbu	1.467	1.521	1.570	1.634	1.694	0.946	1.014	1.074	1.127	1.190	0.713	0.776	0.826	0.902	0.612	0.663	0.723	.0.794	0.558	0.598	0.668	0.711
		Bishop	1.352	1.481	1.567	1.649	1.724	0.886	0.999	1.082	1.150	1.224	0.705	0.788	0.851	0.932	0.610	0.679	0.748	0.823	0.559	0.616	0.693	0.739
	20	Janbu	1.733	1.869	1.980	2.100	2.213	1.181	1.308	1.431	1.538	1.656	0.957	1.083	1.186	1.335	0.848	0.948	1.068	1.206	0.796	0.879	1.018	1.101
		Bishop	1.631	1.836	2.001	2.148	2.283	1.129	1.299	1.455	1.581	1.712	0.960	1.107	1.221	1.376	0.857	0.973	1.100	1.242	0.807	0.904	1.048	1.131
	30	Janbu	2.020	2.219	2.400	2.589	2.765	1.427	1.615	1.800	1.967	2.148	1.212	1.409	1.569	1.802	1.101	1.254	1.438	1.651	1.051	1.180	1.399	1.523
		Bishop	1.923	2.199	2.439	2.658	2.846	1.380	1.610	1.843	2.024	2.220	1.229	1.440	1.608	1.850	1.114	1.284	1.474	1.690	1.070	1.208	1.432	1.553
	40	Janbu	2.357	2.643	2.881	3.168	3.405	1.710	1.961	2.232	2.474	2.738	1.518	1.792	2.023	2.359	1.388	1.613	1.876	2.183	1.362	1.538	1.850	2.024
		Bishop	2.283	2.627	2.946	3.240	3.508	1.651	1.975	2.291	2.541	2.811	1.536	1.822	2.062	2.407	1.414	1.640	1.910	2.222	1.374	1.566	1.885	2.053
30	10	Janbu	3.746	3.794	3.841	3.923	4.006	2.367	2.461	2.515	2.567	2.642	1.683	1.733	1.772	1.851	1.411	1.446	1.498	1.570	1.244	1.265	1.330	1.368
		Bishop	3.460	3.616	3.737	3.868	3.989	2.167	2.326	2.447	2.551	2.657	1.577	1.695	1.780	1.885	1.325	1.424	1.516	1.610	1.168	1.253	1.354	1.415
	20	Janbu	4.108	4.201	4.304	4.444	4.578	2.614	2.763	2.893	3.000	3.135	1.920	2.044	2.149	2.305	1.630	1.731	1.856	2.004	1.467	1.544	1.689	1.781
		Bishop	3.801	4.081	4.268	4.455	4.625	2.435	2.694	2.881	3.044	3.212	1.871	2.061	2.200	2.385	1.600	1.758	1.915	2.079	1.447	1.577	1.747	1.855
	30	Janbu	4.468	4.663	4.828	5.029	5.222	2.901	3.122	3.318	3.492	3.696	2.202	2.410	2.572	2.818	1.893	2.063	2.259	2.489	1.733	1.867	2.095	2.235
		Bishop	4.130	4.540	4.818	5.087	5.323	2.721	3.077	3.347	3.567	3.802	2.182	2.450	2.651	2.917	1.895	2.115	2.337	2.579	1.741	1.923	2.172	2.323
	40	Janbu	4.839	5.183	5.404	5.967	5.957	3.233	3.542	3.822	4.073	4.351	2.546	2.847	3.081	3.428	2.224	2.466	2.745	3.069	2.067	2.262	2.587	2.783
		Bishop	4.518	5.049	5.426	5.801	6.111	3.071	3.503	3.888	4.177	4.492	2.548	2.904	3.175	3.541	2.245	2.530	2.835	3.170	2.089	2.332	2.673	2.876

Table 4.5 Comparison of F. S. obtained by Janbu and Bishop Method for seepage plus earthquake (0.15g) condition (F_B= 0.5m)

C'. (kPa)	φ' (deg)	Method	Factor of Safety (F. S.)																					
			H= 3m					H= 5m					H= 8m				H= 10m				H= 12m			
			1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V
5	10	Janbu	0.636	0.644	0.652	0.663	0.669	0.442	0.465	0.487	0.500	0.517	0.345	0.374	0.392	0.421	0.305	0.328	0.353	0.382	0.287	0.304	0.336	0.349
		Bishop	0.647	0.674	0.683	0.694	0.702	0.446	0.485	0.512	0.525	0.541	0.359	0.392	0.411	0.440	0.319	0.345	0.370	0.398	0.300	0.319	0.352	0.363
	20	Janbu	0.848	0.895	0.934	0.972	0.997	0.606	0.668	0.723	0.761	0.803	0.508	0.554	0.618	0.685	0.463	0.514	0.570	0.632	0.447	0.487	0.559	0.584
		Bishop	0.855	0.928	0.979	1.017	1.038	0.616	0.692	0.757	0.794	0.834	0.528	0.598	0.640	0.706	0.482	0.533	0.588	0.649	0.465	0.504	0.576	0.597
	30	Janbu	1.053	1.140	1.213	1.286	1.335	0.770	0.869	0.969	1.034	1.105	0.683	0.787	0.860	0.971	0.624	0.710	0.803	0.904	0.621	0.682	0.800	0.841
		Bishop	1.071	1.175	1.275	1.340	1.387	0.794	0.904	1.009	1.073	1.141	0.708	0.808	0.880	0.994	0.655	0.732	0.821	0.922	0.643	0.702	0.819	0.853
	40	Janbu	1.293	1.415	1.534	1.643	1.729	0.951	1.108	1.257	1.360	1.464	0.876	1.030	1.147	1.314	0.818	0.940	1.081	1.231	0.825	0.913	1.090	1.149
		Bishop	1.317	1.455	1.611	1.710	1.784	0.983	1.141	1.294	1.401	1.502	0.918	1.059	1.164	1.335	0.851	0.963	1.098	1.249	0.856	0.934	1.109	1.159
10	10	Janbu	1.032	1.024	1.018	1.017	1.014	0.709	0.724	0.735	0.738	0.747	0.522	0.544	0.556	0.579	0.452	0.469	0.490	0.515	0.414	0.425	0.454	0.463
		Bishop	1.056	1.055	1.055	1.056	1.054	0.705	0.748	0.767	0.777	0.788	0.538	0.572	0.586	0.612	0.467	0.495	0.519	0.542	0.429	0.449	0.481	0.489
	20	Janbu	1.288	1.304	1.322	1.345	1.359	0.893	0.944	0.989	1.017	1.052	0.699	0.761	0.798	0.858	0.617	0.667	0.720	0.778	0.582	0.618	0.686	0.711
		Bishop	1.307	1.365	1.384	1.409	1.424	0.903	0.983	1.039	1.066	1.100	0.729	0.798	0.836	0.896	0.648	0.702	0.754	0.812	0.610	0.649	0.718	0.740
	30	Janbu	1.536	1.608	1.654	1.705	1.739	1.085	1.180	1.266	1.320	1.381	0.888	0.993	1.059	1.162	0.800	0.881	0.970	1.066	0.767	0.830	0.941	0.981
		Bishop	1.547	1.671	1.735	1.780	1.811	1.102	1.227	1.326	1.379	1.437	0.929	1.038	1.101	1.202	0.837	0.918	1.005	1.100	0.801	0.862	0.974	1.009
	40	Janbu	1.801	1.919	2.019	2.110	2.178	1.302	1.443	1.576	1.668	1.764	1.106	1.262	1.364	1.523	1.013	1.130	1.262	1.407	0.985	1.075	1.243	1.303
		Bishop	1.823	1.992	2.118	2.213	2.266	1.323	1.498	1.647	1.737	1.830	1.150	1.305	1.408	1.564	1.053	1.171	1.301	1.442	1.022	1.113	1.281	1.330
30	10	Janbu	2.632	2.552	2.491	2.447	2.405	1.811	1.782	1.728	1.689	1.667	1.261	1.230	1.199	1.197	1.067	1.036	1.020	1.022	0.944	0.910	0.908	0.894
		Bishop	2.579	2.545	2.510	2.480	2.447	1.725	1.743	1.738	1.722	1.711	1.223	1.237	1.234	1.244	1.034	1.052	1.059	1.070	0.915	0.929	0.949	0.945
	20	Janbu	2.871	2.822	2.786	2.765	2.741	1.974	1.984	1.974	1.964	1.968	1.416	1.442	1.444	1.482	1.210	1.229	1.260	1.298	1.093	1.100	1.151	1.161
		Bishop	2.895	2.875	2.860	2.848	2.829	1.929	2.017	2.046	2.048	2.059	1.441	1.501	1.523	1.572	1.234	1.289	1.334	1.378	1.120	1.160	1.218	1.231
	30	Janbu	3.164	3.144	3.133	3.136	3.130	2.173	2.228	2.270	2.286	2.320	1.612	1.690	1.729	1.811	1.397	1.459	1.530	1.613	1.282	1.325	1.423	1.455
		Bishop	3.244	3.251	3.253	3.262	3.260	2.167	2.307	2.373	2.406	2.450	1.662	1.774	1.825	1.911	1.450	1.537	1.619	1.698	1.335	1.402	1.504	1.533
	40	Janbu	3.519	3.538	3.555	3.595	3.612	2.429	2.537	2.624	2.680	2.751	1.860	1.995	2.070	2.202	1.625	1.737	1.855	1.983	1.518	1.597	1.749	1.804
		Bishop	3.612	3.684	3.720	3.769	3.786	2.451	2.640	2.757	2.819	2.890	1.928	2.093	2.178	2.311	1.703	1.829	1.950	2.077	1.585	1.688	1.840	1.888

Table 4.6 Comparison of F. S. obtained by Janbu and Bishop Method for seepage plus earthquake (0.25g) condition (F_B= 0.5m)

c'. (kPa)	φ' (deg)	Method	Factor of Safety (F. S.)																					
			H= 3m					H= 5m					H= 8m				H= 10m				H= 12m			
			1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V
5	10	Janbu	0.525	0.526	0.525	0.526	0.525	0.371	0.386	0.399	0.404	0.413	0.288	0.308	0.318	0.338	0.254	0.270	0.287	0.306	0.240	0.250	0.274	0.280
		Bishop	0.557	0.557	0.555	0.558	0.556	0.386	0.410	0.425	0.428	0.433	0.305	0.327	0.336	0.354	0.271	0.288	0.303	0.321	0.255	0.266	0.288	0.292
	20	Janbu	0.714	0.744	0.761	0.776	0.785	0.512	0.555	0.595	0.616	0.640	0.422	0.472	0.500	0.546	0.384	0.420	0.461	0.504	0.370	0.397	0.451	0.465
		Bishop	0.744	0.787	0.802	0.812	0.817	0.535	0.587	0.628	0.645	0.665	0.446	0.494	0.520	0.565	0.405	0.440	0.477	0.519	0.391	0.416	0.467	0.477
	30	Janbu	0.891	0.949	0.999	1.038	1.064	0.648	0.722	0.798	0.839	0.883	0.561	0.643	0.693	0.773	0.514	0.578	0.647	0.719	0.509	0.554	0.643	0.667
		Bishop	0.930	1.002	1.057	1.089	1.102	0.685	0.766	0.838	0.873	0.912	0.592	0.666	0.714	0.793	0.549	0.601	0.663	0.735	0.535	0.576	0.661	0.679
	40	Janbu	1.090	1.181	1.264	1.333	1.381	0.793	0.921	1.032	1.097	1.166	0.721	0.837	0.919	1.042	0.668	0.761	0.866	0.976	0.674	0.738	0.872	0.907
		Bishop	1.141	1.236	1.335	1.392	1.427	0.847	0.961	1.069	1.135	1.201	0.758	0.866	0.939	1.063	0.706	0.785	0.884	0.991	0.704	0.761	0.892	0.914
10	10	Janbu	0.851	0.833	0.818	0.807	0.795	0.602	0.604	0.602	0.596	0.595	0.439	0.451	0.454	0.466	0.380	0.389	0.801	0.415	0.348	0.353	0.371	0.374
		Bishop	0.892	0.872	0.858	0.846	0.833	0.612	0.637	0.639	0.636	0.634	0.461	0.481	0.483	0.495	0.402	0.415	0.428	0.439	0.369	0.378	0.397	0.396
	20	Janbu	1.063	1.065	1.065	1.068	1.066	0.751	0.783	0.810	0.822	0.839	0.584	0.627	0.648	0.688	0.515	0.549	0.585	0.624	0.485	0.510	0.558	0.570
		Bishop	1.127	1.130	1.127	1.131	1.128	0.782	0.831	0.862	0.869	0.881	0.619	0.665	0.683	0.721	0.550	0.585	0.616	0.654	0.517	0.539	0.587	0.595
	30	Janbu	1.295	1.317	1.337	1.358	1.369	0.915	0.983	1.037	1.066	1.101	0.739	0.817	0.858	0.927	0.665	0.723	0.784	0.851	0.637	0.680	0.761	0.782
		Bishop	1.344	1.399	1.410	1.423	1.427	0.959	1.038	1.100	1.120	1.148	0.784	0.858	0.895	0.964	0.704	0.758	0.817	0.880	0.673	0.712	0.792	0.807
	40	Janbu	1.528	1.601	1.653	1.696	1.719	1.096	1.202	1.297	1.352	1.410	0.919	1.035	1.103	1.212	0.838	0.924	1.020	1.121	0.811	0.877	1.002	1.036
		Bishop	1.587	1.692	1.746	1.773	1.786	1.146	1.268	1.367	1.412	1.461	0.967	1.080	1.143	1.250	0.885	0.964	1.054	1.151	0.855	0.913	1.035	1.059
30	10	Janbu	2.182	2.088	2.009	1.948	1.891	1.560	1.490	1.423	1.370	1.332	1.071	1.021	0.980	0.963	0.909	0.863	0.836	0.824	0.805	0.759	0.745	0.723
		Bishop	2.176	2.111	2.047	1.993	1.939	1.511	1.484	1.451	1.412	1.380	1.059	1.045	1.022	1.011	0.898	0.892	0.880	0.872	0.797	0.789	0.789	0.771
	20	Janbu	2.369	2.300	2.240	2.195	2.150	1.684	1.652	1.620	1.587	1.569	1.192	1.196	1.179	1.192	1.020	1.022	1.032	1.047	0.924	0.915	0.943	0.939
		Bishop	2.439	2.378	2.327	2.285	2.239	1.681	1.720	1.703	1.675	1.657	1.242	1.266	1.259	1.276	1.064	1.090	1.103	1.120	0.968	0.979	1.008	1.000
	30	Janbu	2.606	2.558	2.516	2.487	2.454	1.841	1.856	1.860	1.848	1.850	1.351	1.399	1.411	1.457	1.172	1.209	1.251	1.299	1.079	1.099	1.164	1.174
		Bishop	2.739	2.685	2.645	2.613	2.577	1.880	1.961	1.978	1.969	1.972	1.424	1.490	1.503	1.545	1.245	1.291	1.335	1.376	1.144	1.179	1.241	1.242
	40	Janbu	2.907	2.883	2.862	2.852	2.833	2.045	2.107	2.151	2.167	2.194	1.556	1.645	1.687	1.767	1.360	1.437	1.512	1.595	1.272	1.318	1.426	1.451
		Bishop	3.081	3.050	3.030	3.018	2.991	2.120	2.232	2.291	2.300	2.321	1.641	1.748	1.788	1.863	1.454	1.530	1.601	1.677	1.351	1.411	1.509	1.523

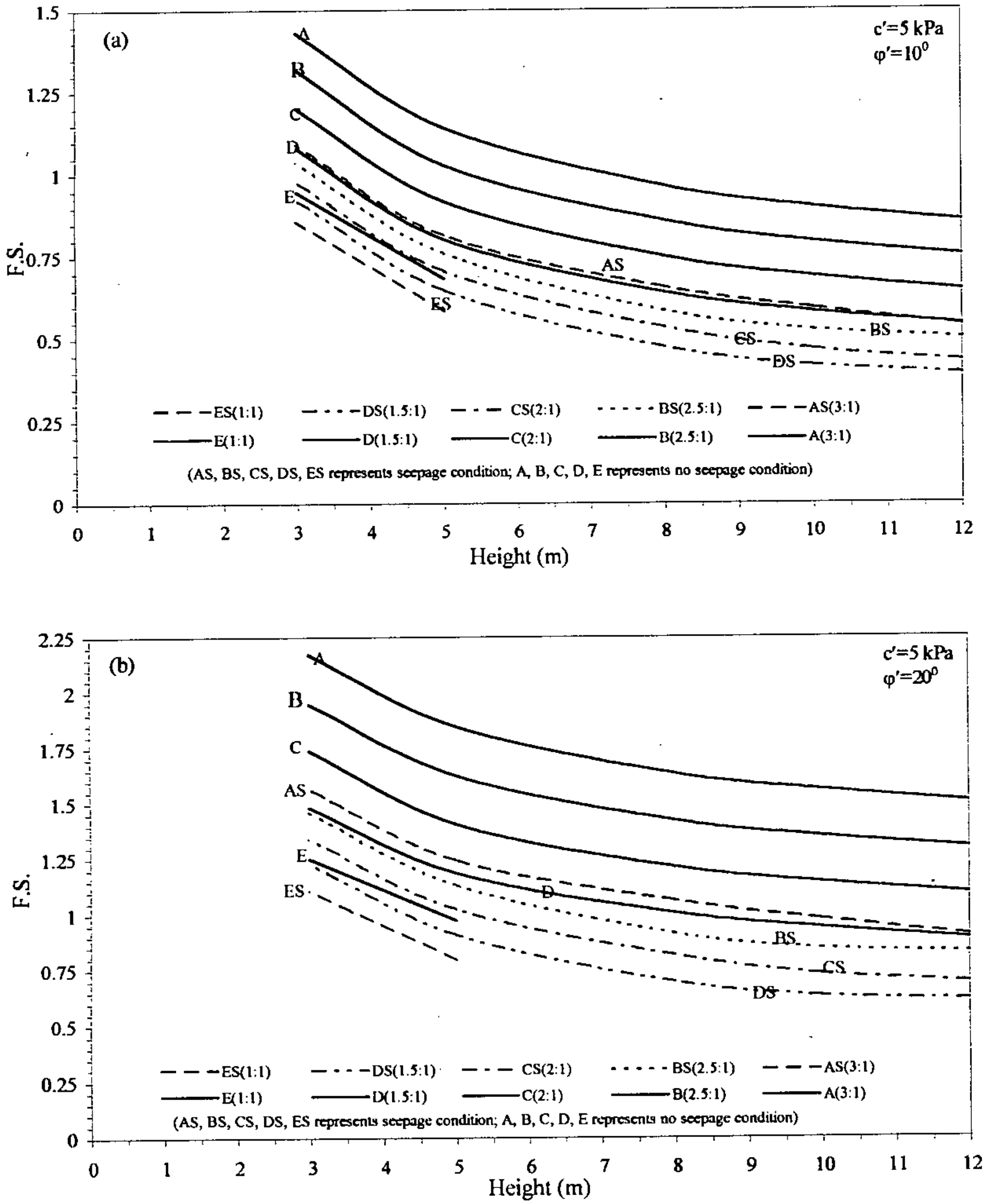


Fig. 4.12 Factor of safety as a function of embankment height and side slope for seepage and no seepage condition. (a) $c'=5kPa$, $\varphi'=10^0$ (b) $c'=5kPa$, $\varphi'=20^0$

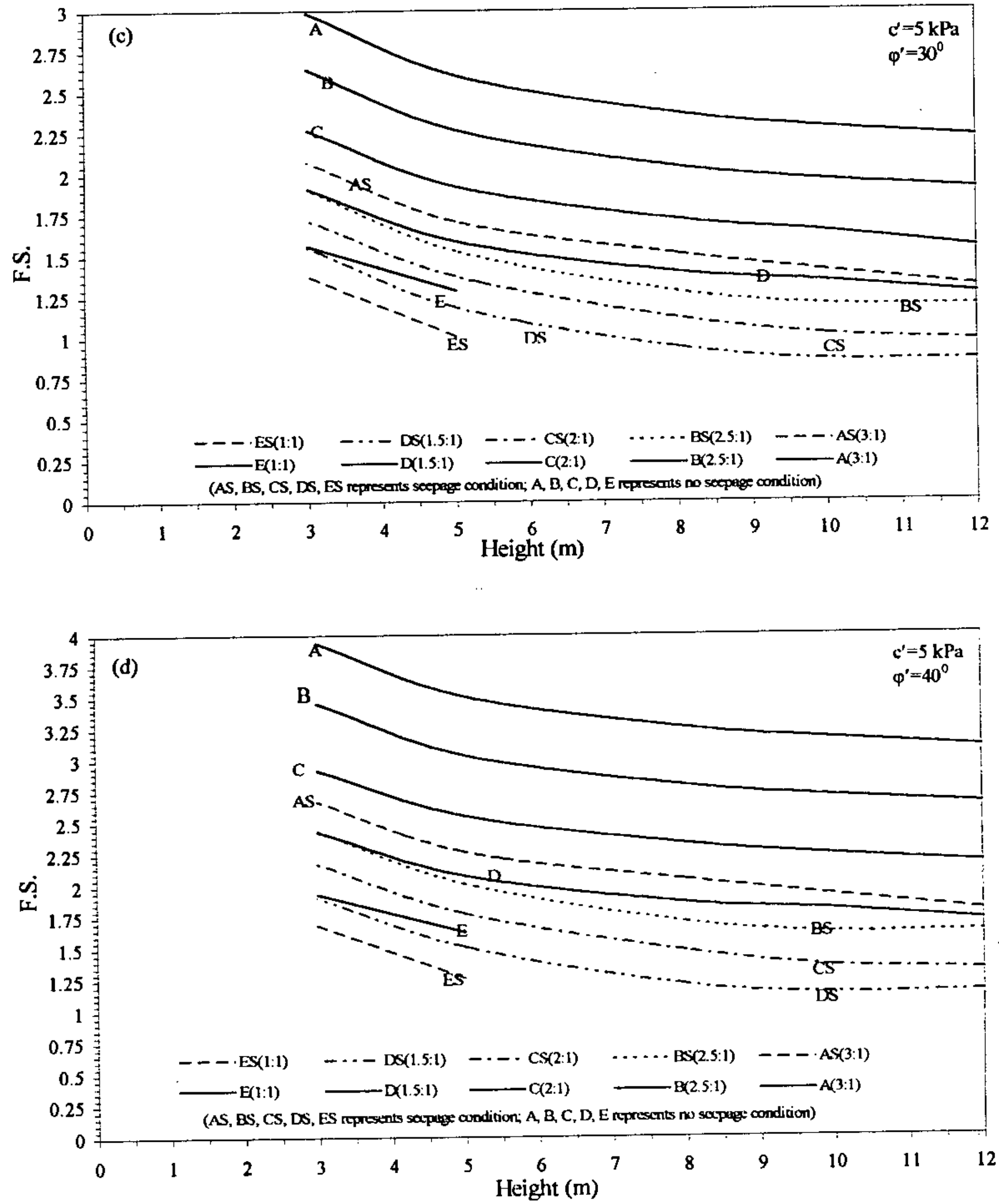
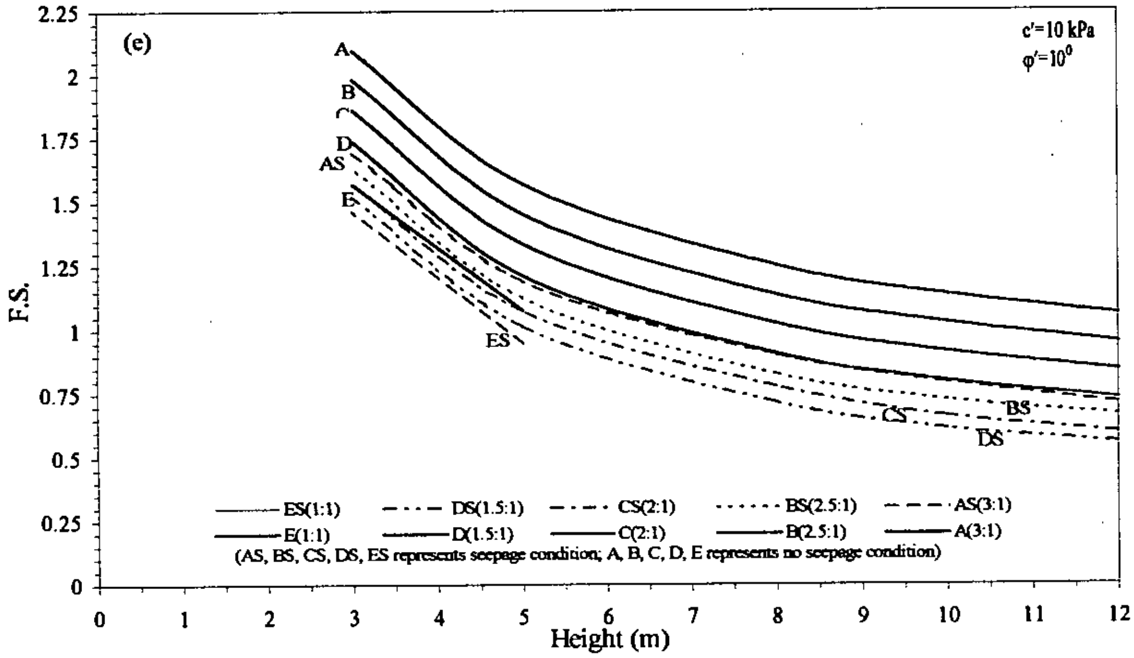


Fig. 4.12 Factor of safety as a function of embankment height and side slope for seepage and no seepage condition. (c) $c'=5$kPa, $\varphi'=30^0$ (d) $c'=5$kPa, $\varphi'=40^0$



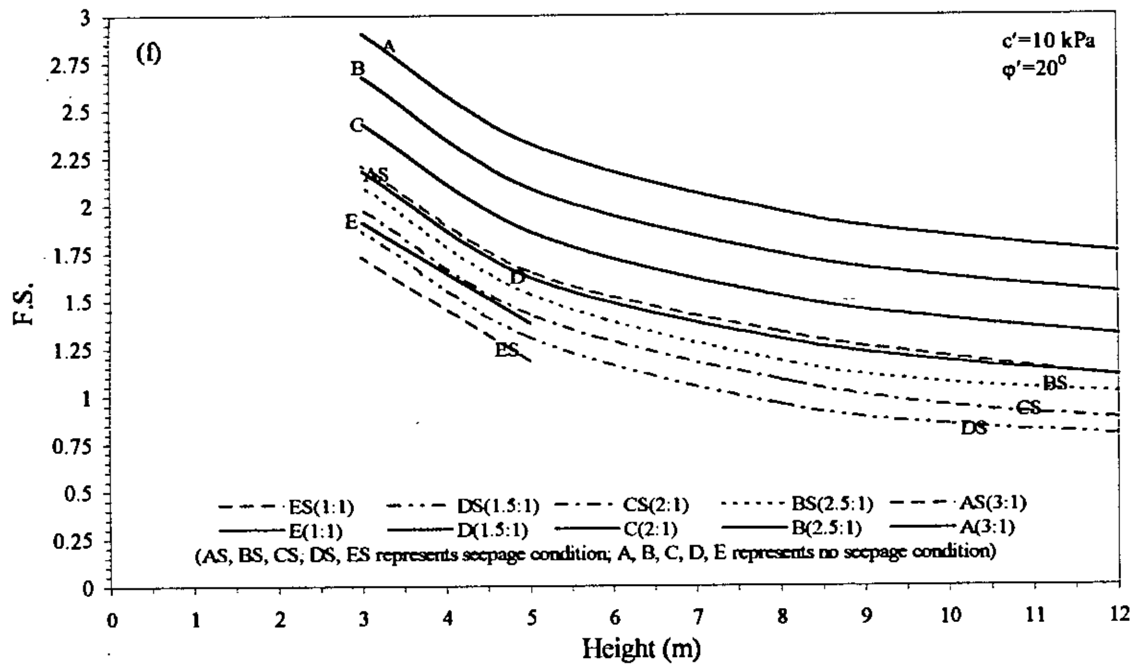
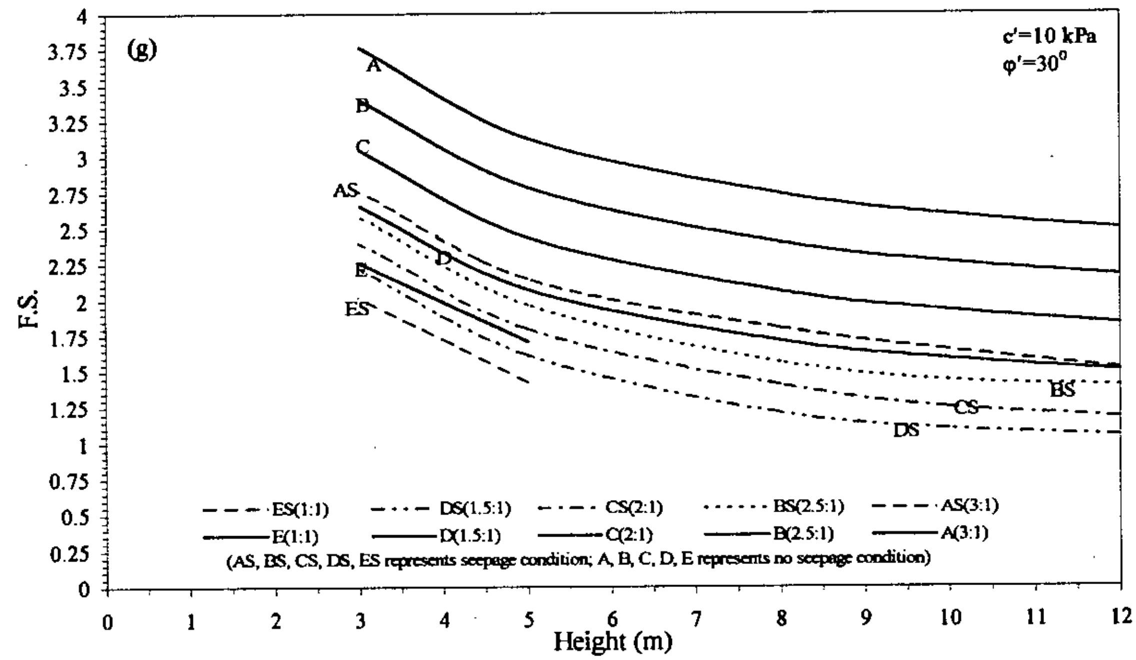


Fig. 4.12 Factor of safety as a function of embankment height and side slope for seepage and no seepage condition. (e) $c'=10kPa$, $\varphi'=10^0$ (f) $c'=10kPa$, $\varphi'=20^0$



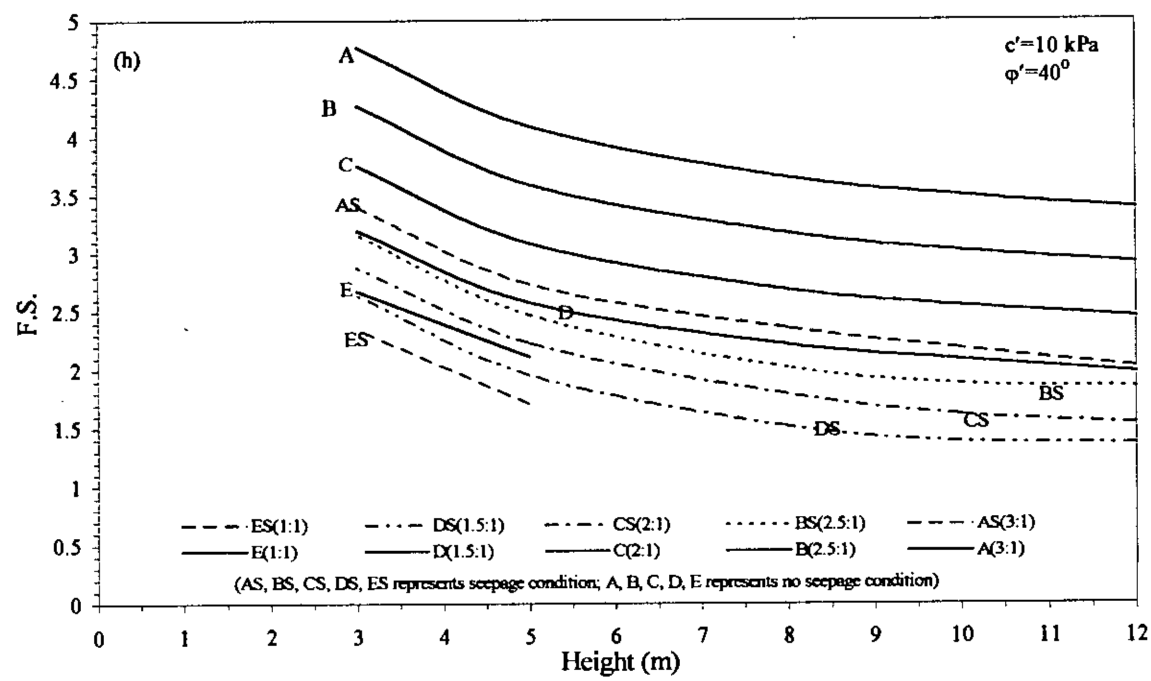
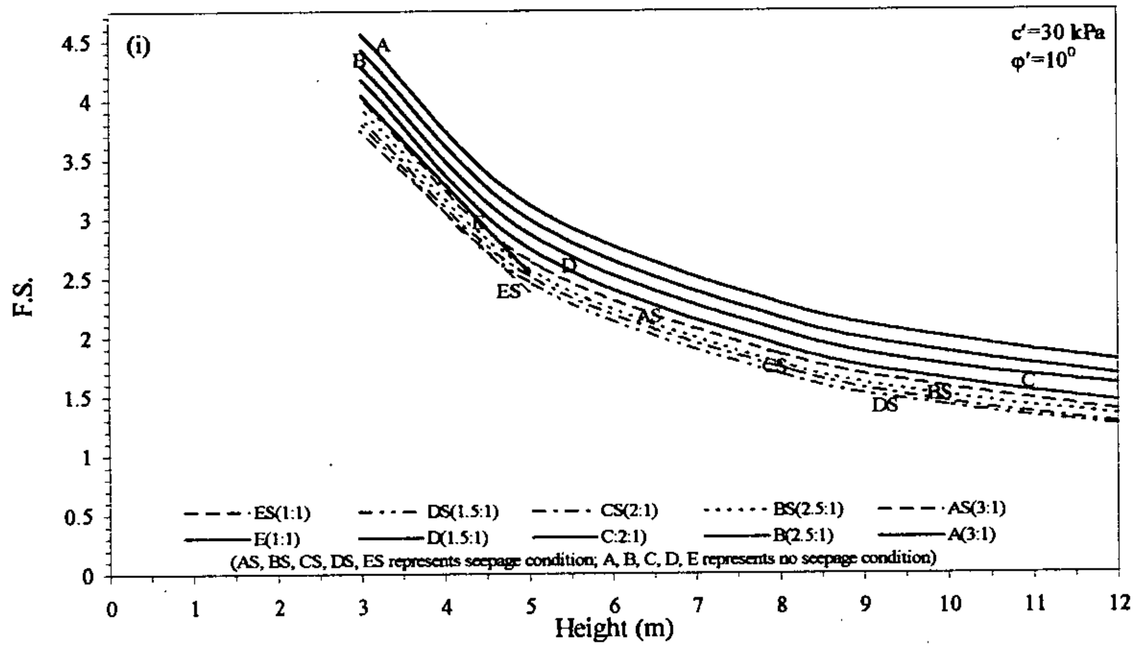


Fig. 4.12 Factor of safety as a function of embankment height and side slope for seepage and no seepage condition. (g) c'=10kPa, φ'=30° (h) c'=10kPa, φ'=40°



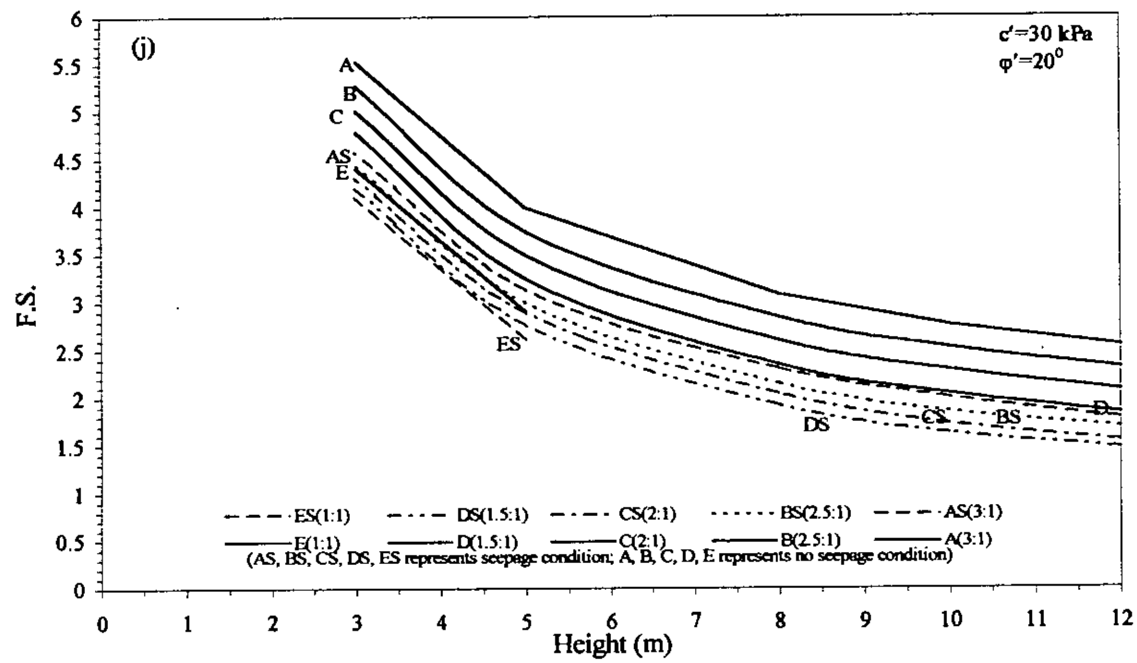


Fig. 4.12 Factor of safety as a function of embankment height and side slope for seepage and no seepage condition. (i) c'=30kPa, φ'=10^0 (j) c'=30kPa, φ'=20^0

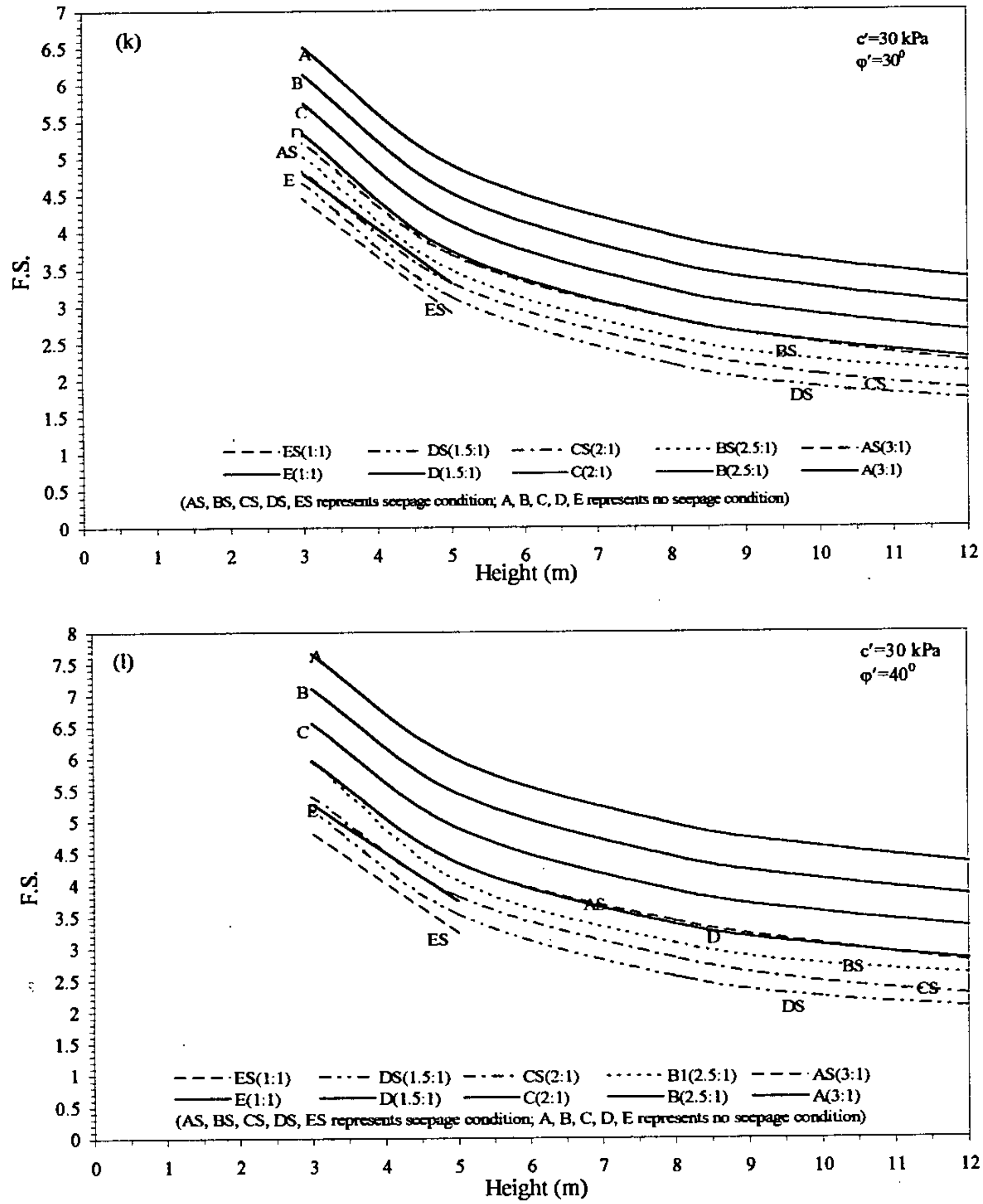


Fig. 4.12 Factor of safety as a function of embankment height and side slope for seepage and no seepage condition. (k) c'=30kPa, φ'=30⁰ (l) c'=30kPa, φ'=40⁰

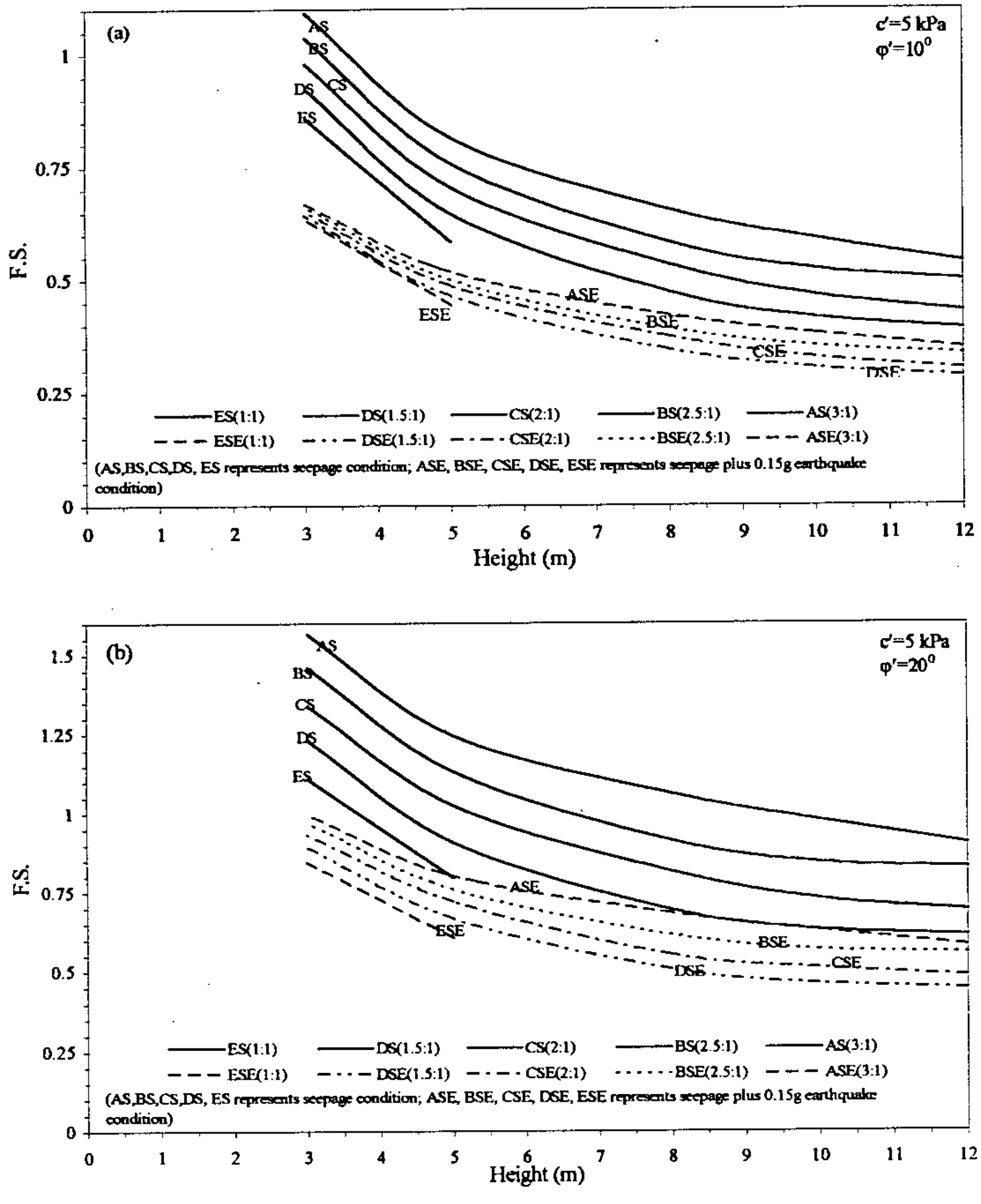


Fig. 4.13 Factor of safety as a function of embankment height and side slope for seepage and seepage plus 0.15g earthquake condition.
(a) c'=5kPa, φ'=10° (b) c'=5kPa, φ'=20°

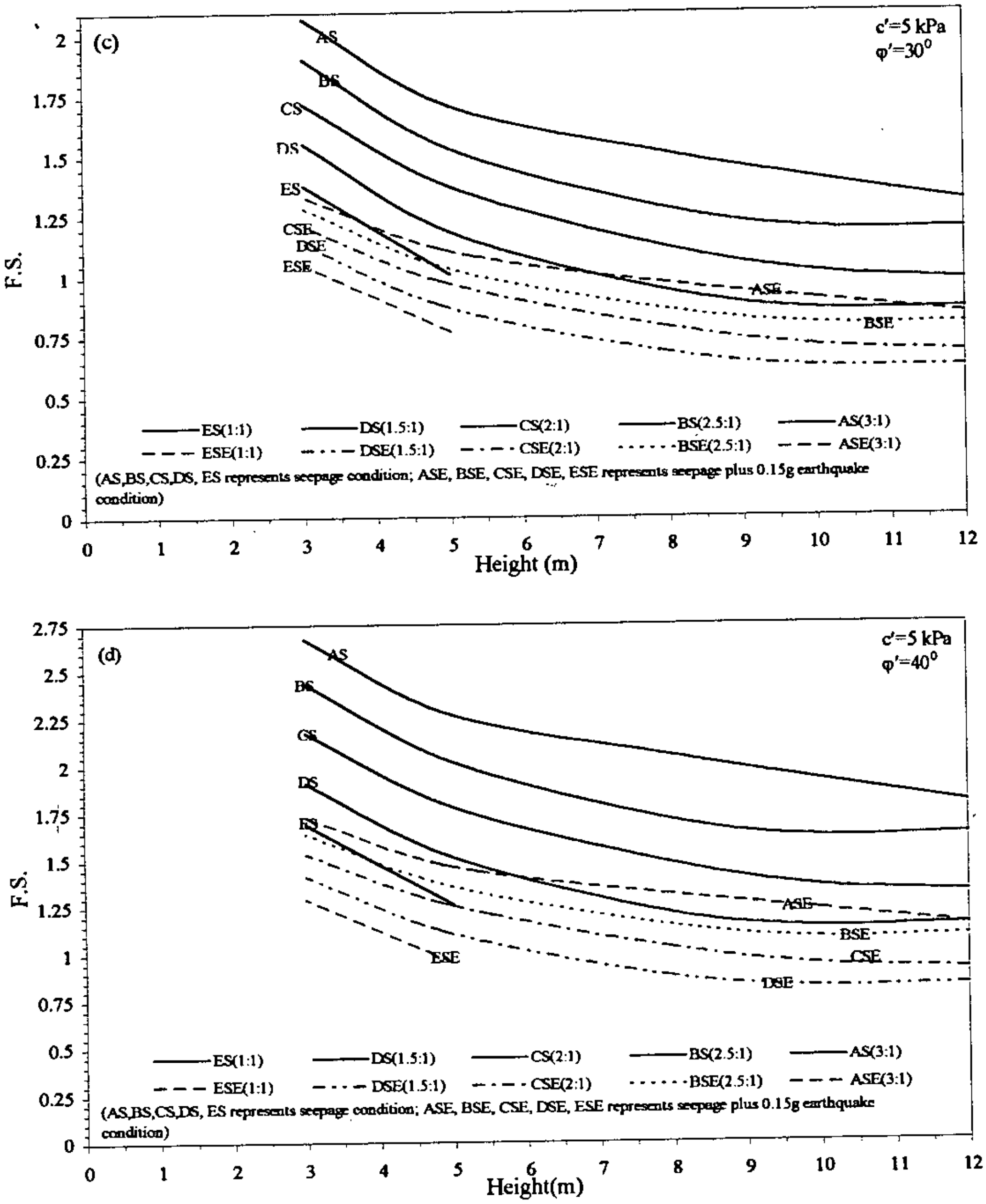
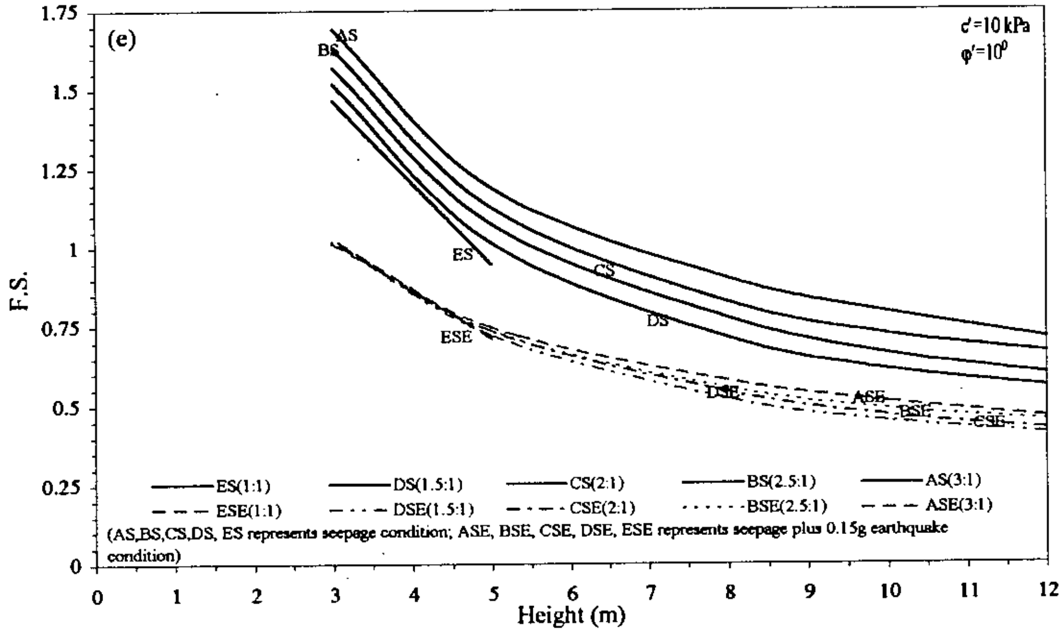


Fig. 4.13 Factor of safety as a function of embankment height and side slope for seepage and seepage plus 0.15g earthquake condition.
(c) c′=5kPa, φ′=30^0 (d) c′=5kPa, φ′=40^0



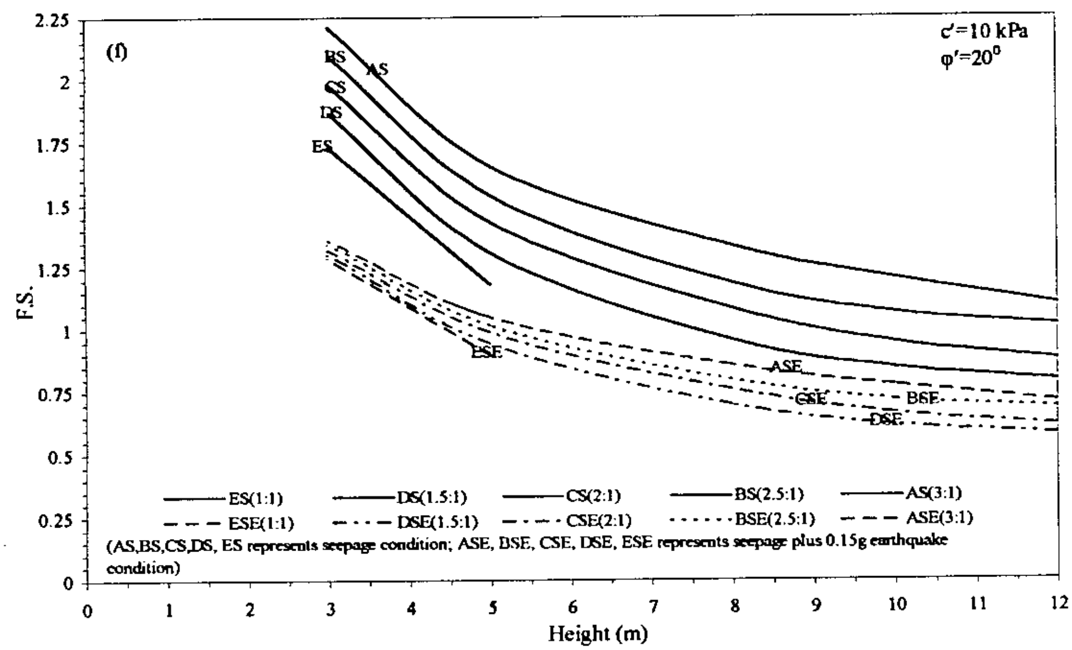
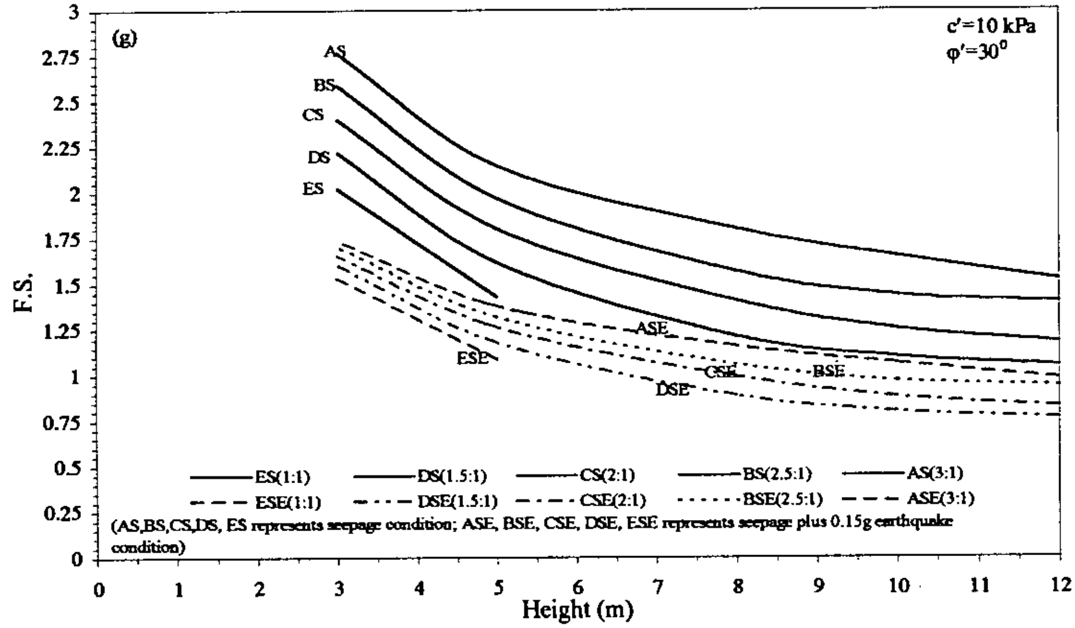


Fig. 4.13 Factor of safety as a function of embankment height and side slope for seepage and seepage plus 0.15g earthquake condition.
(e) c'=10kPa, φ'=10^0 (f) c'=10kPa, φ'=20^0



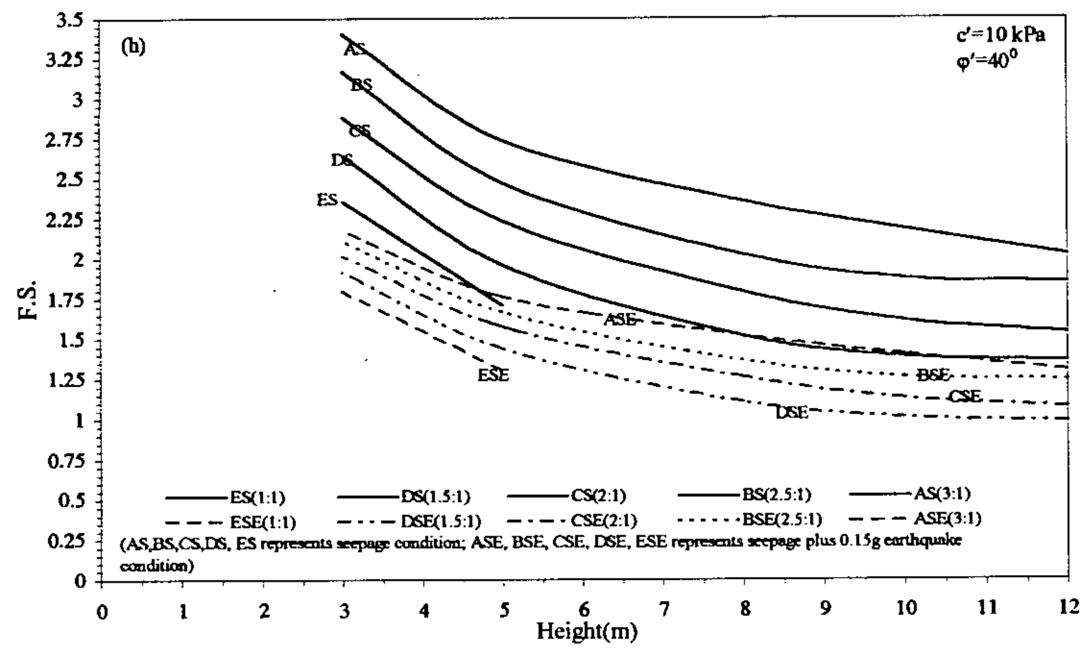
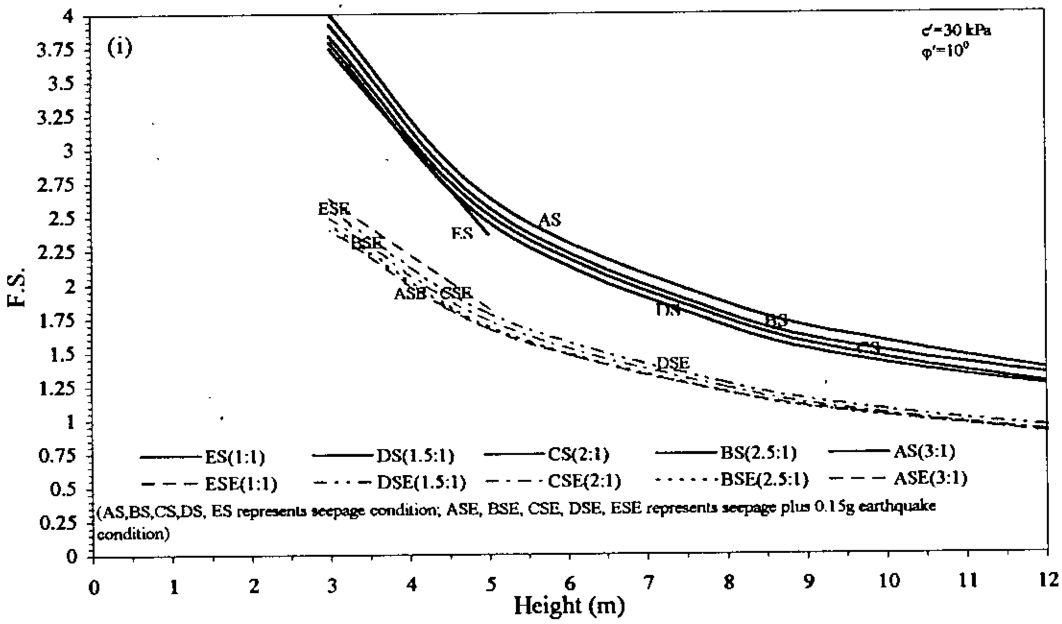


Fig. 4.13 Factor of safety as a function of embankment height and side slope for seepage and seepage plus 0.15g earthquake condition.
(g) c'=10kPa, $\varphi'=30^0$ (h) c'=10kPa, $\varphi'=40^0$



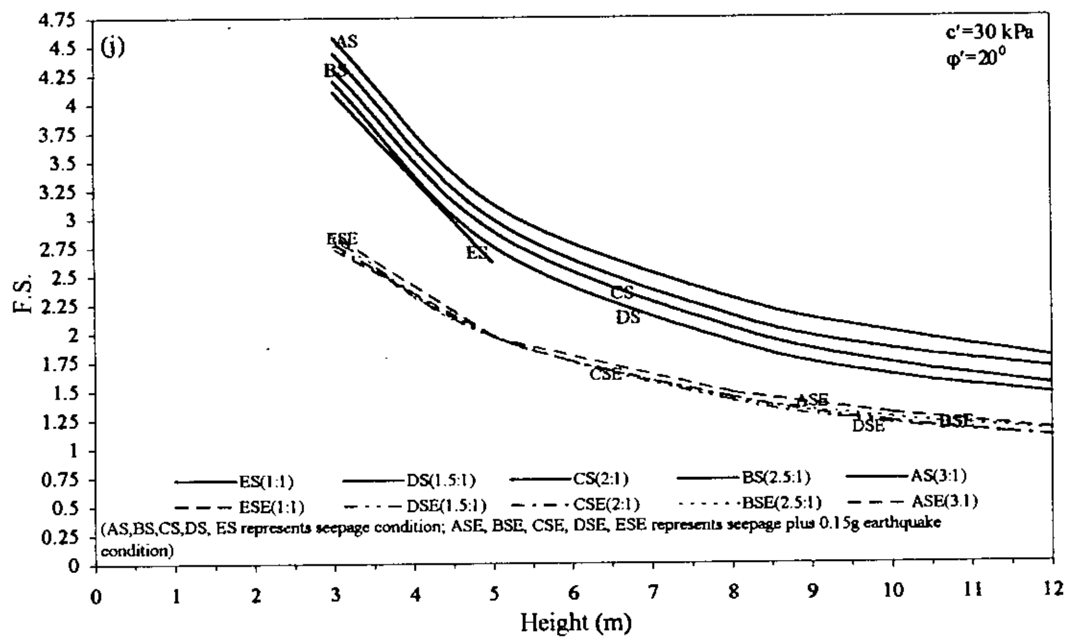
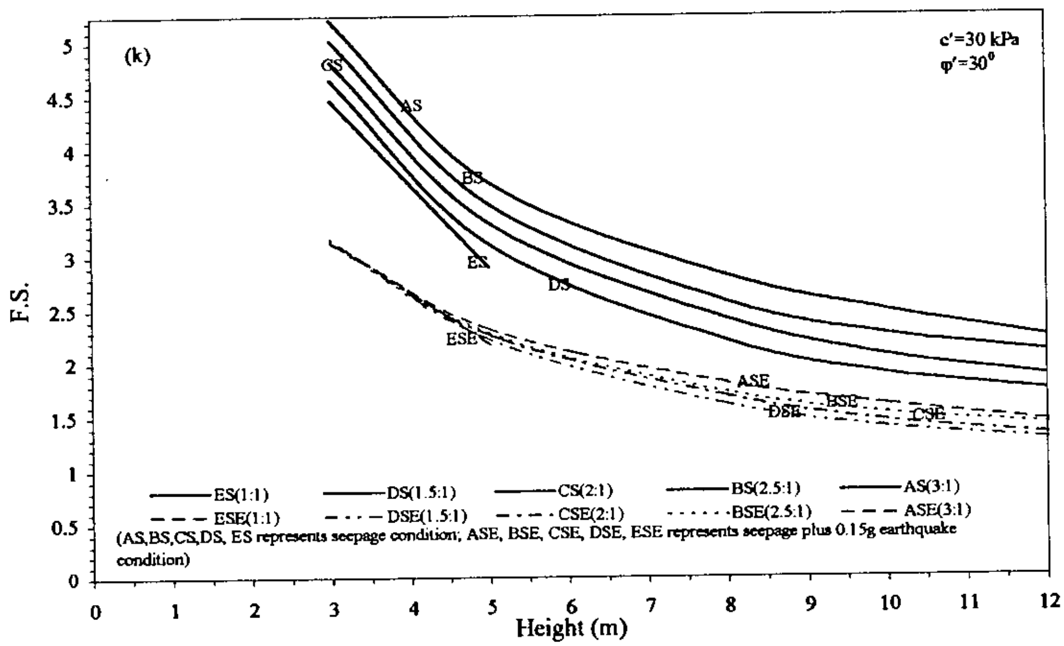


Fig. 4.13 Factor of safety as a function of embankment height and side slope for seepage and seepage plus 0.15g earthquake condition.
(i) c'=30kPa, φ'=10^0 (j) c'=30kPa, φ'=20^0



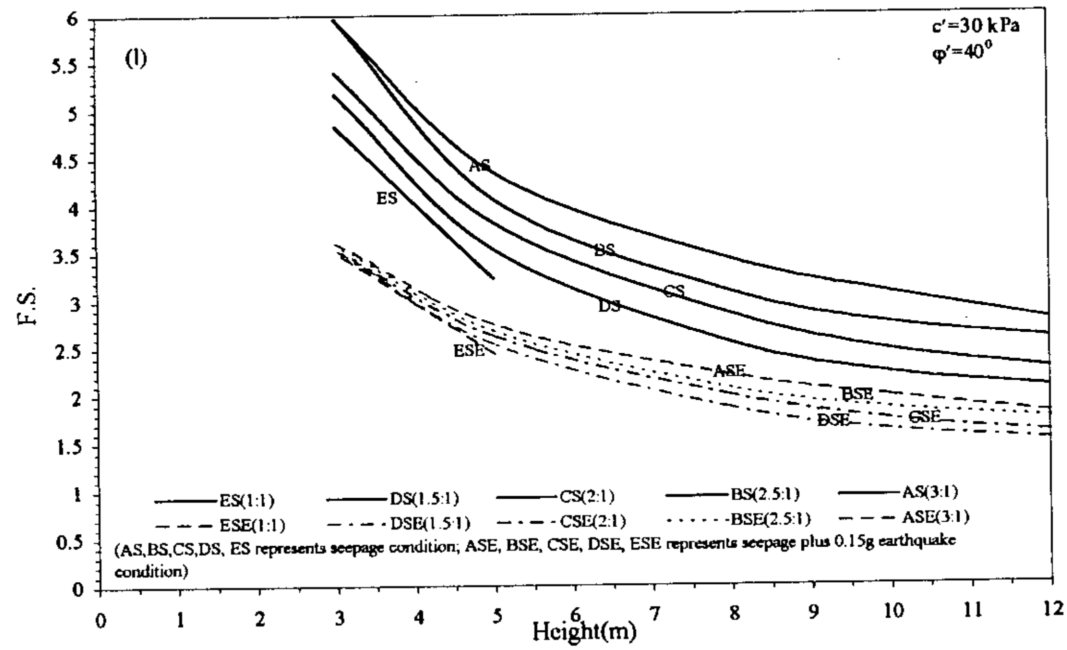
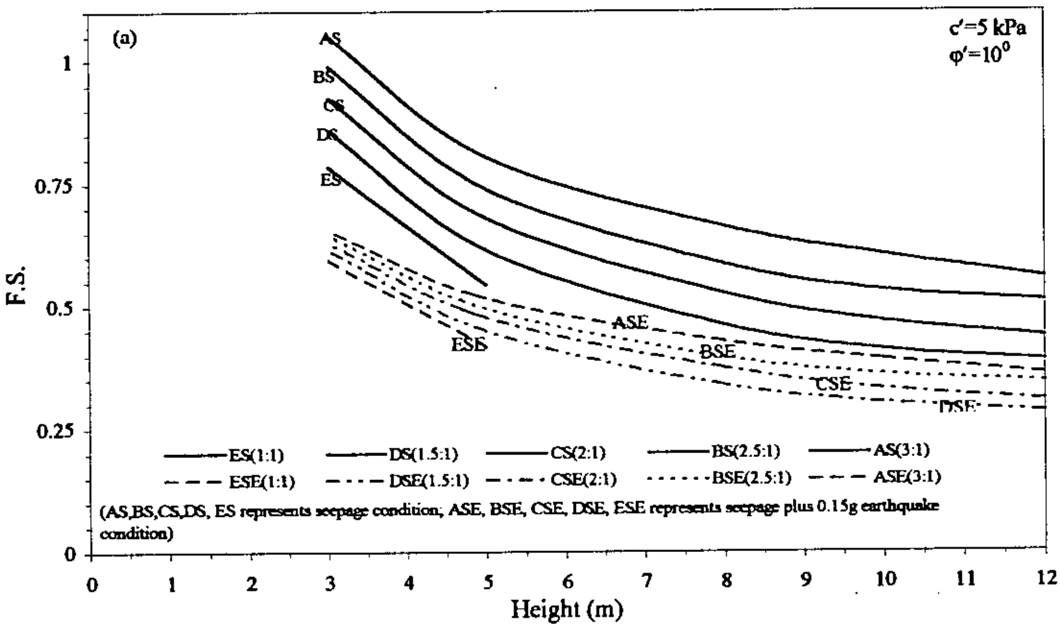


Fig. 4.13 Factor of safety as a function of embankment height and side slope for seepage and seepage plus 0.15g earthquake condition.
(k) c'=30kPa, $\varphi'=30^0$ (l) c'=30kPa, $\varphi'=40^0$



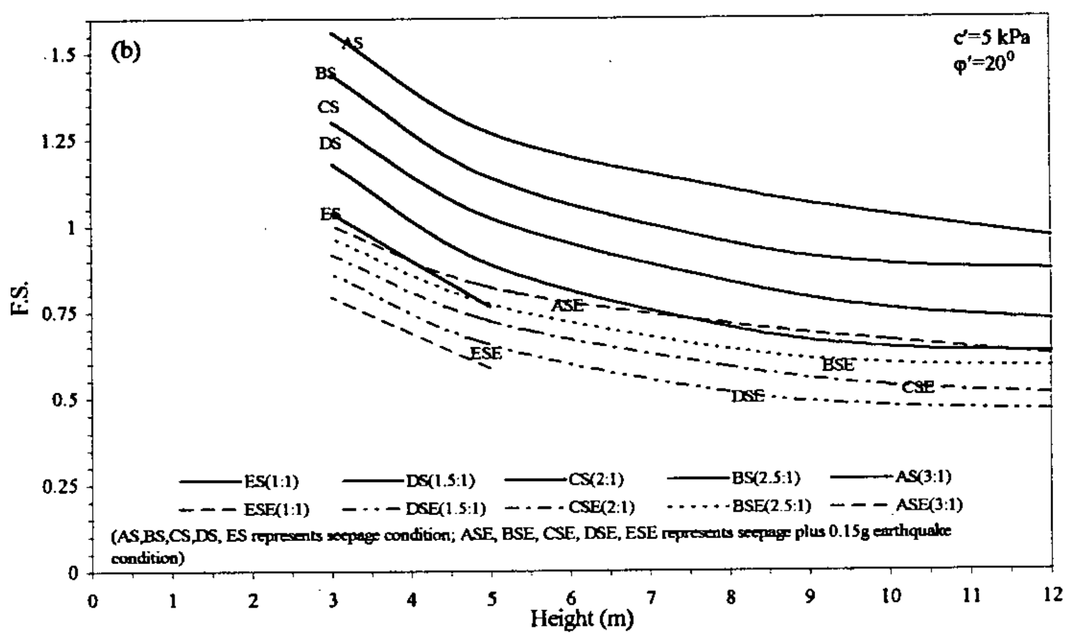
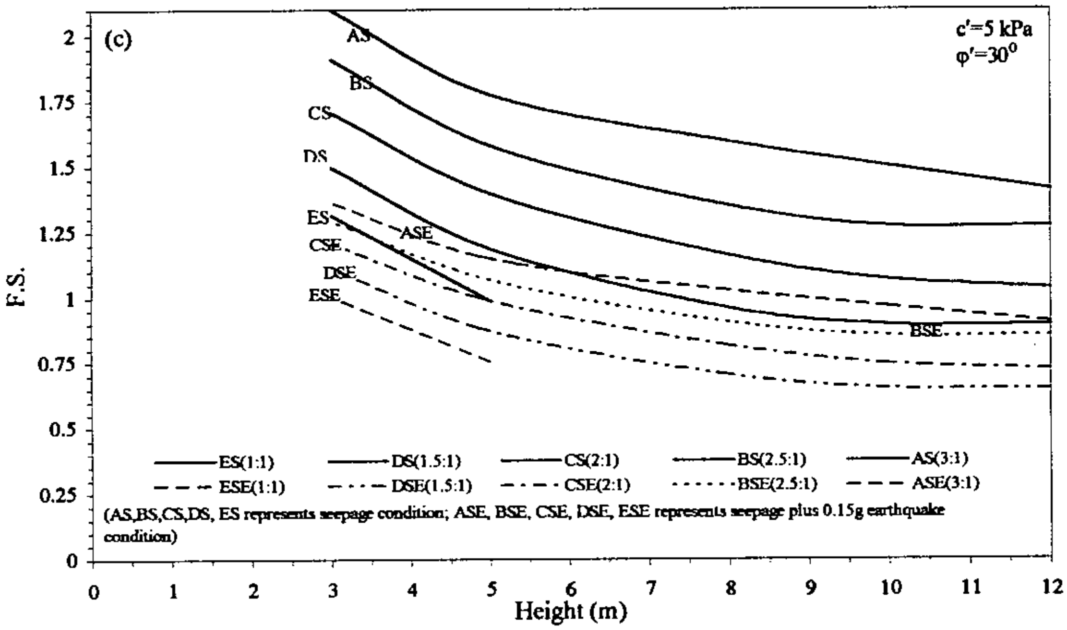


Fig. 4.14 Factor of safety as a function of embankment height and side slope for seepage and seepage plus 0.15g earthquake condition for γ_{moist}=21KN/m^3, γ_{sat}=22KN/m^3 (a) c′=5kPa, φ′=10^0 (b) c′=5kPa, φ′=20^0



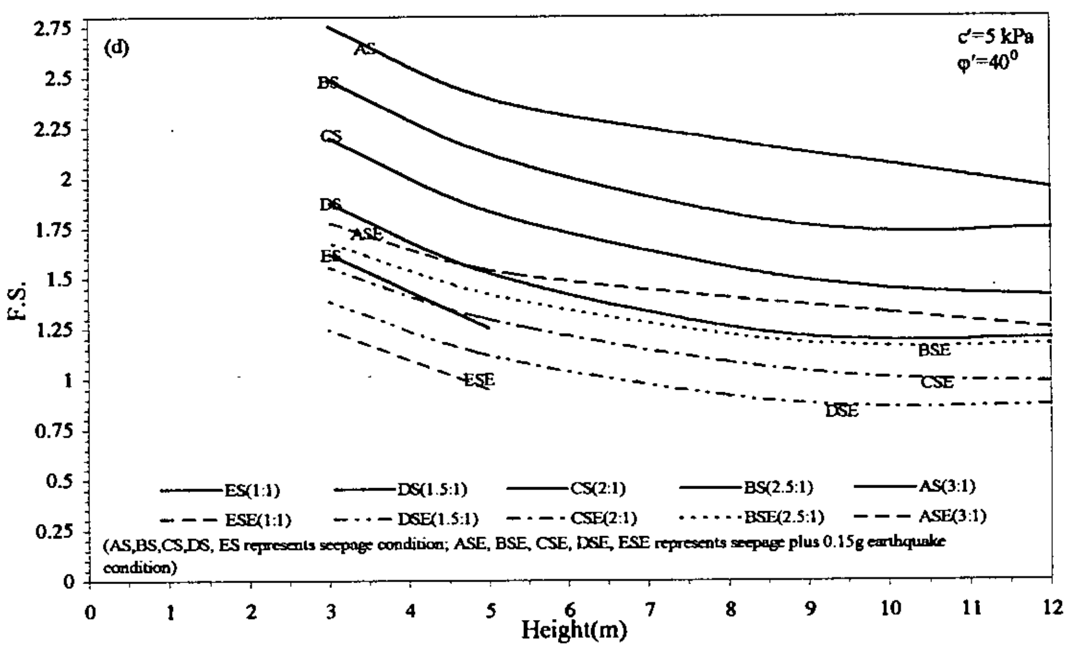


Fig. 4.14 Factor of safety as a function of embankment height and side slope for seepage and seepage plus 0.15g earthquake condition for γ_{moist}=21KN/m^3, γ_{sat}=22KN/m^3. (c) c′=5kPa, φ′=30^0 (d) c′=5kPa, φ′=40^0

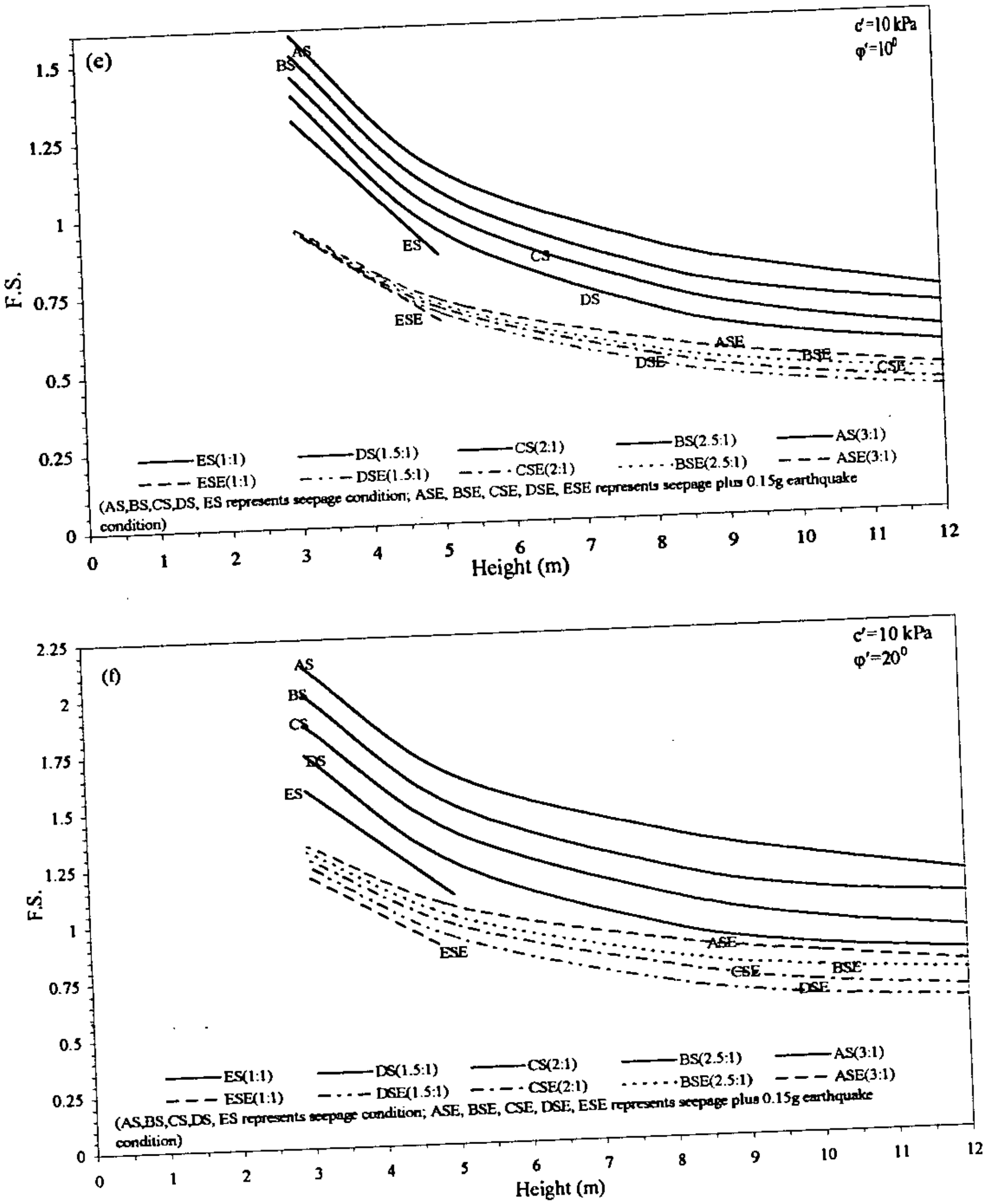
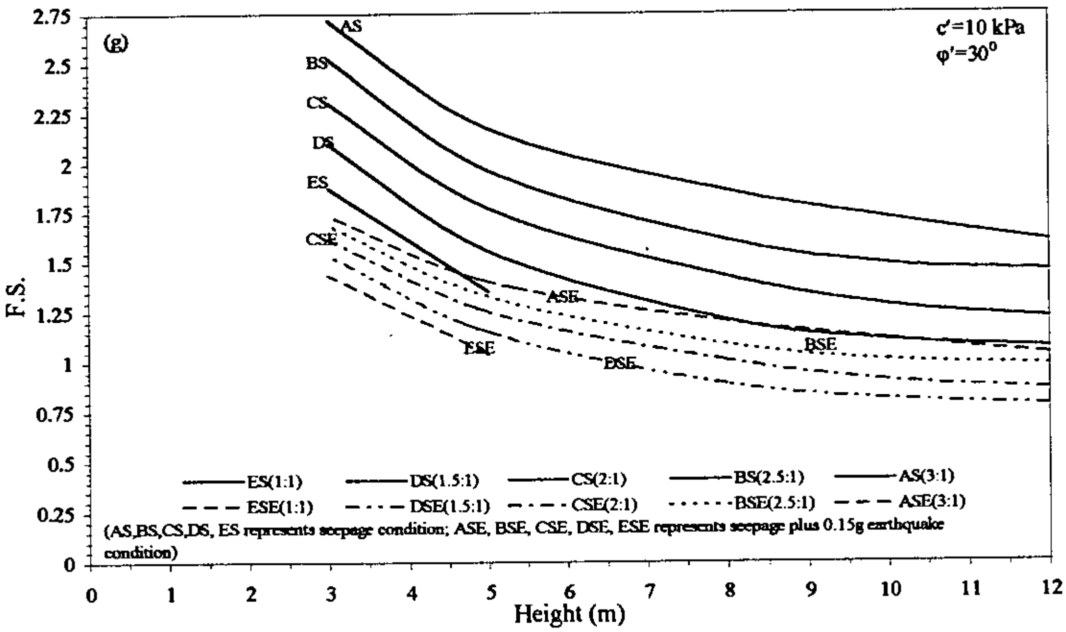


Fig. 4.14 Factor of safety as a function of embankment height and side slope for seepage and seepage plus 0.15g earthquake condition for γ_{moist}=21KN/m^3, γ_{sat}=22KN/m^3
(e) c'=10kPa, φ'=10^0 (f) c'=10kPa, φ'=20^0



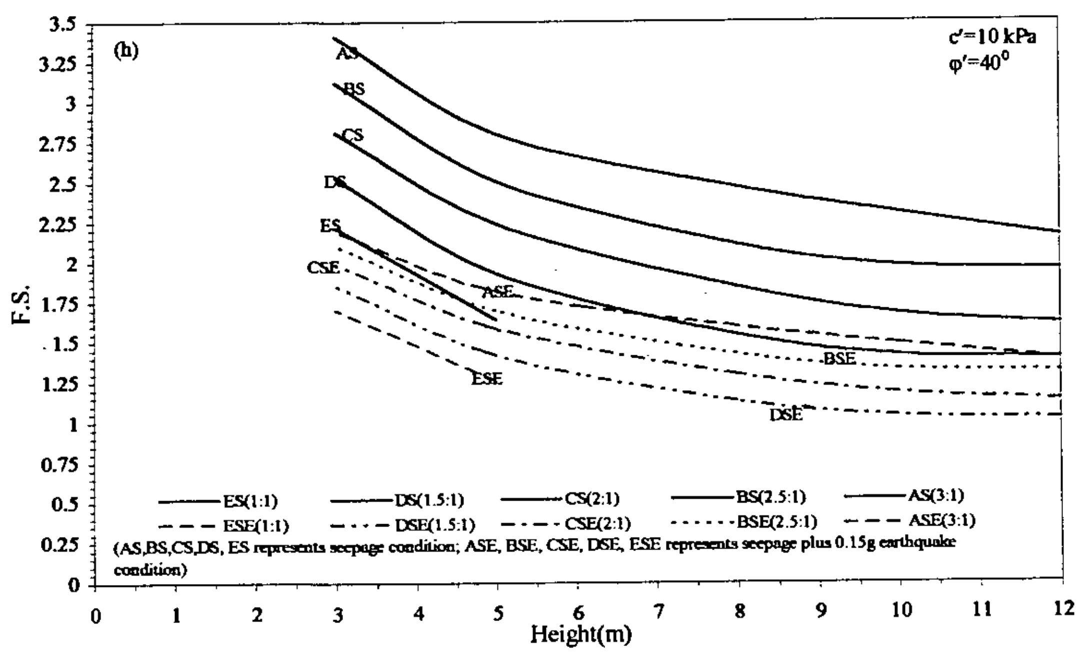
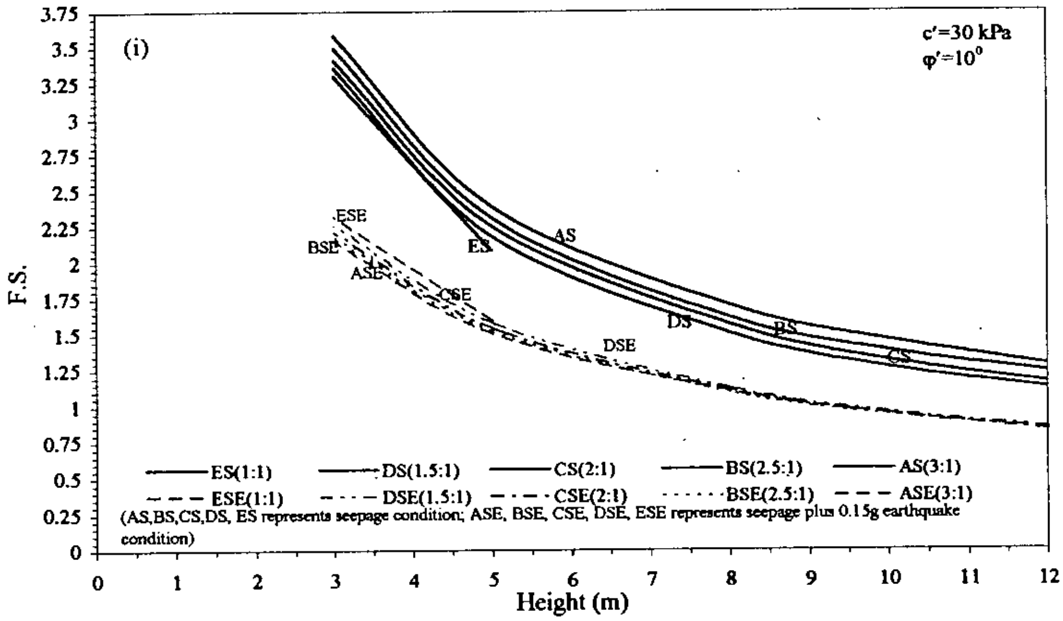


Fig. 4.14 Factor of safety as a function of embankment height and side slope for seepage and seepage plus 0.15g earthquake condition for γ_{moist}=21KN/m^3, $\gamma_{sat.}$=22KN/m^3. (g) c'=10kPa, φ'=30^0 (h) c'=10kPa, φ'=40^0



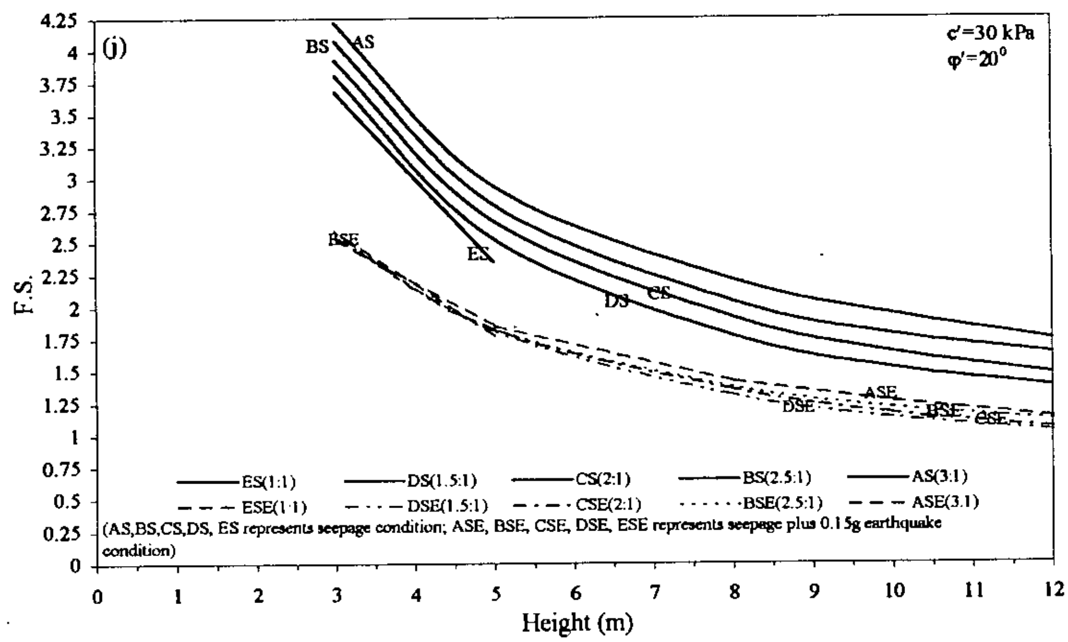
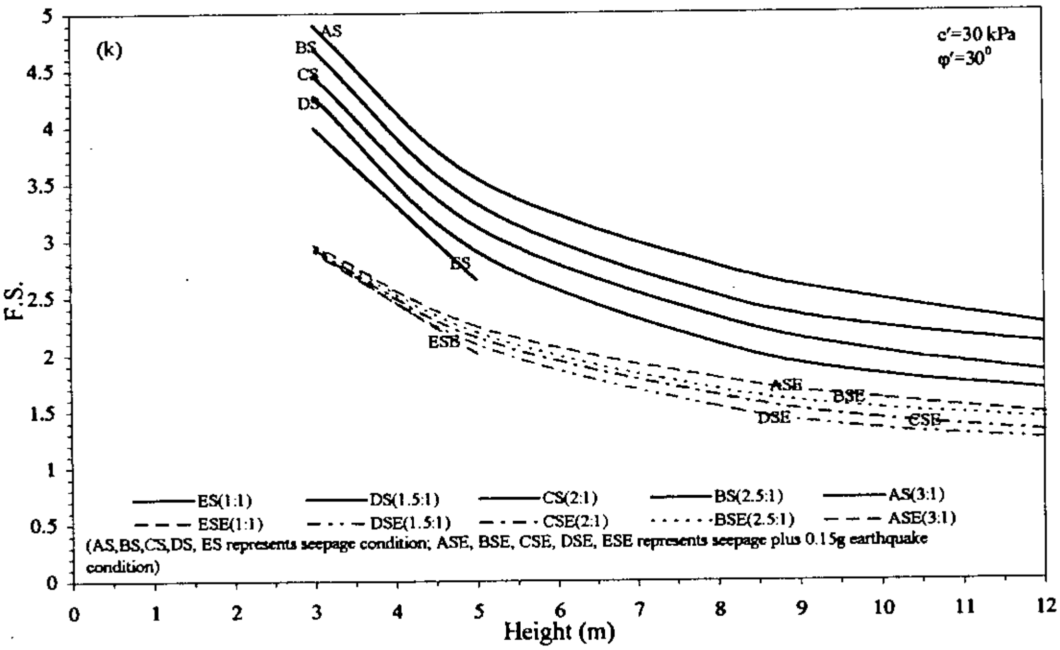


Fig. 4.14 Factor of safety as a function of embankment height and side slope for seepage and seepage plus 0.15g earthquake condition for γ_{moist}=21KN/m^3, γ_{sat}=22KN/m^3. (i) c'=30kPa, φ'=10^0 (j) c'=30kPa, φ'=20^0



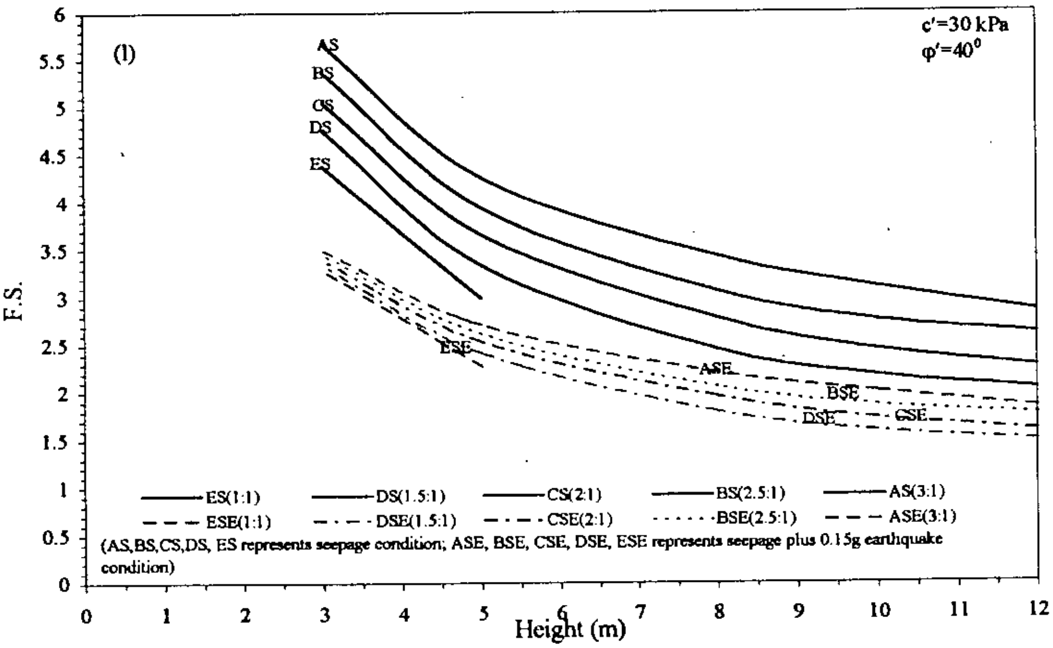


Fig. 4.14 Factor of safety as a function of embankment height and side slope for seepage and seepage plus 0.15g earthquake condition for γ_{moist}=21KN/m^3, γ_{sat}=22KN/m^3. (k) c′=30kPa, φ′=30^0 (l) c′=30kPa, φ′=40^0

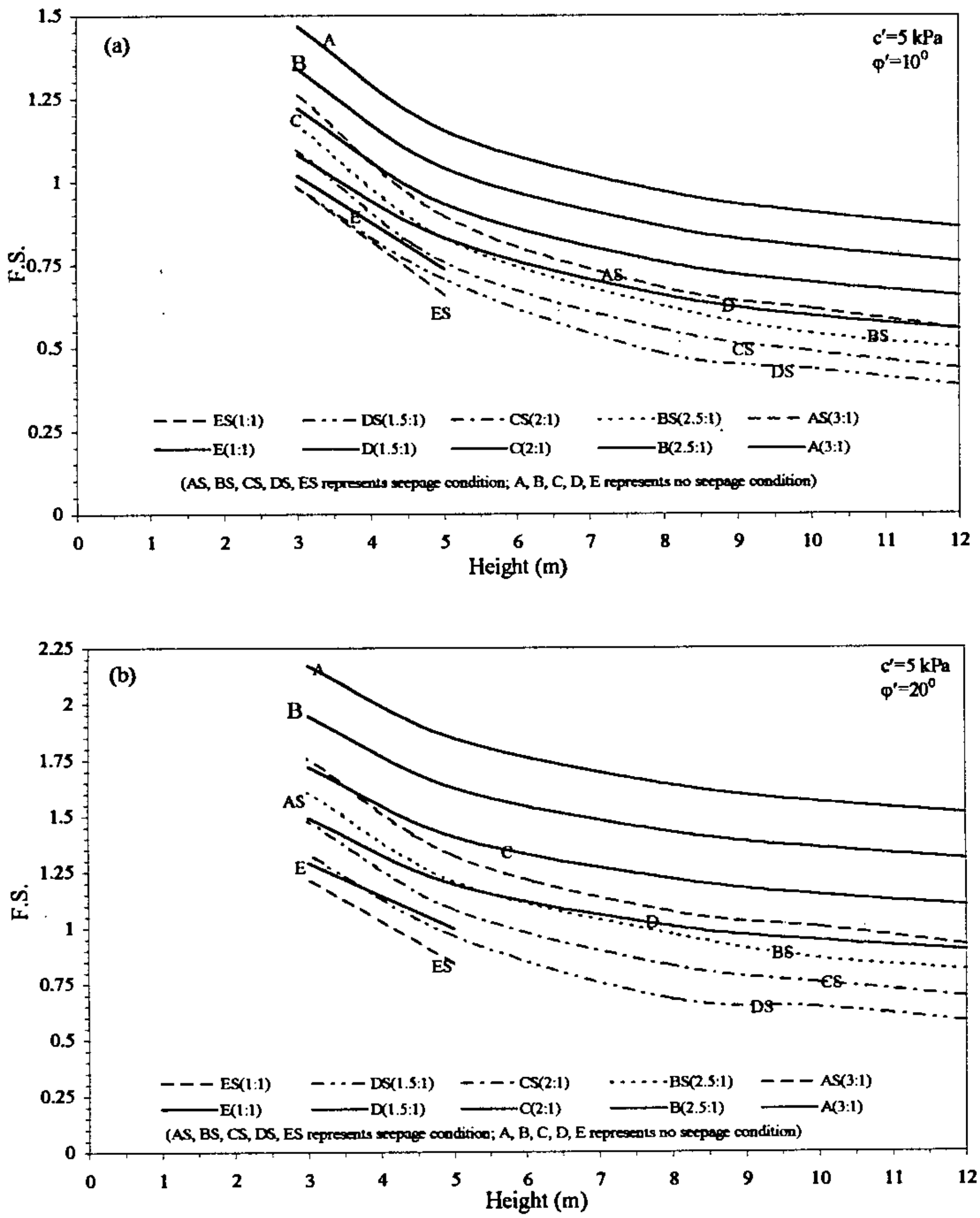


Fig. 4.15 Factor of safety as a function of embankment height and side slope for seepage and no seepage condition for F_B=1.25m.
(a) c'=5kPa, $\varphi'=10^0$ (b) c'=5kPa, $\varphi'=20^0$

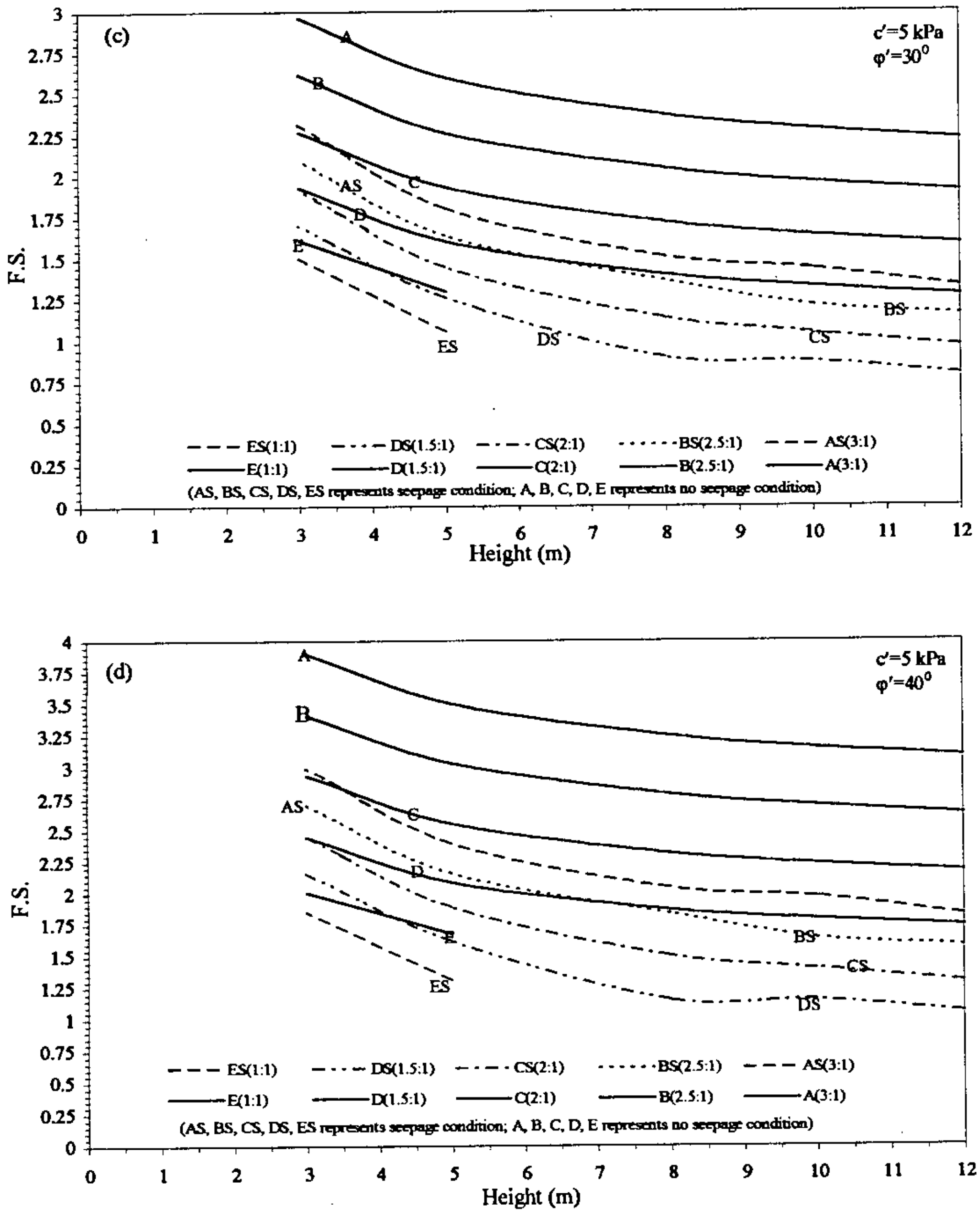


Fig. 4.15 Factor of safety as a function of embankment height and side slope for seepage and no seepage condition for F_B=1.25m.

(c) c'=5kPa, φ'=30^0 (d) c'=5kPa, φ'=40^0

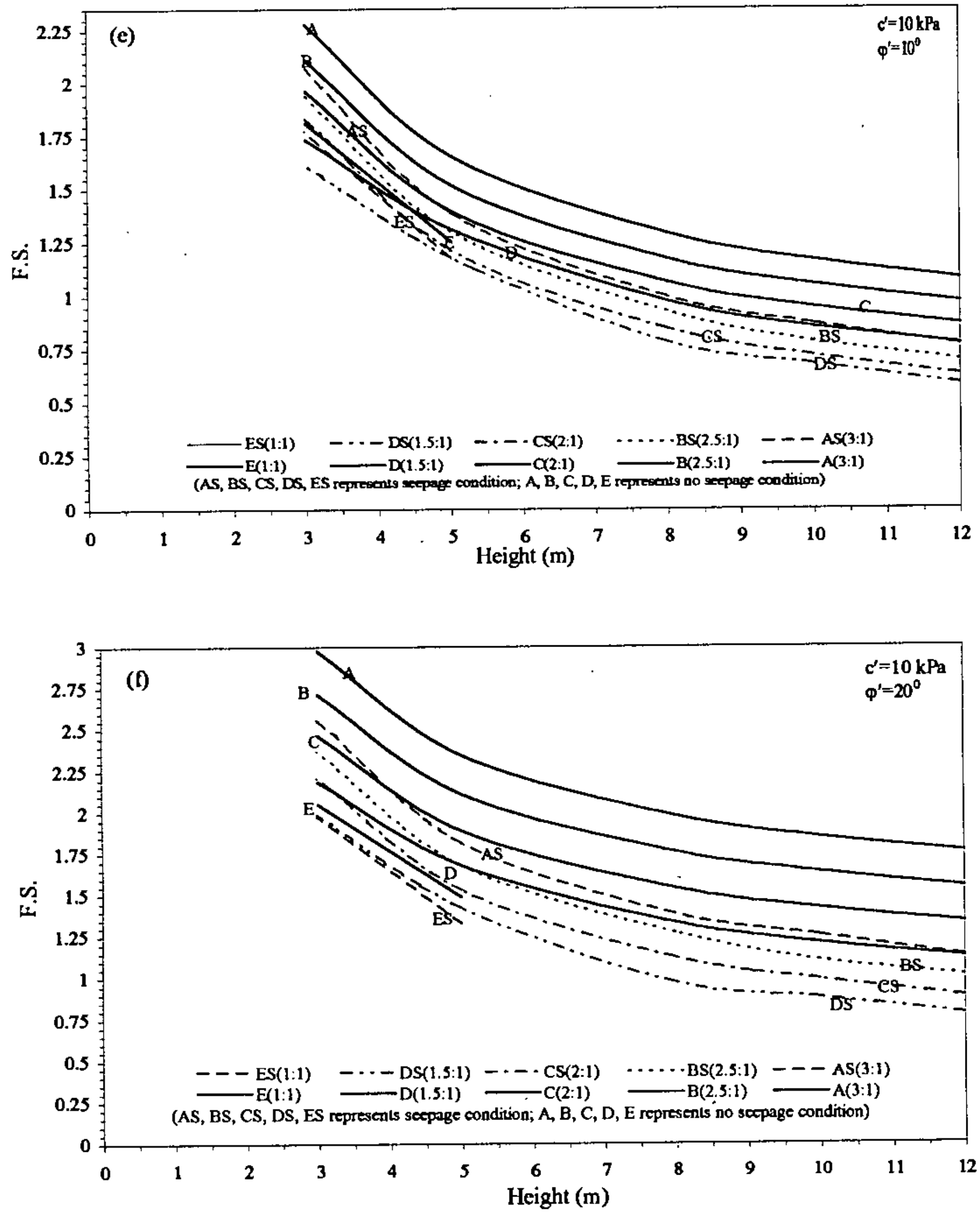


Fig. 4.15 Factor of safety as a function of embankment height and side slope for seepage and no seepage condition for F_B=1.25m.
(e) c'=10kPa, φ'=10^0 (f) c'=10kPa, φ'=20^0

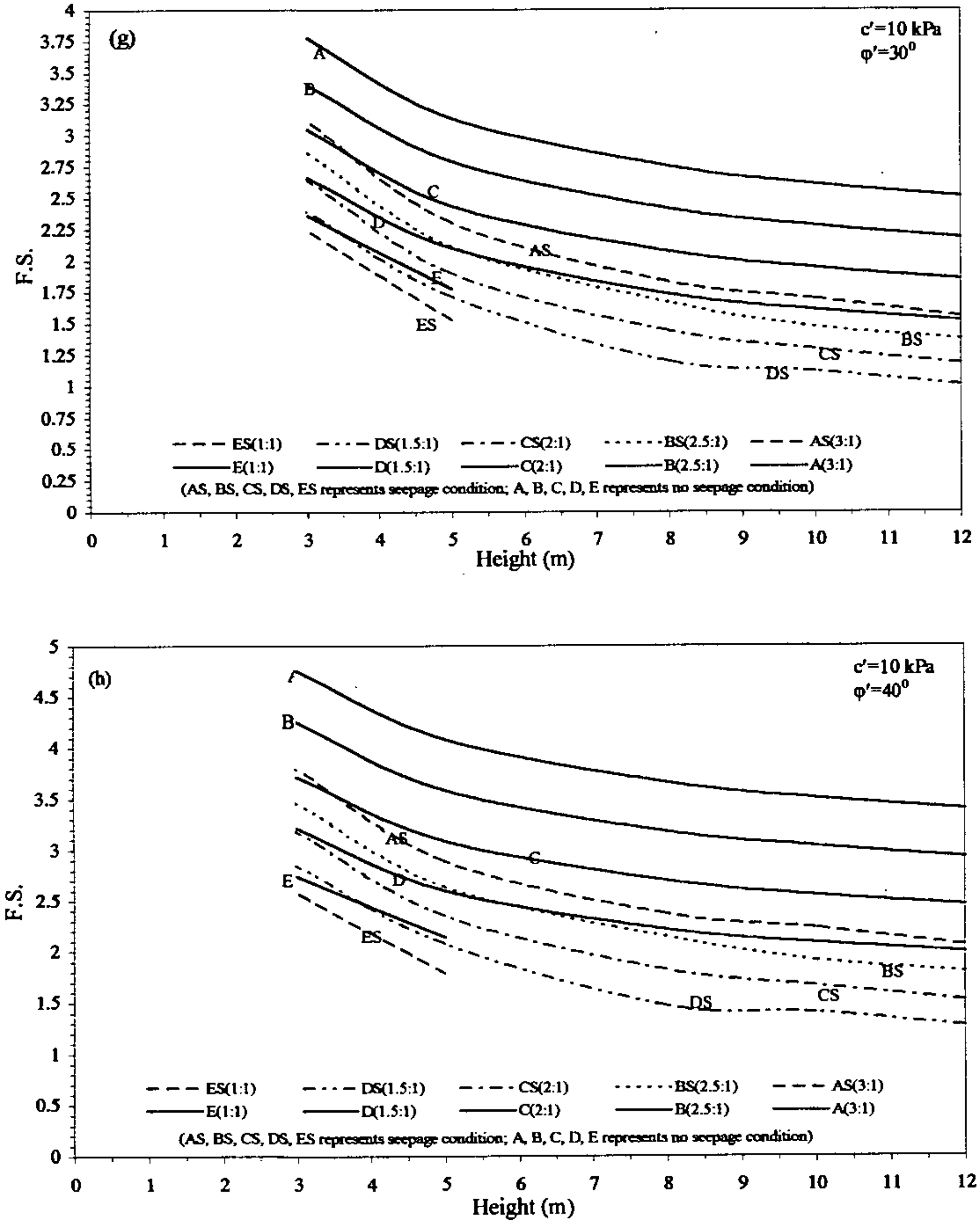
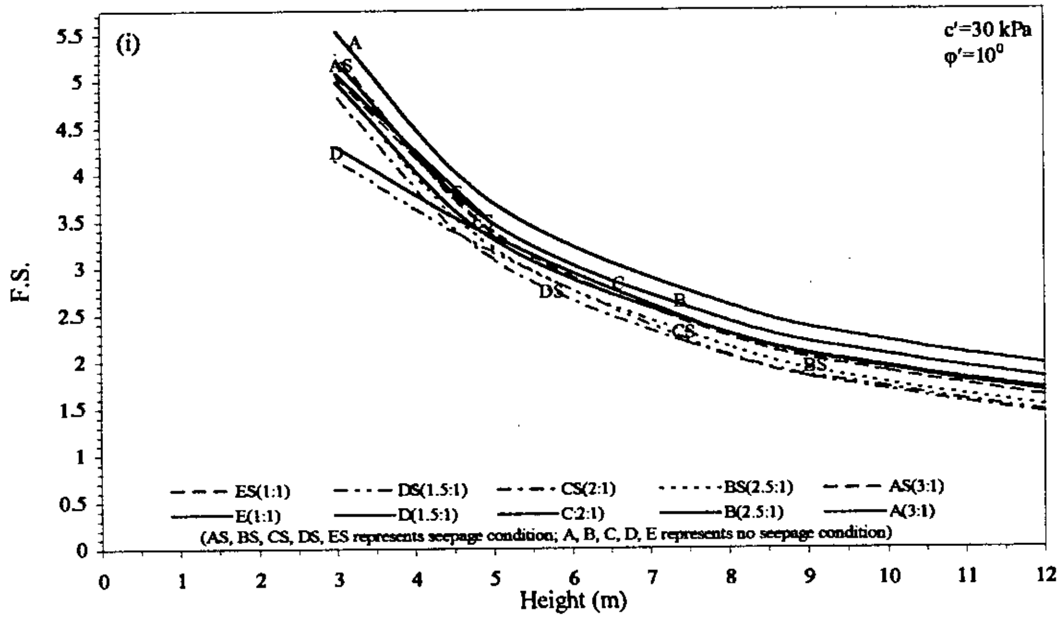


Fig. 4.15 Factor of safety as a function of embankment height and side slope for seepage and no seepage condition for F_B=1.25m.
(g) c'=10kPa, φ'=30^0 (h) c'=10kPa, φ'=40^0



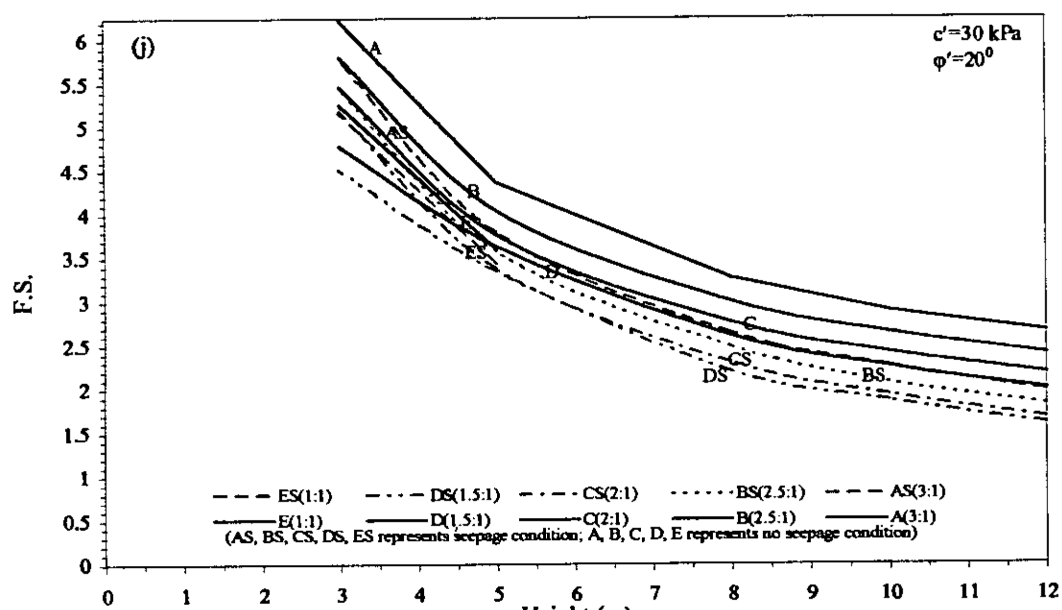
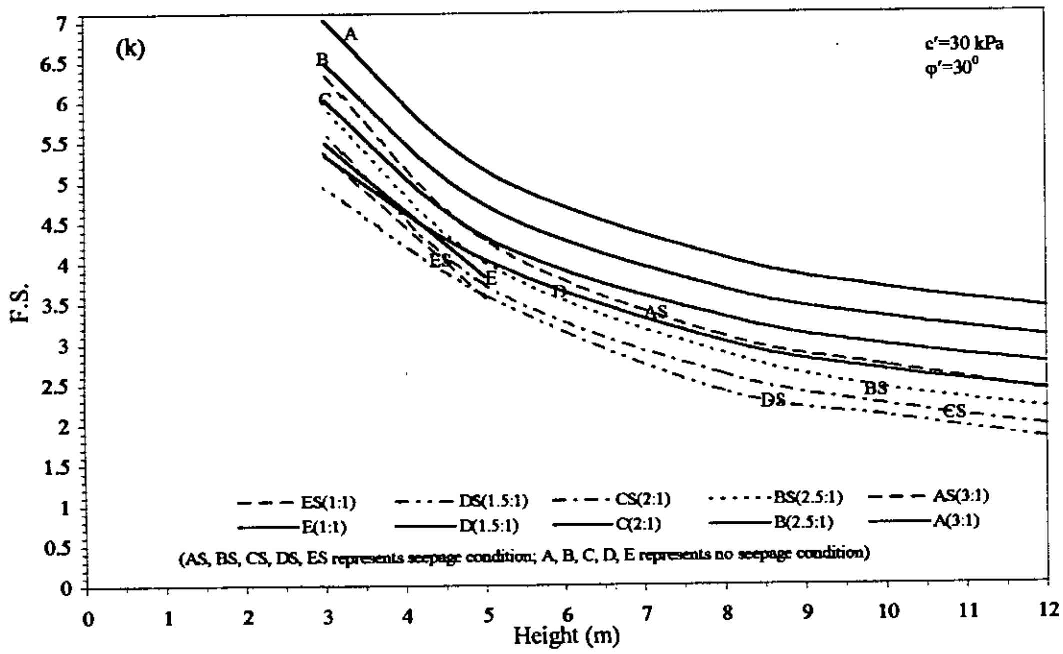


Fig. 4.15 Factor of safety as a function of embankment height and side slope for seepage and no seepage condition for F_B=1.25m.

(i) c'=30kPa, φ'=10^0 (j) c'=30kPa, φ'=20^0



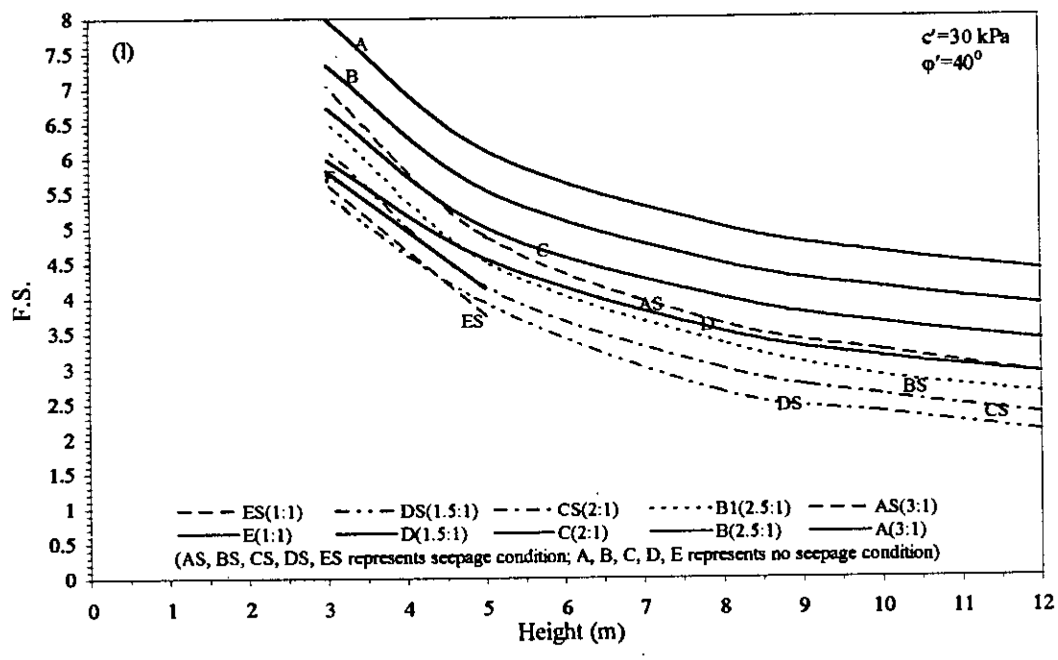


Fig. 4.15 Factor of safety as a function of embankment height and side slope for seepage and no seepage condition for F_B=1.25m.

(k) c′=30kPa, φ′=30° (l) c′=30kPa, φ′=40°

4.13(l) represent seepage and seepage plus 0.15g earthquake condition. Figs. 4.14(a) to Figs 4.14(l) present the same but for higher soil unit weight of γ_{moist}=21 KN/m^3 and γ_{sat}=22 KN/m^3. Figs. 4.15(a) to Figs 4.15(l) show the effect of seepage for 1.25 m freeboard. In these figures, lines A, B, C, D, E represent slopes 3:1, 2.5:1, 2:1, 1.5:1, 1:1 (Hor:Vert) respectively for no seepage condition. AS, BS, CS, DS, ES correspond to seepage condition, while ASE, BSE, CSE, DSE, ESE correspond to seepage plus earthquake condition. For all cases, it is seen that the F. S. decreases as the height is increased for same side slope. The rate of decrease is greater at smaller embankment heights.

From Figs. 4.12 to 4.15, it can be seen that the factor of safety decreases as the slope is steepened for most cases, except for some cases with seepage plus earthquake. In such exceptional cases, the factor of safety slightly increases or remains about the same as the slope is steepened. This is discussed in more detail in a later section.

4.3.3 Effect of Embankment Soil Parameters

Table 4.7 (γ_{moist}=15 KN/m^3, γ_{sat}=16 KN/m^3), Table 4.8 (γ_{moist}=18 KN/m^3, γ_{sat}=19 KN/m^3) and Table 4.9 (γ_{moist}=21 KN/m^3, γ_{sat}=22 KN/m^3) present the factor of safety for different unit weights of soil for the case of no seepage and seepage. Results are presented for various embankment geometries and soil shear strength. Greater unit weight increases driving forces but at the same time, resisting frictional forces will also increase due to increase of normal force. The change in frictional resisting forces will depend on the angle of internal friction φ' and on the relative length of the slip surface. As a result, the F. S. decreases slightly in some cases and increases slightly in other cases for greater weight of soil. The F. S. increases gradually as the angle of internal friction φ' is increased. F. S. also increases with cohesion parameter c' but this increase is much larger. The effect of c' is substantial and it may be stated that values of c' about 30kPa or greater ensures that F.S.>1.2 under conditions of seepage for all embankments, except two cases in Table 4.9 (larger γ, H=12m, φ'=10^0, s $\leq$ 2).

Table 4.7 Factor of safety for no seepage and seepage condition for $\gamma_{moist} = 15\ KN/m^3$, $\gamma_{sat} = 16\ KN/m^3$.

c'	φ'	Case	Factor of Safety (F. S.)																					
			H= 3m					H= 5m					H= 8m				H= 10m				H= 12m			
(kPa)	(deg)		1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V
5	10	seepage	0.959	1.005	1.043	1.094	1.138	0.635	0.681	0.730	0.771	0.820	0.484	0.536	0.574	0.638	0.420	0.459	0.507	0.567	0.391	0.419	0.479	0.509
		no seepage	1.072	1.213	1.337	1.454	1.567	0.763	0.887	1.003	1.118	1.229	0.693	0.804	0.914	1.022	0.626	0.735	0.843	0.950	0.581	0.688	0.794	0.900
	20	seepage	1.204	1.292	1.381	1.481	1.367	0.836	0.927	1.024	1.104	1.200	0.681	0.784	0.865	0.995	0.615	0.689	0.786	0.903	0.589	0.647	0.767	0.823
		no seepage	1.385	1.631	1.880	2.099	2.327	1.054	1.281	1.510	1.732	1.945	1.064	1.278	1.492	1.707	0.988	1.198	1.409	1.619	0.936	1.142	1.353	1.508
	30	seepage	1.463	1.603	1.738	1.888	2.019	1.040	1.183	1.332	1.464	1.608	0.892	1.051	1.175	1.384	0.813	0.934	1.086	1.270	0.806	0.894	1.081	1.164
		no seepage	1.711	2.066	2.437	2.806	3.144	1.386	1.688	2.031	2.377	2.716	1.469	1.789	2.118	2.447	1.396	1.704	2.027	2.351	1.318	1.639	1.961	2.226
	40	seepage	1.772	1.941	2.141	2.363	2.557	1.265	1.472	1.699	1.883	2.096	1.136	1.358	1.551	1.845	1.053	1.226	1.445	1.714	1.058	1.190	1.459	1.574
		no seepage	2.082	2.585	3.087	3.625	4.123	1.741	2.183	2.665	3.153	3.633	1.949	2.409	2.875	3.341	1.873	2.316	2.774	3.235	1.782	2.232	2.705	3.100
10	10	seepage	1.650	1.683	1.725	1.781	1.833	1.059	1.107	1.160	1.199	1.254	0.765	0.816	0.855	0.922	0.650	0.686	0.738	0.799	0.588	0.613	0.675	0.706
		no seepage	1.818	1.994	2.113	2.238	2.357	1.225	1.373	1.498	1.613	1.732	1.009	1.130	1.244	1.357	0.884	1.001	1.115	1.228	0.800	0.915	1.027	1.062
	20	seepage	1.933	2.028	2.107	2.212	2.302	1.281	1.378	1.478	1.564	1.664	0.980	1.087	1.166	1.298	0.851	0.932	1.032	1.154	0.794	0.852	0.975	1.036
		no seepage	2.163	2.452	2.708	2.949	3.182	1.543	1.797	2.036	2.273	2.504	1.411	1.636	1.864	2.087	1.275	1.499	1.721	1.941	1.184	1.404	1.623	1.751
	30	seepage	2.218	2.360	2.498	2.657	2.798	1.514	1.663	1.816	1.950	2.103	1.211	1.373	1.501	1.709	1.073	1.197	1.353	1.540	1.023	1.114	1.308	1.398
		no seepage	2.525	2.943	3.324	3.694	4.073	1.873	2.246	2.623	2.970	3.322	1.832	2.191	2.533	2.876	1.687	2.036	2.375	2.713	1.589	1.927	2.264	2.499
	40	seepage	2.544	2.745	2.947	3.178	3.376	1.787	1.984	2.214	2.399	2.615	1.473	1.707	1.892	2.194	1.332	1.508	1.727	1.999	1.290	1.421	1.698	1.824
		no seepage	2.949	3.491	4.078	4.567	5.081	2.277	2.782	3.298	3.802	4.309	2.345	2.822	3.313	3.801	2.188	2.662	3.145	3.624	2.074	2.548	3.025	3.390
30	10	seepage	4.360	4.380	4.433	4.511	4.591	2.758	2.836	2.873	2.913	2.982	1.929	1.962	1.979	2.052	1.609	1.622	1.656	1.720	1.412	1.408	1.457	1.481
		no seepage	4.766	4.883	4.990	5.132	5.275	2.981	3.202	3.323	3.447	3.571	2.224	2.334	2.459	2.575	1.884	2.005	2.115	2.232	1.663	1.772	1.881	1.787
	20	seepage	4.673	4.740	4.825	4.950	5.071	2.974	3.088	3.191	3.275	3.395	2.116	2.215	2.285	2.427	1.780	1.847	1.947	2.077	1.591	1.628	1.755	1.816
		no seepage	5.153	5.517	5.763	6.014	6.271	3.346	3.710	3.960	4.207	4.458	2.647	2.903	3.146	3.386	2.293	2.541	2.781	3.016	2.054	2.293	2.529	2.546
	30	seepage	5.028	5.142	5.280	5.457	5.623	3.234	3.389	3.562	3.690	3.869	2.347	2.518	2.647	2.863	1.998	2.122	2.287	2.491	1.813	1.899	2.101	2.205
		no seepage	5.543	6.170	6.508	6.908	7.291	3.770	4.228	4.640	5.019	5.405	3.135	3.525	3.900	4.267	2.755	3.138	3.507	3.874	2.501	2.873	3.241	3.370
	40	seepage	5.451	5.633	5.832	6.076	6.299	3.550	3.770	4.010	4.205	4.449	2.648	2.898	3.081	3.397	2.277	2.466	2.700	2.990	2.090	2.233	2.520	2.666
		no seepage	6.031	6.782	7.361	7.925	8.476	4.216	4.829	5.400	5.954	6.491	3.698	4.238	4.772	5.302	3.312	3.837	4.363	4.879	3.046	3.568	4.077	4.342

Table 4.8 Factor of safety for no seepage and seepage condition for $\gamma_{moist} = 18\ KN/m^3$, $\gamma_{sat} = 19\ KN/m^3$.

c′ (kPa)	φ′ (deg)	Case	Factor of Safety (F. S.)																					
			H= 3m					H= 5m					H= 8m				H= 10m				H= 12m			
			1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V
5	10	seepage	0.859	0.925	0.979	1.037	1.092	0.584	0.646	0.705	0.758	0.814	0.472	0.533	0.583	0.655	0.417	0.466	0.524	0.591	0.392	0.431	0.499	0.539
		no seepage	0.947	1.079	1.200	1.319	1.429	0.682	0.801	0.917	1.028	1.141	0.639	0.748	0.856	0.962	0.580	0.687	0.794	0.900	0.542	0.646	0.753	0.856
	20	seepage	1.110	1.233	1.342	1.462	1.567	0.797	0.909	1.026	1.133	1.246	0.695	0.816	0.915	1.063	0.633	0.731	0.846	0.979	0.615	0.694	0.830	0.906
		no seepage	1.253	1.484	1.739	1.950	2.172	0.973	1.187	1.409	1.627	1.849	1.006	1.212	1.423	1.634	0.940	1.144	1.352	1.560	0.891	1.096	1.302	1.508
	30	seepage	1.380	1.556	1.720	1.907	2.076	1.009	1.187	1.375	1.533	1.711	0.939	1.119	1.281	1.511	0.864	1.019	1.200	1.408	0.864	0.981	1.195	1.312
		no seepage	1.564	1.912	2.270	2.640	2.986	1.289	1.590	1.926	2.269	2.604	1.401	1.720	2.044	2.368	1.342	1.648	1.963	2.284	1.270	1.548	1.907	2.226
	40	seepage	1.691	1.910	2.177	2.439	2.675	1.262	1.515	1.783	2.021	2.270	1.212	1.480	1.712	2.052	1.140	1.357	1.620	1.923	1.149	1.321	1.633	1.798
		no seepage	1.935	2.432	2.921	3.457	3.938	1.631	2.083	2.560	3.039	3.511	1.865	2.333	2.798	3.259	1.808	2.247	2.710	3.167	1.719	2.178	2.647	3.100
10	10	seepage	1.467	1.521	1.570	1.634	1.694	0.946	1.014	1.074	1.127	1.190	0.713	0.776	0.826	0.902	0.612	0.663	0.723	0.794	0.558	0.598	0.668	0.711
		no seepage	1.569	1.738	1.863	1.983	2.098	1.076	1.211	1.335	1.453	1.571	0.907	1.024	1.137	1.250	0.800	0.915	1.028	1.139	0.729	0.841	0.952	1.062
	20	seepage	1.733	1.869	1.980	2.100	2.213	1.181	1.308	1.431	1.538	1.656	0.957	1.083	1.186	1.335	0.848	0.948	1.068	1.206	0.796	0.879	1.018	1.101
		no seepage	1.912	2.182	2.432	2.679	2.905	1.381	1.625	1.863	2.092	2.326	1.299	1.525	1.746	1.966	1.181	1.403	1.623	1.840	1.104	1.320	1.539	1.751
	30	seepage	2.020	2.219	2.400	2.589	2.765	1.427	1.615	1.800	1.967	2.148	1.212	1.409	1.569	1.802	1.101	1.254	1.438	1.651	1.051	1.180	1.399	1.523
		no seepage	2.259	2.659	3.049	3.400	3.765	1.708	2.072	2.439	2.790	3.132	1.715	2.058	2.400	2.742	1.591	1.927	2.262	2.597	1.504	1.834	2.169	2.499
	40	seepage	2.357	2.643	2.881	3.168	3.405	1.710	1.961	2.232	2.474	2.738	1.518	1.792	2.023	2.359	1.388	1.613	1.876	2.183	1.362	1.538	1.850	2.024
		no seepage	2.677	3.196	3.756	4.269	4.769	2.115	2.582	3.086	3.590	4.091	2.220	2.689	3.174	3.655	2.089	2.550	3.024	3.499	1.977	2.451	2.920	3.390
30	10	seepage	3.746	3.794	3.841	3.923	4.006	2.367	2.461	2.515	2.567	2.642	1.683	1.733	1.772	1.851	1.411	1.446	1.498	1.570	1.244	1.265	1.330	1.368
		no seepage	4.043	4.174	4.286	4.420	4.557	2.541	2.748	2.869	2.995	3.116	1.920	2.043	2.157	2.278	1.633	1.754	1.867	1.987	1.448	1.588	1.670	1.787
	20	seepage	4.108	4.201	4.304	4.444	4.578	2.614	2.763	2.893	3.000	3.135	1.920	2.044	2.149	2.305	1.630	1.731	1.856	2.004	1.467	1.544	1.689	1.781
		no seepage	4.404	4.785	5.014	5.274	5.530	2.908	3.248	3.496	3.741	3.993	2.344	2.600	2.839	3.078	2.045	2.289	2.526	2.762	1.838	2.077	2.311	2.546
	30	seepage	4.468	4.663	4.828	5.029	5.222	2.901	3.122	3.318	3.492	3.696	2.202	2.410	2.572	2.818	1.893	2.063	2.259	2.489	1.733	1.867	2.095	2.235
		no seepage	4.797	5.338	5.756	6.141	6.516	3.313	3.743	4.146	4.535	4.914	2.824	3.204	3.572	3.946	2.502	2.874	3.241	3.603	2.286	2.652	3.012	3.370
	40	seepage	4.839	5.183	5.404	5.967	5.957	3.233	3.542	3.822	4.073	4.351	2.546	2.847	3.081	3.428	2.224	2.466	2.745	3.069	2.067	2.262	2.587	2.783
		no seepage	5.280	5.969	6.556	7.113	7.659	3.745	4.345	4.898	5.444	5.976	3.376	3.904	4.433	4.950	3.047	3.563	4.083	4.592	2.818	3.333	3.839	4.342

Table 4.9 Factor of safety for no seepage and seepage condition for γ_{moist} =21 KN/m^3, γ_{sat} =22 KN/m^3.

c' (kPa)	φ' (deg)	Case	Factor of Safety (F. S.)																					
			H= 3m					H= 5m					H= 8m				H= 10m				H= 12m			
			1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V
5	10	seepage	0.785	0.860	0.925	0.990	1.051	0.544	0.615	0.680	0.739	0.804	0.460	0.526	0.585	0.662	0.411	0.468	0.532	0.604	0.389	0.437	0.510	0.558
		no seepage	0.854	0.983	1.101	1.219	1.330	0.624	0.740	0.855	0.963	1.074	0.597	0.707	0.813	0.918	0.546	0.652	0.758	0.862	0.512	0.616	0.721	0.824
	20	seepage	1.038	1.180	1.300	1.439	1.561	0.764	0.888	1.023	1.140	1.270	0.704	0.835	0.949	1.105	0.644	0.757	0.886	1.029	0.630	0.725	0.871	0.964
		no seepage	1.155	1.380	1.620	1.844	2.060	0.914	1.115	1.334	1.552	1.769	0.960	1.163	1.373	1.582	0.905	1.104	1.309	1.515	0.856	1.061	1.264	1.468
	30	seepage	1.315	1.498	1.706	1.910	2.099	0.987	1.185	1.397	1.581	1.775	0.959	1.161	1.351	1.595	0.898	1.069	1.273	1.501	0.894	1.036	1.271	1.411
		no seepage	1.457	1.803	2.151	2.520	2.865	1.217	1.519	1.850	2.188	2.519	1.353	1.669	1.990	2.311	1.300	1.604	1.918	2.236	1.235	1.546	1.869	2.185
	40	seepage	1.621	1.875	2.197	2.488	2.757	1.253	1.527	1.835	2.120	2.396	1.262	1.554	1.820	2.181	1.194	1.446	1.734	2.069	1.204	1.414	1.749	1.948
		no seepage	1.828	2.323	2.802	3.337	3.800	1.543	2.012	2.485	2.957	3.425	1.800	2.272	2.743	3.200	1.745	2.198	2.665	3.119	1.673	2.138	2.596	3.046
10	10	seepage	1.308	1.390	1.448	1.518	1.583	0.863	0.942	1.009	1.071	1.140	0.672	0.743	0.802	0.882	0.583	0.643	0.711	0.786	0.536	0.586	0.662	0.713
		no seepage	1.392	1.555	1.682	1.798	1.914	0.966	1.096	1.218	1.336	1.451	0.831	0.946	1.059	1.172	0.739	0.852	0.964	1.073	0.678	0.788	0.897	1.006
	20	seepage	1.584	1.739	1.873	2.008	2.133	1.102	1.246	1.382	1.503	1.636	0.933	1.072	1.193	1.351	0.837	0.953	1.085	1.235	0.791	0.890	1.041	1.140
		no seepage	1.725	1.990	2.234	2.476	2.708	1.264	1.503	1.740	1.963	2.190	1.215	1.442	1.660	1.878	1.113	1.333	1.551	1.764	1.044	1.260	1.475	1.687
	30	seepage	1.875	2.102	2.305	2.526	2.719	1.355	1.559	1.773	1.965	2.176	1.212	1.427	1.609	1.858	1.107	1.284	1.491	1.721	1.069	1.219	1.455	1.603
		no seepage	2.070	2.448	2.851	3.190	3.545	1.593	1.946	2.302	2.649	2.995	1.632	1.963	2.301	2.638	1.521	1.849	2.182	2.513	1.444	1.767	2.098	2.425
	40	seepage	2.219	2.528	2.812	3.123	3.407	1.642	1.933	2.243	2.509	2.802	1.545	1.842	2.109	2.467	1.419	1.678	1.974	2.305	1.399	1.611	1.951	2.160
		no seepage	2.464	2.980	3.517	4.056	4.545	2.000	2.439	2.936	3.439	3.931	2.123	2.590	3.067	3.545	2.020	2.468	2.937	3.407	1.907	2.373	2.843	3.309
30	10	seepage	3.303	3.361	3.413	3.499	3.585	2.085	2.190	2.257	2.318	2.396	1.507	1.569	1.622	1.706	1.268	1.320	1.385	1.461	1.123	1.163	1.238	1.285
		no seepage	3.525	3.668	3.782	3.910	4.044	2.227	2.424	2.546	2.669	2.791	1.702	1.825	1.942	2.063	1.455	1.576	1.690	1.808	1.294	1.406	1.520	1.635
	20	seepage	3.679	3.807	3.925	4.076	4.222	2.353	2.530	2.669	2.799	2.945	1.774	1.920	2.041	2.213	1.519	1.644	1.783	1.944	1.376	1.478	1.635	1.744
		no seepage	3.870	4.237	4.480	4.737	4.985	2.596	2.911	3.165	3.404	3.651	2.128	2.381	2.617	2.851	1.864	2.106	2.342	2.575	1.685	1.922	2.154	2.385
	30	seepage	3.993	4.277	4.457	4.689	4.897	2.657	2.907	3.128	3.330	3.552	2.085	2.319	2.510	2.770	1.815	2.013	2.231	2.476	1.674	1.837	2.084	2.251
		no seepage	4.266	4.791	5.191	5.579	5.958	2.976	3.398	3.793	4.177	4.547	2.595	2.967	3.336	3.702	2.319	2.682	3.046	3.402	2.132	2.491	2.843	3.198
	40	seepage	4.381	4.767	5.062	5.378	5.678	2.992	3.339	3.662	3.946	4.261	2.455	2.783	3.062	3.435	2.175	2.450	2.761	3.111	2.033	2.269	2.620	2.853
		no seepage	4.744	5.390	5.974	6.535	7.066	3.398	3.975	4.527	5.064	5.601	3.146	3.664	4.183	4.696	2.846	3.362	3.872	4.377	2.655	3.153	3.660	4.156

4.3.4 Effect of Seepage

The effect of seepage may be observed in Figs. 4.12, 4.14, 4.15 and Tables 4.7, 4.8, 4.9, 4.10. Seepage effect reduces the factor of safety, the effect is larger for greater φ'. Inspection of Table 4.8 which corresponds to a reasonable value of unit weight (γ_{moist}=18KN/m^3, γ_{sat}=19KN/m^3) and critical seepage condition (F_B=0.5m) reveals that embankment slope stability may be of concern for low values of c' (5 kPa, 10 kPa or less). Minimum slope required according to φ' and c' values, and embankment height can be obtained from this table. As for example for 3m high embankments, φ' should be at least 20^0 and slope at least 1.5:1 if c' =5 kPa or less.

Table 4.10 Presents values of F. S. for a freeboard of 1.25m. Comparison between Tables 4.8 and 4.10 show that the F. S. is slightly larger for larger freeboard. This is due to the fact that the seepage induced pore water pressures are smaller for a lower phreatic surface.

4.3.5 Effect of Earthquake

In this study, two horizontal earthquake coefficients of 0.15g and 0.25g are used to represent the case of earthquake occurrence in zones 2 and 3. The vertical coefficient is not considered. Occurrence of earthquake during the flood seasons further lowers the factor of safety of the embankment slopes. Fig. 4.13 and Fig. 4.14 show the combined effect of 0.15g earthquake and seepage. Complete results for both 0.15g and 0.25g earthquakes are presented in Tables. 4.11 and 4.12 for two different unit weights of soil. It may be observed that 0.15g earthquake, which may be considered as a quite likely event in zones 2 and 3 of Bangladesh, can result in unsafe slopes even for c'=30 kPa.

As discussed in Sec.4.3.3, for some cases with seepage plus earthquake, the F.S. decreases as the slope is made milder. This happens particularly for large c' (30 kPa) and low friction angle φ'(10°, 20°) and this effect is more for smaller embankment

Table 4.10 Factor of safety for no seepage and seepage conditions for free board $F_B = 1.25$m.

c′ (kPa)	φ′ (deg)	Case	Factor of Safety (F. S.) H= 3m					H= 5m					H= 8m				H= 10m				H= 12m			
			1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V
5	10	seepage	0.985	0.983	1.096	1.172	1.265	0.657	0.706	0.758	0.83	0.896	0.48	0.553	0.626	0.681	0.434	0.487	0.543	0.617	0.385	0.437	0.498	0.557
		no seepage	1.02	1.082	1.22	1.34	1.469	0.737	0.83	0.933	1.04	1.154	0.657	0.757	0.864	0.97	0.596	0.696	0.803	0.906	0.554	0.655	0.758	0.862
	20	seepage	1.222	1.333	1.48	1.608	1.757	0.84	0.966	1.082	1.209	1.323	0.681	0.832	0.973	1.072	0.645	0.756	0.864	1.004	0.582	0.691	0.812	0.924
		no seepage	1.292	1.492	1.719	1.947	2.171	0.996	1.197	1.409	1.626	1.845	1.008	1.22	1.429	1.638	0.943	1.149	1.358	1.564	0.899	1.101	1.308	1.512
	30	seepage	1.504	1.703	1.921	2.101	2.315	1.055	1.263	1.45	1.639	1.806	0.903	1.14	1.366	1.513	0.879	1.053	1.22	1.442	0.799	0.969	1.16	1.332
		no seepage	1.611	1.935	2.269	2.616	2.965	1.301	1.605	1.934	2.265	2.599	1.402	1.72	2.047	2.373	1.331	1.644	1.966	2.287	1.28	1.589	1.908	2.227
	40	seepage	1.854	2.158	2.453	2.701	2.996	1.314	1.62	1.893	2.161	2.394	1.156	1.505	1.84	2.044	1.155	1.406	1.647	1.973	1.058	1.299	1.579	1.825
		no seepage	2.006	2.446	2.934	3.412	3.899	1.677	2.084	2.556	3.032	3.504	1.869	2.319	2.785	3.248	1.792	2.234	2.696	3.153	1.74	2.174	2.63	3.087
10	10	seepage	1.776	1.616	1.836	1.938	2.068	1.176	1.183	1.218	1.304	1.392	0.783	0.847	0.929	0.995	0.679	0.721	0.786	0.87	0.587	0.63	0.7	0.769
		no seepage	1.814	1.737	2.966	2.113	2.279	1.257	1.316	1.4	1.523	1.659	0.974	1.065	1.174	1.293	0.855	0.948	1.054	1.167	0.774	0.869	0.973	1.082
	20	seepage	1.982	1.987	2.213	2.371	2.56	1.324	1.425	1.535	1.681	1.818	0.972	1.123	1.274	1.386	0.881	0.99	1.106	1.257	0.782	0.89	1.015	1.136
		no seepage	2.054	2.188	2.47	2.716	2.979	1.488	1.681	1.895	2.116	2.35	1.335	1.543	1.763	1.98	1.212	1.421	1.639	1.853	1.128	1.337	1.55	1.764
	30	seepage	2.245	2.391	2.65	2.865	3.119	1.525	1.716	1.902	2.112	2.303	1.199	1.44	1.667	1.83	1.121	1.296	1.471	1.697	1.007	1.18	1.374	1.554
		no seepage	2.358	2.663	3.046	3.406	3.778	1.777	2.099	2.436	2.793	3.138	1.734	2.072	2.409	2.751	1.607	1.943	2.275	2.606	1.52	1.849	2.179	2.508
	40	seepage	2.585	2.858	3.19	3.471	3.804	1.786	2.083	2.355	2.641	2.898	1.474	1.824	2.15	2.374	1.411	1.666	1.915	2.236	1.276	1.527	1.807	2.062
		no seepage	2.747	3.215	3.72	4.253	4.76	2.146	2.606	3.092	3.587	4.09	2.219	2.701	3.183	3.663	2.087	2.558	3.037	3.508	1.997	2.458	2.93	3.401
30	10	seepage	5.036	4.15	4.828	5.011	5.282	3.352	3.165	3.087	3.212	3.38	2.063	2.05	2.148	2.258	1.716	1.687	1.765	1.886	1.457	1.439	1.52	1.619
		no seepage	5.086	4.297	4.985	5.222	5.532	3.455	3.325	3.302	3.469	3.688	2.288	2.293	2.424	2.591	1.918	1.941	2.063	2.214	1.666	1.702	1.819	1.96
	20	seepage	5.179	4.517	5.186	5.437	5.772	3.425	3.356	3.383	3.582	3.803	2.2	2.304	2.484	2.644	1.873	1.935	2.077	2.269	1.605	1.677	1.826	1.983
		no seepage	5.261	4.786	5.466	5.814	6.225	3.601	3.641	3.773	4.051	4.372	2.61	2.766	3.005	3.274	2.249	2.417	2.645	2.898	2.005	2.181	2.402	2.664
	30	seepage	5.378	4.943	5.605	5.925	6.33	3.561	3.606	3.734	4.007	4.286	2.393	2.608	2.873	3.086	2.084	2.231	2.438	2.708	1.806	1.961	2.181	2.399
		no seepage	5.498	5.333	6.027	6.494	7.018	3.823	4.037	4.325	4.72	5.154	3.014	3.318	3.672	4.055	2.659	2.968	3.312	3.678	2.411	2.732	3.07	3.424
	40	seepage	5.657	5.467	6.128	6.528	7.017	3.749	3.939	4.174	4.533	4.88	2.651	2.992	3.353	3.63	2.362	2.604	2.883	3.247	2.074	2.319	2.62	2.911
		no seepage	5.825	5.976	6.727	7.333	7.994	4.136	4.568	5.015	5.546	6.118	3.519	3.999	4.495	5.014	3.154	3.632	4.134	4.636	2.907	3.383	3.886	4.38

Table 4.11 Factor of safety for conditions of seepage and occurrence of 0.15g and 0.25g earthquakes for γ_{moist} =18 KN/m^3, γ_{sat} =19 KN/m^3.

C′ (kPa)	φ′ (deg)	Case	Factor of Safety (F. S.) H= 3m 1H:1V	H= 3m 1.5H:1V	H= 3m 2H:1V	H= 3m 2.5H:1V	H= 3m 3H:1V	H= 5m 1H:1V	H= 5m 1.5H:1V	H= 5m 2H:1V	H= 5m 2.5H:1V	H= 5m 3H:1V	H= 8m 1.5H:1V	H= 8m 2H:1V	H= 8m 2.5H:1V	H= 8m 3H:1V	H= 10m 1.5H:1V	H= 10m 2H:1V	H= 10m 2.5H:1V	H= 10m 3H:1V	H= 12m 1.5H:1V	H= 12m 2H:1V	H= 12m 2.5H:1V	H= 12m 3H:1V
5	10	Seepage	0.859	0.925	0.979	1.037	1.092	0.584	0.646	0.705	0.758	0.814	0.472	0.533	0.583	0.655	0.417	0.466	0.524	0.591	0.392	0.431	0.499	0.539
		S+0.15g EQ	0.636	0.644	0.652	0.663	0.669	0.442	0.465	0.487	0.500	0.517	0.345	0.374	0.392	0.421	0.305	0.328	0.353	0.382	0.287	0.304	0.336	0.349
		S+0.25g EQ	0.525	0.526	0.525	0.526	0.525	0.371	0.386	0.399	0.404	0.413	0.288	0.308	0.318	0.338	0.254	0.270	0.287	0.306	0.240	0.250	0.274	0.280
	20	Seepage	1.110	1.233	1.342	1.462	1.567	0.797	0.909	1.026	1.133	1.246	0.695	0.816	0.915	1.063	0.633	0.731	0.846	0.979	0.615	0.694	0.830	0.906
		S+0.15g EQ	0.848	0.895	0.934	0.972	0.997	0.606	0.668	0.723	0.761	0.803	0.508	0.554	0.618	0.685	0.463	0.514	0.570	0.632	0.447	0.487	0.559	0.584
		S+0.25g EQ	0.714	0.744	0.761	0.776	0.785	0.512	0.555	0.595	0.616	0.640	0.422	0.472	0.500	0.546	0.384	0.420	0.461	0.504	0.370	0.397	0.451	0.465
	30	Seepage	1.380	1.556	1.720	1.907	2.076	1.009	1.187	1.375	1.533	1.711	0.939	1.119	1.281	1.511	0.864	1.019	1.200	1.408	0.864	0.981	1.195	1.312
		S+0.15g EQ	1.053	1.140	1.213	1.286	1.335	0.770	0.869	0.969	1.034	1.105	0.683	0.787	0.860	0.971	0.624	0.710	0.803	0.904	0.621	0.682	0.800	0.841
		S+0.25g EQ	0.891	0.949	0.999	1.038	1.064	0.648	0.722	0.798	0.839	0.883	0.561	0.643	0.693	0.773	0.514	0.578	0.647	0.719	0.509	0.554	0.643	0.667
	40	Seepage	1.691	1.910	2.177	2.439	2.675	1.262	1.515	1.783	2.021	2.270	1.212	1.480	1.712	2.052	1.140	1.357	1.620	1.923	1.149	1.321	1.633	1.798
		S+0.15g EQ	1.293	1.415	1.534	1.643	1.729	0.951	1.108	1.257	1.360	1.464	0.876	1.030	1.147	1.314	0.818	0.940	1.081	1.231	0.825	0.913	1.090	1.149
		S+0.25g EQ	1.090	1.181	1.264	1.333	1.381	0.793	0.921	1.032	1.097	1.166	0.721	0.837	0.919	1.042	0.668	0.761	0.866	0.976	0.674	0.738	0.872	0.907
10	10	Seepage	1.467	1.521	1.570	1.634	1.694	0.946	1.014	1.074	1.127	1.190	0.713	0.776	0.826	0.902	0.612	0.663	0.723	0.794	0.558	0.598	0.668	0.711
		S+0.15g EQ	1.032	1.024	1.018	1.017	1.014	0.709	0.724	0.735	0.738	0.747	0.522	0.544	0.556	0.579	0.452	0.469	0.490	0.515	0.414	0.425	0.454	0.463
		S+0.25g EQ	0.851	0.833	0.818	0.807	0.795	0.602	0.604	0.602	0.596	0.595	0.439	0.451	0.454	0.466	0.380	0.389	0.401	0.415	0.348	0.353	0.371	0.374
	20	Seepage	1.733	1.869	1.980	2.100	2.213	1.181	1.308	1.431	1.538	1.656	0.957	1.083	1.186	1.335	0.848	0.948	1.068	1.206	0.796	0.879	1.018	1.101
		S+0.15g EQ	1.288	1.304	1.322	1.345	1.359	0.893	0.944	0.989	1.017	1.052	0.699	0.761	0.798	0.858	0.617	0.667	0.720	0.778	0.582	0.618	0.686	0.711
		S+0.25g EQ	1.063	1.065	1.065	1.068	1.066	0.751	0.783	0.810	0.822	0.839	0.584	0.627	0.648	0.688	0.515	0.549	0.585	0.624	0.485	0.510	0.558	0.570
	30	Seepage	2.020	2.219	2.400	2.589	2.765	1.427	1.615	1.800	1.967	2.148	1.212	1.409	1.569	1.802	1.101	1.254	1.438	1.651	1.051	1.180	1.399	1.523
		S+0.15g EQ	1.536	1.608	1.654	1.705	1.739	1.085	1.180	1.266	1.320	1.381	0.888	0.993	1.059	1.162	0.800	0.881	0.970	1.066	0.767	0.830	0.941	0.981
		S+0.25g EQ	1.295	1.317	1.337	1.358	1.369	0.915	0.983	1.037	1.066	1.101	0.739	0.817	0.858	0.927	0.665	0.723	0.784	0.851	0.637	0.680	0.761	0.782
	40	Seepage	2.357	2.643	2.881	3.168	3.405	1.710	1.961	2.232	2.474	2.738	1.518	1.792	2.023	2.359	1.388	1.613	1.876	2.183	1.362	1.538	1.850	2.024
		S+0.15g EQ	1.801	1.919	2.019	2.110	2.178	1.302	1.443	1.576	1.668	1.764	1.106	1.262	1.364	1.523	1.013	1.130	1.262	1.407	0.985	1.075	1.243	1.303
		S+0.25g EQ	1.528	1.601	1.653	1.696	1.719	1.096	1.202	1.297	1.352	1.410	0.919	1.035	1.103	1.212	0.838	0.924	1.020	1.121	0.811	0.877	1.002	1.036
30	10	Seepage	3.746	3.794	3.841	3.923	4.006	2.367	2.461	2.515	2.567	2.642	1.683	1.733	1.772	1.851	1.411	1.446	1.498	1.570	1.244	1.265	1.330	1.368
		S+0.15g EQ	2.632	2.552	2.491	2.447	2.405	1.811	1.782	1.728	1.689	1.667	1.261	1.230	1.199	1.197	1.067	1.036	1.020	1.022	0.944	0.910	0.908	0.894
		S+0.25g EQ	2.182	2.088	2.009	1.948	1.891	1.560	1.490	1.423	1.370	1.332	1.071	1.021	0.980	0.963	0.909	0.863	0.836	0.824	0.805	0.759	0.745	0.723
	20	Seepage	4.108	4.201	4.304	4.444	4.578	2.614	2.763	2.893	3.000	3.135	1.920	2.044	2.149	2.305	1.630	1.731	1.856	2.004	1.467	1.544	1.689	1.781
		S+0.15g EQ	2.871	2.822	2.786	2.765	2.741	1.974	1.984	1.974	1.964	1.968	1.416	1.442	1.444	1.482	1.210	1.229	1.260	1.298	1.093	1.100	1.151	1.161
		S+0.25g EQ	2.369	2.300	2.240	2.195	2.150	1.684	1.652	1.620	1.587	1.569	1.192	1.196	1.179	1.192	1.020	1.022	1.032	1.047	0.924	0.915	0.943	0.939
	30	Seepage	4.468	4.663	4.828	5.029	5.222	2.901	3.122	3.318	3.492	3.696	2.202	2.410	2.572	2.818	1.893	2.063	2.259	2.489	1.733	1.867	2.095	2.235
		S+0.15g EQ	3.164	3.144	3.133	3.136	3.130	2.173	2.228	2.270	2.286	2.320	1.612	1.690	1.729	1.811	1.397	1.459	1.530	1.613	1.282	1.325	1.423	1.455
		S+0.25g EQ	2.606	2.558	2.516	2.487	2.454	1.841	1.856	1.860	1.848	1.850	1.351	1.399	1.411	1.457	1.172	1.209	1.251	1.299	1.079	1.099	1.164	1.174
	40	Seepage	4.839	5.183	5.404	5.967	5.957	3.233	3.542	3.822	4.073	4.351	2.546	2.847	3.081	3.428	2.224	2.466	2.745	3.069	2.067	2.262	2.587	2.783
		S+0.15g EQ	3.519	3.538	3.555	3.595	3.612	2.429	2.537	2.624	2.680	2.751	1.860	1.995	2.070	2.202	1.625	1.737	1.855	1.983	1.518	1.597	1.749	1.804
		S+0.25g EQ	2.907	2.883	2.862	2.852	2.833	2.045	2.107	2.151	2.167	2.194	1.556	1.645	1.687	1.767	1.360	1.437	1.512	1.595	1.272	1.318	1.426	1.451

Table 4.12 Factor of safety for conditions of seepage and occurrence of 0.15g and 0.25g earthquakes for γ_{moist} =21 KN/m^3, γ_{sat} =22 KN/m^3.

c′ (kPa)	φ′ (deg)	Case	Factor of Safety (F. S.)																					
			H= 3m					H= 5m					H= 8m				H= 10m				H= 12m			
			1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V
5	10	Seepage	0.785	0.860	0.925	0.990	1.051	0.544	0.615	0.680	0.739	0.804	0.460	0.526	0.585	0.662	0.411	0.468	0.532	0.604	0.389	0.437	0.510	0.558
		S+0.15g EQ	0.595	0.616	0.630	0.646	0.657	0.417	0.450	0.477	0.495	0.516	0.339	0.373	0.395	0.428	0.303	0.331	0.360	0.392	0.286	0.309	0.345	0.362
		S+0.25g EQ	0.498	0.503	0.508	0.514	0.516	0.352	0.374	0.391	0.400	0.411	0.283	0.307	0.321	0.342	0.253	0.273	0.293	0.314	0.239	0.255	0.280	0.290
	20	Seepage	1.038	1.180	1.300	1.439	1.561	0.764	0.888	1.023	1.140	1.270	0.704	0.835	0.949	1.105	0.644	0.757	0.886	1.029	0.630	0.725	0.871	0.964
		S+0.15g EQ	0.797	0.863	0.918	0.966	1.002	0.585	0.655	0.725	0.774	0.823	0.516	0.589	0.642	0.715	0.474	0.534	0.598	0.666	0.461	0.510	0.588	0.622
		S+0.25g EQ	0.678	0.722	0.755	0.781	0.796	0.496	0.548	0.597	0.627	0.658	0.429	0.485	0.520	0.571	0.393	0.439	0.484	0.531	0.382	0.416	0.475	0.495
	30	Seepage	1.315	1.498	1.706	1.910	2.099	0.987	1.185	1.397	1.581	1.775	0.959	1.161	1.351	1.595	0.898	1.069	1.273	1.501	0.894	1.036	1.271	1.411
		S+0.15g EQ	1.006	1.114	1.208	1.295	1.363	0.753	0.873	0.990	1.070	1.150	0.705	0.817	0.907	1.028	0.654	0.749	0.855	0.967	0.649	0.724	0.854	0.907
		S+0.25g EQ	0.853	0.930	0.997	1.051	1.088	0.634	0.727	0.813	0.867	0.920	0.582	0.668	0.731	0.819	0.539	0.611	0.689	0.768	0.532	0.590	0.687	0.720
	40	Seepage	1.621	1.875	2.197	2.488	2.757	1.253	1.527	1.835	2.120	2.396	1.262	1.554	1.820	2.181	1.194	1.446	1.734	2.069	1.204	1.414	1.749	1.948
		S+0.15g EQ	1.249	1.389	1.556	1.679	1.780	0.951	1.120	1.298	1.425	1.544	0.917	1.088	1.224	1.405	0.862	1.008	1.164	1.329	0.870	0.984	1.173	1.250
		S+0.25g EQ	1.060	1.163	1.283	1.364	1.424	0.794	0.931	1.062	1.150	1.230	0.757	0.887	0.984	1.117	0.707	0.819	0.934	1.054	0.714	0.799	0.942	0.990
10	10	Seepage	1.308	1.390	1.448	1.518	1.583	0.863	0.942	1.009	1.071	1.140	0.672	0.743	0.802	0.882	0.583	0.643	0.711	0.786	0.536	0.586	0.662	0.713
		S+0.15g EQ	0.949	0.948	0.948	0.954	0.956	0.649	0.676	0.694	0.705	0.719	0.495	0.524	0.541	0.569	0.431	0.456	0.482	0.510	0.397	0.416	0.450	0.465
		S+0.25g EQ	0.784	0.772	0.763	0.757	0.750	0.550	0.563	0.570	0.570	0.573	0.416	0.434	0.441	0.457	0.363	0.378	0.394	0.411	0.335	0.346	0.368	0.375
	20	Seepage	1.584	1.739	1.873	2.008	2.133	1.102	1.246	1.382	1.503	1.636	0.933	1.072	1.193	1.351	0.837	0.953	1.085	1.235	0.791	0.890	1.041	1.140
		S+0.15g EQ	1.203	1.248	1.280	1.312	1.336	0.842	0.913	0.969	1.008	1.050	0.688	0.759	0.806	0.873	0.615	0.674	0.736	0.800	0.581	0.630	0.705	0.739
		S+0.25g EQ	1.010	1.019	1.031	1.045	1.050	0.713	0.759	0.794	0.815	0.837	0.575	0.626	0.654	0.698	0.513	0.555	0.597	0.640	0.485	0.519	0.572	0.591
	30	Seepage	1.875	2.102	2.305	2.526	2.719	1.355	1.559	1.773	1.965	2.176	1.212	1.427	1.609	1.858	1.107	1.284	1.491	1.721	1.069	1.219	1.455	1.603
		S+0.15g EQ	1.441	1.540	1.618	1.689	1.738	1.037	1.153	1.256	1.331	1.405	0.891	1.008	1.090	1.205	0.814	0.908	1.009	1.115	0.781	0.861	0.984	1.038
		S+0.25g EQ	1.219	1.284	1.320	1.352	1.371	0.879	0.962	1.035	1.077	1.121	0.743	0.830	0.884	0.961	0.677	0.746	0.817	0.890	0.649	0.706	0.795	0.827
	40	Seepage	2.219	2.528	2.812	3.123	3.407	1.642	1.933	2.243	2.509	2.802	1.545	1.842	2.109	2.467	1.419	1.678	1.974	2.305	1.399	1.611	1.951	2.160
		S+0.15g EQ	1.705	1.856	1.989	2.108	2.193	1.259	1.423	1.590	1.701	1.818	1.129	1.300	1.425	1.595	1.039	1.183	1.331	1.490	1.018	1.134	1.314	1.394
		S+0.25g EQ	1.448	1.552	1.637	1.706	1.751	1.063	1.189	1.309	1.381	1.454	0.936	1.069	1.152	1.273	0.861	0.968	1.076	1.187	0.839	0.925	1.061	1.109
30	10	Seepage	3.303	3.361	3.413	3.499	3.585	2.085	2.190	2.257	2.318	2.396	1.507	1.569	1.622	1.706	1.268	1.320	1.385	1.461	1.123	1.163	1.238	1.285
		S+0.15g EQ	2.324	2.263	2.217	2.185	2.154	1.593	1.585	1.550	1.525	1.511	1.129	1.115	1.097	1.103	0.959	0.946	0.943	0.951	0.852	0.835	0.845	0.841
		S+0.25g EQ	1.926	1.850	1.788	1.739	1.693	1.370	1.326	1.275	1.235	1.207	0.958	0.926	0.897	0.888	0.815	0.788	0.772	0.767	0.725	0.698	0.693	0.680
	20	Seepage	3.679	3.807	3.925	4.076	4.222	2.353	2.530	2.669	2.799	2.945	1.774	1.920	2.041	2.213	1.519	1.644	1.783	1.944	1.376	1.478	1.635	1.744
		S+0.15g EQ	2.598	2.568	2.549	2.543	2.532	1.775	1.815	1.831	1.836	1.852	1.311	1.355	1.379	1.424	1.131	1.168	1.213	1.264	1.027	1.055	1.116	1.140
		S+0.25g EQ	2.143	2.093	2.050	2.018	1.986	1.513	1.518	1.502	1.485	1.477	1.103	1.123	1.127	1.145	0.953	0.971	0.993	1.019	0.868	0.878	0.914	0.920
	30	Seepage	3.993	4.277	4.457	4.689	4.897	2.657	2.907	3.128	3.330	3.552	2.085	2.319	2.510	2.770	1.815	2.013	2.231	2.476	1.674	1.837	2.084	2.251
		S+0.15g EQ	2.914	2.922	2.930	2.957	2.968	1.996	2.089	2.151	2.196	2.245	1.534	1.632	1.691	1.786	1.338	1.425	1.514	1.605	1.239	1.305	1.417	1.466
		S+0.25g EQ	2.411	2.381	2.360	2.346	2.327	1.688	1.740	1.766	1.776	1.789	1.289	1.351	1.380	1.435	1.125	1.180	1.236	1.292	1.043	1.082	1.157	1.180
	40	Seepage	4.381	4.767	5.062	5.378	5.678	2.992	3.339	3.662	3.946	4.261	2.455	2.783	3.062	3.435	2.175	2.450	2.761	3.111	2.033	2.269	2.620	2.853
		S+0.15g EQ	3.298	3.344	3.397	3.458	3.501	2.272	2.422	2.542	2.619	2.712	1.802	1.965	2.066	2.213	1.598	1.731	1.868	2.013	1.498	1.605	1.774	1.851
		S+0.25g EQ	2.725	2.731	2.737	2.749	2.747	1.915	2.012	2.083	2.118	2.163	1.509	1.620	1.680	1.776	1.337	1.429	1.521	1.616	1.253	1.325	1.445	1.486

heights. This may be explained as follows. As the slope is made flatter, the volume of sliding soil mass subjected to gravity force as well as horizontal inertia force becomes larger reducing the F.S. The increase in horizontal seismic force due to flatter slope has a larger effect for smaller embankment heights. On the other hand, increased length of the slip circle due to flatter slope results in increased resisting shear force along the slip surface both from cohesion and friction. The resisting force due to cohesion c′ depends only on the length of slip circle and does not depend on the weight. The resisting force due to friction increases with the length as well as the weight. For cases where c′ is large and φ′ is small, the cohesion effect plays a major role in the resisting shear force. The increased weight does not contribute as much, since φ′ is small. The net effect of all these factors is that the increase in driving forces due to increased weight and increased horizontal force almost balance the contribution of the increased resistance. It should be noted that in the cases where the F.S. reduces with more gentle slopes, the variation of F.S. is slight and may be neglected for practical purposes. This effect is evident in Figs 4.13(i, j). The values of F.S. may be obtained from Table 4.11

4.4 DEVELOPMENT OF DESIGN CHARTS AND DESIGN AIDS

The numerical results obtained for the critical seepage condition (F_B=0.5 m) are used to develop design charts, similar to those of Bishop and Morgenstern (1960). Results for two different unit weights of γ_{moist}=18 KN/m^3, γ_{sat}=19 KN/m^3 and γ_{moist}=21 KN/m^3, γ_{sat}=22 KN/m^3 have been incorporated. These design charts are based on dimensionless parameters $c'/\gamma H$, slope and φ' for the four conditions of no seepage, seepage and seepage with 0.15g and 0.25g earthquakes. The design charts are a convenient means of calculating the factor of safety under extreme conditions of seepage and for extreme conditions of seepage plus earthquake.

The dimensionless parameters are chosen to be $c'/\gamma H$, φ' and slope. Values of $c'/\gamma H$ for different parameters are shown in Table 4.13. For a particular slope, we have 30 values of $c'/\gamma H$ with 30 values of factor of safety for two unit weights of γ_{moist}=18 KN/m^3 and 21 KN/m^3. Plots of F.S. vs. $c'/\gamma H$ for different slopes are presented in Figs. 4.16(a) to

Table 4.13 Values of $c'/\gamma H$ for different parameters.

γ =18 KN/m^3					
c′ (kPa)	H (m)				
	3	5	8	10	12
5	0.0926	0.055	0.0347	0.027	0.023
10	0.1852	0.11	0.069	0.055	0.046
30	0.555	0.33	0.208	0.166	0.1388
γ =21KN/m^3					
5	0.0794	0.0476	0.0298	0.0238	0.0198
10	0.1587	0.0952	0.0595	0.0476	0.0397
30	0.4762	0.2857	0.1786	0.1428	0.1190

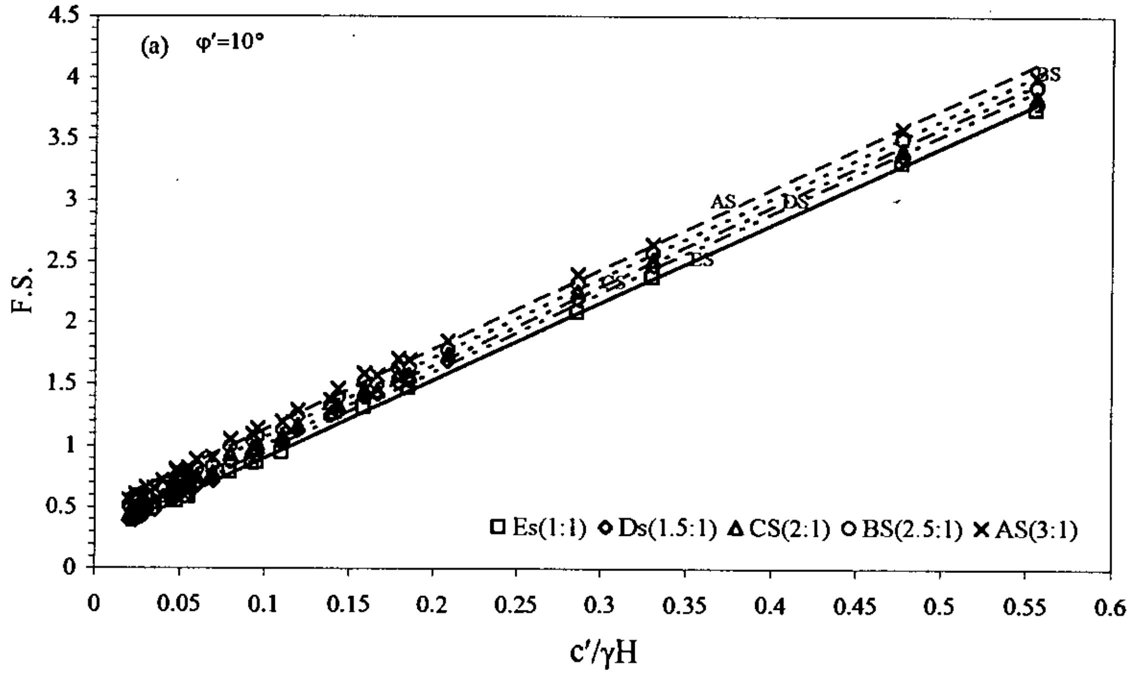
4.16(d) for critical seepage condition. Figs. 4.16 (a), (b), (c), (d) correspond to φ' $=10^0, 20^0, 30^0, 40^0$ respectively. Best fit curves are drawn based on linear regression of data for a particular slope. As a result, five such curves AS to ES are obtained for the five slopes. These curves are then used to obtain the design charts of Fig. 4.17 (a), (b), (c), (d) which correspond to $c'/\gamma H$=0.025, 0.05, 0.1, 0.2 respectively. In these design charts, the F.S. is presented as a function of φ' and slope for a particular value of stability number ($c'/\gamma H$). For stability number other than the specified values, linear interpolation may be used.

In a similar manner, design charts are developed for the other conditions of no seepage, seepage with 0.15g earthquake and seepage with 0.25g. Figs. 4.18, 4.19, 4.20 present design charts for these cases.

For use as a design aid, another type of chart (Figs. 4.21 to 4.24) is developed which gives the minimum slope required to achieve a specified factor of safety for various embankment heights and soil properties. As discussed earlier in Chapter 2, embankments are normally designed for F.S. values greater than about 1.2 to 1.5, but for earthquakes lower values of F.S. can be considered. Figs. 4.21 and 4.22 present the case of critical seepage condition for F.S. value of 1.2 and 1.5 respectively. Figs. 4.23 and 4.24 present the case of critical seepage plus 0.15g earthquake condition for F.S. value of 1.0 and 1.2 respectively.

It must be noted that this study considers slopes of 1:1 to 3:1 for 3m to 5m high embankments, and slopes of 1.5:1 to 3:1 for 8m to 12 m high embankments. In other words, it is assumed that the minimum slope for 3m to 5m high embankment should be 1:1, while that for 8m to 12 m high embankments should be 1.5:1. Slopes milder than 3:1 have not been considered in this study. As a result, points are missing in the plots for cases where the required F.S. was not obtained in the slope range studied.

It may be observed that for low values of c' (5 kPa, 10 kPa) and low values of φ' (10^o, 20^o), slopes of 3:1 may not be sufficient for seepage conditions. For seepage plus



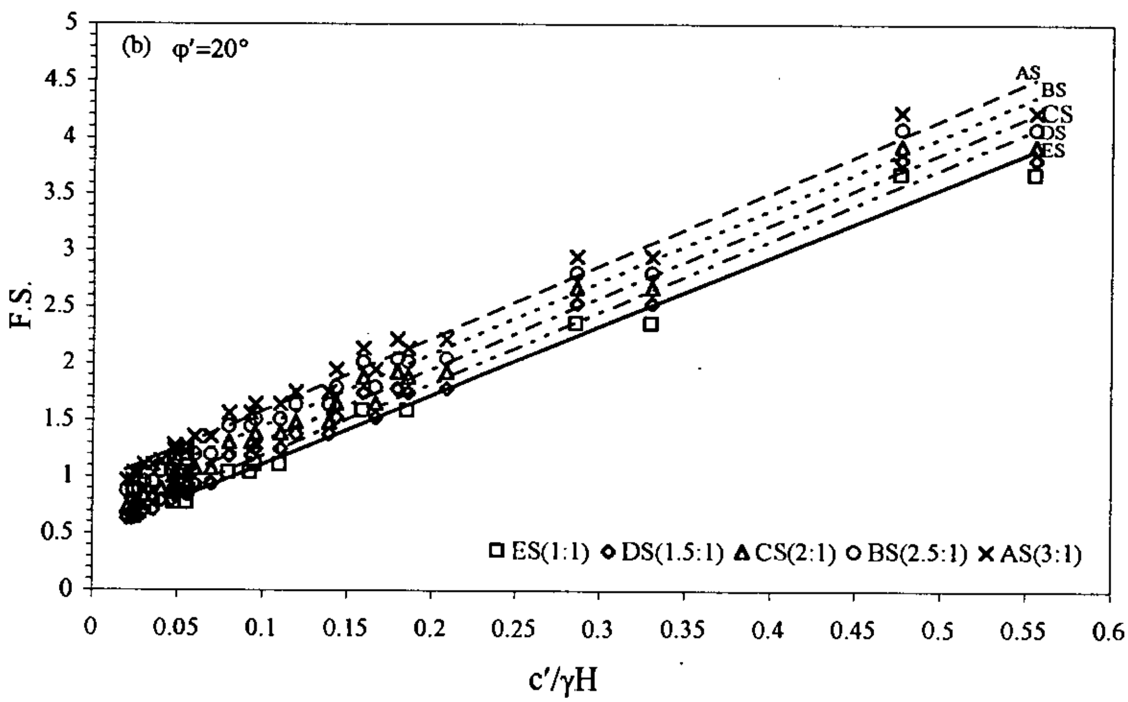
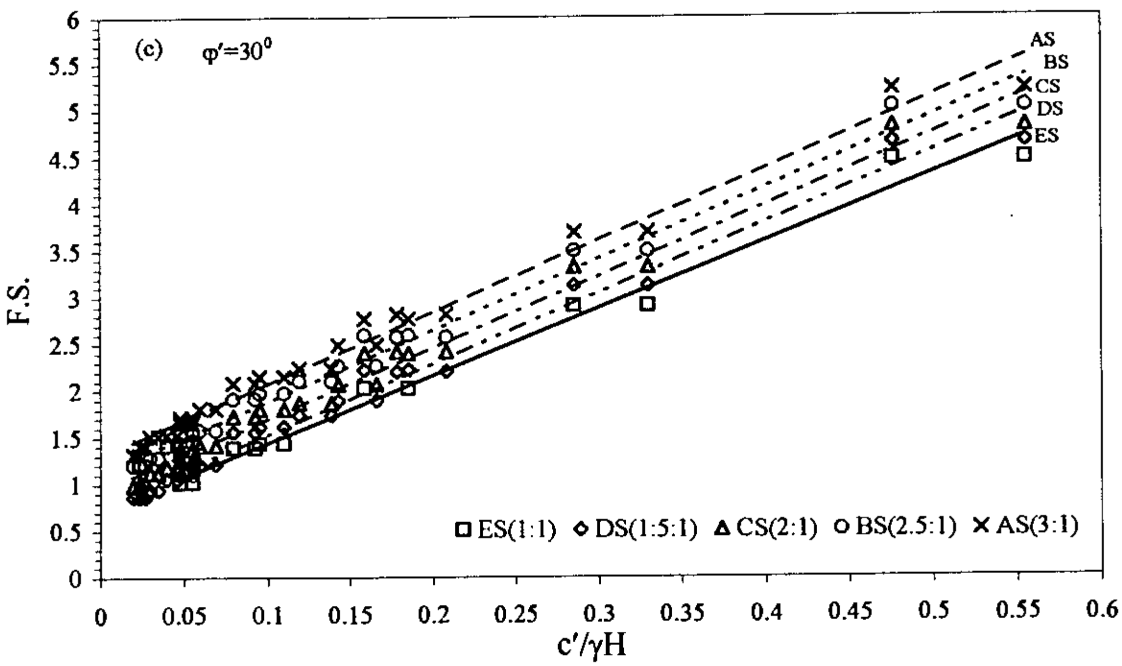


Fig. 4.16 Linear Regression curves for Embankment factor of safety under seepage condition as a function of stability number for (a) $\varphi'=10^0$ (b) $\varphi'=20^0$.



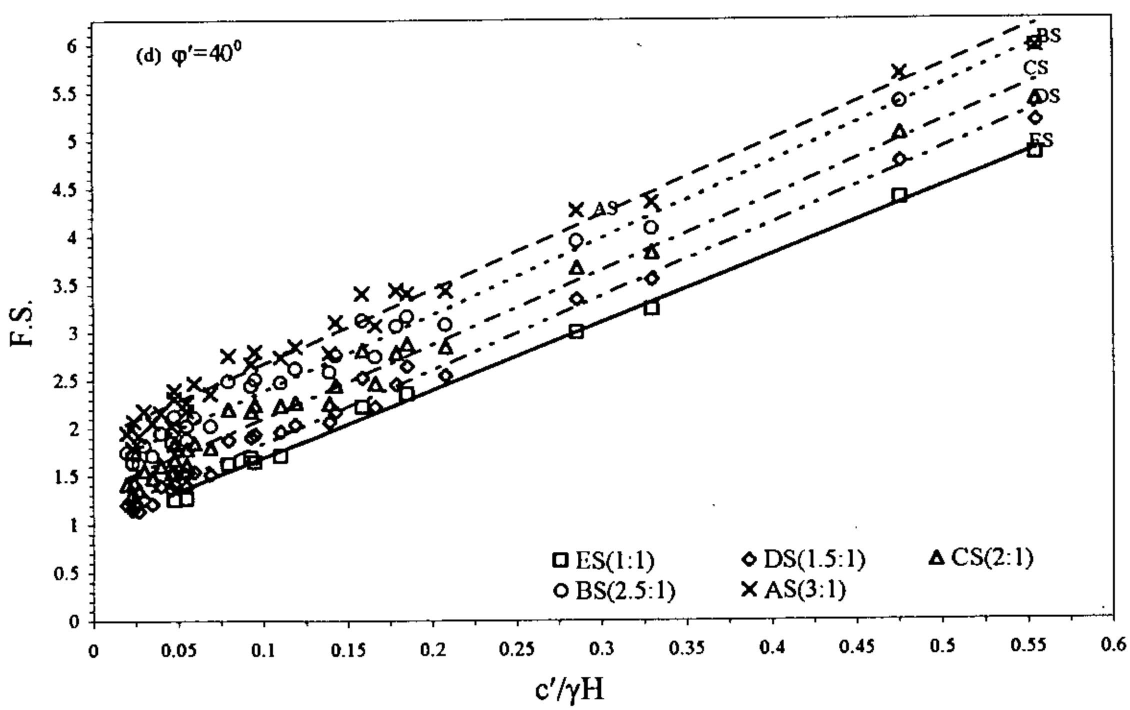


Fig. 4.16 Linear Regression curves for Embankment factor of safety under seepage condition as a function of stability number for (c) $\varphi'=30^0$ (d) $\varphi'=40^0$.

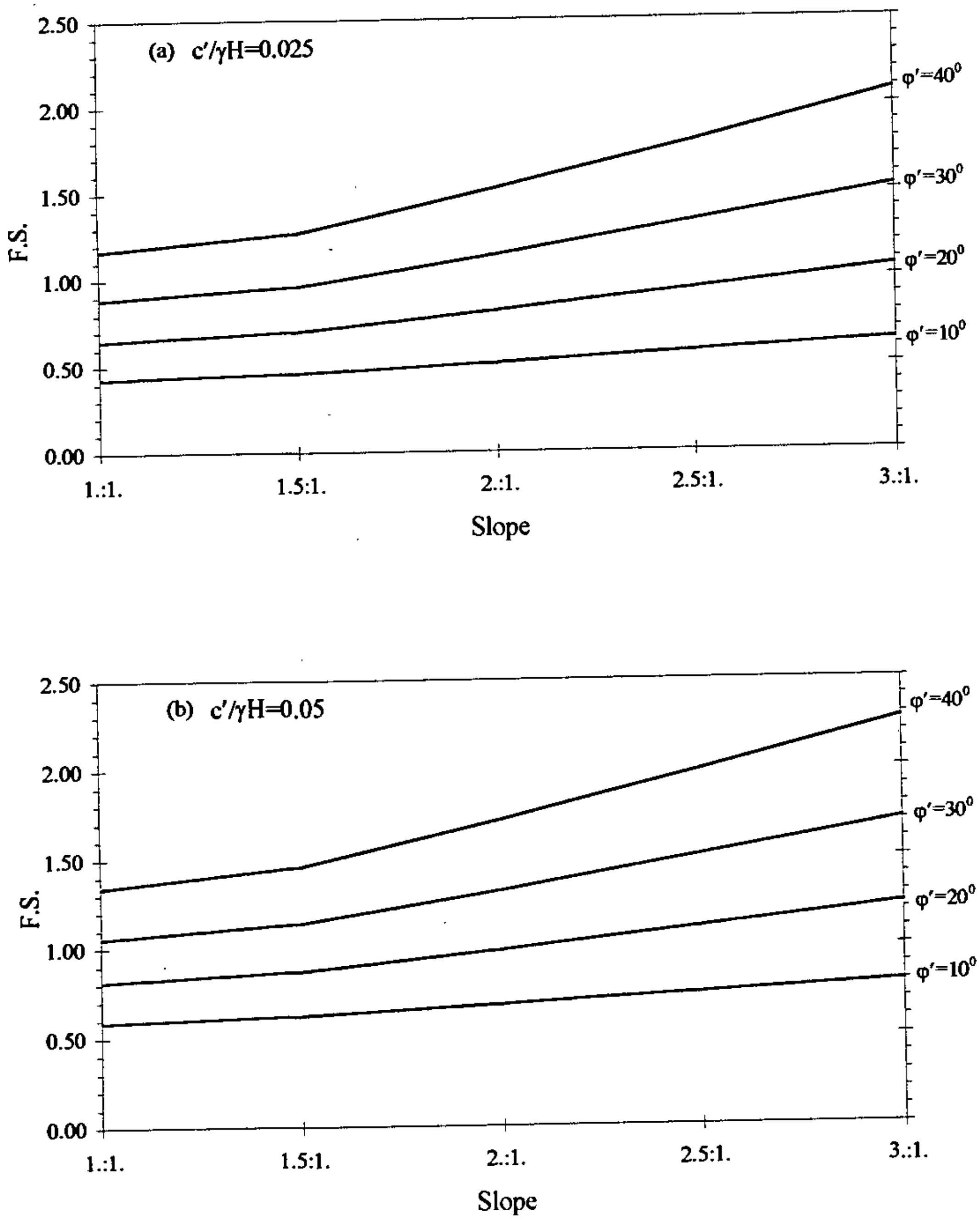


Fig. 4.17 Design Chart for embankment stability under critical seepage condition for (a) $c'/\gamma H=0.025$ (b) $c'/\gamma H=0.05$.

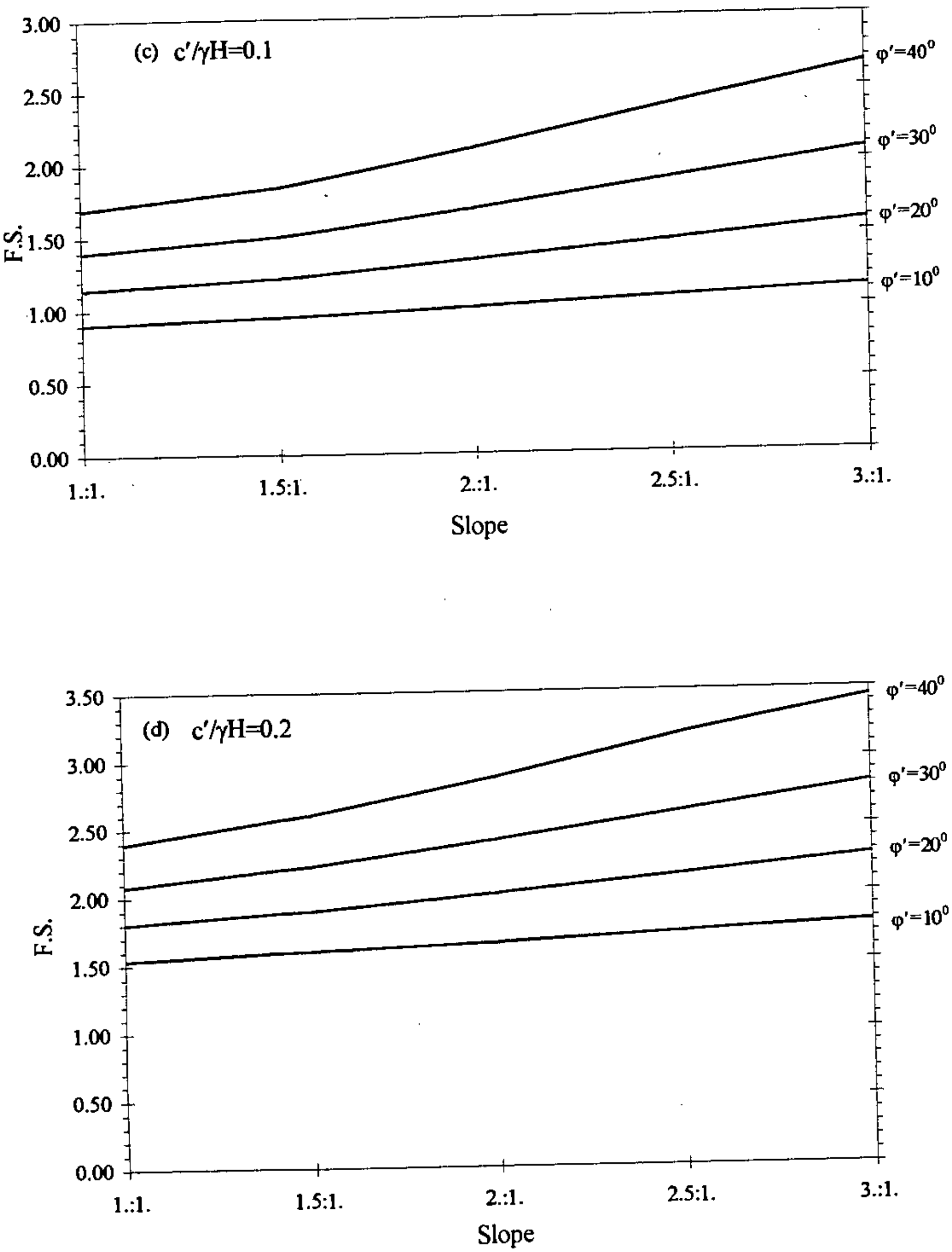
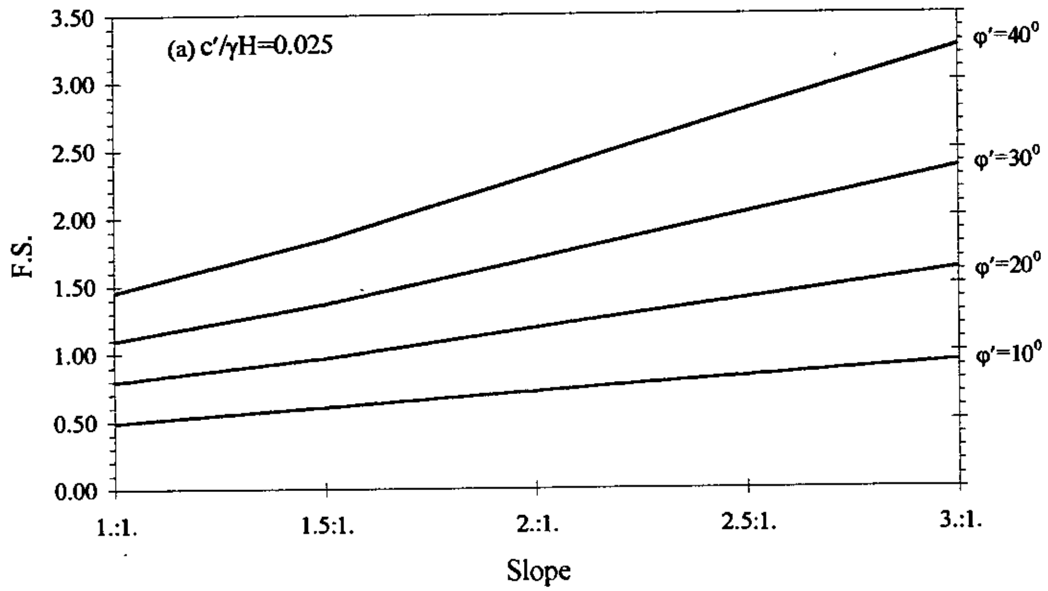


Fig. 4.17 Design Chart for embankment stability under critical secpage condition for (c) $c'/\gamma H=0.1$ (d) $c'/\gamma H=0.2$.



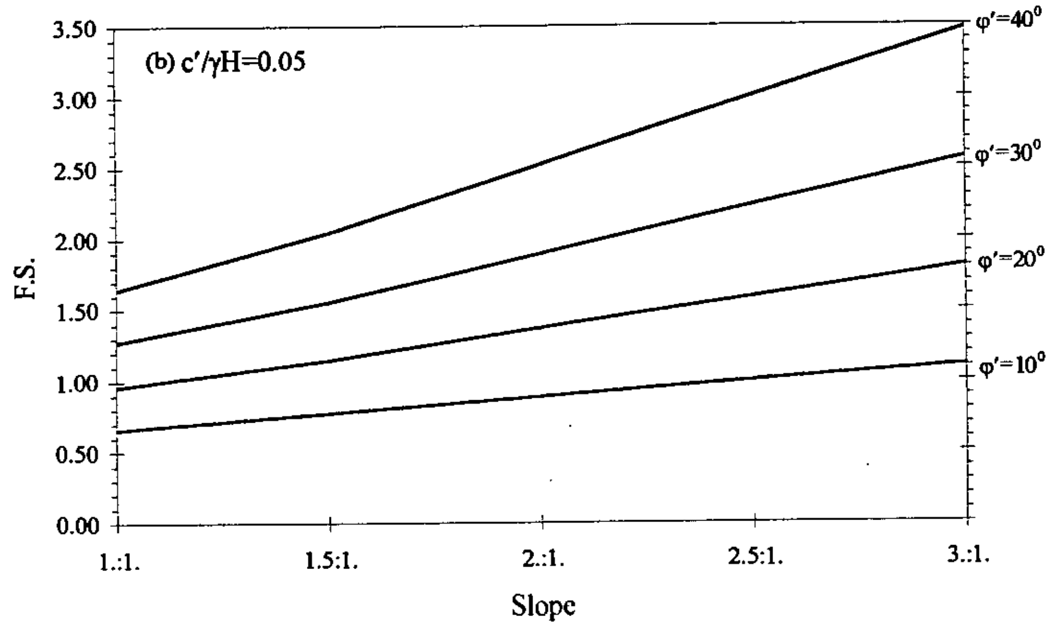
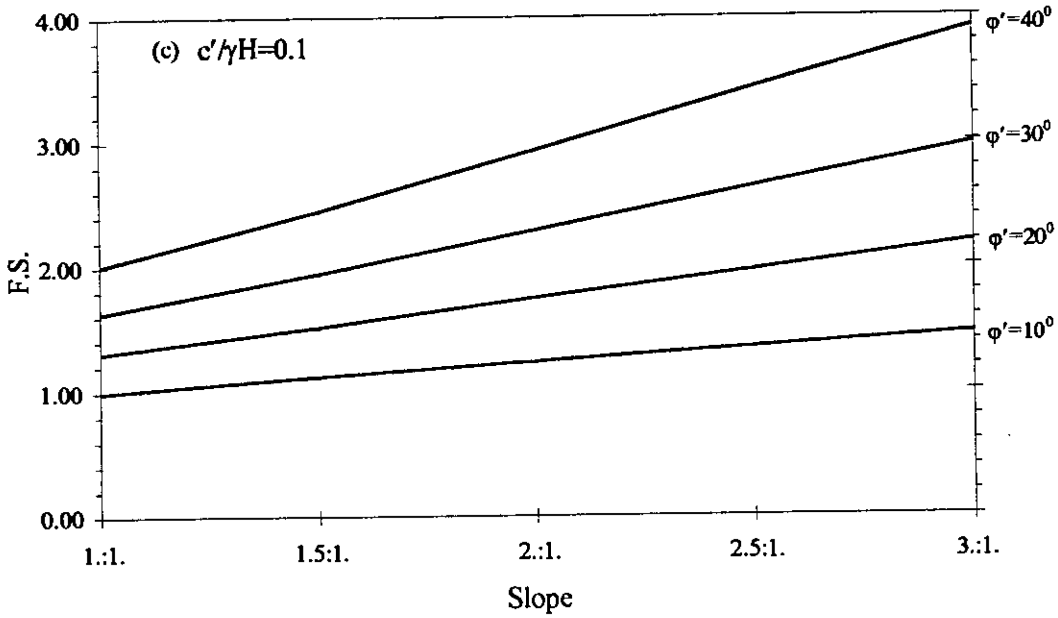


Fig. 4.18 Design Chart for embankment stability under no seepage condition for (a) $c'/\gamma H$=0.025 (b) $c'/\gamma H$=0.05.



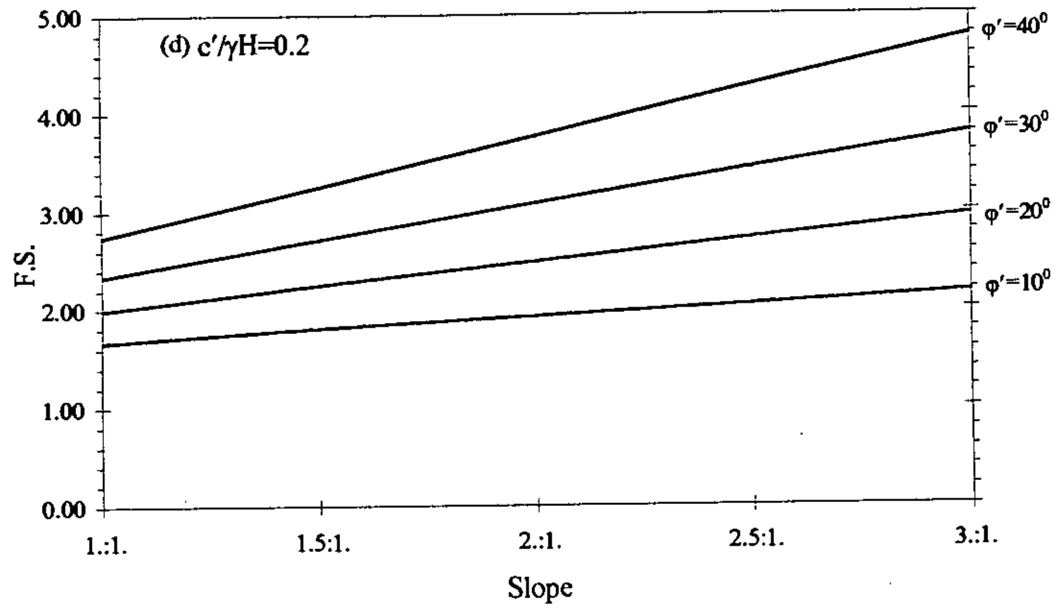


Fig. 4.18 Design Chart for embankment stability under no seepage condition for (c) $c'/\gamma H$=0.1 (d) $c'/\gamma H$=0.2.

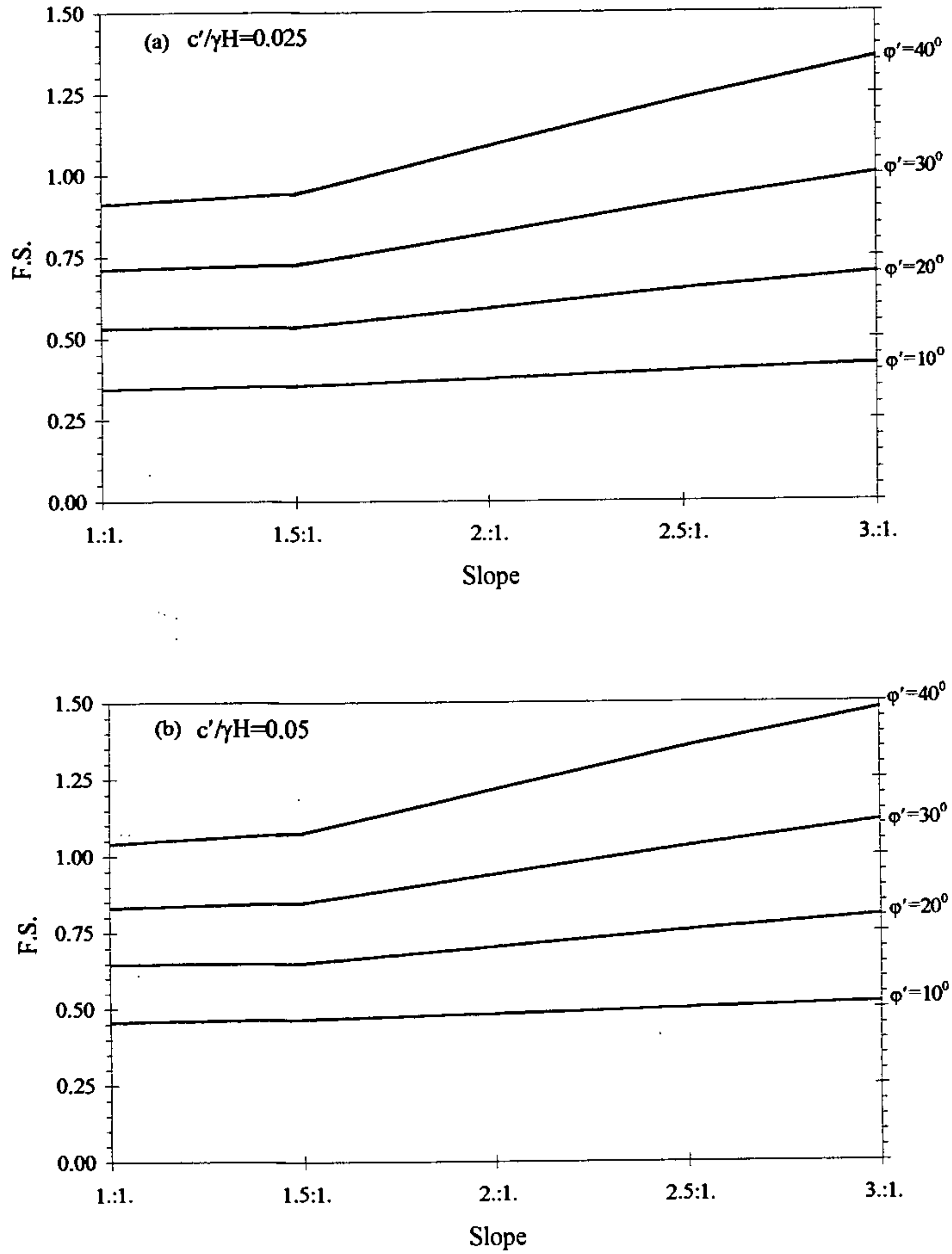


Fig. 4.19 Design Chart for embankment stability under seepage with 0.15g earthquake condition for (a) $c'/\gamma H=0.025$ (b) $c'/\gamma H=0.05$

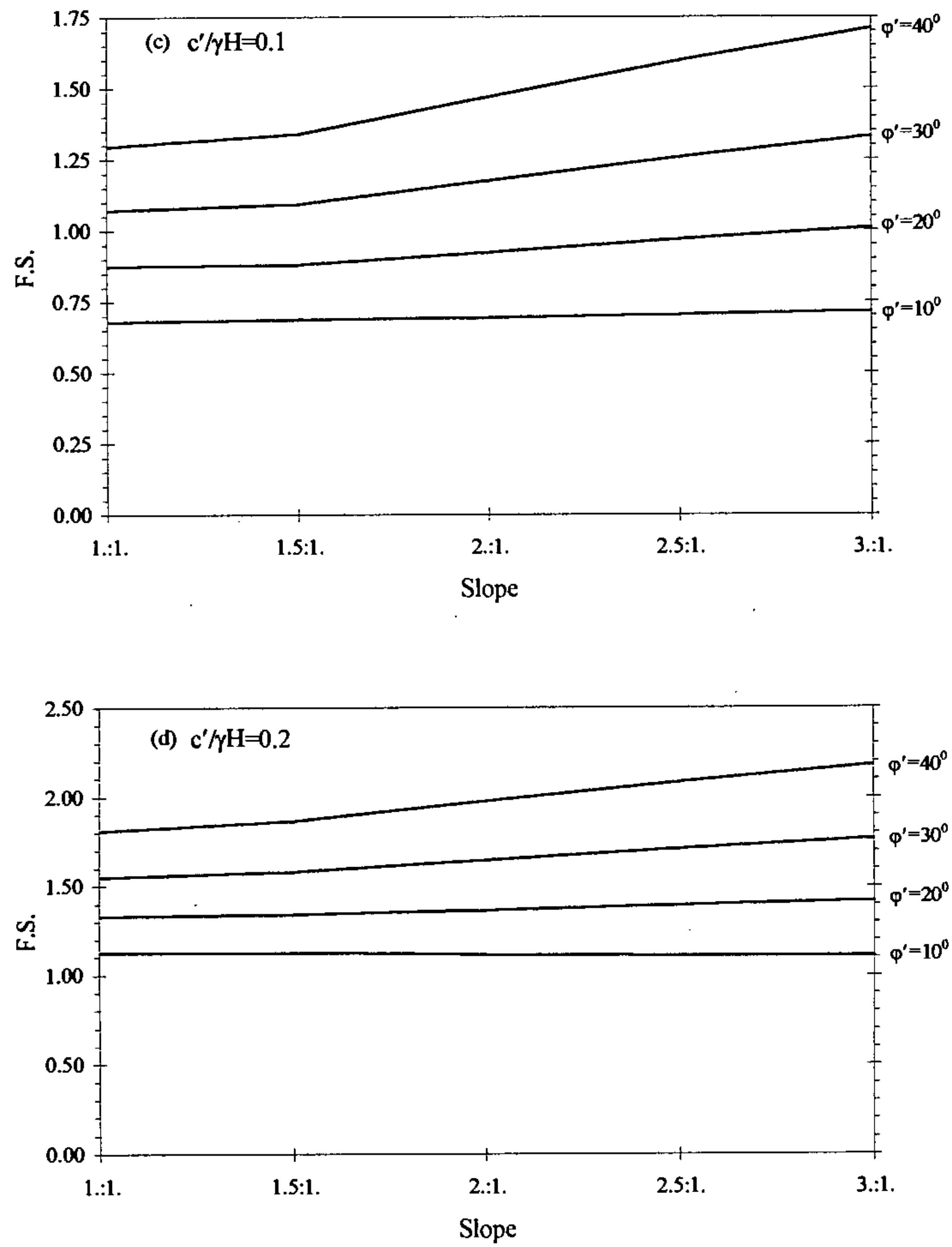


Fig. 4.19 Design Chart for embankment stability under seepage with 0.15g earthquake condition for (c) $c'/\gamma H=0.1$ (d) $c'/\gamma H=0.2$

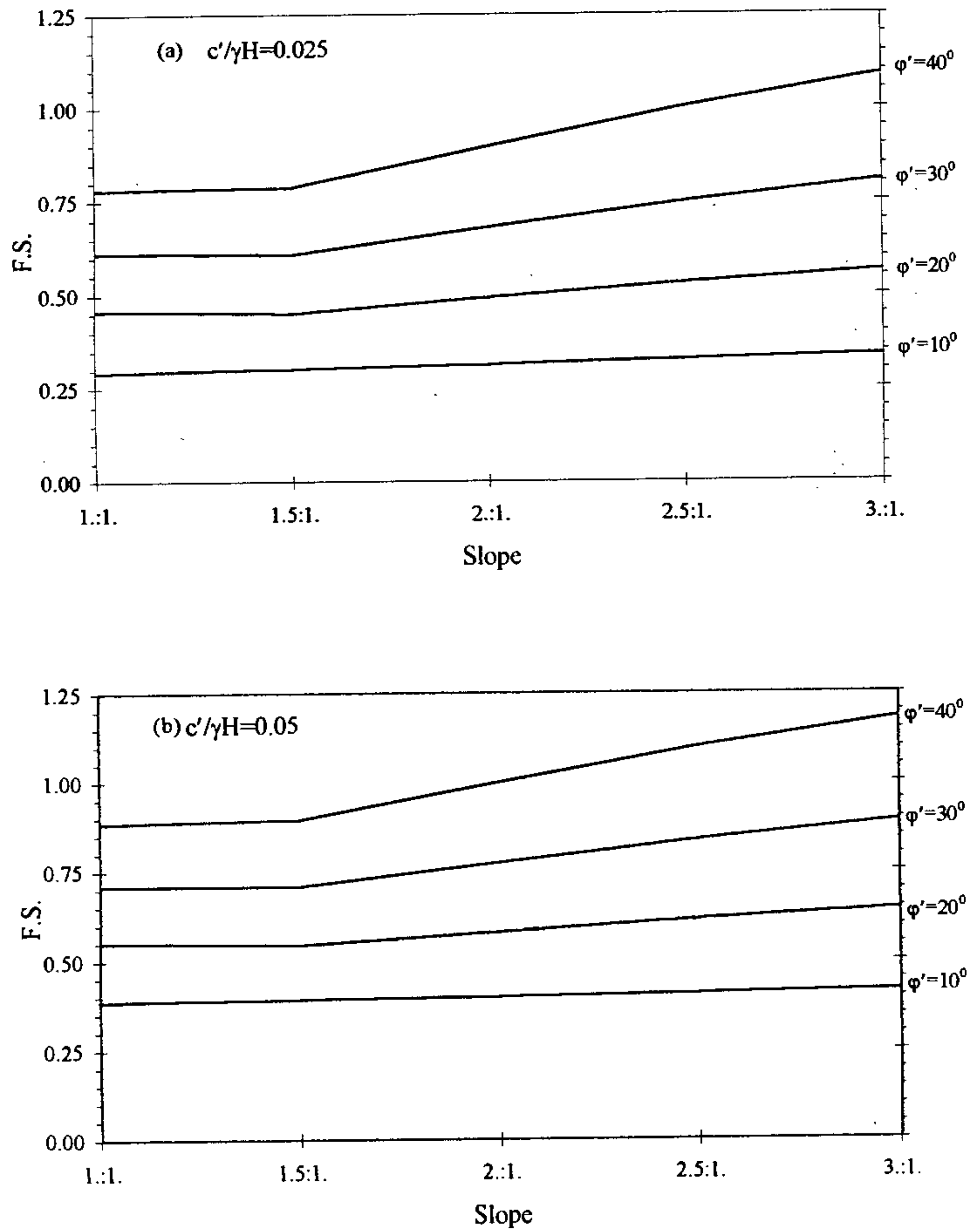


Fig. 4.20 Design Chart for embankment stability under seepage with 0.25g earthquake condition for (a) $c'/\gamma H$=0.025 (b) $c'/\gamma H$=0.05

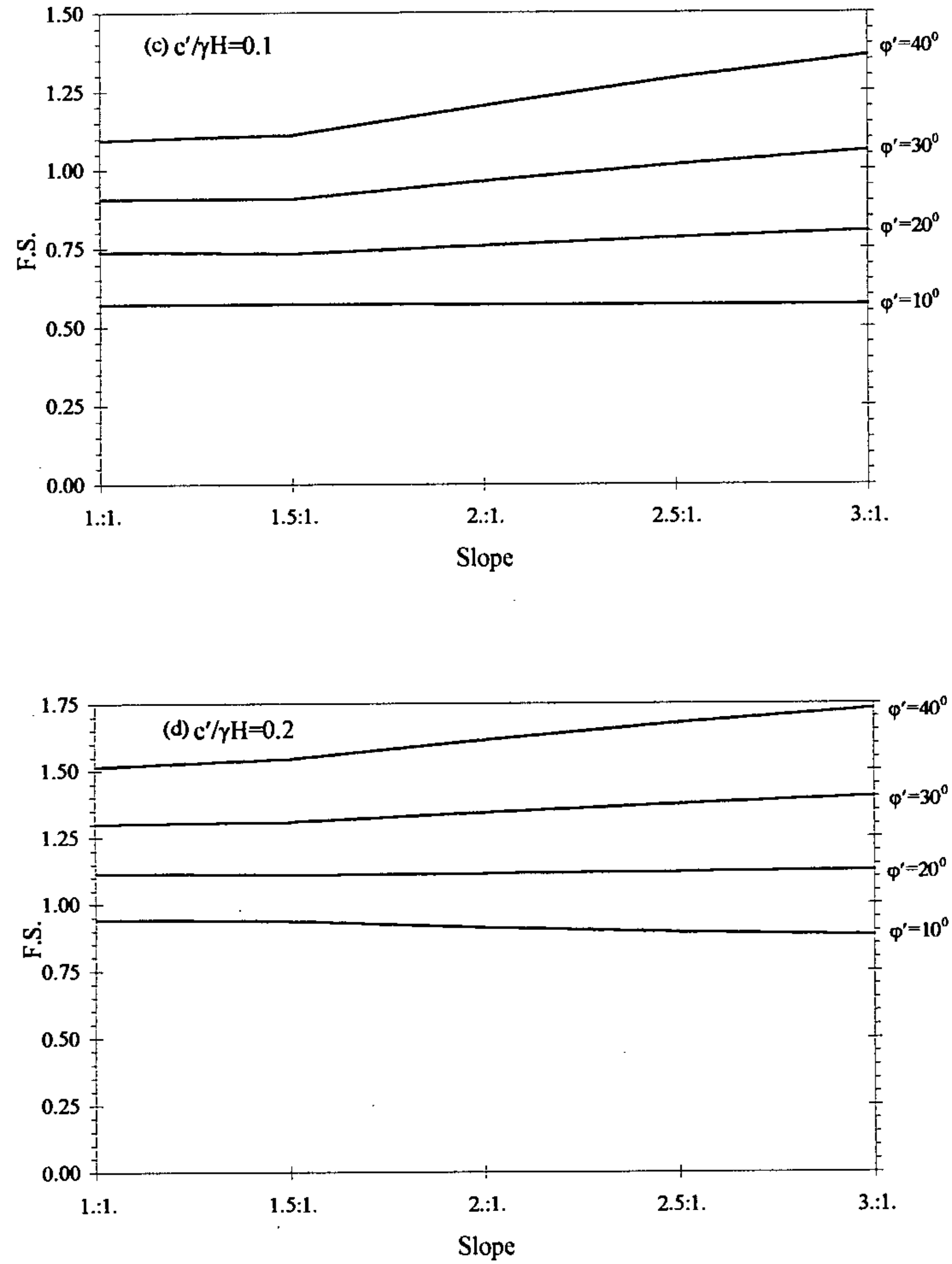


Fig. 4.20 Design Chart for embankment stability under seepage with 0.25g earthquake condition for (c) $c'/\gamma H=0.1$ (d) $c'/\gamma H=0.2$

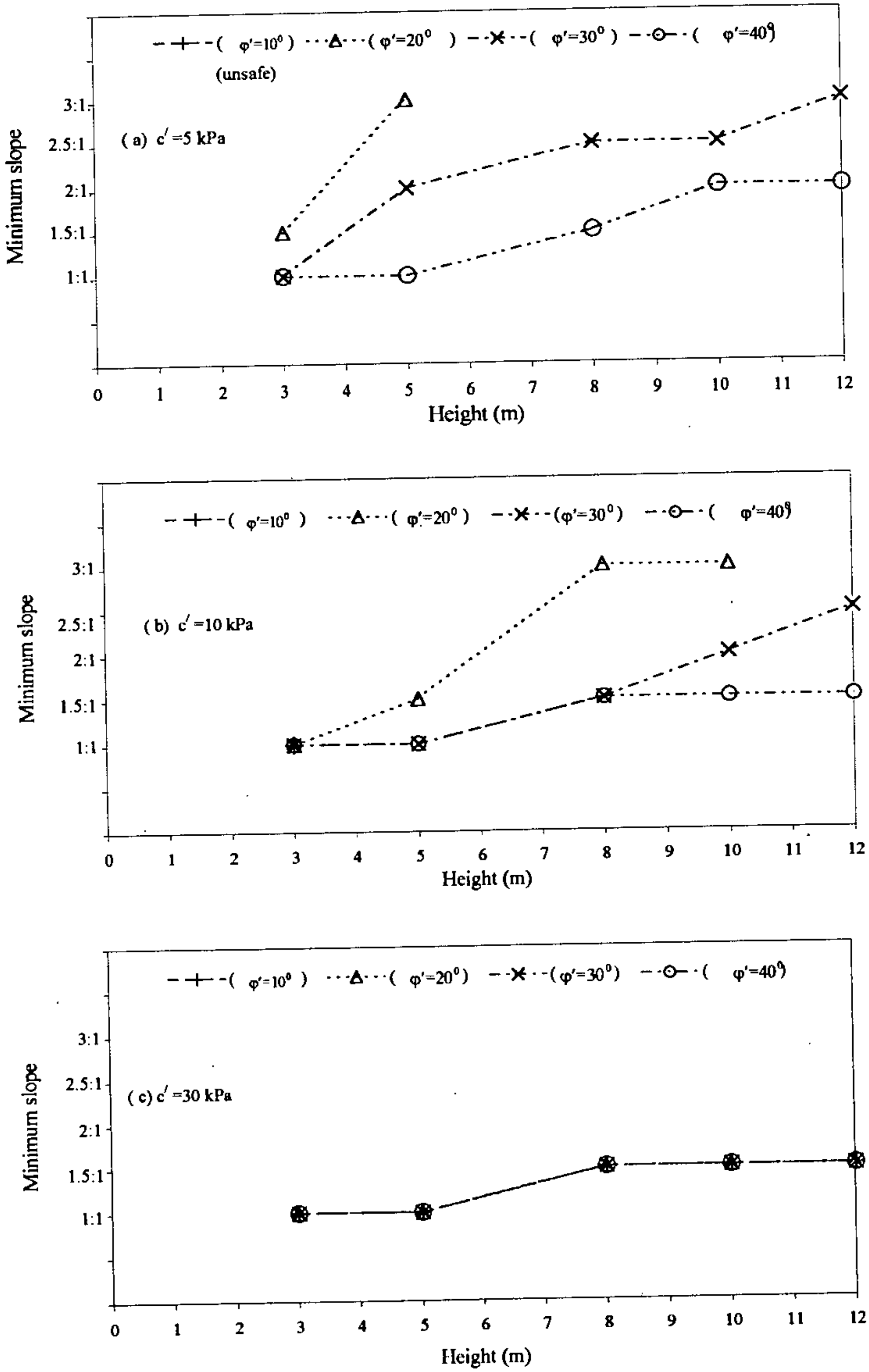


Fig. 4.21 Minimum slope required for F.S.≥ 1.2 under critical seepage condition for (a) c′=5 kPa (b) c′=10 kPa (c) c′=30 kPa.

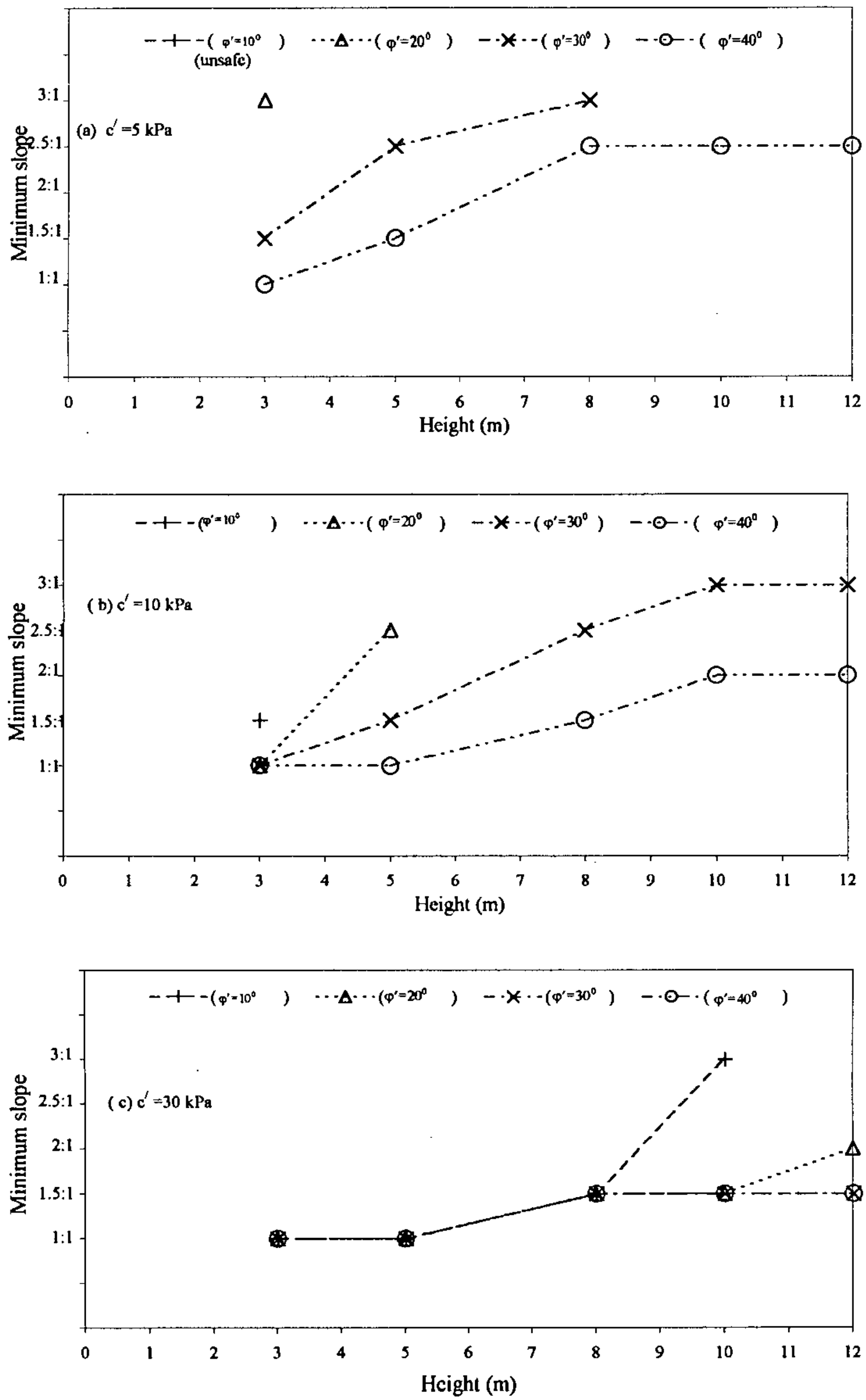


Fig. 4.22 Minimum slope required for F.S.≥ 1.5 under critical seepage condition for (a) c'=5 kPa (b) c'=10 kPa (c) c'=30 kPa.

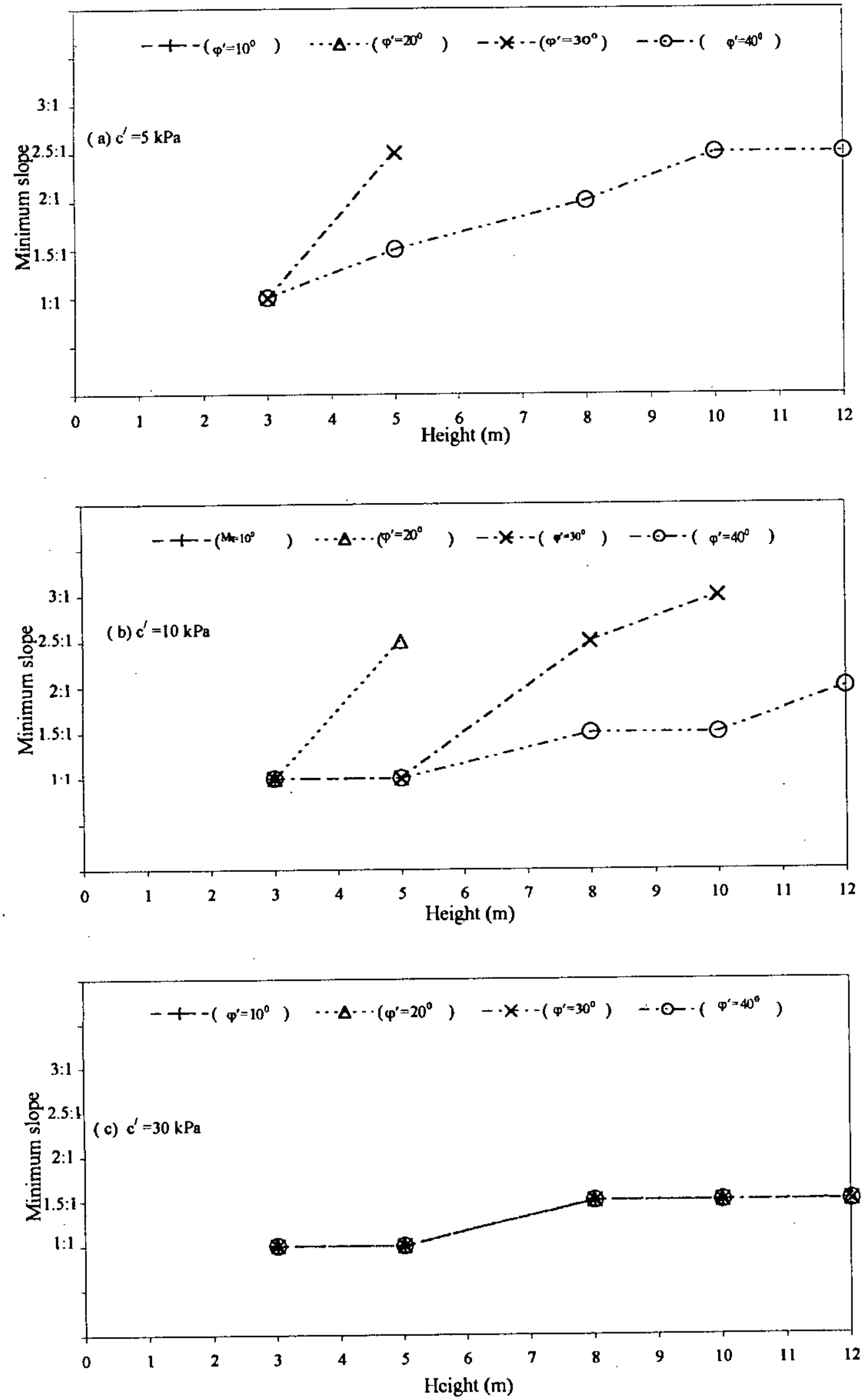
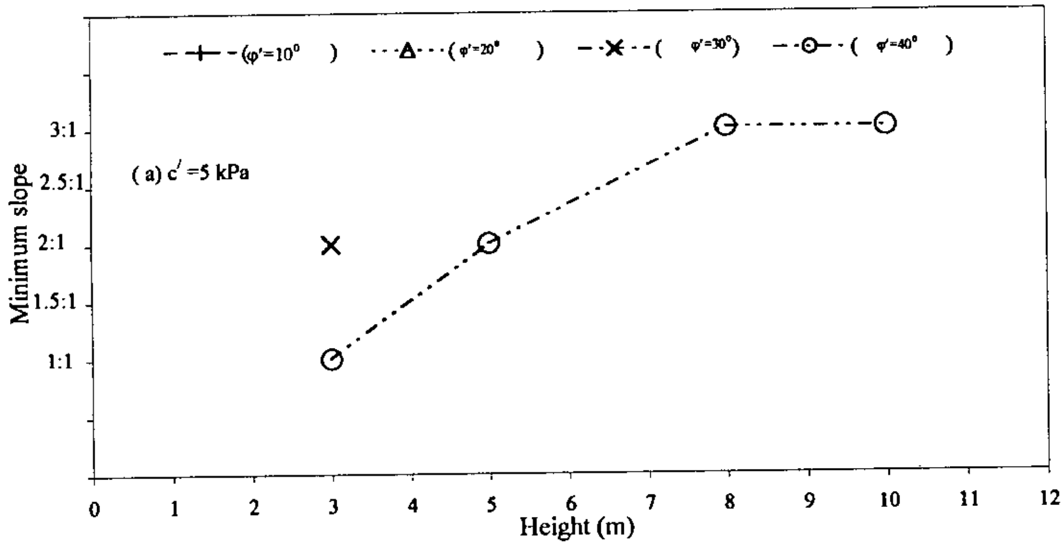
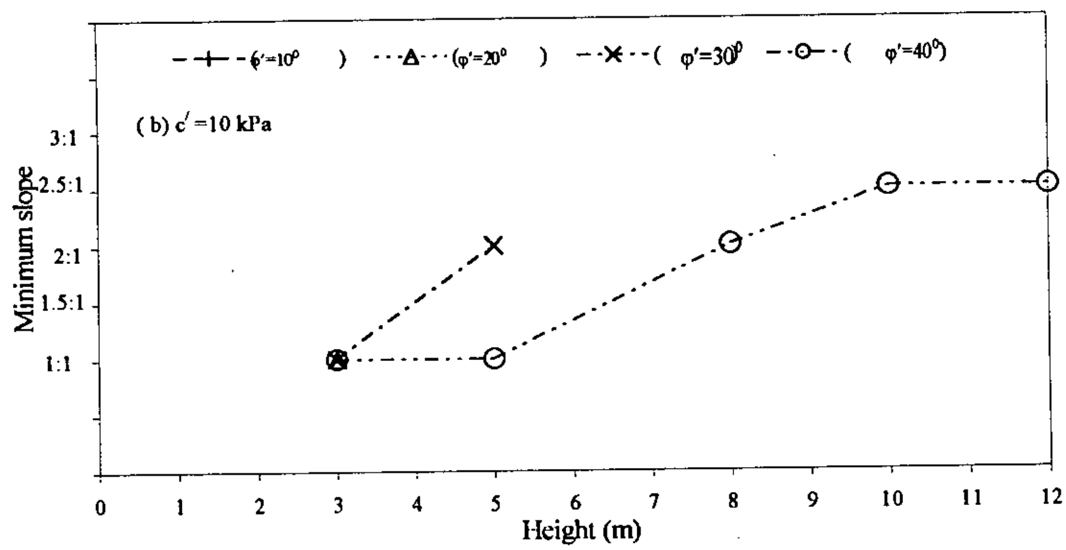


Fig. 4.23 Minimum slope required for F.S.≥ 1. under critical seepage with 0.15g earthquake condition for (a) c'=5 kPa (b) c'=10 kPa (c) c'=30 kPa.





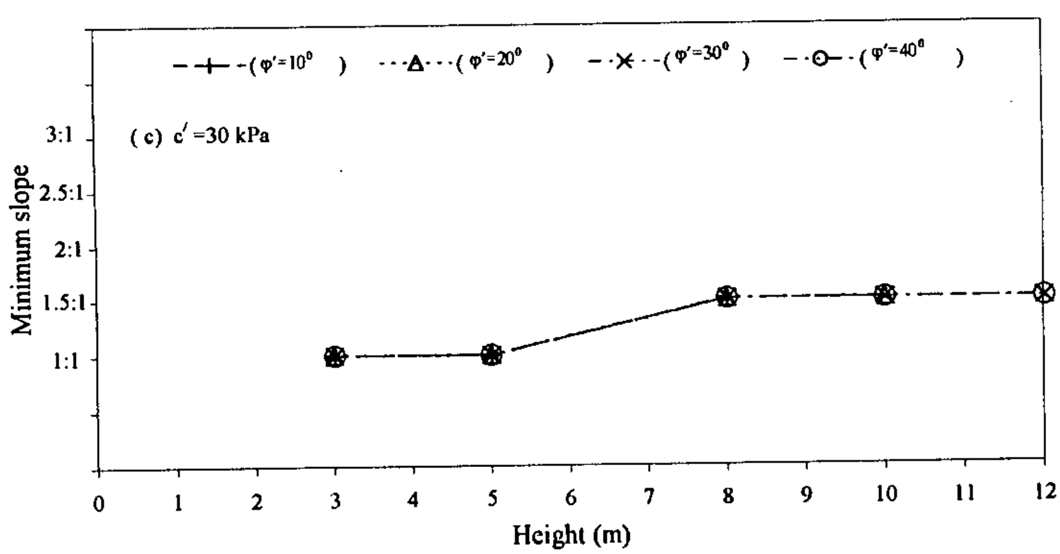


Fig. 4.24 Minimum slope required for F.S.≥ 1.2 under critical seepage with 0.15g earthquake condition for (a) c′=5 kPa (b) c′=10 kPa (c) c′=30 kPa.

earthquake conditions, slopes of 3:1 may not be safe for higher values of φ' (30°) with c'=5 kPa, 10 kPa. Even for c'=30 kPa, slopes of 3:1 may not be adequate with 8m to 12m high embankments in several cases for φ'=10°, 20°, 30°.

Results presented in Figs. 4.21 to 4.24 are presented in tabular form in Tables 4.14 to 4.17. Note that for seepage condition, as shown in Tables 4.14 and 4.15, for some cases s>3 is stated, this means slopes milder than 3:1 is needed,. This statement can be made since the F.S. increases as 's' is increased. This may not be true for the case of seepage plus earthquake. In fact for some cases, it is observed that the F.S. doesn't change much with milder slope. Moreover, it may reduce slightly in some cases as the slope is made milder. As for example, for c'=30 kPa and φ'=10°, the F.S. reduces gradually from 1.811 to 1.667 as the slope for a 5 m high embankment is flattened from 1:1 to 3:1 . The effect of making the slope milder is thus uncertain in some cases. Thus, in Tables 4.16 and 4.17, no remark about 's' required is made for cases where the required F.S. is not attained using the range of slopes studied.

4.5 EMBANKMENT OVER SOFTER GROUND

When embankments are placed on soft cohesive soils, the safety of the embankment is likely to depend on the properties of the foundation soils. Usually, complications arise due to variations in soil properties, depth to firm substratum, and embankment geometry. For control of the rate of construction of embankments on soft soils, Lobdell (1959) advocates a c-φ analysis using the procedure of slices, so that the pore pressure variation with depth and position under slope can be introduced. The stability at various stages of construction is analyzed on the basis of field pore pressure measurements.

Slope stability analysis is carried out for the case of embankment over a softer foundation soil. The values of the shear strength parameter c′ considered for the underlying foundation are to be 2.5 kPa, 5 kPa and 15 kPa which are half of the value of c′ for embankment soil. Fig. 4.25 shows a schematic diagram of the problem.

Table 4.14 Minimum value of slope parameter 's' to obtain F. S ≥1.2 for critical conditions of seepage.

	H (m)				
	3	5	8	10	12
	c′= 5 kPa				
φ′=10°	>3	>3	>3	>3	>3
φ′=20°	1.5	3	>3	>3	>3
φ′=30°	1	2	2.5	2.5	3
φ′=40°	1	1	1.5	2	2
c′= 10 kPa					
φ′=10°	1	>3	>3	>3	>3
φ′=20°	1	1.5	3	3	>3
φ′=30°	1	1	1.5	2	2.5
φ′=40°	1	1	1.5	1.5	1.5
c′= 30 kPa					
φ′=10°	1	1	1.5	1.5	1.5
φ′=20°	1	1	1.5	1.5	1.5
φ′=30°	1	1	1.5	1.5	1.5
φ′=40°	1	1	1.5	1.5	1.5

Table 4.15 Minimum value of slope parameter 's' to obtain F. S ≥1.5 for critical conditions of seepage.

	H (m)				
	3	5	8	10	12
	c′= 5 kPa				
φ′=10°	>3	>3	>3	>3	>3
φ′=20°	>3	>3	>3	>3	>3
φ′=30°	1.5	2.5	3	>3	>3
φ′=40°	1	1.5	2.5	2.5	2.5
c′= 10 kPa					
φ′=10°	1.5	>3	>3	>3	>3
φ′=20°	1	2.5	>3	>3	>3
φ′=30°	1	1.5	2.5	3	3
φ′=40°	1	1	1.5	2	2
c′= 30 kPa					
φ′=10°	1	1	1.5	3	>3
φ′=20°	1	1	1.5	1.5	2
φ′=30°	1	1	1.5	1.5	1.5
φ′=40°	1	1	1.5	1.5	1.5

Table 4.16 Minimum value of slope parameter 's' to obtain F. S ≥1.0 for critical seepage plus 0.15g earthquake conditions.

	H (m)				
	3	5	8	10	12
	c′= 5 kPa				
φ′=10°	-	-	-	-	-
φ′=20°	-	-	-	-	-
φ′=30°	1	2.5	-	-	-
φ′=40°	1	1.5	2	2.5	2.5
c′= 10 kPa					
φ′=10°	1	-	-	-	-
φ′=20°	1	2.5	-	-	-
φ′=30°	1	1	2.5	3	-
φ′=40°	1	1	1.5	1.5	2
c′= 30 kPa					
φ′=10°	1	1	1.5	1.5	-
φ′=20°	1	1	1.5	1.5	1.5
φ′=30°	1	1	1.5	1.5	1.5
φ′=40°	1	1	1.5	1.5	1.5

Table 4.17 Minimum value of slope parameter 's' to obtain F. S ≥1.2 for critical seepage plus 0.15g earthquake conditions.

	H (m)				
	3	5	8	10	12
	c′= 5 kPa				
φ′=10°	-	-	-	-	-
φ′=20°	-	-	-	-	-
φ′=30°	2	-	-	-	-
φ′=40°	1	2	3	3	-
c′= 10 kPa					
φ′=10°	-	-	-	-	-
φ′=20°	1	-	-	-	-
φ′=30°	1	2	-	-	-
φ′=40°	1	1	2.	2.5	2.5
c′= 30 kPa					
φ′=10°	1	1	1.5	-	-
φ′=20°	1	1	1.5	1.5	-
φ′=30°	1	1	1.5	1.5	1.5
φ′=40°	1	1	1.5	1.5	1.5

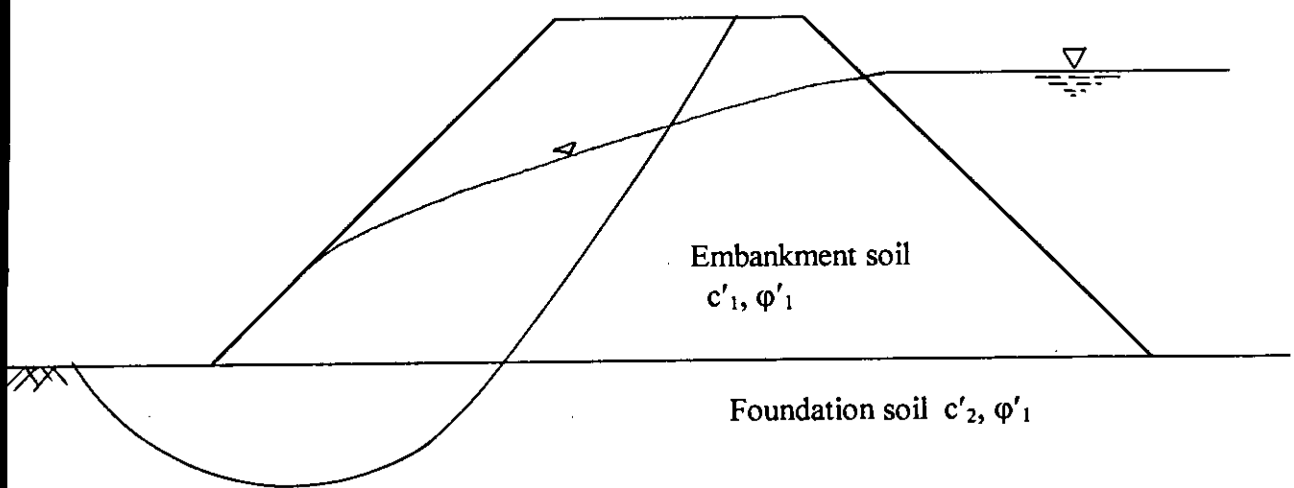


Fig. 4.25 Schematic diagram of the problem : Slope Stability Analysis of Embankment over softer ground.

Problems considered are those where the factor of safety is less than 2 in the homogeneous case.

It is assumed that the top flow line obtained by Casagrande's (1937) method may not be proper for this layered soil case. The effect of seepage induced pressure for F_B= 0.5 m is considered using pore pressure ratio instead of specifying the phreatic surface (water table). Table 4.18 presents equivalent pore pressure ratio r_u obtained by trials that gives similar F.S. as that using water table for different soil parameters for homogeneous case. An average 'r_u' is determined for a particular embankment geometry, which is later used to represent the seepage effects on embankments on softer foundation.

Using the average r_u thus obtained, slope stability analysis is performed for the case of embankment on softer soil. Table. 4.19 represents results on values of F. S. obtained for no seepage and seepage condition for the case of softer foundation. Comparing results of Table 4.18 and 4.19, the case of softer foundation (c'_2=0.5 c'_1) gives slightly lower F. S. than the case of homogeneous soil (c'_2=c'_1).

Table 4.18 Determination of equivalent pore pressure ratio r_u representing seepage condition for homogeneous soil.

H (m)	c' (kPa)	φ' (deg)	slope 1.:1				slope 1.5:1				slope 2.:1				slope 2.5:1				slope 3.:1			
			F. S. using water table	F. S. using r_u	r_u	Avg. r_u	F. S. using water table	F. S. using r_u	r_u	Avg. r_u	F. S. using water table	F. S. using r_u	r_u	Avg. r_u	F. S. using water table	F. S. using r_u	r_u	Avg. r_u	F. S. using water table	F. S. using r_u	r_u	Avg. r_u
3	5	10	0.938	0.936	0.170	0.166	0.919	0.918	0.300	0.285	1.016	1.015	0.340	0.335	1.087	1.083	0.390	0.383	1.184	1.180	0.400	0.388
		20	1.649	1.652	0.160		1.908	1.907	0.250		2.113	2.107	0.320		2.336	2.320	0.375		2.658	2.660	0.375	
	10	30	2.095	2.110	0.160		2.198	2.200	0.275		2.401	2.398	0.335		2.598	2.600	0.375		2.869	2.859	0.380	
	30	40	5.424	5.420	0.175		5.151	5.142	0.315		5.737	5.733	0.345		6.107	6.106	0.390		6.618	6.617	0.395	
5	5	10	0.627	0.629	0.220	0.224	0.667	0.666	0.310	0.306	0.716	0.719	0.360	0.358	0.779	0.777	0.410	0.400	0.850	0.851	0.420	0.416
		20	1.193	1.197	0.225		1.458	1.456	0.295		1.708	1.713	0.345		1.943	1.948	0.385		2.191	2.190	0.410	
	10	30	1.433	1.434	0.225		1.598	1.595	0.310		1.771	1.776	0.360		1.954	1.949	0.400		2.157	2.161	0.415	
	30	40	3.600	3.599	0.225		3.748	3.751	0.310		3.970	3.978	0.365		4.287	4.291	0.405		4.656	4.673	0.420	
8	5	10					0.475	0.475	0.360	0.358	0.531	0.531	0.400	0.394	0.587	0.588	0.430	0.426	0.650	0.648	0.450	0.443
		20					1.142	1.138	0.350		1.421	1.422	0.380		1.659	1.655	0.415		1.904	1.908	0.430	
	10	30					1.185	1.178	0.360		1.374	1.374	0.395		1.547	1.550	0.425		1.731	1.737	0.440	
	30	40					2.627	2.620	0.360		2.881	2.885	0.400		3.167	3.160	0.435		3.477	3.477	0.450	
10	5	10					0.408	0.408	0.380	0.376	0.468	0.457	0.430	0.410	0.523	0.523	0.440	0.434	0.588	0.586	0.450	0.441
		20					1.023	1.030	0.370		1.317	1.324	0.390		1.568	1.564	0.420		1.838	1.846	0.425	
	10	30					1.036	1.036	0.375		1.236	1.236	0.405		1.410	1.411	0.435		1.607	1.612	0.440	
	30	40					2.230	2.228	0.380		2.508	2.504	0.415		2.786	2.790	0.440		3.108	3.104	0.450	
12	5	10					0.362	0.365	0.390	0.353	0.420	0.420	0.430	0.425	0.479	0.477	0.450	0.443	0.543	0.544	0.450	0.443
		20					0.943	0.940	0.390		1.213	1.216	0.416		1.489	1.483	0.430		1.775	1.773	0.430	
	10	30					0.932	1.176	0.230		1.124	1.121	0.425		1.313	1.319	0.440		1.512	1.516	0.445	
	30	40					1.960	1.950	0.400		2.231	2.235	0.430		2.525	2.521	0.450		2.842	2.836	0.445	

Table 4.19 Factor of safety for embankment on softer foundation (c'_2=0.5 c'_1) for seepage and no seepage conditions.

c'_1 (kPa)	φ'_1 (deg)	c'_2 (kPa)	φ'_2 (deg)		H= 3m					H= 5m					H= 8m				H= 10m				H= 12m			
				Slope	1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V	1.5H:1V	2H:1V	2.5H:1V	3H:1V
Embankment		Foundation		r_u avg.	0.166	0.285	0.335	0.383	0.388	0.224	0.306	0.358	0.4	0.416	0.358	0.394	0.426	0.443	0.376	0.41	0.434	0.441	0.353	0.425	0.443	0.443
					Factor of Safety (F. S.)																					
5	10	2.5	10	seepage	0.898	0.796	0.853	0.886	0.951	0.602	0.592	0.614	0.651	0.705	0.422	0.463	0.504	0.553	0.367	0.41	0.455	0.51	0.347	0.372	0.42	0.478
				no seepage	0.985	0.959	1.056	1.14	1.233	0.717	0.758	0.827	0.912	1.003	0.615	0.693	0.778	0.867	0.565	0.645	0.732	0.821	0.529	0.611	0.7	0.789
	20		20	seepage	1.085	1.068	1.14	1.211	1.336	0.758	0.81	0.889	0.965	1.068	0.618	0.717	0.803	0.897	0.553	0.652	0.748	0.855	0.543	0.604	0.706	0.821
				no seepage	1.257	1.388	1.556	1.727	1.897	0.982	1.134	1.307	1.492	1.677	0.978	1.158	1.343	1.532	0.922	1.104	1.29	1.477	0.883	1.067	1.253	1.44
	30		30	seepage	1.327	1.376	1.481	1.589		0.947	1.065	1.202	1.329	1.486	0.835	0.995	1.137	1.288	0.756	0.918	1.07	1.243	0.753	0.858	1.018	1.202
				no seepage	1.585	1.841				1.29	1.553	1.836			1.377	1.672			1.315	1.61	1.908		1.274	1.568	1.864	
	40		40	seepage	1.611	1.727				1.177	1.371	1.573	1.761		1.093	1.329	1.535	1.753	0.996	1.235	1.458	1.704	1.003	1.16	1.396	1.659
				no seepage						1.664					1.859				1.789				1.737			
10	10	5	10	seepage	1.672	1.305	1.451	1.483	1.552	1.103	1.005	0.984	1.02	1.076	0.674	0.696	0.737	0.788	0.564	0.595	0.641	0.698	0.516	0.525	0.574	0.635
				no seepage	1.758	1.48				1.224	1.175	1.201	1.282	1.376	0.883	0.932	1.013	1.105	0.781	0.839	0.922	1.012	0.713	0.777	0.86	0.95
	20		20	seepage	1.802	1.609				1.212	1.196	1.244	1.321	1.43	0.856	0.94	1.026	1.126	0.745	0.833	0.928	1.04	0.705	0.757	0.858	0.976
				no seepage						1.449	1.537	1.682			1.25	1.415	1.589	1.774	1.15	1.318	1.497	1.681	1.078	1.249	1.434	1.618
	30		30	seepage						1.383	1.44	1.554	1.676		1.077	1.229	1.365	1.517	0.955	1.113	1.261	1.433	0.93	1.025	1.184	
				no seepage						1.742					1.665				1.559	1.842			1.484	1.77		
	40		40	seepage						1.613	1.752				1.353	1.579	1.781		1.21	1.441	1.666		1.193	1.339	1.576	
				no seepage																			1.965	1.249	2.827	
30	10	15	10	seepage														1.753	1.503	1.394	1.417		1.292	1.19	1.222	1.282
				no seepage															1.715	1.643			1.498	1.449	1.511	1.6
	20		20	seepage															1.57	1.59			1.402	1.383	1.482	1.608
				no seepage																						
	30		30	seepage															1.729				1.591	1.631		
				no seepage																						
	40		40	seepage																						
				no seepage																						

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

Earthen embankments of Bangladesh, which are often built improperly, need to withstand the effects of high flood levels. In addition, there is the risk of major earthquakes in a major part of the country. Extensive numerical investigations, using limiting equilibrium methods, are performed to study the effect of seepage induced pore pressures on the stability of slopes of earthen embankments. In addition, the effect of earthquakes is studied in a simplified manner using the pseudo-static procedure.

A wide range of parameters are used in this study. Homogeneous soil is assumed with soil shear strength parameters in the range of c'=5 to 30 kPa and φ'=10° to 40°. Embankment height in the range of 3m to 12m and slope in the range of 1:1 to 1:3 are considered. Soil unit weight is varied in the range of 15 to 22 KN/m^3. A freeboard of only 0.5 m represents the most critical condition for seepage. Circular slip surfaces are considered. The factor of safety of the country side slope against general slope failure is determined using the computer program PC-STABL. Three cases were studied: (i) no seepage (ii) seepage and (iii) Seepage with earthquake.

The main conclusions of this study may be summarised as follows:

- Embankment construction practice in Bangladesh is still primitive. This often results in embankments with uncompacted material of low shear strength and/or steep slopes. These embankments have a low factor of safety against slope failure even under dry condition and when subjected to high flood levels they are vulnerable to developing breaches or complete failure.

- Several trials are needed to determine the specifications for trial failure surface generation using PC-STABL that would give the lowest factor of safety.

- Factor of safety obtained using Simplified Janbu's method (with correction factor) agrees reasonably well with that using the Simplified Bishop's method. For majority of cases, the Bishop's method yields slightly higher factor of safety.

- Seepage induced pore pressures result in lowering the factor of safety of the embankment slope against general slope failure.

- The factor of safety decreases if the embankment height is increased without changing the slope. This rate of decrease is greater at smaller heights.

- The factor of safety generally increases as the side slopes are made less steeper.

- For some cases with seepage plus earthquake, for large c' and small φ', the effect of embankment slope can be almost negligible.

- The effect of the cohesion intercept c' on slope stability is significant. In fact, value of c' around 30 kPa or more will ensure stability under seepage conditions for most cases.

- Earthquakes can have a significant effect on the factor of safety. A horizontal motion of 0.15g can result in unsafe slopes for many cases even for c' around 30 kPa.

- Based on the numerical results, four sets of design charts have been developed that use dimensionless parameters $c'/\gamma H$, slope and φ'. The four sets correspond to the four conditions of (i) no seepage (ii) seepage (iii) seepage plus 0.15g earthquake and (iv) seepage plus 0.25g earthquake. These charts are expected to be useful for rapid preliminary design or stability reassessment of flood control embankments in Bangladesh.

- Minimum slope required for different embankment heights and soil properties have been presented graphically and in tabular form as design aids for the conditions of (i) seepage and (ii) seepage plus 0.15g earthquake. In some cases, slopes of 3:1 are found to be not safe.

5.2 RECOMMENDATIONS FOR FUTURE STUDY

The following recommendations for future study on slope stability of earthen embankments can be made from the present research:

- The case of layered soils may be studied subjected to the critical conditions of this study.

- The case of nonhomogeneous embankment body with impermeable core at the centre and granular material around the core may be studied.

- The effect of seepage pressures at the exit point on the exposed country side slope and possible piping action may be analysed.

- Study of flow nets through the embankment need to be carefully done for different variations of soil parameters including anisotropic permeability and their effect on seepage pressures assessed.

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APPENDIX – A-1

INPUT FILE

```
PROFIL        C:JCC23.IN  PCSTABL Version 5M
PCSTABL5M PROBLEM WITH JANBU CIRCULAR METHOD
3 3
0. 10. 5. 10. 1
5. 10. 21. 18. 1
21. 18. 27. 18. 1
SOIL
1
18. 19. 10. 30. 0. 0. 1
WATER
1 9.81
15
0. 10.
5. 10.
6. 10.5
7. 11.
8. 11.5
10. 12.4
12. 13.3
12.5 13.5
13. 13.75
13.75 14.
14.5 14.3
15.5 14.65
25.75 16.55
27.75 17.1
27.95 17.45
CIRCLE-Janbu circular, search.
1 2
7 50
1. 5. 21. 25. 0. 0.5 0. 0.
```

OUTPUT FILE

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

Run Date:
Time of Run:
Run By:
Input Data Filename: C:JCC23.IN
Output Filename: C:JCC23.OUT
Plotted Output Filename: C:JCC23.PLT

PROBLEM DESCRIPTION PCSTABL5M PROBLEM WITH JANBU CIRCULAR ME
THOD

BOUNDARY COORDINATES

3 Top Boundaries
3 Total Boundaries

Boundary No.	X-Left (m)	Y-Left (m)	X-Right (m)	Y-Right (m)	Soil Type Below Bnd
1	.00	10.00	5.00	10.00	1
2	5.00	10.00	21.00	18.00	1
3	21.00	18.00	27.00	18.00	1

1

ISOTROPIC SOIL PARAMETERS

1 Type(s) of Soil

Soil Type No.	Total Unit Wt. (KN/m3)	Saturated Unit Wt. (KN/m3)	Cohesion Intercept (kPa)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (kPa)	Piez. Surface No.
1	18.0	19.0	10.0	30.0	.00	.0	1

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 9.81

Piezometric Surface No. 1 Specified by 15 Coordinate Points

Point No.	X-Water (m)	Y-Water (m)
1	.00	10.00
2	5.00	10.00
3	6.00	10.50
4	7.00	11.00
5	8.00	11.50
6	10.00	12.40
7	12.00	13.30
8	12.50	13.50
9	13.00	13.75
10	13.75	14.00
11	14.50	14.30
12	15.50	14.65
13	25.75	16.55
14	27.75	17.10
15	27.95	17.45

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

Janbus Empirical Coef. is being used for the case of c & phi both >

0

350 Trial Surfaces Have Been Generated.

50 Surfaces Initiate From Each Of 7 Points Equally Spaced Along The Ground Surface Between X = 1.00 m.
and X = 5.00 m.

Each Surface Terminates Between X = 21.00 m.
and X = 25.00 m.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = .00 m.

.50 m. Line Segments Define Each Trial Failure Surface.

1

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Janbu Method * *

Failure Surface Specified By 47 Coordinate Points

Point No.	X-Surf (m)	Y-Surf (m)
1	4.33	10.00
2	4.78	9.78
3	5.24	9.58
4	5.70	9.39
5	6.18	9.23
6	6.65	9.08
7	7.14	8.95
8	7.63	8.85
9	8.12	8.76
10	8.61	8.69
11	9.11	8.64
12	9.61	8.61
13	10.11	8.60
14	10.61	8.61
15	11.11	8.64
16	11.61	8.69
17	12.10	8.76
18	12.59	8.85
19	13.08	8.96
20	13.56	9.09
21	14.04	9.24
22	14.51	9.40
23	14.98	9.59
24	15.43	9.79
25	15.88	10.01
26	16.32	10.25
27	16.75	10.51
28	17.17	10.78
29	17.58	11.07
30	17.97	11.38
31	18.36	11.70
32	18.73	12.04
33	19.08	12.39
34	19.42	12.75
35	19.75	13.13
36	20.06	13.52
37	20.36	13.93
38	20.64	14.34
39	20.90	14.76
40	21.15	15.20

41	21.38	15.64
42	21.59	16.10
43	21.78	16.56
44	21.95	17.03
45	22.11	17.50
46	22.24	17.99
47	22.25	18.00

*** 1.409 ***

Individual data on the 59 slices

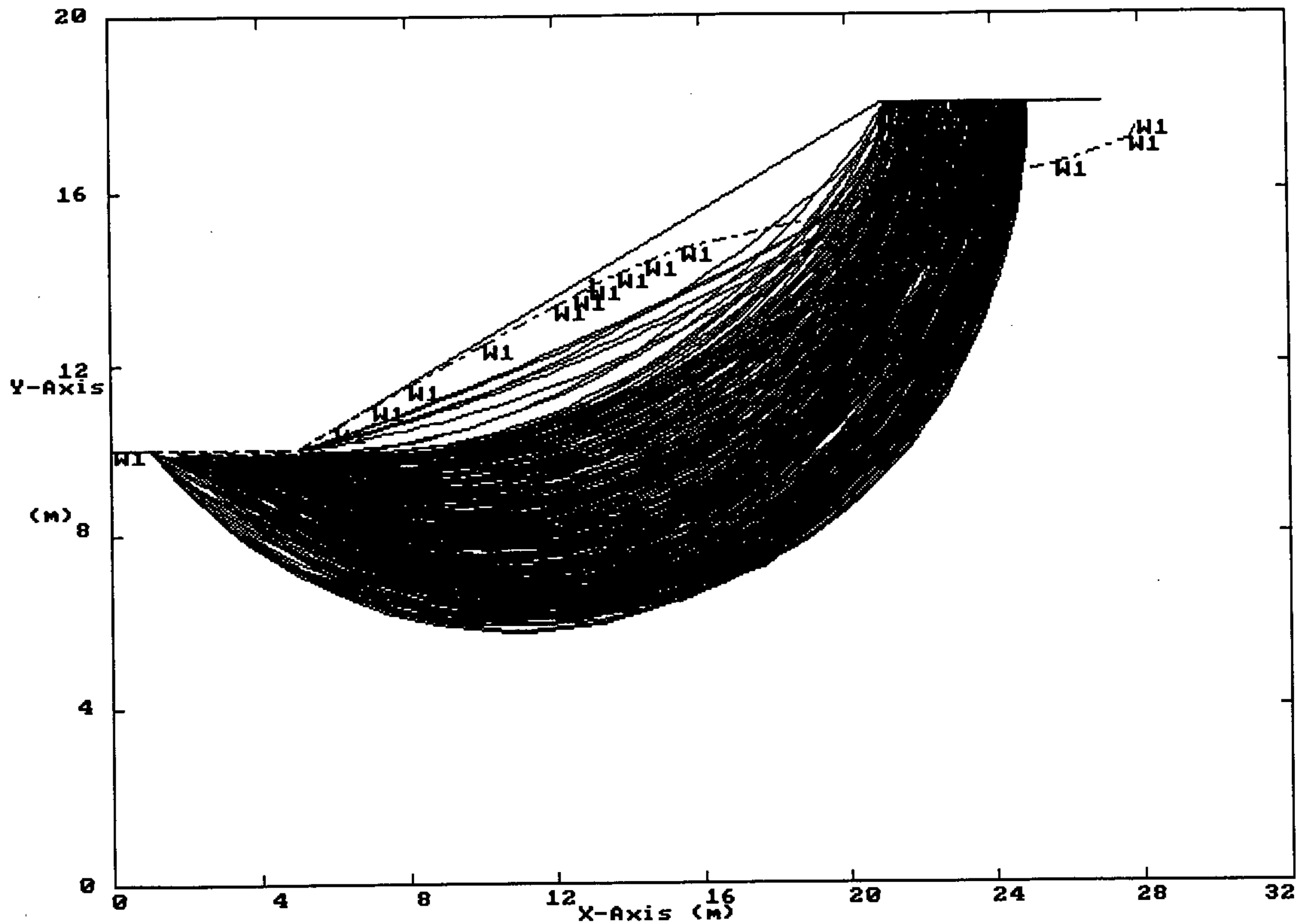
Slice No. kg(Lbs)	Width m(Ft)	Weight kg(Lbs)	Water Force Top kg(Lbs)	Water Force Bot kg(Lbs)	Tie Force Norm kg(Lbs)	Tie Force Tan kg(Lbs)	Earthquake Force Hor kg(Lbs)	Earthquake Force Ver kg(Lbs)	Surcharge Load
1	.4	.9	.0	.5	.0	.0	.0	.0	.0
2	.2	1.1	.0	.6	.0	.0	.0	.0	.0
3	.2	1.9	.0	1.0	.0	.0	.0	.0	.0
4	.5	6.6	.0	3.3	.0	.0	.0	.0	.0
5	.3	6.1	.0	3.0	.0	.0	.0	.0	.0
6	.2	4.3	.0	2.1	.0	.0	.0	.0	.0
7	.5	14.0	.0	6.9	.0	.0	.0	.0	.0
8	.3	12.3	.0	5.9	.0	.0	.0	.0	.0
9	.1	5.4	.0	2.6	.0	.0	.0	.0	.0
10	.5	21.1	.0	10.1	.0	.0	.0	.0	.0
11	.4	18.3	.0	8.7	.0	.0	.0	.0	.0
12	.1	6.2	.0	3.0	.0	.0	.0	.0	.0
13	.5	27.7	.0	13.2	.0	.0	.0	.0	.0
14	.5	30.7	.0	14.5	.0	.0	.0	.0	.0
15	.5	33.5	.0	15.7	.0	.0	.0	.0	.0
16	.4	27.9	.0	13.0	.0	.0	.0	.0	.0
17	.1	8.2	.0	3.8	.0	.0	.0	.0	.0
18	.5	38.5	.0	17.8	.0	.0	.0	.0	.0
19	.5	40.6	.0	18.7	.0	.0	.0	.0	.0
20	.5	42.4	.0	19.5	.0	.0	.0	.0	.0
21	.4	34.8	.0	16.0	.0	.0	.0	.0	.0
22	.1	9.2	.0	4.3	.0	.0	.0	.0	.0
23	.4	36.5	.0	17.1	.0	.0	.0	.0	.0
24	.1	8.7	.0	3.9	.0	.0	.0	.0	.0
25	.4	38.3	.0	17.5	.0	.0	.0	.0	.0
26	.1	7.8	.0	3.7	.0	.0	.0	.0	.0
27	.5	46.9	.0	22.5	.0	.0	.0	.0	.0
28	.2	18.2	.0	8.8	.0	.0	.0	.0	.0
29	.3	29.0	.0	13.6	.0	.0	.0	.0	.0
30	.5	45.9	.0	21.7	.0	.0	.0	.0	.0
31	.0	1.4	.0	.7	.0	.0	.0	.0	.0
32	.5	47.1	.0	22.7	.0	.0	.0	.0	.0
33	.5	46.6	.0	22.5	.0	.0	.0	.0	.0
34	.1	6.7	.0	3.3	.0	.0	.0	.0	.0
35	.4	39.2	.0	19.6	.0	.0	.0	.0	.0

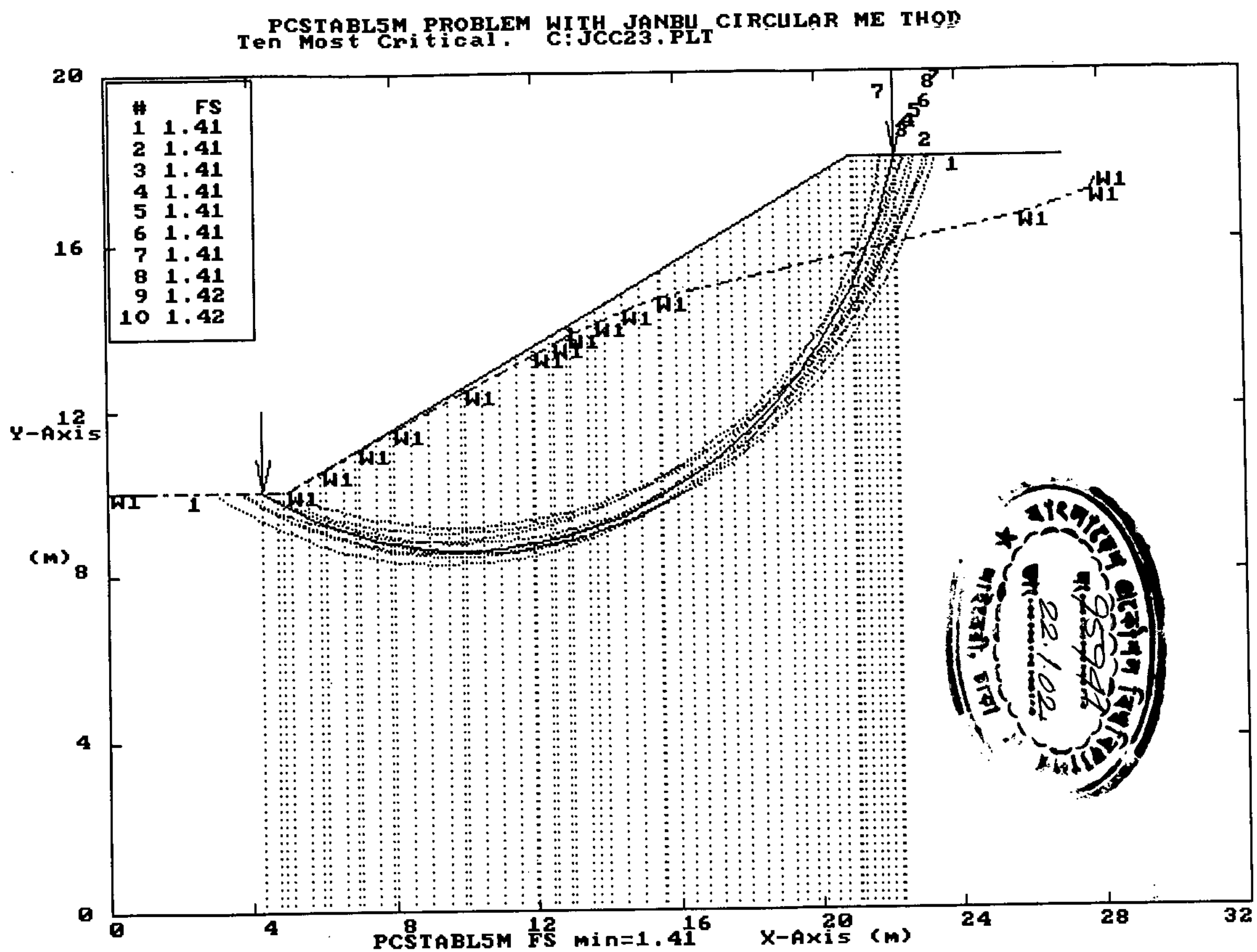
36	.4	44.8	.0	22.3	.0	.0	.0	.0	.0
37	.4	43.5	.0	21.5	.0	.0	.0	.0	.0
38	.4	41.9	.0	20.6	.0	.0	.0	.0	.0
39	.4	40.2	.0	19.6	.0	.0	.0	.0	.0
40	.4	38.2	.0	18.5	.0	.0	.0	.0	.0
41	.4	36.1	.0	17.4	.0	.0	.0	.0	.0
42	.4	33.8	.0	16.1	.0	.0	.0	.0	.0
43	.4	31.5	.0	14.8	.0	.0	.0	.0	.0
44	.3	29.0	.0	13.4	.0	.0	.0	.0	.0
45	.3	26.4	.0	11.9	.0	.0	.0	.0	.0
46	.3	23.8	.0	10.3	.0	.0	.0	.0	.0
47	.3	21.2	.0	8.7	.0	.0	.0	.0	.0
48	.3	18.6	.0	7.0	.0	.0	.0	.0	.0
49	.3	16.1	.0	5.2	.0	.0	.0	.0	.0
50	.1	5.6	.0	1.6	.0	.0	.0	.0	.0
51	.1	7.9	.0	1.8	.0	.0	.0	.0	.0
52	.2	10.7	.0	1.4	.0	.0	.0	.0	.0
53	.0	2.0	.0	.1	.0	.0	.0	.0	.0
54	.2	6.1	.0	.0	.0	.0	.0	.0	.0
55	.2	5.8	.0	.0	.0	.0	.0	.0	.0
56	.2	3.8	.0	.0	.0	.0	.0	.0	.0
57	.2	2.0	.0	.0	.0	.0	.0	.0	.0
58	.1	.6	.0	.0	.0	.0	.0	.0	.0
59	.0	.0	.0	.0	.0	.0	.0	.0	.0

```
              Y          A     X     I     S         M

               .00      3.96      7.92      11.88     15.84     19.80
 X         .00 +---------+---------+----*----+---------+---------+
               -
               -                        ..
               -                       ...
               -                     .....
               -                    .....0
      3.96     +                   .....03
               -                  .....031
               -                 ......41*.
               -                 .....011..W
               -                ......418...
               -                .....0418...W
 A    7.92     +               ......012.... W
               -               ......416.....
               -               ......416......
               -               ......416...... W
               -               ......416.......
               -               ......413.......
 X    11.88    +               ......012........  W
               -               .......126........ W
               -               .......413........  W
               -               .......0126........ W
               -                .......413........  W
               -                ........113........  W
 I    15.84    +                 ........13.........
               -                 ........511.........
               -                  ........4133........
               -                  .........5113........
               -                   .........9213........
               -                    ..........2113........
 S    19.80    +                     ..........52113.......
               -                      ..........9521137......
               -                        ...........95211317...*
               -                         ............9922211117
               -                           ..............922341
               -                             ..............9222
      23.76    +                                ..............
               -                                   ...........
               -                                         .....
               -                                         W
               -
               -                                            *
 M    27.72    +                                          WW
               -
               -
               -
               -
               -
      31.68    +
```

PCSTABL5M PROBLEM WITH JANBU CIRCULAR METHOD
All surfaces evaluated. C:JCC23.PLT





PCSTABL5M PROBLEM WITH JANBU CIRCULAR METHOD
Ten Most Critical. C:JCC23.PLT
FS
1 1.41
2 1.41
3 1.41
4 1.41
5 1.41
6 1.41
7 1.41
8 1.41
9 1.42
10 1.42
Y-Axis (m)
PCSTABL5M FS min=1.41 X-Axis (m)

165