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CAUSES OF FAILURE OF EARTH EMBANKMENTS WITH REFERENCE TO GUMTI PROJECT

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CAUSES OF FAILURE OF EARTH EMBANKMENTS WITH REFERENCE TO GUMTI PROJECT

A project report

by

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B.Sc. Engg. (Civil)

Submitted to the department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka in partial fulfilment of the requirements for the degree

of

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BANGLADESH UNIVERSITY OF ENGINEERING AND TECHNOLOGY

DHAKA

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ABSTRACT

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CAUSES OF FAILURE OF EARTH EMBANKMENTS WITH REFERENCE TO GUMTI PROJECT

Md. Mojibur Rahman Shikder

This study gives an account of investigations carried out into the causes of failure of earth embankment of Gumti (Phase-I) flood control, drainage and irrigation project during the flood of 1987. The study comprises of the collection of soil and hydrological data, and analyses of slope stability, seepage and piping for the embankment.

Stability analyses of the slope was done using various methods (Swedish slip circle method, Taylor's method, Bishop's method, and Bshop's Simplified method) considering different drawdown conditions. Flownet method was used for seepage analysis.

The analyses show that a high exit gradient of flow occured during high flood condition. Consequently a significant seepage pressure developed at the countryside toe of the embankment. This

might be a reason to initiate the failure by toe erosion. It is observed that the toe erosion occured mostly in places where the soil has a low plasticity.

The analysis also indicate that the seepage through the embankment is critical with a factor of safety of unity. All the methods, except Bishop's method suggest that the slope of the embankment is safe against stability. However, Bishop's method shows a factor of safety less than unity. On review of the methods of construction and maintenance, it appears that these factors have been no less significant in the initiation of failure of the embankment of Gumti Project.

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CHAPTER 1

INTRODUCTION

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1.1 GENERAL

Gumti Phase-I Project was under taken (1986) by the Bangladesh Water Development Board (BWDB) in order to facilitate flood protection, irrigation and drainage in certain parts of Comilla district. The project mainly consists of construction of earth embankments, regulators, sluice gates, bridges and drainage canals. From Indian boarder to Jaffargonj,the embankment was started to be constructed from British period. From Jaffargonj to Debidwar, the embankment was constructed in 1986-87, while Debidwar to Companigonj, the construction was done during 1988-89. Before the implementation of embankment on the banks of the Gumti river by BWDB,there was and even now there is public dyke constructed by local bodies. But the public dykes are usually inadequate in section and technically unsound in alignment. Due to the 1987 flood of Bangladesh, even the earthen embankments constructed by the BWDB, failed at several locations. In this study an attempt is made to investigate the causes of failure of these embankments.

Gumti Phase-I project is located on the southern part of Gumti river and eastern part of the Meghna Ferryghat via Elliotgonj to Comilla road. The project area, as shown in Fig. 1.1, covers Debidwar, Muradnagar, Burichang and Daudkandi upazilas under

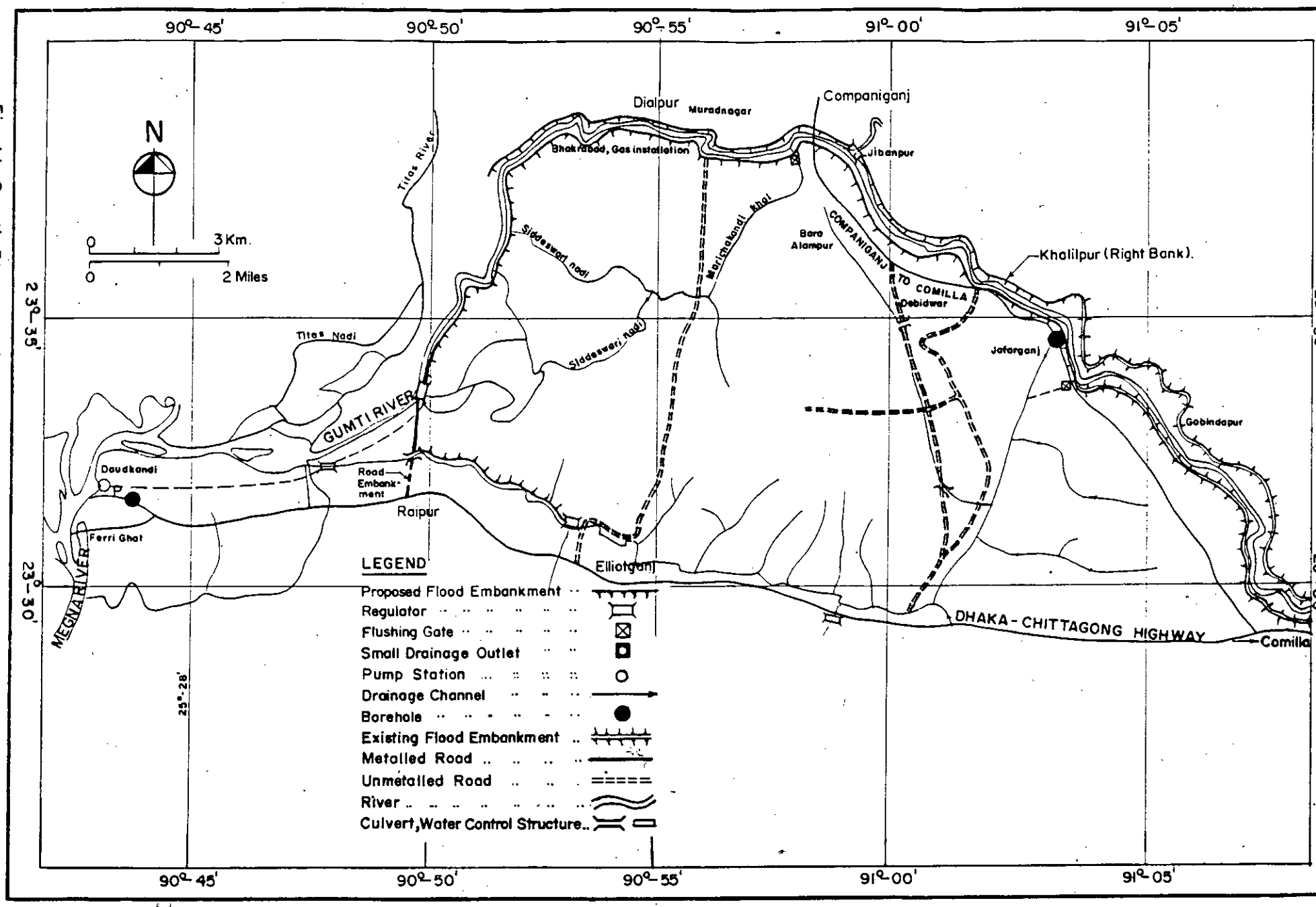

Fig. 1.1 Gumti Phase I Project Area.

Comilla district. The area consists of 37,340 hectares of land enclosed by embankments which is to receive irrigation benefit during dry periods (October-March) and flood protection during the monsoon (April-September). The main objectives of the project were to provide flood control and drainage facilities to the entire project area and irrigation facilities to 11,000 hectares of land. The project includes a pumping plant which drains out monsoon water from within and pumps river water to an elevated canal head which distributes the water by gravity flow to various locations of the project area within and outside the enclosure by a system of main canals, laterals, sublaterals and control structures.

New embankments were constructed in certain areas. The existing road system of Dhaka-Chittagong highway were also used as flood embankments by closing bridge openings, where found necessary.

The failure of the embankments occured at the newly constructed portion. The causes of failure are, however, yet not reported by any agencies. The failure of any earthen embankments depends on many factors. Of them, the properties of soil, the geology and climate of the area and the geometry of the embankments are normally considered to be the major factors.

1.2 GEOLOGY AND CLIMATOLOGY OF THE PROJECT AREA

The entire project area is consisted of the alluvial deposit of the river Gumti and Meghna. The subsurface conditions were investigated by the water Development Board (1985), and the locations of the borings are indicated in Fig. 1.1.

The subsoil mainly consists of very fine silty sand with trace of mica. Peat and soft clay layers were found at depths in excess of 130 m from ground level.

The area has a typical tropical monsoon climate of Bangladesh with hot and humid summers and dry cool winter seasons. The most characteristic feature of the climate is the rainfall regime. The mean annual rainfall in the project area ranges from about 2,000 mm in the West to nearly 2,200 mm in the East. About 75 percent of the total annual rainfall occurs during the true monsoon period. By April to May the south-west monsoon, which originates from the Indian ocean, carries warm and moist air, causing local rain showers. For more details of the climate reference can be made to Manalo (1975) and Karim and Akhand (1982).

1.3 OBJECTIVES OF THE PRESENT STUDY

The failure of any civil engineering structures involves a huge cost interms of money, man and materials and it is not desirable. However, if properly monitored, the failure may sometimes be helpful in developing the technical knowledge of the professionals. The investigation of the causes of failure of Gumti project is expected to yield important information for the concerned people. This particular aspect is set as the primary objectives of the present research.

The main two objectives of the present study are:

i) The review of the existing literature on the causes of failure embankments.

ii) The collection and analyses of the available soil exploration and other related data of the project site in order to identify the parameters responsible for the failure of embankments of Gumti project.

CHAPTER 2

REVIEW OF LITERATURE

2.1 INTRODUCTION

The present study is primarily concerned with the effect of seepage and piping on the stability of slopes of the embankments. In this chapter the important literature related to the present investigation concerning stability,seepage and piping analyses are briefly reviewed.

2.2 CAUSES OF FAILURE OF EARTH EMBANKMENTS

Earth embankments are less rigid and hence, more susceptable to failure,(Garg,1976). Every past failure of such an embankment has contributed to an increase in the knowledge of the earth embankment designers. Earthen embankment may fail, like other engineering structures, due to improper design, faulty construction, lack of maintenance, etc. The various causes leading to the failure of earth embankments can be grouped into four classes:

- Stability failures
- Seepage failures
- Piping failures
- Hydraulic failures

2.2.1 STABILITY FAILURES

About 25 percent of the embankments failures have been attributed to stability failures. Stability failures are generally caused by shear failures, causing slides, Garg (1976).

a) **Foundation slide:** When the foundation of earth embankments are made of soft soils, such as fine silt, soft clay, etc. the entire embankment may slide over the foundations. Sometimes seams of fissured rocks, shales or soft clay, etc. may exist under the foundations and the embankment may slide over some of them, causing its failure. In this type of failure, the top of embankment gets cracked and subsides, the lower slope moves outward forming large mud waves near the heel as shown in Fig.2.1 .

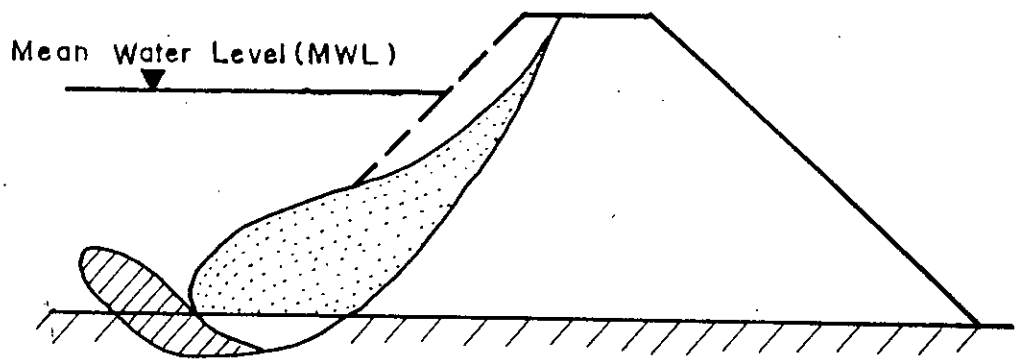

Fig.2.1 Sliding of the Embankment due to Soft or Weak Foundations

Excessive pore water pressure in confined seams of sand and silt, artesian pressure in abutments, or hydrostatic excess developed due to consideration of clay seams embedded between sands or

silts, etc. may reduce the shear strength of the soil, until it becomes incapable of resisting the induced shear stresses, leading to the failure of embankment foundations without warning, (Garg,1976). Loose sand foundations may fail by the liquefaction or flow slides.

b) Slide in Embankments: When the embankment slopes are too steep to resist by the strength of the soil, they may slide causing embankment failure. The most critical condition of the slide of the riverside slope is the sudden drawdown of the reservoir, Fig.2.2, and countryside slope is most likely to slide, when the reservoir is full, Fig.2.3. The riverside slope failures seldom lead to catastrophic failures, but the countryside slope failures are very serious. These failures, generally, occur due to development of excessive unaccounted pore pressure which may reduce the shearing strength of the soils as explained in the previous section. Many embankments may fail during the process of consolidation, at the time of construction or after the construction.

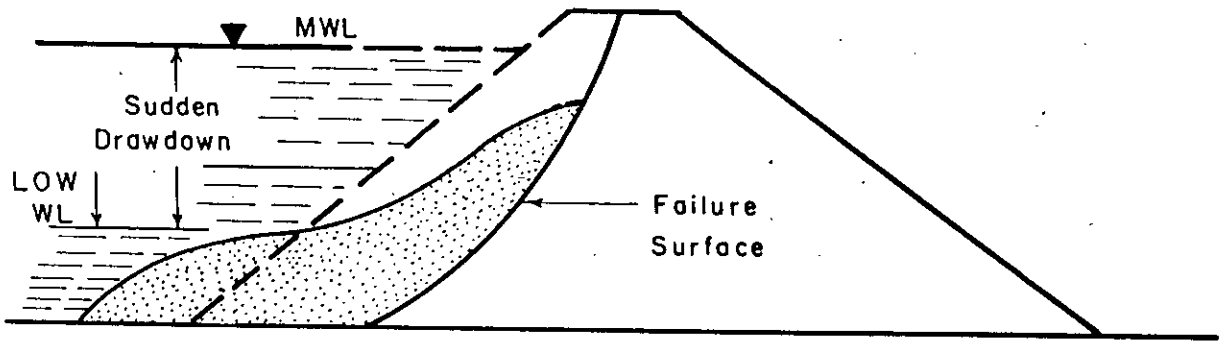

Fig.2.2 Riverside Slope Slide due to Sudden Drawdown

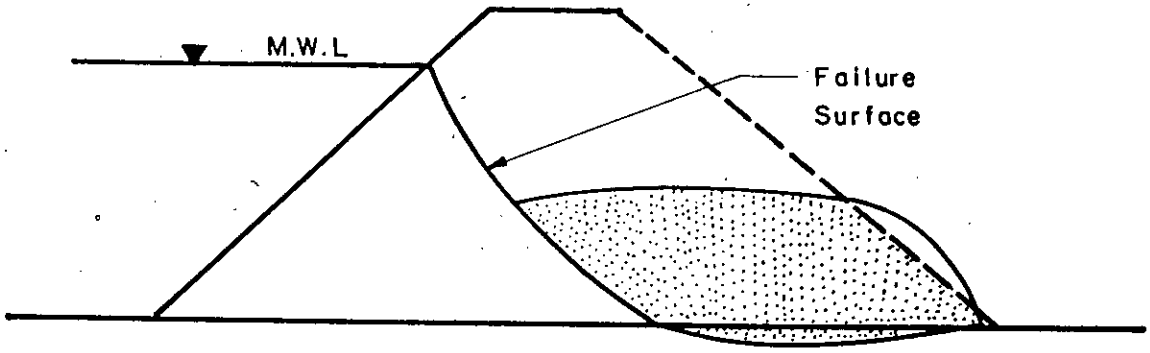

Fig.2.3 Countryside Slope Slide due to Sudden Drawdown

Generally, there are three modes of failure of the slopes of an embankment:

i) Sliding down of the slope.

ii) Gradual erosion from near the landside toe of the slope.

iii) Piping through opening on the landside toe of the slope.

A comprehensive list of the modes of unsatisfactory performance of embankments in the U.S.A is reported by Middle-brook (1953). Some of the common modes of failure include sliding of the side slopes or foundation, seepage through embankment creating seepage sloughs and removal of material from the toe, piping through the embankment and foundation and excessive settlement leading to longitudinal crack opening exposing the embankment to leakage.

In order to establish the causes of sliding, two methods of analyses were used: the effective stress method and the total

stress method. In the effective stress analysis pore pressures that developed during the highest flood condition were calculated from the flownets. Three distinct aspects enter into stability computation, (Lowe III,1967) :

1. A method of analysing the forces causing and resisting stability failure.

2. A method of determining and expressing the shear strength of the materials of the embankments and its foundations.

3. The application of the methods of analysing the forces and determining the shear strength to popular conditions of loading of the embankment.

Using limit equillibrium method the overall stability of sliding may be analysed. Various failure surfaces are considered for the sliding mass. The basis for once a very popular method of analysis called the Ø-circle method. The method is well described in Taylor(1937). Spencer (1969) described another type of failure surface, the logarithmic spiral surface. Huang et. al.(1976) reported that the stability can also be calculated using logarithmic spiral surface and extension of Taylor´s concept. In the circumstances where failure surface deviate greatly from cylindrical, a method based on the stability of earth slices

above the failure plane can be used to determine stability. The method was originated by Fellenius (1927)*. The method has since then been modified by Bishop (1955), Janbu (1954), Lowe and Karafiath (1960), Spencer (1967), Morgenstern (1963), Morgenstern and Price (1965). Winterkorn and Fang (1975) provided a summary of various methods of analyses. Whitman and Baily (1967) carried out an investigation on the various methods of stability analyses used. It was found that ordinary method of slices (Fallenius method) may err by as much as 60 percent while in simplified Bishop method the error is less than 5 percent, (Safiullah, 1982). Classification of slope stability analyses by limit equilibrium method, type of solutions, basic assumptions and references are presented in the Table 2.1.

---

* Cited by Safiullah (1982)

Table 2.1 Classification of slope stability analyses by limit equilibrium method
(After Winterkorn and Fang,1975)

Type of Failure plane	Name of method	Type of solution	Basic Assumptions	References
Straight Line	Culmann Method	Analytical	Failure occurs on a plane through the toe of the slope (Fig.2.4a).	Culmann(1866)
	Method of infinite slope	Analytical	The slope is constant with unlimited extent.	Resal(1910)
			A vertical column is typical of the entire mass.	Frontard(1922)
			No cohesion may be depended upon within the depth to which tension occurs.	
	Wedge Method	Semigrapical Analytical	Sliding block mechanism is assumed,with lateral earth forces.	Culmann(1866) Terzaghi and peck(1967) Lambe and Whitman(1969)
Circular Arc	Slices Method	Semigraphical Numerical	The lateral forces are equal on two sides of each slice (Fig.2.4b).	Fellenius(1927)
	Bishop's Method**	Analytical Numerical	Oblique side forces on each slice are consider (Fig.2.4c).	Bishop(1955)
	Ø-Circle Method**	Analytical Graphical	Resultant acting on rupture arc in tangential to a concentric circle with radius=R sinØ (Fig.2.4f).	Taylor(1937,1948)
	Lowe and Karafiath Method**	Graphical Analytical	Earth forces as well as water forces on the the sides of the slices are considered (Fig.2.4d).	Lowe and Karafiath(1960) Giger(1970
	Spencer Method**	Graphical Analytical	Assuming inter-slice forces are parallal (Fig.2.4e).	Spencer(1967)
Logarithmic spiral	Log-Spiral Method	Analytical	No assumptions required to make statically determinate.	Rendulic(1935) Taylor(1937)
Irregular Failure plane**		Analytical Numerical	General slip surface. Forces between slices are considered.	Morgenstern and Price(1965) Janbu(1954) Monveiller(1965)

** Long term stability analysis (seepage considered)

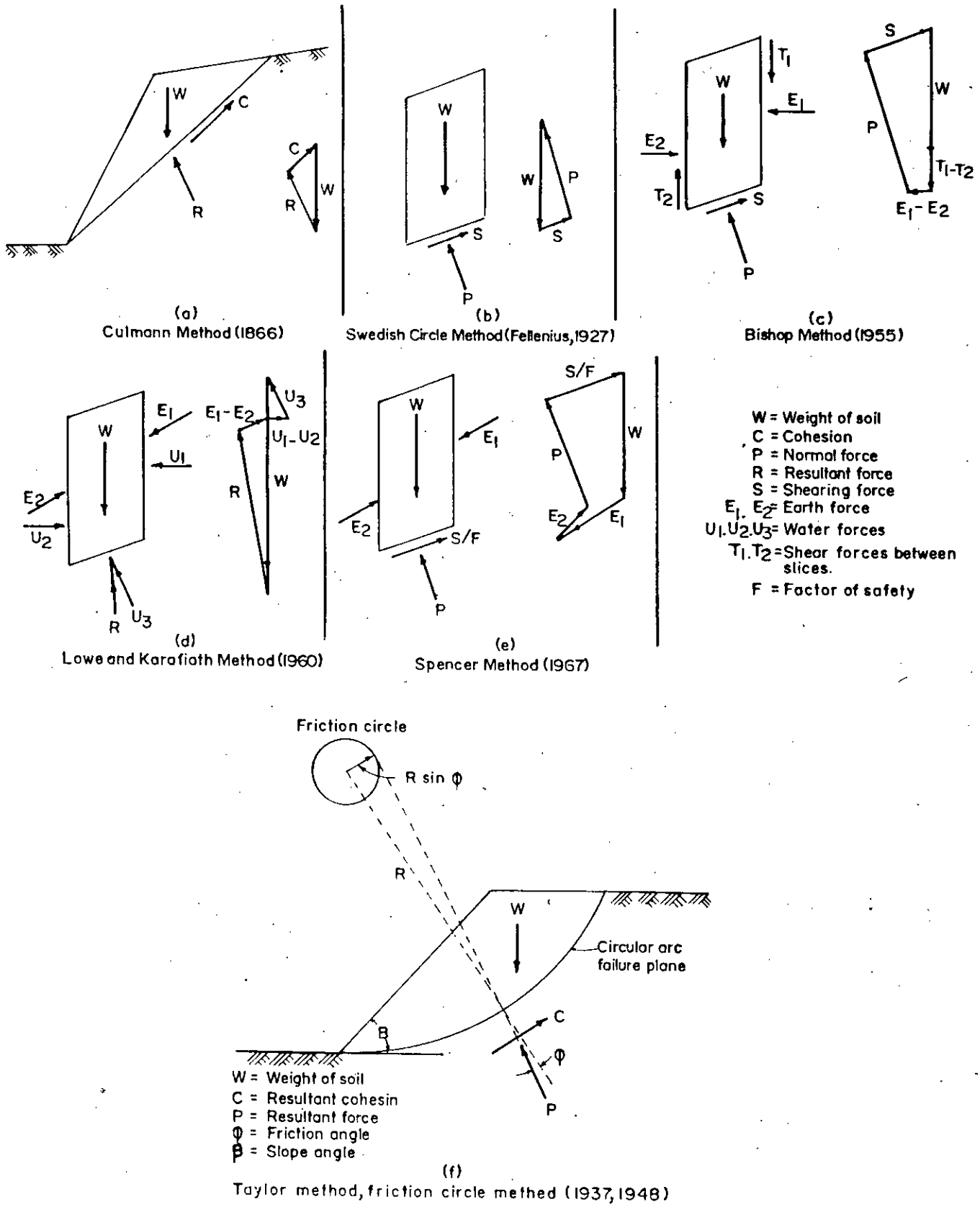

Fig. 2.4 Force Acting on a Free Body

2.2.2 SEEPAGE FAILURES

Controlled seepage or limited uniform seepage is inevitable in all earth embankments, (Eusufzai and Safiullah, 1974). And ordinarily, if it seeps through the embankment body or through the foundations, that may lead to progressive erosion and subsequent removal of the soil grains from within the body of the embankment or the foundations of the embankment. Sloughing is the progressive removal of soil from the wet countryside face. More than one third of failures of earth embankments is due to seepage reasons.

Usually, high seepage pressure exists at the exposed surface near the toe of the embankment. The seepage pressure being proportional to the slope in any flood embankment, if the phreatic line touches the embankment slope at the exist point, the slope become hydrostatically unstable. This force can be resisted only by soil cohesion as the firction between grains, cannot develop due to lack of confinement. In soils with low plasticity little or no amount of resistance can develop and soil erosion is bound to occur at the spot where free waterline touches the slope.

It is not possible to quantify such pressure in relation to soil cohesion as no relation between these two quantities are available. The general practice is to provide filters so that the phreatic line does not touch the slope surface.

2.2.3 PIPING FAILURES

There are some cases of piping failure on various embankments. A grain size analysis of the soil which shows that the soil lies close to the gradation range of soils susceptable to form transverse crack in the embankment,(Sherard et.al. 1963). Transverse crack in embankment made of above type of soil may occur if there is differential settlement. There is no surface evidence of such differential settlement, the possibility of existance of internal transverse crack can not be excluded.

Another reason for piping could be non uniform compaction at the spot. The soil which has high uniformity coefficient, if compacted properly is not likely to form piping holes unless initial openings made by transverse crack or non uniform compaction exist.

Sheraj (1979), reported that failure of the embankment occur at the middle of embankment section. The bottom portion of the embankment section is compacted due to the weight of the upper portion. The top portion of the embankment section is compacted due to rain, sunshine wind etc. The middle portion of the embankment section is not fully compacted due to shortage of adequate surcharge. Aitchison and Wood (1965) reported from a large number of Australian cases observed that piping is initiated by deflocculation of clay particles in soils with high

sodium adsorption ratio.Recent investigations by Jaworski et.al. (1981)* have shown that piping by hydraulic fracturing where discontinuities within the embankment are present such discontinuities may be due to settlement cracks. Improper compaction or due to any pre existing crack. Although their investigations suggest that a condition of hydrolic fracturing may be related to tensile strength of the soil, the development of an appropriate theory to take this into consideration is highly desirable.

Piping failure through the embankments may occur in various modes:

a) **Piping through foundations:** Sometimes when highly permeable cavities or fissures or strata of coarse sand or gravel are

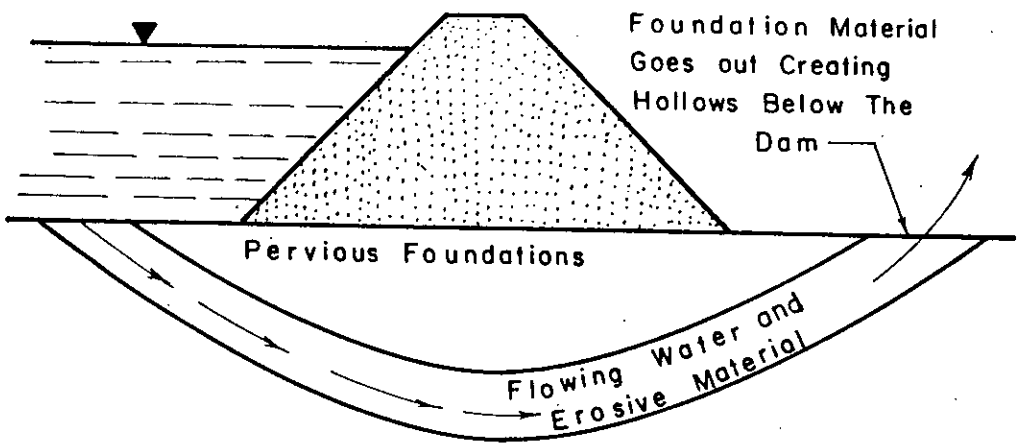

Fig.2.5 Piping through the Foundations of the Embankment

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* Cited by Safiullah(1982)

present in the foundations of embankment, water may start seeping at a huge rate through them, Fig.2.5. This concentrated flow at a high gradient, may erode the soil. This leads to flow of water and soil, ultimately resulting in a rush of water and soil, thus, creating hollows below the foundations. The embankment may sink down into the hollow so formed, causing its failure.

b) Piping through dam body: When the concentrated flow channels get developed in the body of an embankment, Fig.2.6 , soil may be removed in the same manner as is explained in the previous section. This may cause subsiding of the embankment. The flow channels may develop due to faulty construction, insufficient compaction, cracks developed in embankment owing to foundation settlement, shrinkage cracks, animal burrows, etc. All these causes can be removed by better construction and better maintenance of the embankments.

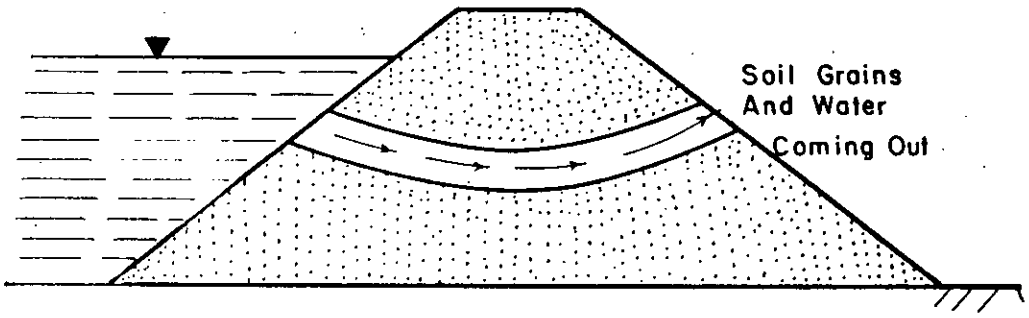

Fig.2.6 Piping through the Body of the Embankment

Piping through the embankment body, generally, get developed near the pipe conduits passing through the dam body. Contact seepage along the outer side of conduits may either develop into piping, or seepage through leaks in the conduits. This can be avoided by compacting the soil properly near the outlet conduits, by preventing the possibilities of leakage through conduits, by preventing the formation of cracks in the conduits, (Garg,1978). These cracks in the conduits are caused by differential settlement and by overloading from the embankment. When these factors are controlled, automatically, the possibility of piping due to leakage through the conduits is reduced.

c) **Sloughing of downstream toe:** The process behind the sloughing of the toe is somewhat similar to that of piping. The process of failure due to sloughing starts when the downstream toe becomes saturated and gets eroded, producing a small slump or a miniature slide. The miniature slide leaves a relatively steep face which becomes saturated by the seepage from the reservoir and slumps, forming a more unstable surface. The process continues till the remaining portion of the embankment is too thin to withstand the horizontal water pressure, leading to sudden failure of the embankment.

2.2.4 HYDRAULIC FAILURES

About 40 percent of earth embankment failures have been attributed to hydraulic causes, Garg(1976). The failure under this category, may occur due to the following reasons:

a) By overtopping: Water may overtop the embankment, if the design flood is under estimated or if the spillway is of insufficient capacity or if the spillway gates are not properly operated. Sufficient freeboard should, therefore, be provided as an additional safety measure.

b) Erosion of riverside face: The waves develop near the top water surface due to wind, try to notch out the soil from the riverside face and may even, sometimes, cause the slip of the riverside slope. Riverside stone pitching or riprap is usually provided to avoid such failures.

c) Cracking due to frost action: Frost in the upper portion of the embankment may cause heaving and cracking of the soil with dangerous seepage and consequent failure. An additional freeboard allowance upto a maximum of 1.5 m should, therefore, be provided for embankment in areas of low temperatures, (Garg,1976).

d) Erosion of countryside face by gully formation: Heavy rains falling directly over the countryside face and the erosive action of the moving water, may lead to the formation of gullies on the countryside face, ultimately leading to the embankment failure,

Fig.2.7. This can be avoided by proper maintenance, filling the cuts from time to time especially during rainy season, by grassing the slopes and by providing proper berms at suitable heights (Fig.2.8), so that the water does not flow for considerable distances. The proper drainage arrangements are made for the removal of the rain water collected on the horizontal berms.

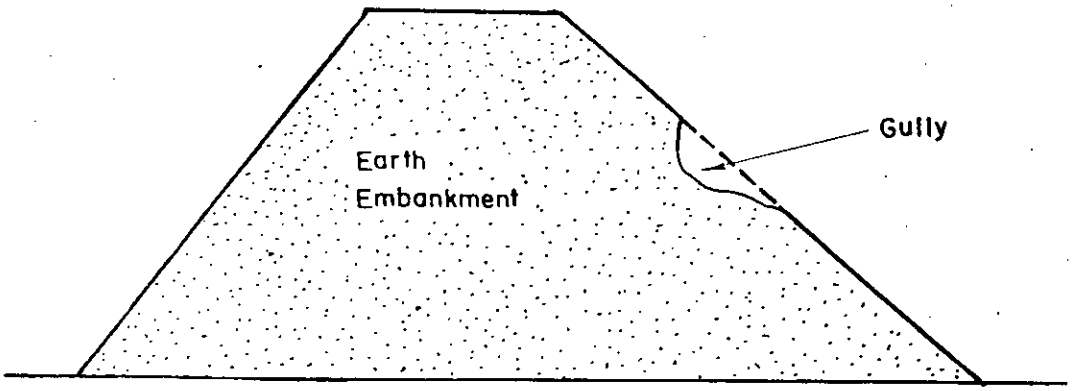

Fig.2.7 Erosion of Countryside Face by Gully Formation

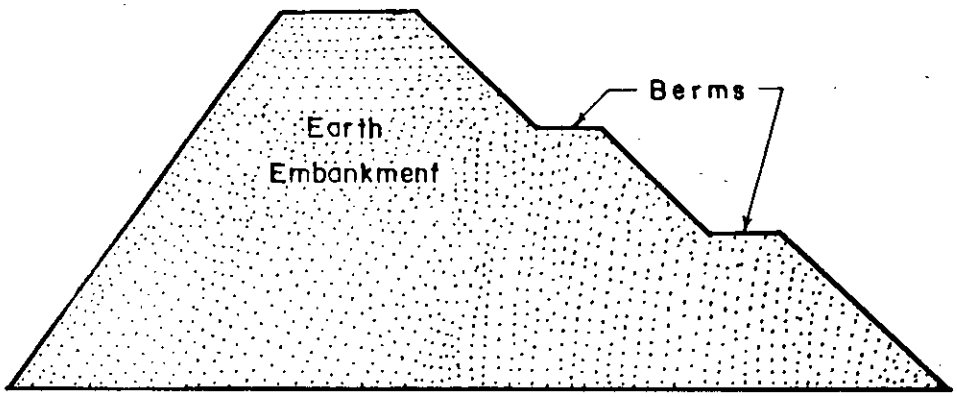

Fig.2.8 Remady From Gully Formation of Countryside Face by Providing Proper Berms

Since the provision of berms ensures the collection and removal of water before it acquires high downward velocities, the consequent erosion caused by the moving water (run-off) is considerably reduced, (Garg,1976).

e) Erosion of the countryside toe: The countryside toe of the earth embankment may get eroded due to two reasons, i.e. (i) the erosion due to cross currents that many come from the spillway buckets (ii) the erosion due to tail water. This erosion of the toe can be avoided by providing a countryside slope pitching or a riprap up to a height slightly above the normal tail water depth. Side walls of the spillway (called diaphragm walls) must be of sufficient height and length so as to prevent the possibility of the cross flow towards the earthen embankment.

2.3 CASE STUDIES

There are several case studies of embankments failure throughout the world, some of them are briefly described in the following sections.

2.3.1 CASE STUDIES OF U.S.A

In U.S.A. Rector Geek Dam, completed in California in 1946, is an instructive early example of a dam build by methods consisted with modern practice in which important transverse cracks

developed but where simple repairs were adequate to prevent leakage, (Sherard, 1973). The Dam is an essentially homogeneous embankments with a height of 200 ft and thin zones of rock fill on both sides. Additional details of construction and measurements of the crest movements are given in Sherard et. al. (1963)*.

The embankments were compacted in 6-in layers with 10 passes of a heavy sheepsfoot roller. The crest of the dam settled about 3 ft in the first 10 years of operation or about 1.5 percent of the height, which is more than would have been expected. A three fourth inch wide, essentially vertical crack developed diagonally across the crest in February,1947 during the first resirvoir filling. The crack was about 100 ft from the end of the dam over the left abutment, which was steeper than the right abutment. A 24-ft deep test pit was excavated to trace the crack down into the dam. At the bottom of the test pit the crack was about half inch wide and extended below to an unknown depth. After several months the crack had not increased in width and it was filled with cement grout.

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* Cited by Sherard (1973)

2.3.2 CASE STUDIES OF U.K.

In U.K, the Balderhead Dam in England constructed in 1961-1965, is a rockfill dam with a narrow, verticle earth core and a maximum height of 48 metres, (Sherard, 1973). In April 1967, after the reservoir had been full continuous for more than 2 years, a subsidence crater appeared near the upstream edge of the crest. Extensive investigations led to the conclusion that concentrated leaks had developed through horizontal differential settlement cracks in the core and that a considerable portion of the core material had been damaged by porgressive erosion.

The core was constructed of a gravelly, sandy clay (boulder clay) with proper gradation. The clay fraction was tough at the plastic limit. The liquid limit was 30 to 40, the plastic limit was 12 to 22. The material was compacted in 6-in layers with a tamping roller at an average water content near standard AASHO optimum.

The shape comprising the shells of the dam was excavated by scrapers after ripping. The specifications required that the material be placed in 9 inch layers and compacted to 95 percent standard AASHO maximum density.

2.3.3 CASE STUDIES OF BANGLADESH

In Bangladesh Dhaka-Narayangonj-Demra (DND) is a earth embankment constructed for protecting land against flood discharges and tidal inundation. The failure of the embankment slopes of the DND project during the high flood of 1974 has been investigated as a case study by Safiullah (1982). The earth embankments forming the enclosure of the project area are part of the old road system and a railway embankment in which all the bridge openings were closed in 1968. The embankment thus formed functioned well until August, 1974. With the abnormal rise in water level in the surrounding rivers, the side slopes of the embankments at some points started to show signs of distress in the form of either erosion near the toe or sudden sliding down. During the later part of August the situation became very serious and embankments were threatened with complete breach at different locations but fortunately saved by the efforts made by the engineers of the project.

The case study of failure of the DND embankment slopes revealed some of the soil mechanics aspect of the problem. The modes of failure can be classified into four categories :

1. Progressive erosion from near the country side toe of the embankment occured at places where the soil cohesion was low and can be attributed to high seepage pressure at the exit point, low cohesion of the soil at places where failure occured and nonprovision of filter at such locations.

2. Sudden sliding down of the embankment slope occured on relatively old portion of the embankment (Dhaka-Narayanganj portion) and can be attributed to reduction in soil strength due to increase in pore water pressure set up by the seepage flow during high flood.

3. Sliding failure in thin layer in the new portion of the embankment (Dhaka-Demra portion) can be attributed to shrinkage cracks (due to high clay content and OCR) exposed on the surface of the slope.

4. Piping failure occuring at one location could probably be due to non unoform compaction or effect of tunnels created by burrowing animals. However, no definite criteria could be established in this respect.

The analyses revealed that some aspects of the failures could be predicted in advance if stability computations were performed with proper soil parameters obtained from routine laboratory tests. In the case under study, limit equilibrium methods of analyses with effective stress conditions obtained from seepage flow nets provided realistic failure criteria for stability of slopes. In an old embankment like DND the Ø=0 soil parameters did not provided the required failure criteria.

Some amount of research is necessary to provide a quantitative relation between limiting seepage pressure and available soil

cohesion at exit point of the seepage line resist erosion in various clay soils.

In embankment soil with high clay content (CH group) shrinkage cracks on the slope surface are possible and may create stability problem. Any stability analysis on such soils should take into consideration the nature of the cracks without which the factor of safety calculation may be in error. It has been observed that soil movements in such soils were in thin layers.

2.4 METHODS OF ANALYSES OF EARTH EMBANKMENTS

There are various methods available for examining the stability of slopes of the embankment. They are -

- o Swedish Slip Circle method
- o Taylor's method
- o Bishop's method
- o Bishop's simplified method
- o Bishop and Morgenstern method
- o Spencer method
- o Hunter and Schuster method
- o Morgenstern method

Conventionally, Laplacian two dimensional equation is used to calculate the quantity of seepage through the embankment. Of the

various methods of piping analysis mention can be made of Sherard et. al. (1963).

Details of the methods are provided in the chapter 3.

2.5 DESIGN CRITERIA FOR EARTH EMBANKMENTS

The major design parameters for the commencement of embankment design are the selection of the magnitude of flood flows, corresponding flood levels and their frequency of occurrence. The latter depends on the degree of protection desired. Whatever may be the degree of protection, an embankment must be safe and stable during all phases of construction and operation.

To accomplish this, the design of an embankment must consider the following criteria, Garg (1976).

i) The embankment must not be overtopped during the passage of the design flood i.e it should have sufficient freeboard.

ii) A fill of sufficiently low permeability should be provided out of available materials, so as to serve the intended purpose with minimum cost. Borrow pits should be as close to embankment sit as possible, so as to reduce the leads.

iii) Sufficient freeboard must be provided for wind set-up, wave action, frost action and earthquake motions.

iv) The seepage line (i.e phreatic line) should remain well within the downstream face of the embankment, so that no sloughing of the face occurs and the factor of safety against boiling is not less than 1.5.

v) There should be no possibility of free flow of water from upstream river side to downstream country side face.

vi) There is a little harm in seepage through a flood control embankment, if the stability of foundations and embankments is not impaired, by piping, sloughing, etc, but a conservation embankment must be as watertight as possible.

vii) The riverside slope must be protected against wave action and the crest and counteryside slope must be protected against erosion by wind and rain.

viii) The portion of the embankment, downstream of the impervious core should be properly designed by providing suitable horizontal filter drain or toe drain or chimney drain etc.

ix) The countryside slope should remain stable during steady seepage at design high flood level.

x) The riverside slope must be stable during rapid drawdown conditions where these prevail.

xi) The upstream and downstream slope should be flat enough so as to provide suficient base at the foundation level, such that the maximum shear stress developed remains well below the corresponding maximum shear strength for the soil, so as to provide suitable factor of safety.

xii) It is understood that the consolidation of the soil does not take place instantaneously when the compaction is done by external loadings. It takes place slowly as the excess pore water pressure goes out and the load is transferred to the soil grains. In coarse gravels, the void openings are large enough so as to permit rapid escape of confined water and air and full compaction may occur before the construction is over. But in fine grained impervious soils, the consolidation is slow. It is therefore, becomes necessary in such cases to provide an additional height (between 2 to 3 percent of embankment height) of the fill.

All the above criteria must be satisfied and accounted for, in order to obtain a safe design and construction of an earth embankment.

CHAPTER 3

METHODS OF ANALYSES

3.1 INTRODUCTION

Gravitational and seepage forces tend to cause instability in natural and man made slopes. The most important types of slope failure are illustrated in Fig.3.1. In rotational slip the shape of the slip surface in section may be a circular arc or a non-circular curve. In general, circular slips are associated with homogeneous soil conditions and non circular slips with non-homogeneous conditions. Tanslational 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 parallal to the slope. Compound slips usually occur where the adjacent stratum is at grater 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

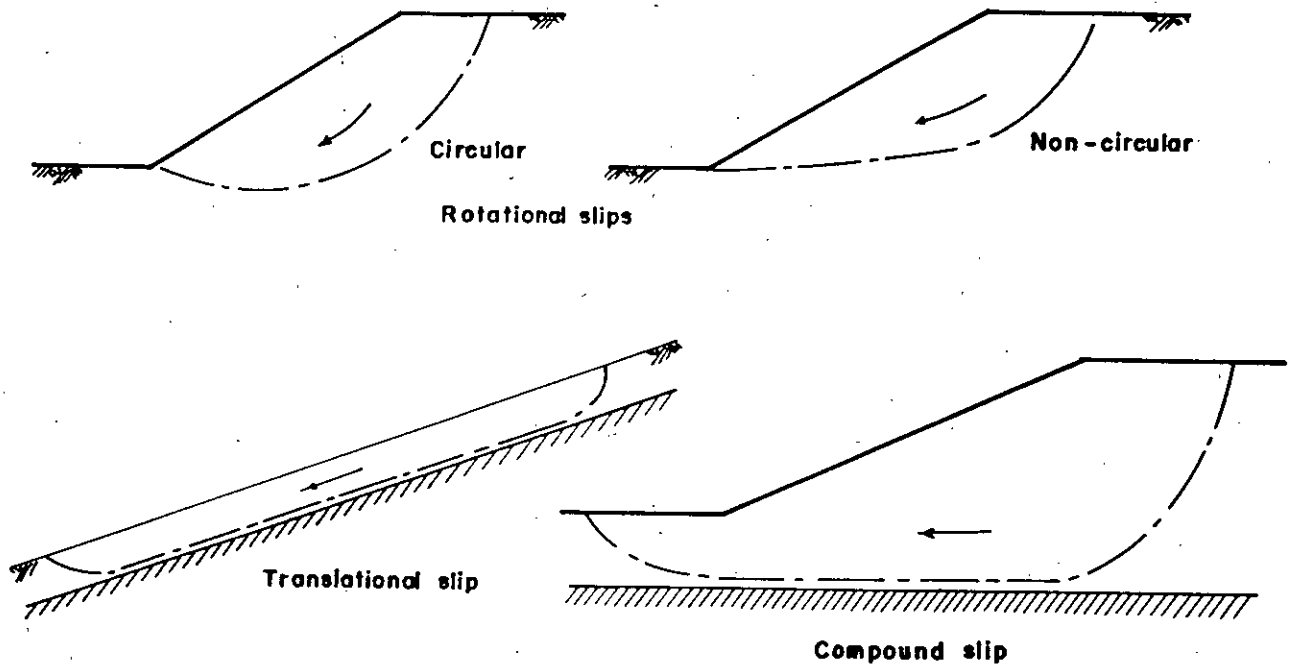


Fig.3.1 Types of Slope Failures

the failure surface. It has been shown that a two dimensional analysis gives a conservative result for a failure on a three dimensional (dish shaped) surface, (Craig,1974).

The failure of a mass of soil in a drawdown and outward movement of a slope is called slope failure. Two types of slope stability problems usually occur in clays; short term stability (end-of-construction case) and long term stability (steady seepage case). Short term case applies after a cut is made in a slope. The long term case is also encountered in natural slopes and should also be considered in analysing the stability of embankments. In this case pore pressure may assumed to be in equilibrium and are determined from considerations of steady seepage, thus no excess pore pressures are included.

The quantity of seepage through an embankment is of prime importance to the designers because of its involvement in the erosion and piping failure.

In the following sections various methods of analyses of slope stability, seepage and piping are briefly described.

3.2 SLOPE STABILITY ANALYSES

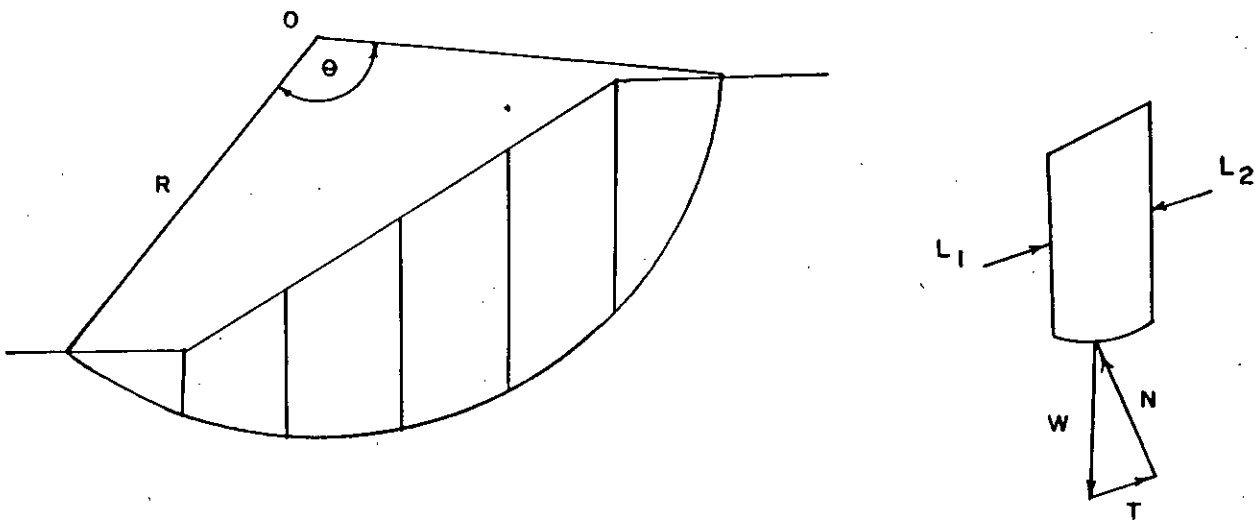
The following aspects of the stability of an embankment are normally considered, in design (BWDB, 1987A) :

- stability of the countryside slope during steady seepage flow;
- stability of the riverside slope during sudden drawdown;
- resistance of the foundation to shear failure;
- stability of the foundation under the weight of the embankment.

Various methods have been proposed for computing embankment stability. In general, these methods are based upon certain assumptions with respect to character of an embankment failure.

3.2.1 SWEDISH SLIP CIRCLE METHOD

An earth embankment usually fails, because of the sliding of a large soil mass along a curved surface, BWDB (1987A). It has been established by actual investigation of sides of railway embankments in Sweden that the surface of slip is usually close to cylindrical i.e an arc of a circle in cross section. This method assumes the surface of rupture to be a cylindrical one, (Terzaghi and Peck, 1967). This is a comparatively simple method of analysing embankment stability. For the analysis, the location of the centre of the possible failure surface is usually assumed. The earth mass is divided into a number of vertical segments called slices, Fig 3.2. These verticles are usually equally



a) Failure Surface divided into Wedges b)Equilibrium of a Wedge

Fig.3.2 Slpoe Stability Analysis by Swedish Slip Circle Method

spaced, through it is not necessary to do so. Depending upon the accuracy desired, six to twelve slices are generally sufficient, (Garg,1976). The center of the most critical circle can only be

found by trail and error, various slip circles being analysed and the minium factor of safety eventually obtained, (Smith,1968).

The factor of safety of slope failure is defined as the ratio of resisting moment to disturbing moment and is given by

$$F.S. = \frac{\text{Resisting moment}}{\text{Disturbing moment}} = \frac{M_r}{M_d}$$

$$F.S = \frac{CL + \tan\emptyset(\Sigma N_s - \Sigma U_s)}{\Sigma T_s} \qquad (3.1)$$

where, F.S= factor of safety

C= cohesion of the soil used for embankment fill

L= length of arc intersecting the embankment

Ø= angle of internal friction of soil used for embankment fill

ΣN_s= sum of the normal forces for all the slices

ΣT_s= sum of the tangential forces tending to produce moment of the soil along the circumference

ΣU_s= sum of uplift presures

Details of the method can be found in BWDB (1987A).

The Swedish slip circle method considers the effect of pore water pressure on the shear strength of soil.

3.2.1.1 PHREATIC OR SEEPAGE LINE

The phreatic or seepage line or saturation line is defined as the line within the embankment section below which there is a positive hydrostatic pressure. The hydrostatic pressure on the phreatic line is equal to atmospheric pressure and hence, equal to zero. Above the phreatic line, there is a zone of capillary saturation called capillary fringe, in which the hydrostatic pressure is negative. The appreciable flow through the embankment body, below the phreatic line, reduce the effective weight of this soil and thus reduces the shear strength of the soil. On the other hand, the insignificant flow through the capillary fringe, leads to greater shear strength, because the capillary tension in water leads to increased intergranular pressure. As the effects of the capillary fringe are on a slightly safer side, they are usually neglected.

It is, therefore, essential to determine the position of the phreatic line because it:

- gives a divide line between the dry (or moist) and submerged soil. The soil above the phreatic line will be taken as dry and the soil below the phreatic line taken as submerged for computation of the shear strength of the soil.

- represents the top streamline and so assists in drawing the flow net.

- helps to ensure that it does not cut the downstream face of the embankment. This is necessary for preventing, softening or sloughing of the embankment.

For stability analysis of an embankment, determination of phreatic line, i.e. seepage gradient, along the embankment cross-section is the prime requirement, (BWDB,1987A). The phreatic line is assumed to be a base parabola with its focus at F, Fig.3.3, the extreme end of the downstream toe of the embankment. The equation of the base parabola is given by

$$(x^2+y^2)^{1/2} = x+S \qquad (3.2)$$

where,

S is the distance to the point (x, y) from the directrix, called focal distance.

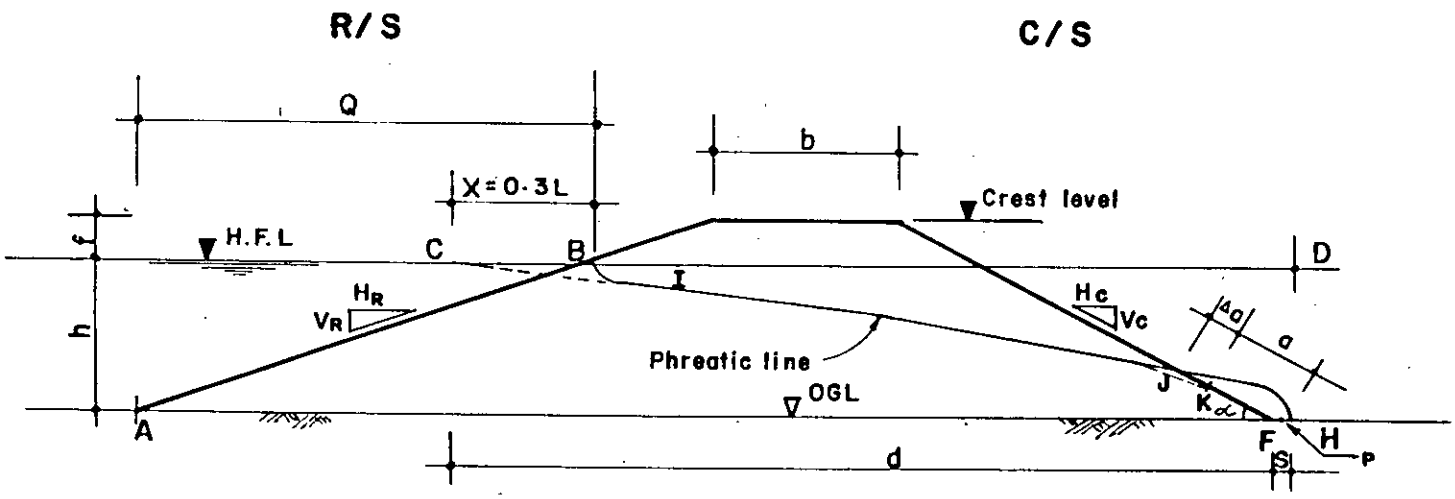


Fig.3.3 Location of the Phreatic Line through an Embankment

The following procedure is used in locating the phreatic line:

a) It is assumed that the horizontal projection of the upstream face of the embankment AB as shown in Fig.3.3 above water level be L. On the water surface, a distance BC = 0.3L is considered, the point C is the starting point of the base parabola.

b) To locate the directrix of the parabola, an arc is drawn taking the point C as centre and with a radius CF to cut the horizontal line through CB at D. A vertical tangent is drawn to the curve FD at D. The vertical line DH is the directrix.

c) The co-ordinate of the point C(d,h) with respect to F are obtained from the geometry of the embankment section. Substituting these co-ordinates in the equation of the base parabola the value of S is obtained. The vertex P of the base parabla is to be situated at a distance equal to 0.5 S from F, beyond the downstream toe of the embankment.

d) Using the equation of base parabola with value of S determined in the previous step, a few more co-ordinates of the base parabola at known distances (X_i) are computed and the parabola drawn.

e) The parabola has now to be corrected at the entry and exit points. The phreatic line must be from B and not from C. The phreatic line is a flow line and must start perpendicular to

the upstream face AB which is an equipotential line. Hence the portion phreatic line at B is sketched free hand, as a reverse curvature BI, in such a way that it starts perpendicularly to AB.

At the exit the base parabola will cut the downstream slope at J and extended beyond the limits of the embankment as shown by the borken line in Fig.3.3 . But according to exit conditions, the pheratic line must emerge at some point K, meeting the downstream face tangentally 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 downwards, can be determined easily by using the equation given by

$$a = (a + \Delta a)(180 - \alpha)/400 \qquad (3.3)$$

where a, Δa and α (in degrees) are as shown in Fig.3.3 .

Having computed Δa, the point K is plotted and the phreatic line BIK completed.

A general analytical method for computation of 'a' is as follows:

The countryside slope, is computed by using the relation,

$$\alpha = \tan^{-1}\frac{V_c}{H_c} \qquad (3.4)$$

where V_c: H_c = country sideslope

$$\text{Now } d = \frac{H_c}{V_c} h + b + \left(\frac{H_c}{V_c} + \frac{H_R}{V_R} \right) f + X \qquad (3.5)$$

where,

V_R: H_R = river sideslope

$$Q = \frac{H_R}{V_R} h$$

$X = 0.3Q$

b = crest width

f = freeboard

$d' = d - 0.3Q$

$$\text{Hence, } a = \frac{d'}{\cos\alpha} - \left(\frac{d'^2}{\cos^2\alpha} - \frac{h^2}{\sin^2\alpha} \right)^{1/2} \qquad (3.6)$$

3.2.2 TAYLOR'S METHOD

By this method rapid determination of the factor of safety (F.S) is possible in case of a homogeneous slope. The Taylor's method is based on the friction circle method (Taylor, 1937, 1948). Taylor (1948) prepared two curves that relate the stability number (N) to the angle of slope (β): the first is for the general case a C-Ø soil with an angle of slope (β) less than 53°, and the second one is for a soil with $Ø=0^o$. The curves are shown in Appendix-A (Fig.A.1 and Fig.A.2). The stability number (N) is given by

$$\text{stability number, } N = \frac{C_m}{\gamma H} \qquad (3.7)$$

where,

C_m = cohesion mobilised with respect to total stress

γ = density of soil

H = vertical height of embankment.

Details of the method can be found in Smith (1968).

3.2.3 BISHOP'S METHOD

Bishop(1955) used the effective stress concept in his analysis of stability of slope of embankments. If a slope consists of several types of material with different values of C and Ø and if the pore pressures u in the slope are known or can be estimated, the Bishop's method of slices can be used.

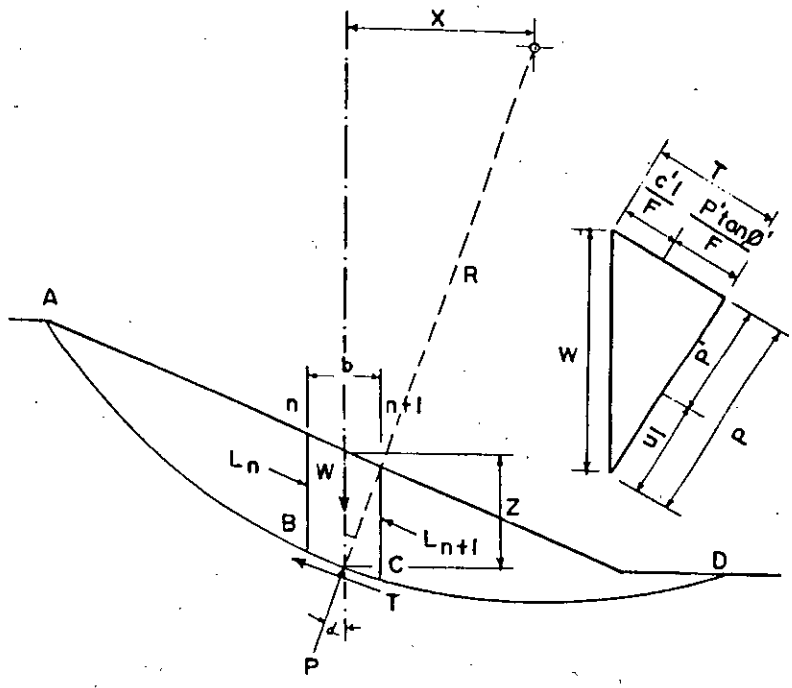


Fig.3.4 Effective Stress Analysis: Bishop's method

In this method the failure surface is considered as a circular arc (ABCD, Fig.3.4). The soil mass bounded by the failure surface and the slope lines are divided into several vertical slices and the forces acting on the vertical slices are considered in examining the slope stability.

The factor of safety is given by the relation,

$$F.S = \frac{1}{\Sigma W \sin\alpha} \Sigma [c' l + W(\cos\alpha - r_u \sec\alpha) \tan\emptyset'] \qquad (3.8)$$

where,

F.S= factor of safety

ΣW = weight of slices

l = length of BC (taken as a straight line)

α = angle between p and vertical

c' = effective cohesion of the soil of embankment fill

$\emptyset'$ = effective angle of internal friction of embankment fill

r_u= pore pressure ratio

The pore pressure ratio is given by

$$r_u = \frac{u}{\gamma z} \qquad (3.9)$$

where,

u= pore water pressure

γ= unit weight of soil

z= height of slice

The details of the Bishop's method can be found in Smith (1968).

3.2.4 BISHOP'S SIMPLIFIED METHOD

The simplified Bishop method, for factor of safety against sliding, uses the concept of slices. The equilibrium of the mass of soil above a circular arc of sliding is investigated by considering unit length along the bank. Using the simplified approach, the forces on the vertical surfaces between the slices are assumed to have no influence on the forces on the base of each slice. Where there are no external forces on the slope, the overall summation of the side forces will sum to zero, Fig.3.5. The simplified Bishop method sums the balancing forces for each

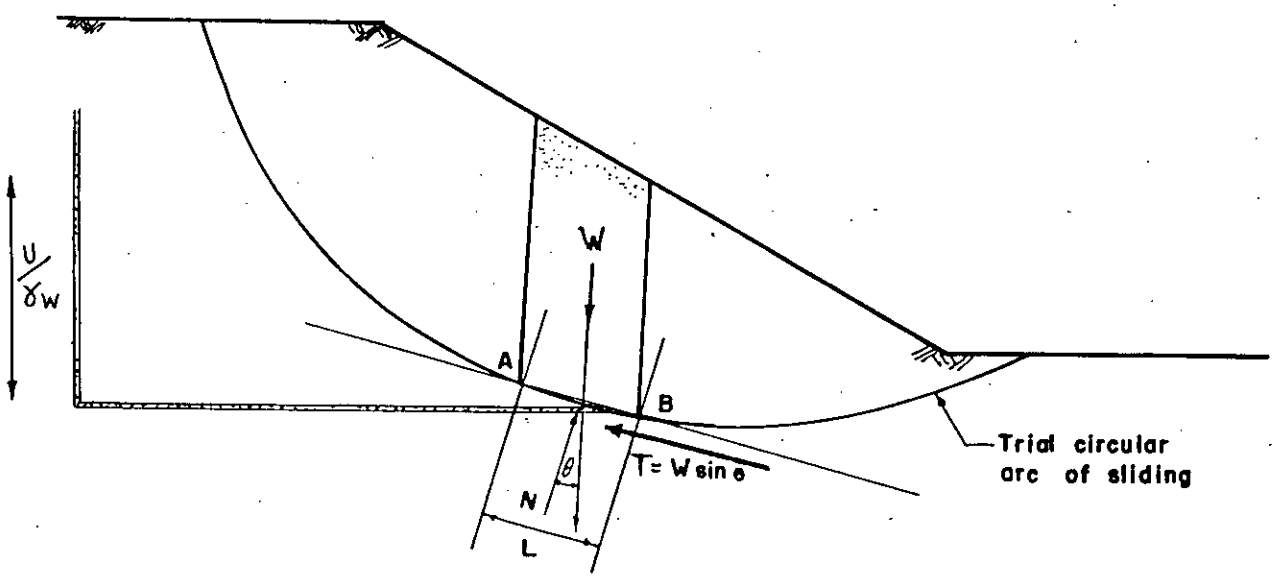


Fig.3.5 Slope Stability Analysis by Bishop's Simplified Method

slice, and the factor of safety is determined using the following equations,

$$F.S = \frac{N}{T} \qquad (3.10)$$

$$T = W \times \sin\theta \tag{3.10a}$$

$$N = \frac{\{C \times L + \tan\phi \times (W \times \cos\theta - U \times L)\}}{G} \tag{3.10b}$$

$$G = 1 + \frac{\tan\theta \times \tan\phi}{F} \tag{3.10c}$$

where,

W = total weight of sliice (kN/m^3)

θ = angle between N and vertical (degrees)

u = pore water pressure at surface AB (kN/m^2)

γ_w= unit weight of water (kN/m^3)

ϕ = angle of friction (degrees)

C = cohesion (kN/m^2)

F.S = Factor of safety

F = assumed factor of safety

By trial and error the value of F is determined.

3.2.5 BISHOP AND MORGENSTERN METHOD

The Bishop and Morgenstern method (1960) is based on the Bishop's method of slices and considers the pore water pressure by means of the pore pressure ratio r_u, which is same as the equation 3.9.

The pore pressure ratio is assumed to be constant throughout the cross section which is called homogeneous pore pressure

distribution. If there are minor variations in r_u throughout the soil layer, an average value of r_u is used. For steady state seepage, a weighted average of r_u over the layer is assumed (Winterkorn and Fang,1975).

The factor of safety F.S is defined as:

$$F.S = m - nr_u \tag{3.11}$$

where, m and n are called the stability coefficients. Bishop and Morgenstern prepared charts of m and n for three sets of $C'/\gamma H$ values. The charts are provided in the Appendix-A (Fig.A.3, Fig.A.4 and Fig.A.5).

Bishop and Morgenstern (1960)* claim that the averaging technique for estimating the pore pressure ratio tends to give an over estimation for the factor of safety which in an extreme case will be of the order of 7 percent.

The details of the method can be found in Winterkorn and Fang (1975) and Smith (1968).

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* Cited by Winterkorn and Fang (1975)

3.2.6 SPENCER METHOD

Spencer's method (1967) is based on the work of Fellenius (1927) and Bishop (1955). A cylindrical slip surface is assumed and the earth mass within the surface is divided into small vertical slices. The analysis is done in terms of effective stress and satisfies two equations of equilibrium, the first with respect to forces and the second with respect to moments. The inter slice forces are assumed to be parallel.

The forces acting on a typical slice are shown in Fig.2.4e . The factor of safety F.S is defined as the quotient of shear strength (S) available divided by the shear strength mobilized (τ).

$$F.S = \frac{S}{\tau} \qquad (3.12)$$

The mobilized angle of shear resistance $\emptyset'_m$ is the angle whose tangent is $\tan\emptyset'/F.S$. The depth factor D is assumed to be large. A homogeneous pore pressure distribution is assumed and the pore pressure ratio r_u proposed by Bishop and Morgenstern (1960) is used. The stability factor R_s is defined as,

$$R_s = \frac{C'}{(F.S)\ \gamma Y} \qquad (3.13)$$

where,

C' = cohesion with respect to effective stress

F.S = factor of safety

γ = unit weight of soil

Y = mean height of slice

Spencer provides charts for a range of stability factors R_s from 0 to 0.12 with mobilized friction angle $\emptyset_m$ varying from 10° to 40° and slope angle β up to 34° .Three pore pressure ratios r_u with values of 0, 0.25, and 0.5 are provided. Values of r_u failing between the charts can be obtained sufficiently accurate for practical purposes by linear interpolation.

Details are available in Winterkorn and Fang (1975).

3.2.7 HUNTER AND SCHUSTER METHOD

The Hunter and Schuster method (1968, 1971)* is based on Hunter's work (1968)* and assumes that the potential sliding surface is a circular arc. The soil is saturated to the surface through capillarity and is assumed to be a normally consolidated unfissured clay when $\emptyset$ =0. The cohesion 'C' varies linearly with depth.

The method considers the depth of the water table by using water table ratio M, defined as:

$$M = \left(\frac{J}{E}\right)\left(\frac{\gamma_w}{\gamma'}\right) \qquad (3.14)$$

---

* Cited by Winterkorn and Fang (1975)

where,

J = depth from top of slope to the water table during consolidation

E = height of cut

γ_w = unit weight of water

γ' = sumberged or buoyant unit weight of soil

The factor of safety F.S is defined as

$$F.S = \left(\frac{c}{p'}\right)\left(\frac{\gamma'}{\gamma}\right) R_s \qquad (3.15)$$

where,

c = cohesion

p' = effective vertical stress.

R_s = stability factor

The c/p' ratio can be estimated from Skempton's (1957)* formula

$$c/p' = 0.11 + 0.0037(PI)$$

where,

PI = plasticity index of the soil in percent

R_s are calculated from Charts, which are presented in Appendix-A (Fig.A.6). The method also considers the effect of deep hard strata by using the concept of the depth factor D developed by Taylor. The charts furnished by Hunter and Schuster show the water table ratio varying from 0 to 2 and the depth factor varying from 0 to 4.

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* Cited by Winterkorn and Fang (1975)

For details of the method reference can be made to Winterkorn and Fang (1975).

3.2.8 MORGENSTERN METHOD

The Morgenstern method (1963) is based on the Bishop and Morgenstern method of slices. The primary use of this method is to compute the factor of safety of earth slopes during rapid drawdown. As the reservoir level is lowered, the factor of safety decreases if it is assumed that no dissipation of pore pressure occurs during drawdown. Morgenstern assumed that the slope is simple slope of homogeneous material resting on a rigid impermeable layer at the toe of the slope.

The pore pressure parameter $B = \Delta u/\Delta \sigma_1$ (Bishop, 1954) is assumed to be unity during drawdown, the unit weight γ of the soil is assumed constant at twice the unit weight of water or 124.8 pcf. In addition, it is assumed that the pore pressure can be approximated by the product of the height of the soil above a given point times the unit weight of water.

The drawdown ratio is defined as I/H, where I is the amount of drawdown and H is the original height of the slope, Fig.A.7 .

All assumed potential sliding circles must be tangent to the base of the section. This means that the value of H in the stability factor $C'/\gamma H$ and in the I/H must be adjusted for intermediate levels of tangency.

Morgenstern's charts cover a range of stability factor $C'/\gamma H$ from 0.125 to 0.050 for slopes of 2:1 to 5:1, which are presented in the Appendix-A (Fig.A.7, Fig.A.8 and Fig.A.9).

The details of the method can be found in Winterkorn and Fang (1975).

3.3 COMPARISON OF THE METHODS OF SLOPE STABILITY ANALYSES

The slope stability analysis of an embankment can be done by various methods such as listed in article 3.2 . A comparison of the analyses by each method is given below.

In the Swedish slip circle method it is assumed that the lateral forces are equal on two sides of slice and the failure surface is assumed to be a cylindrical one. Actually below the slices, the forces are distributed load but in analysis it is considered that the forces are concentrated at the middle of the slices. The normal forces which develop along the sliding surface in the shell are much higher in the slices method with lateral earth

forces and a higher factor of safety is obtained (Lowe III,1967). The increase in factor of safety may be as much as 30 percent or more. Lowe III (1967) reported that the slices method of stability analysis considering earth forces on sides of slices is more accurate and preferred to the slices method without earth forces particularly where analysis of an earth embankment with a strong shell or where determination of stresses along the failure arc is of interest.

In the Taylor's method it is assumed that the resultant reaction acting on rupture arc is tangential to a concentric circle with a radius of R sinØ. It gives better result for purely cohesive soil. Stability analysis can be facilitated greatly by the use of stability charts. Most popular of these are Taylor's stability charts, (Lowe III,1967). The charts can be used for nonhomogeneous materials if proper average values of unit weight and shear strength parameters are used.

In the Bishop's method it is assumed that the oblique side forces exist on each slice and the tangential components of forces at the base of the slice are determined by dividing the limiting forces by a assumed factor of safety. The effect of lateral forces are ignored resulting in some loss in accuracy.

In the Bishop's simplified method it is assumed that the lateral earth forces are horizontal, althrough it is not always accurate.

The formula for Bishop's method of analysis can give errors up to 15 percent in the estimated value of factor of safety, although the error is on the safe side, (Smith,1968). Calculation of the analytical solution are adaptable to the computer. When the analytical solutions are used, it is well to plot the lateral earth forces obtained to check their reasonableness.

Bishop and Morgenstern method is based on the Bishop's method of slices. In Bishop and Morgenstern method it is assumed that the oblique side forces on each slice exist and the pore water pressure is considered by means of pore pressure ratio. The pore pressure ratio is assumed to be constant throughout the cross section. By this method the factor of safety of an embankment is determined from charts using soil parameters. The averaging technique for estimating the factor of safety which, in an extreme case will be on the order of 7 percent, (Winterkorn and Fang,1975).

In the Spencer method it is assumed that the inter slice forces are parallel and a cylindrical slip circle is consider. The analysis is in terms of effective stress and satisfy two equations of equalibrium. By this method the factor of safety of an embankment is determined from charts using soil parameters.

In the Hunter and Schuster method it is assumed that the potential sliding surface is a circular arc and the soil is a

normally consolided unfissured clay when $\emptyset=0^{o}$ and the cohesion C varies with depth, (Winterkorn and Fang,1975). By the method the factor of safety of an embankment is determined from charts using soil parameters, depth factor and water table ratio.

In the Morgenstern method it is assumed that the oblique side forces on each slice are consider, no dissipation of the pore water pressure occurs during drawdown. It is also assumed that the embankment slope is simple slope of homogeneous material resting on a rigid impermeable layer at the toe of the slope and the pore pressure parameter B is to be unity during drawdown and the unit weight of the soil is consider at twice the unit weight of water. By the method the factor of safety of an enbankment is determined from charts using soil parameters and drawdown ratio.

3.4 SEEPAGE ANALYSES

Seepage quantity and uplift forces are the major considerations in the design of an earth embankments.

3.4.1 SEEPAGE QUANTITY

The method involves drawing a flow net for the embankment under the design flood condition and taking values of N_f and N_d,the number of flow channels and the number of potential drops

respectively, from the flow net. The seepage through a pervious soil material for two dimensional flow is given by Laplacian equation as

$$\frac{\partial^2 \emptyset}{\partial x^2} + \frac{\partial^2 \emptyset}{\partial y^2} = 0 \qquad (3.16)$$

where, $\emptyset$ = k.h = velocity potential

k = permeability of the soil

h = head causing flow.

Garg (1976) suggested a graphical solution of the above equation considering some assumptions. The solution consists of two sets of curves as equipotential lines and stream lines or flow lines, mutually perpendicular to each other, Fig.3.6 . It is assumed that the coefficient of permeability of the foundation materials and the embankment materials are the same for drawing the flow net.

Seepage quantity, q is given by

$$q = K\ h\ \frac{N_f}{N_d} \qquad (3.17)$$

where,

K = coefficient of permeability

h = total head loss

$= h_2 - h_1$

h_2=height of water level at riverside

h_1=height of water level at countryside

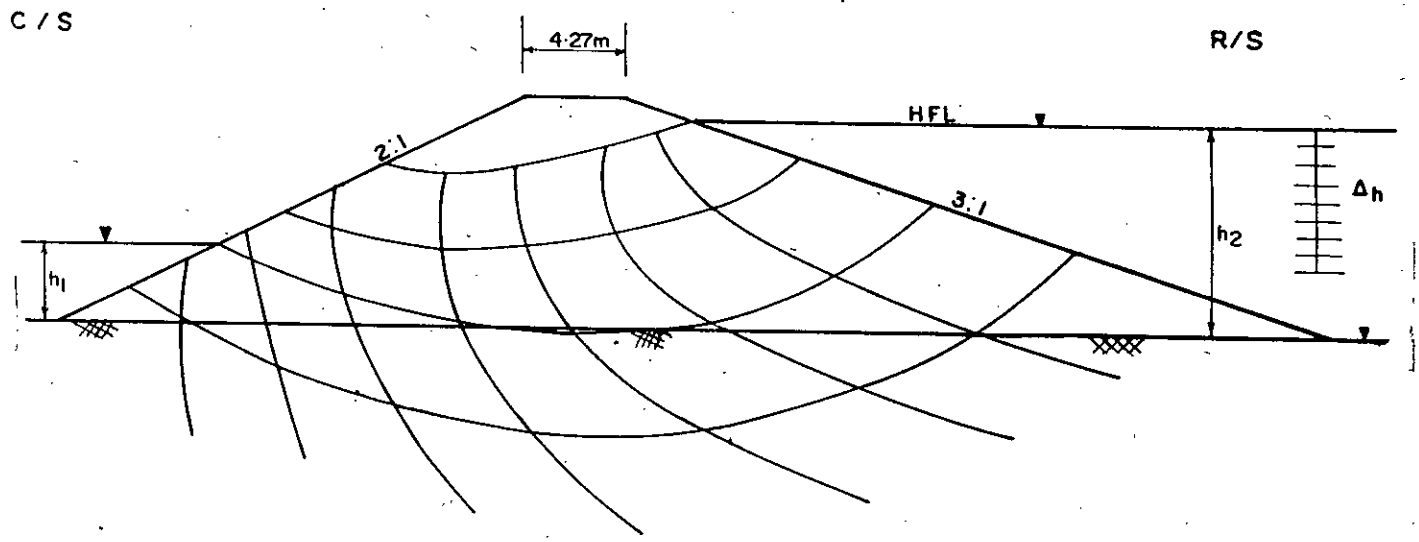


Fig.3.6 Seepage Analysis through an Embankment

N_f= No. of significant flow channels

N_d= No. of potential drops

The limiting value of q should not exceed 1 m^3/day/lin m of embankment. If the limiting value is exceeded in any case , a filter may be provided to protect the discharge face, (BWDB,1987A).

3.4.2 UPLIFT STABLITY

The factor of safety against uplift stability is conventionally calculated by using the following relation,

$$\text{Factor of safety, } F.S = \frac{W}{F_u} < 1.5 \qquad (3.18)$$

where,

W = Buoyant weight of soil

$= \gamma_{sat.} - \gamma_w$

$\gamma_{sat.}$ = saturated unit weight of soil

γ_w = submerged unit weight of soil

F_u = Maximum uplift force

$= \gamma_w i_{max} V$

γ_w = submerged unit weight of soil

i_{max} = maximum hydraulic gradient

$= \tan\beta$

β = side slope of the embankment

V = Volume of the soil = 1 m^3

BWDB(1987A) uses a factor of safety of grater than 1.5 against uplift stability.

3.5 ANALYSIS OF PIPING FAILURE

Safiullah (1982) reported that for the performance of DND project, a possible reason for piping in embankments is differential settlement cracks but there was no surface evidence of any such movement although the possibility of existence of internal transverse crack cannot be excluded. Non uniform compaction could create discontinuity within the embankment providing a condition for piping. This, however, was difficult to

confirm. The low permeability ($7x10^{-7}$ cm/sec) of the soil precludes deflocculation to be the cause of piping, (Arnold,1961). The possibility of existence of tunnels created by burrowing animals cannot be excluded and many have contributed to the causes of failure of earth embankments. The grain size analysis of the soil where piping occured was compared with the gradation criteria provided by Sherard et. al. (1963) for soils susceptible to piping.

CHAPTER 4

ANALYSES OF DATA AND THEIR INTERPRETATION

4.1 INTRODUCTION

The present study is mainly concerned with the causes of failure of earth embankments of Gumti Project during the flood of 1987. It is understood that the structural failure of any embankment is attributed mainly to stability, seepage or erosion and piping. The causes of failure depends on the methods of construction and the post construction maintenance. It is to be maintioned that the embankment of Gumti Project failed only at Khalilpur near Jafforgonj, Fig.1.1 . In the following sections the results of analyses of the available data on the above mentioned aspects are presented and are discussed. The soil and hydrological data of the site were collected from Bangladesh Water Development Board (BWDB), whereas the climatological data as provided by Manalo(1975) was used for the purpose. The data on construction method and maintenance were collected from Engineering and Planning Consultant, EPC(1989). Slope stability, Seepage and Piping are analysed at two locations of Jafforgonj (Khalilpur) and Daudkandi.

4.2 SOIL DATA

The physical and engineering properties of the site soils are presented in Table 4.1 . The soil of the site is predominantly silt of low plasticity mixed with low plastic clay, Fig.4.1 .

Table 4.1 Properties of soils of Gumti Embankment (After BWDB,1987B)

Name of tests/parameters	Values
Natural moisture content (%)	16-39
Liquid limit (%)	31-43
Plasticity index (%)	5-12
Wet unit weight (kN/m^3)	19.6
Dry unit weight (kN/m^3)	15.3
Sp.gravity	2.67
Unconfined compressive strength, q_u (kN/m^2)	84.2
Compression index, C_c	0.204
Cohesion, c (kN/m^2)	20
Angle of internal friction, Ø(degree)	10

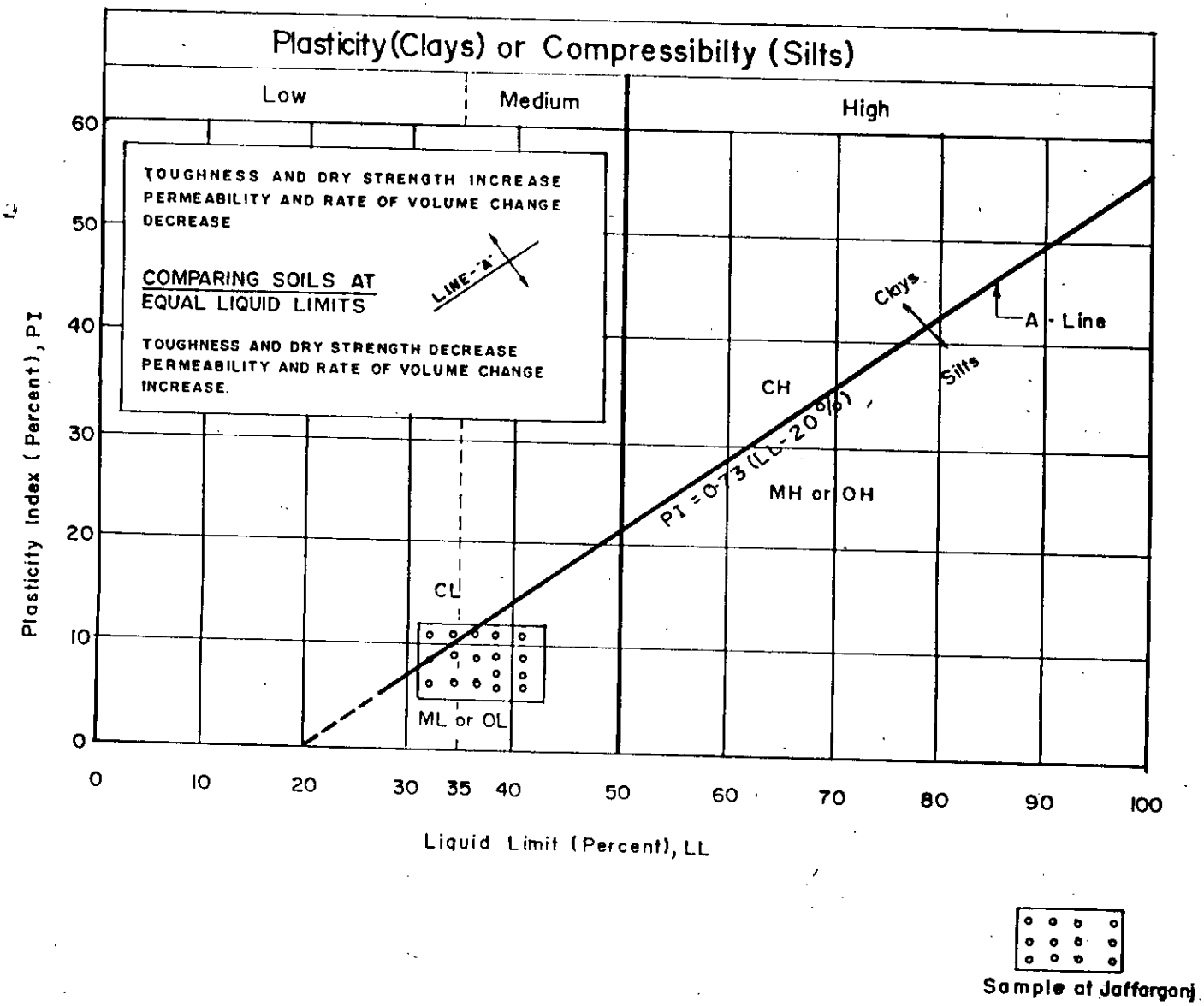


Fig. 4·1 Classifification of Gumti Embankment Soil Using Plasticity Chart.

4.3 HYDRAULIC DATA

Representative hydraulic data consisting of mean daily water level, ten daily average water level and monthly mean and extrems, annual high water level and annual low water level of the site is presented in Table 4.2. The monthly maximum, mean and minimum water levels at the site are shown in Fig.4.2. The data of Comilla is taken as the representative for the site, as there was no available gauge station at the site.

4.4 GEOMETRY OF THE EMBANKMENT

The embankment was designed and constructed by BWDB (1986). A typical section of the embankment is shown in Fig. 4.3. The crest width of the embankment were found to vary from 3.8 m to 4.5 m. The variation of the slope ranges from 1:2 to 1:4. However, in actual design an average crest width of 4.27 m and slope of 1:3 were considered.

4.5 CONSTRUCTION METHODS AND MAINTENANCE

According to Engineering and Planning Consultant (1989), Gumti embankment was constructed without proper compaction. During the construction, earth work was done by heaping soils at places without breaking clods. The flood way of 300 m were used by the people to cultivate Jute, Dhanicha etc. which obstruct the flow of the river.

Table 4.2 Mean daily water level (Gumti-Burinadi, Station Comilla).

Date	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar
1	7.704	8.010	1.812	9.898	9.116	8.410	10.448	8.226	8.106	7.976	7.822	7.748
2	7.726	8.004	8.290	9.158	9.004	8.254	10.394	8.238	8.120	7.970	7.760	7.542
3	7.716	7.990	7.916	8.794	8.884	8.276	10.682	8.216	8.106	7.980	7.732	7.632
4	7.732	8.042	7.834	8.508	8.800	8.264	10.292	8.218	6.096	8.004	7.628	7.604
5	7.744	8.132	7.780	8.584	8.780	8.824	9.948	8.212	8.088	8.008	7.802	7.548
6	7.746	8.380	7.764	8.528	8.830	8.766	9.826	8.208	8.080	7.982	7.648	7.618
7	7.788	8.788	7.716	8.606	8.848	8.470	9.858	8.224	8.072	7.976	7.718	7.698
8	7.744	8.606	7.662	8.862	8.620	8.448	10.136	8.220	8.068	7.946	7.690	7.350
9	7.792	8.366	7.362	8.664	8.476	9.464	10.036	8.516	8.062	7.942	7.652	7.532
10	7.882	8.232	8.130	8.602	8.372	9.248	10.250	10.382	8.036	7.970	7.684	7.564
11	7.912	8.140	7.742	8.464	8.260	9.120	10.028	9.714	8.044	7.954	7.632	7.924
12	7.880	8.026	7.694	8.428	8.294	9.052	9.714	9.330	8.044	7.936	7.662	7.774
13	7.856	8.044	7.824	8.316	8.610	8.880	9.458	9.064	7.042	7.934	7.652	8.020
14	7.856	7.944	7.808	8.916	8.968	8.728	9.284	8.900	8.042	7.936	7.608	8.510
15	7.834	7.980	7.796	8.092	9.018	8.718	9.140	8.764	8.042	7.952	7.606	8.010
16	7.816	8.268	7.792	8.024	8.814	8.566	9.146	8.638	8.052	7.922	7.564	7.866
17	7.772	8.334	7.868	7.948	8.552	8.354	9.048	8.534	8.032	7.930	7.644	7.774
18	7.770	8.402	7.904	7.866	8.374	8.526	8.878	8.466	8.024	7.916	7.492	7.860
19	7.868	8.560	7.854	7.824	8.368	8.872	8.764	8.382	8.028	7.900	7.664	7.894
20	8.608	8.560	7.770	7.820	8.574	8.626	8.656	8.330	8.020	7.872	7.654	7.746
21	8.382	8.480	7.740	7.862	8.512	10.090	8.578	8.282	8.004	7.882	7.594	7.798
22	8.138	8.338	7.686	7.858	8.416	10.016	8.496	8.282	7.996	7.884	7.632	7.688
23	8.070	8.308	7.694	8.340	8.464	9.574	8.434	8.248	8.000	7.898	7.666	7.750
24	7.930	8.132	7.682	10.034	8.582	9.326	8.378	8.228	7.990	7.892	7.528	7.880
25	8.056	8.048	7.652	10.010	8.526	9.058	8.336	8.204	7.994	7.884	7.530	7.700
26	8.388	7.714	7.650	10.692	8.538	8.888	8.304	8.192	7.982	7.828	7.506	7.656
27	8.258	7.834	7.642	10.572	8.720	8.994	8.278	8.168	7.984	7.572	7.626	7.528
28	8.128	8.160	7.666	10.254	8.478	10.228	8.252	8.166	7.980	7.354	7.574	7.344
29	8.030	7.914	7.684	10.064	8.470	10.000	8.242	8.146	7.994	7.770	-	7.770
30	7.942	7.616	8.710	9.660	8.368	10.044	8.240	8.130	7.986	7.802	-	7.798
31	-	7.860	-	9.340	8.428	-	8.222	-	7.990	7.890	-	7.614
						Ten Daily Average						
Av1	7.757	8.255	7.827	8.820	8.773	8.642	10.187	8.466	8.083	7.975	7.714	7.584
Av2	7.917	8.226	7.805	8.098	8.583	8.744	9.212	8.812	8.037	7.925	7.618	7.938
Av3	8.132	8.037	7.781	9.517	8.500	9.622	8.342	8.205	7.991	7.787	7.582	7.684
						Monthly Means & Extremes						
Max	8.620	8.800	10.240	10.780	9.160	10.510	10.720	10.460	8.130	8.010	7.950	8.570
Mean	7.936	8.168	7.804	8.834	8.615	9.003	9.218	8.494	8.036	7.892	7.642	7.734
Min	7.700	7.510	7.340	7.810	8.150	8.180	8.210	8.120	7.960	7.320	7.320	7.260

Annual HWL : 10.780 Date : 26/7/1986 Annual LWL : 7.260 Date : 8/3/1987

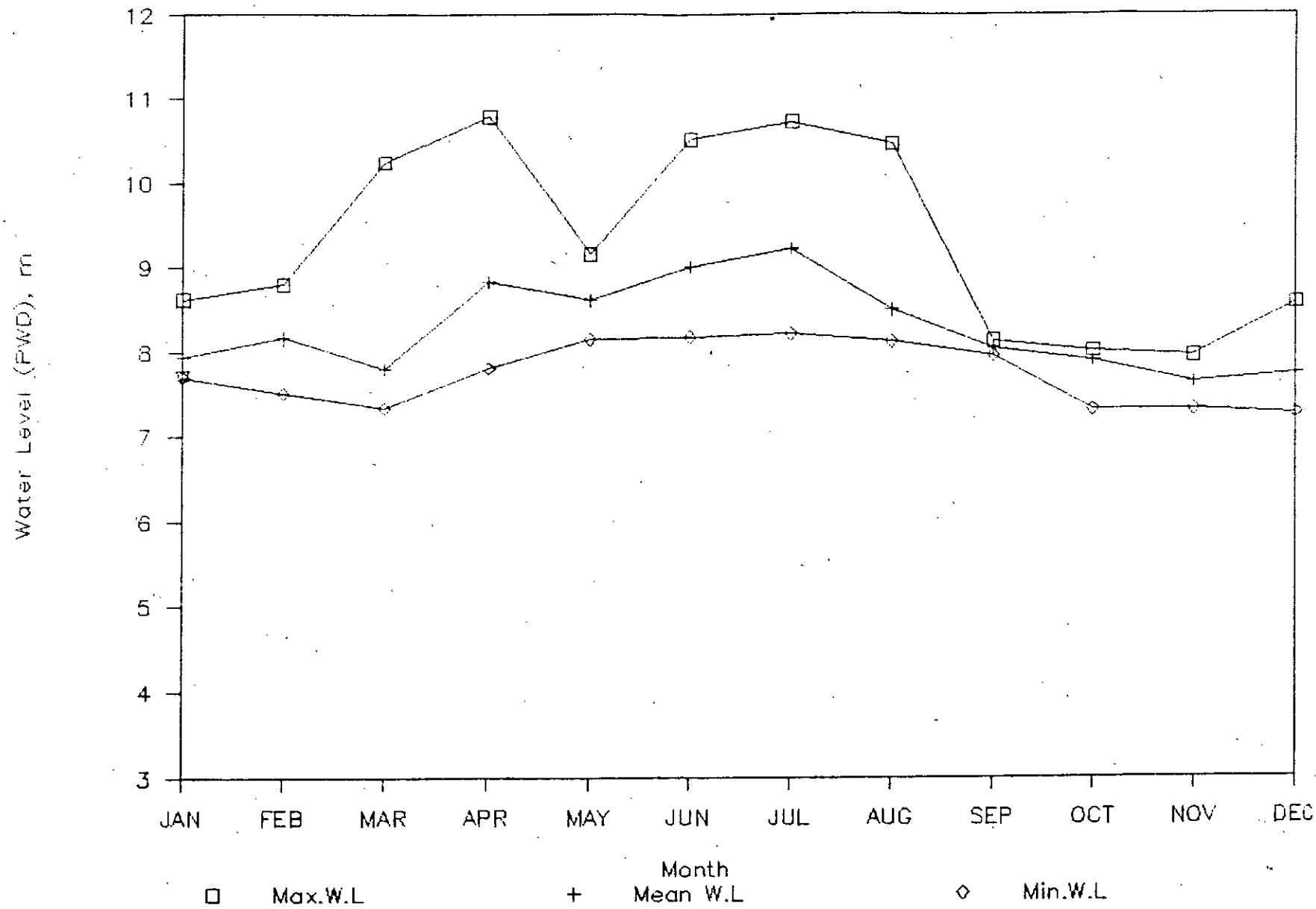
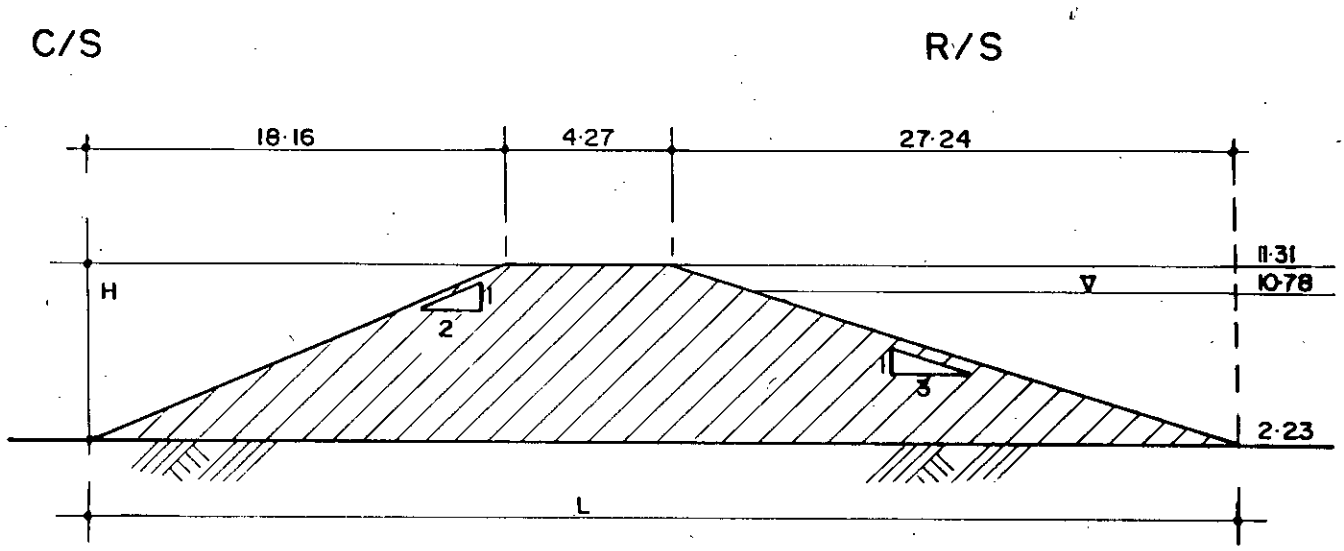


Fig.4.2 Water Level at Jafforgonj



H = Embankment Height
L = 49·67

All Dimensions in Metres

Fig. 4·3 Typical Embankment Section of Gumti Project.

4.6 SLOPE STABILITY ANALYSES

Slope stability analyses of the embankment were done using various methods like Swedish slip circle method, Taylor's method, Bishop's method and Bishop's Simplified method. A computer programme for Bishop's Simplified method was developed and used to compute the factor of safety. The programme was originated by Provis (1989) and modified for the present use. The programme listing and the programme documentation are presented in Appendix-B and Appendix-C respectably. For the other methods, manual computations were used. The sample calculations are provided in the Appendix-D. The factor of safety as obtained by using different methods are presented in Table 4.3 . Table 4.3 shows that the factor of safety of the slopes against sliding is always greater than unity except for the Bishop's method, which gives a factor of safety of approximately one. This dictates that the failure of the embankment was not initiated due to the stability of the slopes.

4.7 SEEPAGE ANALYSES

4.7.1 ANALYSIS OF SEEPAGE QUANTITY

The quantity of seepage through the embankment was estimated using the graphical solution of Laplacian two dimensional equation. The quantity of seepage through the embankment as obtainted are presented in Table 4.4. From Table 4.4, it appears that the quantity of seepage at Khalilpur near Jafforgonj is

0.9914 m^3/day/m, which is very close to allowable limit of 1.0 m^3/day/m as suggested by BWDB(1987A). From the quantity of seepage it can be concluded that the embankment section was not stable at Khalilpur near Jafforgonj. The sample calculations of quantity of seepage are presented in the Appendix-D.

4.7.2 UPLIFT ANALYSIS

Uplift analysis of the embankment was done using the conventional method. Sample calculation is shown in Appendix-D. The factor of safety as obtained by this method is presented in Table 4.5. From Table.4.5 it is found that the embankment section was stable due to uplift force, the factor of safety being grater than 1.5. The factor of safety for uplift are the same both for Jafforgonj and Daudkandi.

4.8 PIPING ANALYSIS

Piping analysis of the embankment soil was done by comparing the gradation of the soil with criteria provided by Sherard et.al.(1963) for soils susceptable to piping. The comparison is presented in Fig.4.4. From the analysis of the piping it is clear that the piping failure at the locations i.e Khalilpur near Jafforgonj and Daudkandi were occured. The comparison, Fig.4.4, shows that the soil at the site Khalilpur near Jafforgonj and Daudkandi are quite susceptable to piping. This might be a possible reason for the failure of the embankment.

Table 4.3 Factor of safety (F.S) against slope stability at different locations of Gumti Project.

Sl. No.	Name of the method	Soil properties at Jafforgonj			Soil properties at Daudkandi			F.S at Jafforginj	F.S at Daudkandi
		C (C′) kN/m^2	Ø (Ø′) o	$\gamma_{sat.}$ kN/m^3	C (C′) kN/m^2	Ø (Ø′) o	$\gamma_{sat.}$ kN/m^3		
1	Swedish slip circle method	20	10	19.6	15	25	19.1	1.86	1.79
2	Taylor′s method	20	10	19.6	15	25	19.1	1.70	1.99
3	Bishop′s method	20	10	19.6	15	25	19.1	0.86	1.00
4	Bishop′s simplified method by computer programme	20	10	19.6	15	25	19.1	1.44	1.76

Table 4.4 Quantity of seepage at different locations of Gumti Project.

Sl. No.	Location	No.of significant flow line(N_f)	No.of Potential drops(N_d)	Total head loss(h) m	Permiability K Cm/sec	Allowable quantity of seepage(After BWDB,1987A) m^3/day/m	Quantity of Seepage m^3/day/m
1	Jafforganj	3	8	6.12	$5x10^{-4}$	1.0	0.9914
2	Daudkandi	3	8	6.03	$5x10^{-4}$	1.0	0.9768

Table 4.5 Factor of safety (F.S) against uplift stability at different locations of Gumti project.

No. Sl.	Location	Countryside embankment slope (β)	Maximum upward hydraulic gradient i_{max}= tanβ	Maximum uplift force $F_u = \gamma_w i_{max} V$ (where V= 1 m^3) kN	Buoyant weight of siol $W = \gamma_{sat} - \gamma_w$ kN	F.S= $\frac{W}{F_u}$
1	Jafforgonj	1:2	0.5	4.9	9.8	2
2	Daudkandi	1:2	0.5	4.9	9.8	2

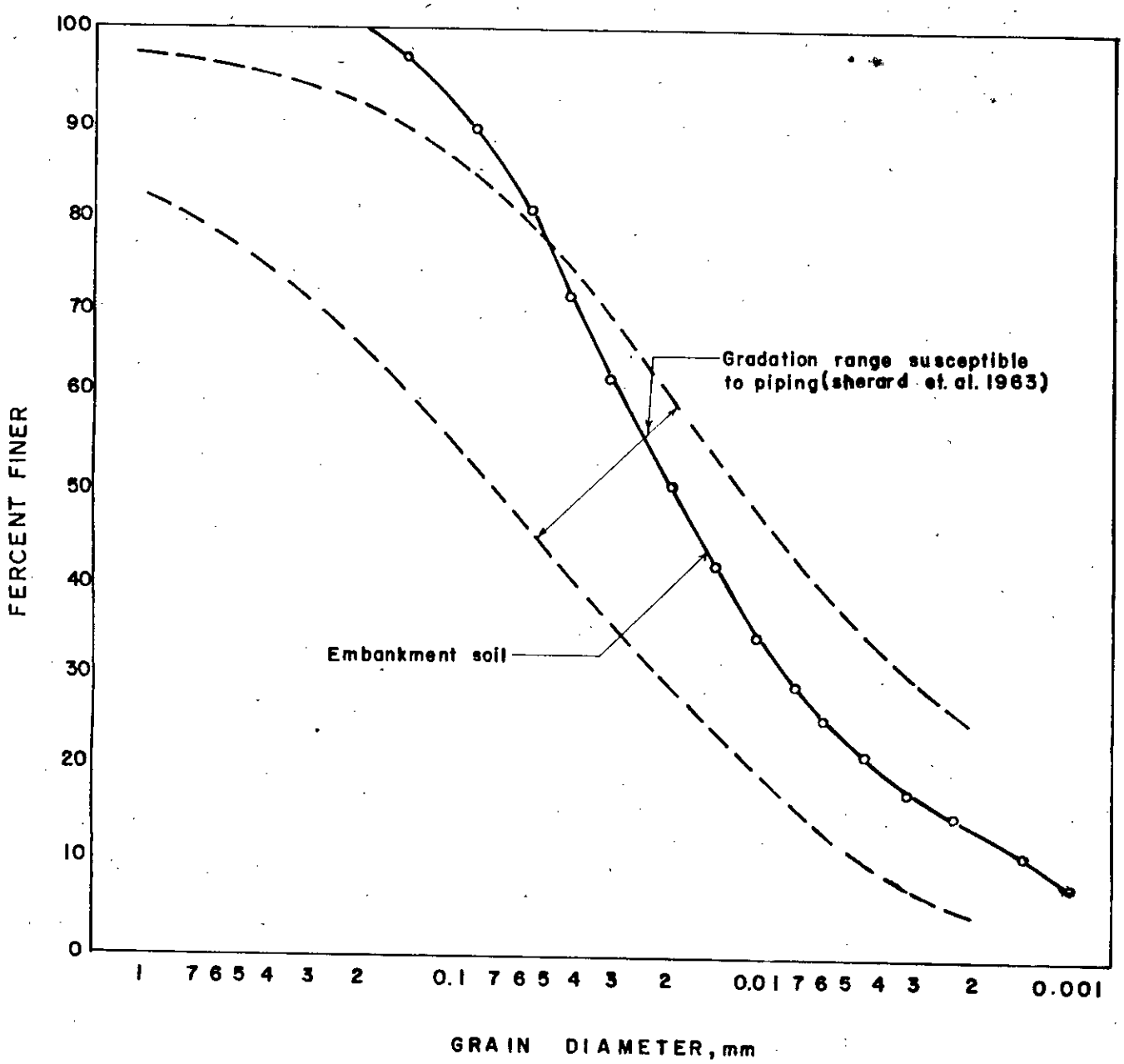


Fig. 4.4 Gradation Range of Soils Susceptable to Piping (After Sherard et. al; 1963)

CHAPTER 5

CONCLUSIONS

5.1 CONCLUSIONS

On the basis of the study related to the causes of failure of earthen embankment of Gumti Project, the following conclusions can be drawn :

1. Progressive erosion from near the countryside toe of the embankment might occur at the location as the soil cohesion was low and can be attributed to high seepage pressure at the exit point.

2. Failure occured at the location where the soil has low cohesion and proper filter was not provided.

3. Sliding failure of the embankment, occured at new portion of the embankment (Khalilpur), can be attributed to high clay content of the soil resulting in shrinkage cracks responsible for such movements.

4. Gumti basin is a hilly river basin, but the river is passing over plain land having slope 0.12 m/Km . Hence, flashy flood is occuring. This may be a cause of failure of Gumti Embankment..

5. Due to the meandering of Gumti river, direct attack of flow occurs on the bank of the river, consequently erodes the bank and thus causing the failure of embankment.

6. Gumti Embankment was constructed without any compaction and voids were present within the embankment. This might cause piping. Moreover, tunnels might be created by burrowing animals which accelerates the process.

7. Inside the flood way of 300 m of Gumti river people cultivating Jute,Dhanicha etc. which obstructs the flow of the Gumti river causing rise in water level. The actual level might be higher than that recorded at the gauge station. This could be a cause of difference of predicated and actual thrust on the embankment.

It is understood from the above that in embankment soils with silts of low plasticity (ML) mixed with low plastic clay (CL), (Fig.4.1), shrinkage cracks on the slope surface are possible and may create stability problems. Any stability analysis on such soils should consider the nature of the cracks without which the factor of safety calculated may be in error.

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APPENDICES

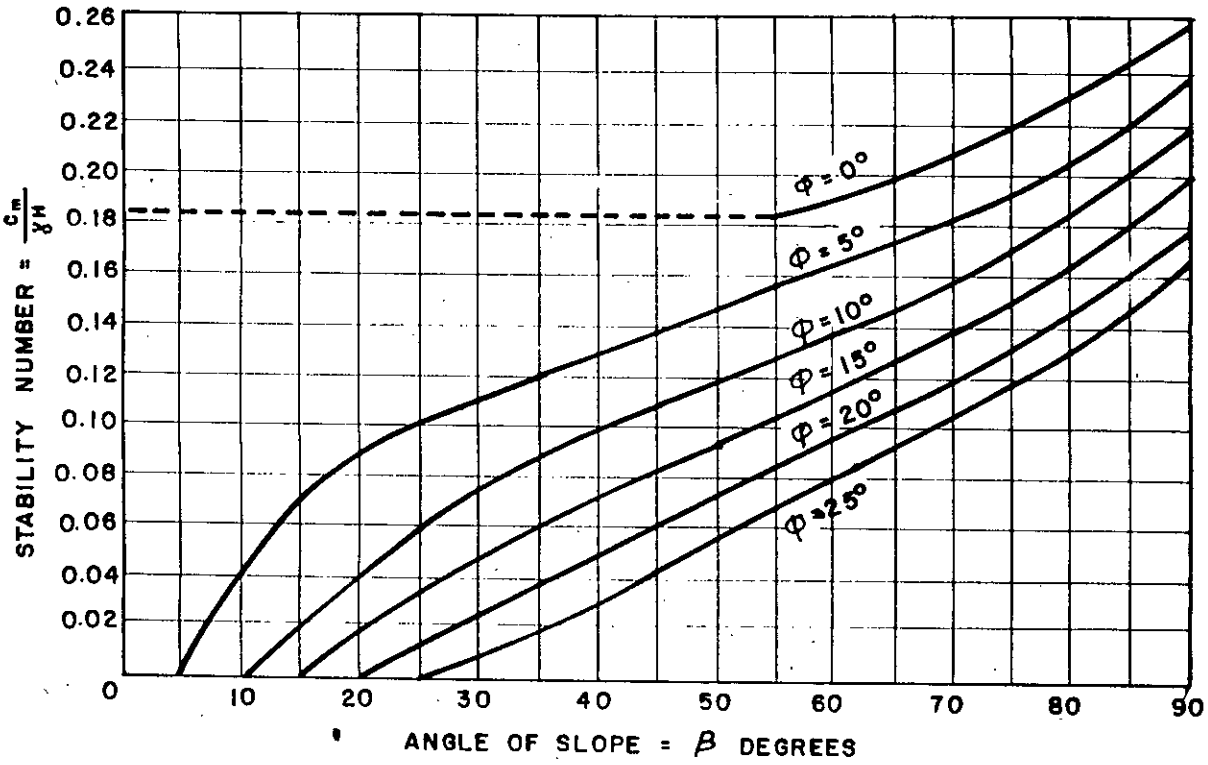

Fig. A-1 Taylor's curves for total stress analysis (for ϕ = 0° and β < 53°.

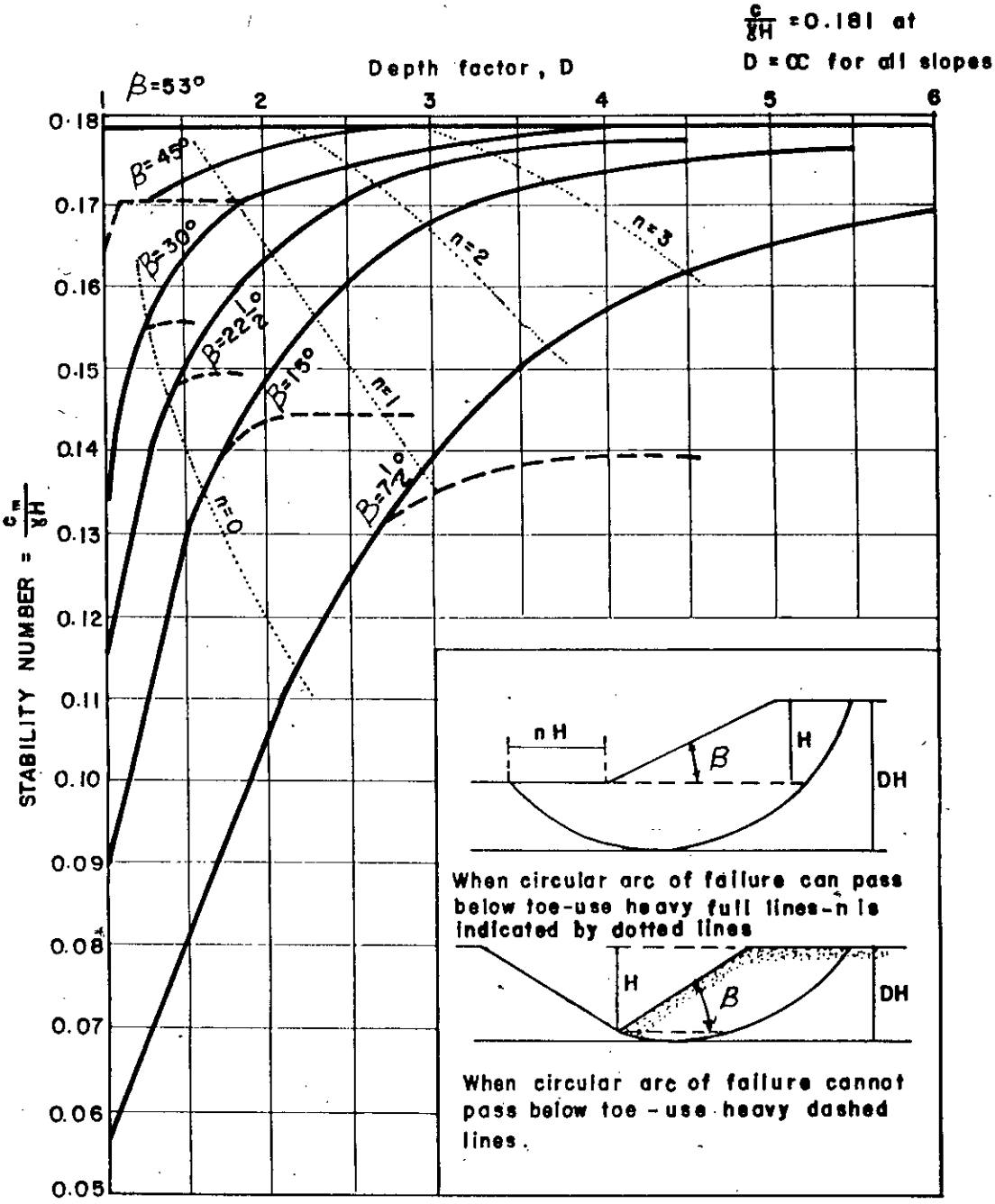

Fig. A·2 Effect of depth limitation on Taylor's curves (for β>53°,

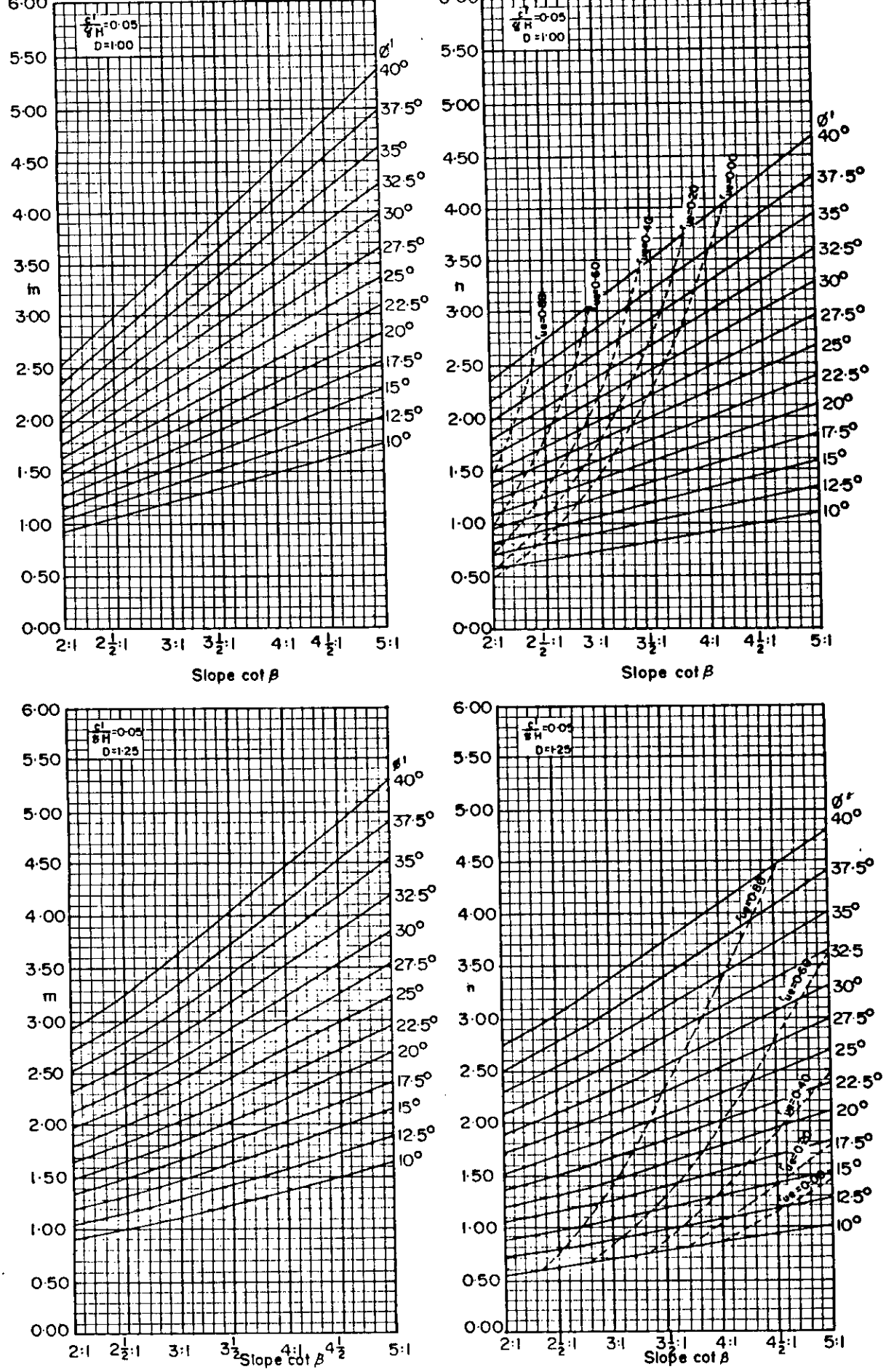

Fig. A·3 Stability Coefficients for Earth Slopes by A.W. Bishop and Norbert Morgenstern

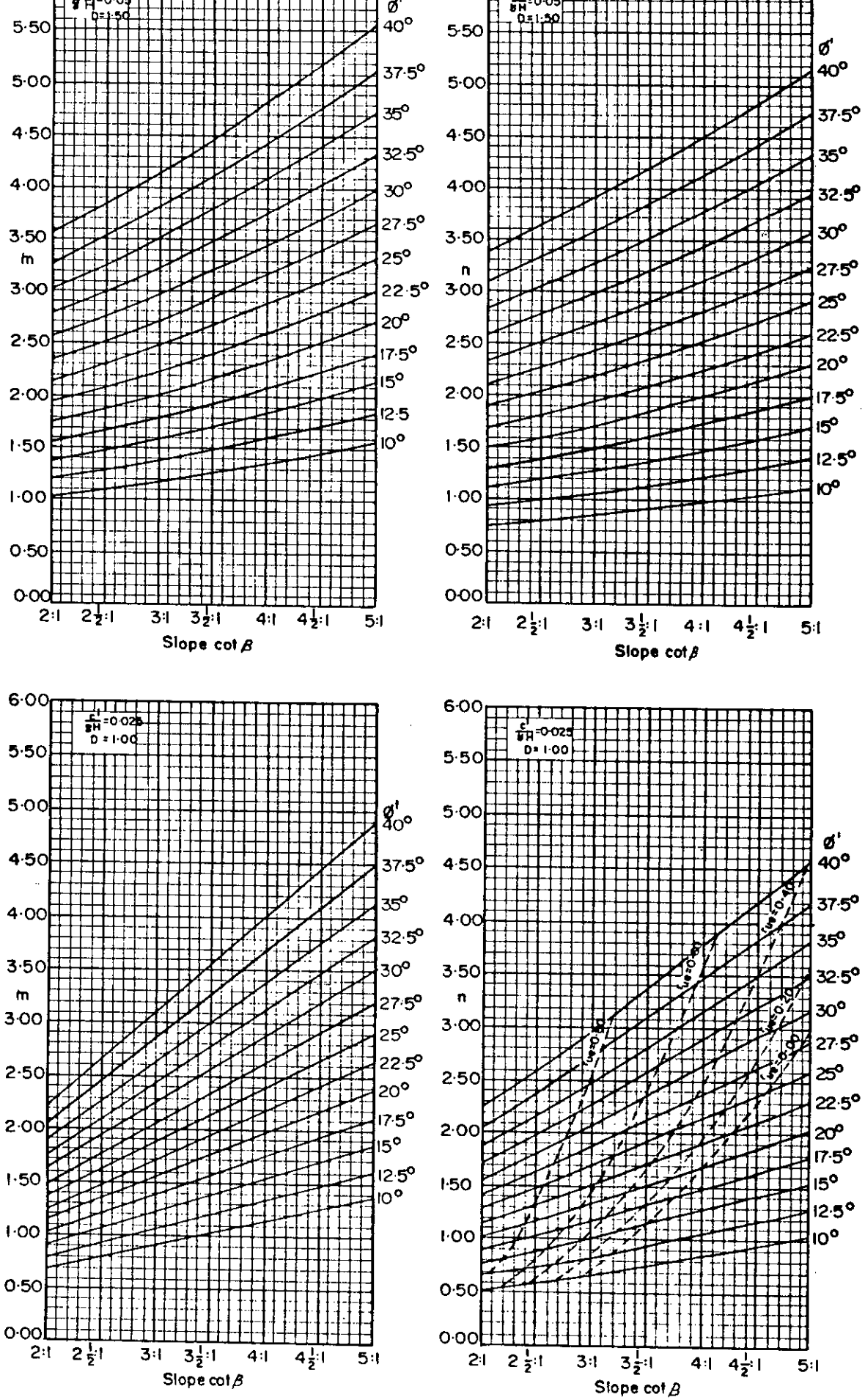

Fig. A·4 Stability Coefficients for Earth Slopes by A.W. Bishop and Norbert Morgenstern

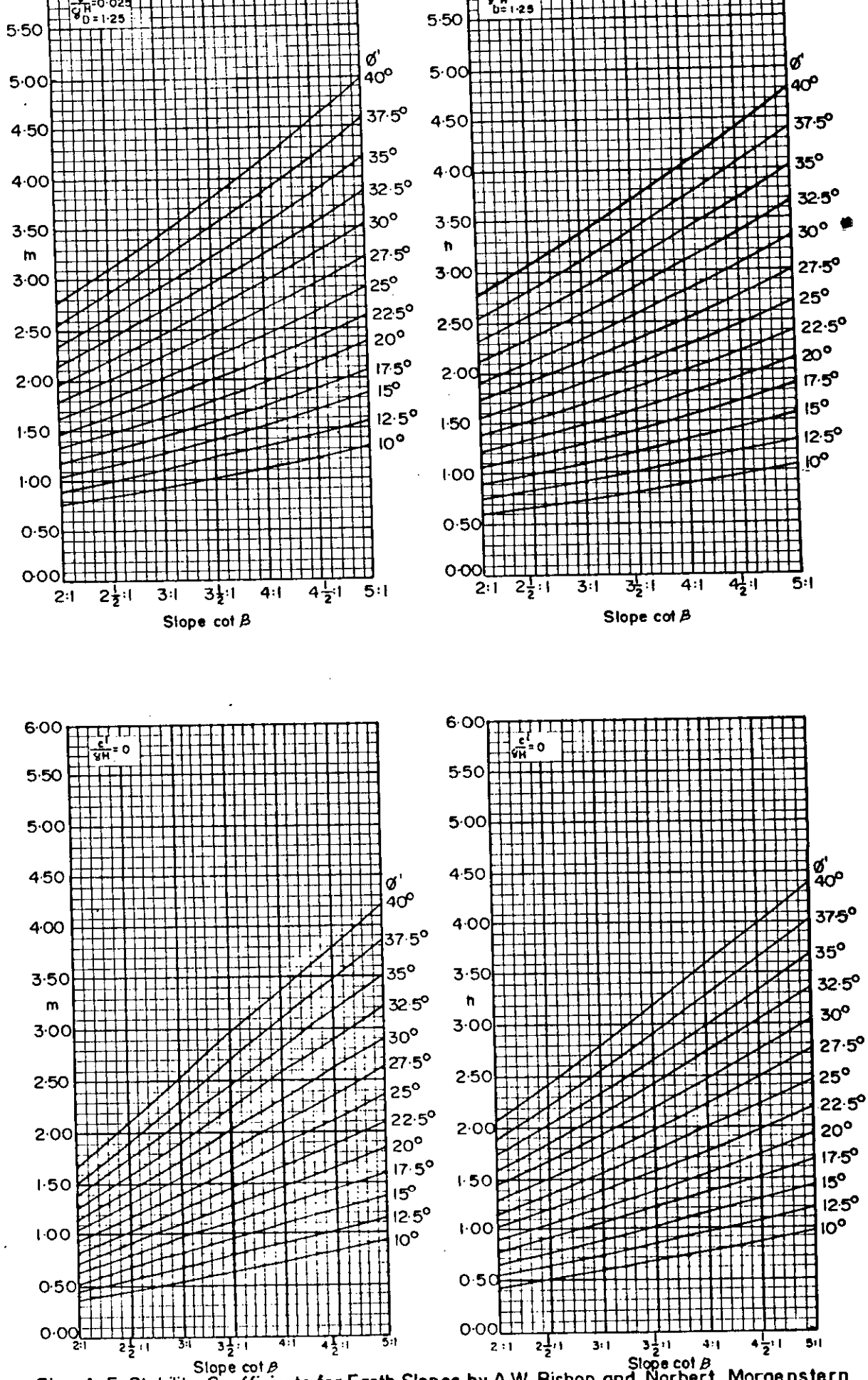

Fig. A·5 Stability Coefficients for Earth Slopes by A.W. Bishop and Norbert Morgenstern

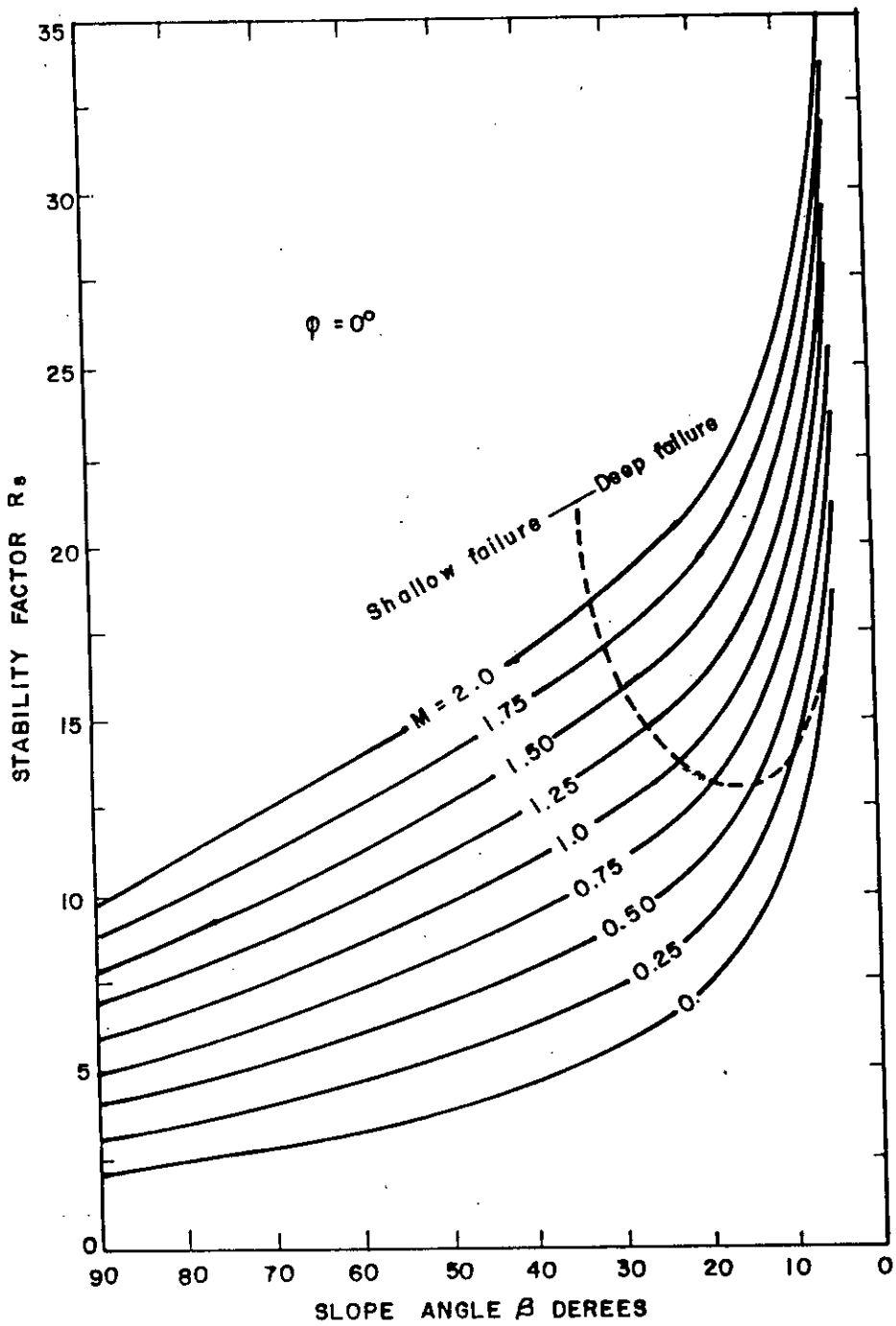

Fig.A-6 Relationship between slope angle β and stability factor R_s for various water table ratios M. (After Hunter and Schuster, 1968)

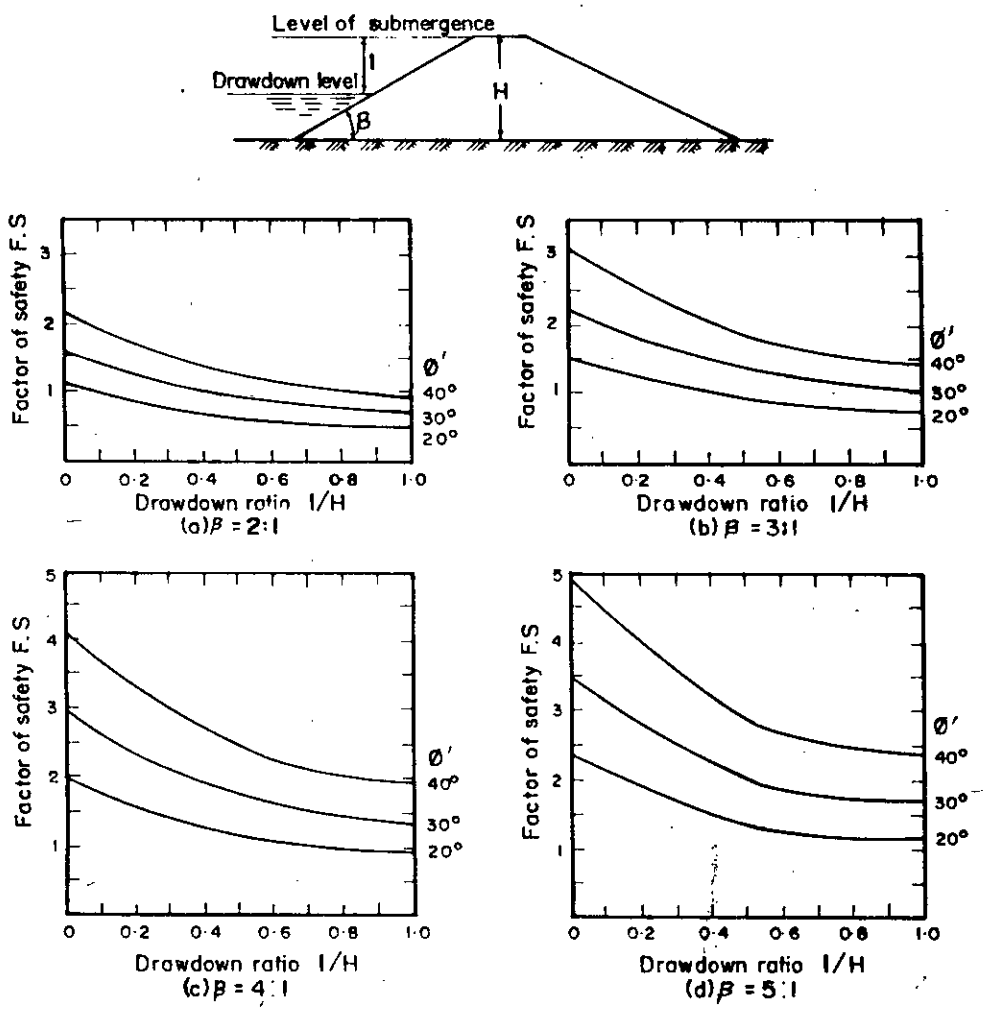

Fig. A·7 Drawdown stability chart for $c'/\gamma H = 0{\cdot}0125$ (After Morgenstern, 1963)

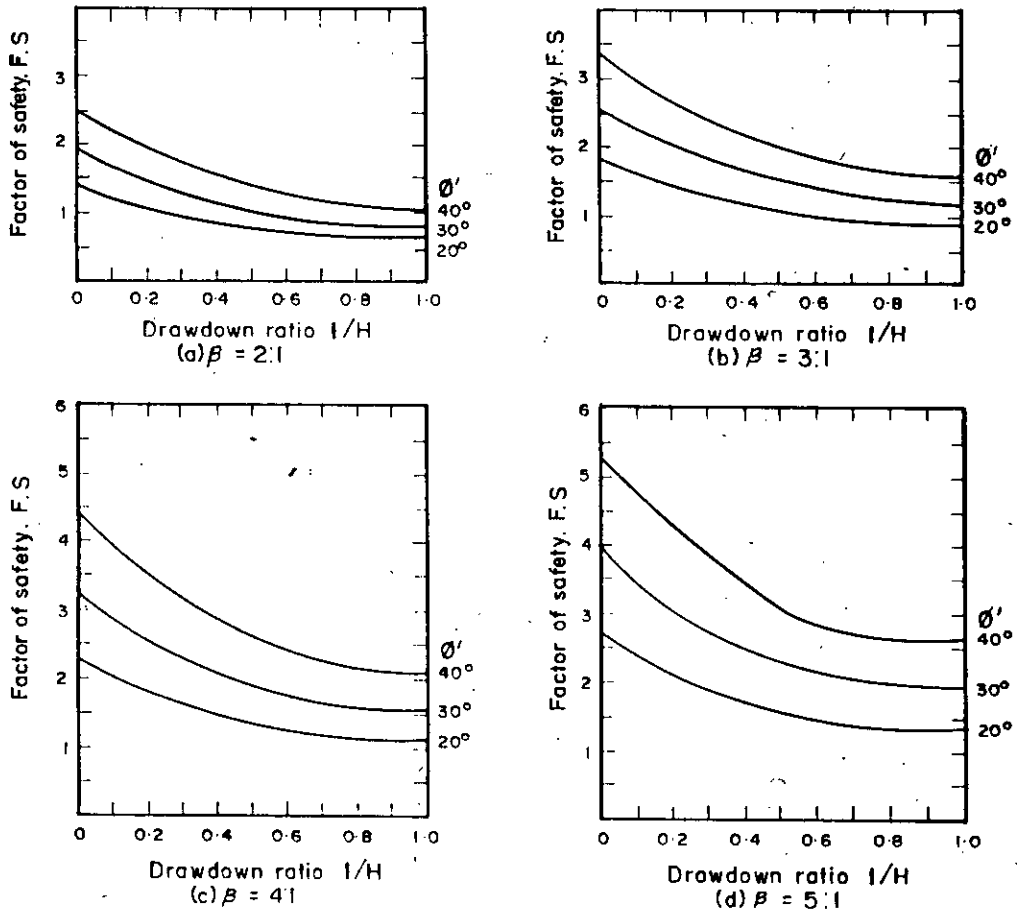

Fig A·8 Drawdown stability chart $c'/\gamma H = 0{\cdot}025$. (After Morgenstern, 1963)

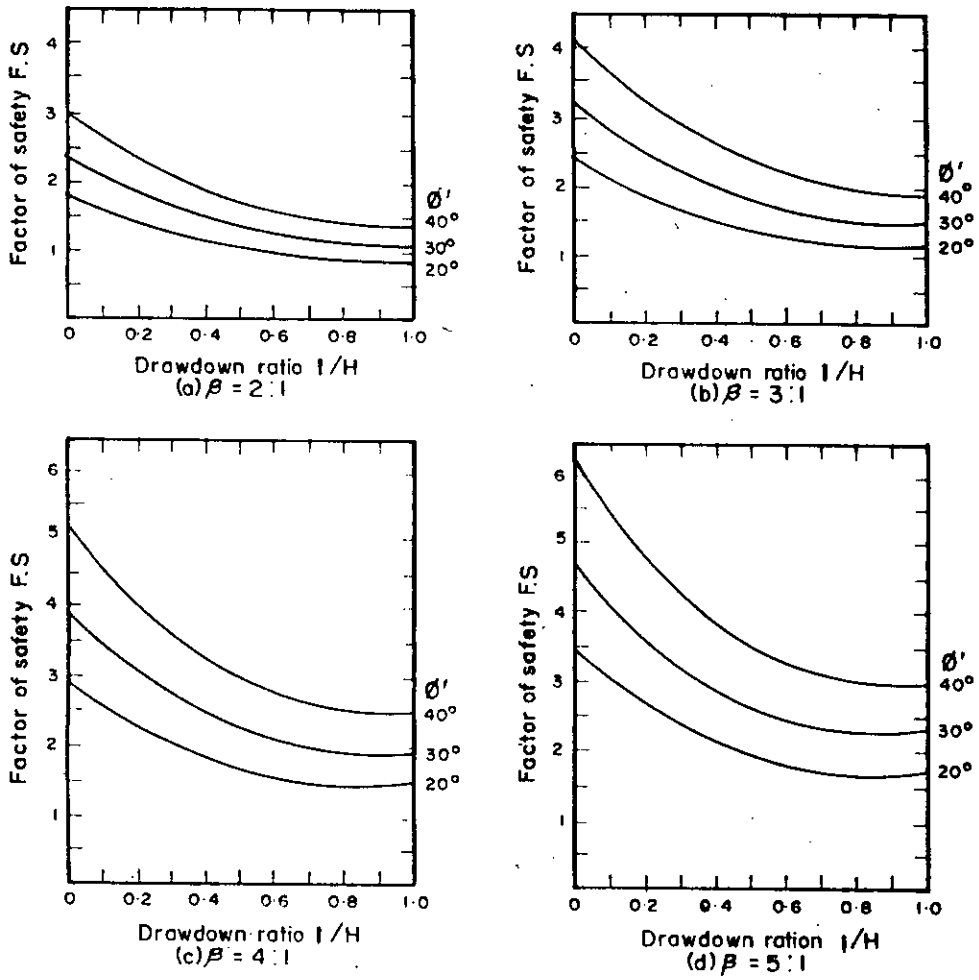

Fig. A·9 Drawdown stability chart for $c'/\gamma H = 0{\cdot}05$. (After Morgenstern, 1963)

PROGRAMME LISTING

```
10 REM ***BISHOP'S SIMPLIFIED METHOD***
20 REM ***FOR FACTOR OF SAFETY AGAINST
30 REM *** EMBANKMENT COLLAPSE BY
40 REM *** SLIPPING
50 DATA 5 :REM NR SLICES
60 DATA 0,0,0,0,0,0  :REM WATER LEVELS
70 DATA 20,20,20,20,20 : REM COHESION
80 DATA 10,10,10,10,10 :  REM ANGLE FRICTION
90 DATA 19.6,19.6,19.6,19.6,19.6  : REM UNIT SAT SOIL WT
100 DATA 6 :REM WR OF EMBANKMENT COORDS
110 REM * EMBANKMENT COORDS *
120 DATA 3.33,10.65,6.2,10.65,13.33,7.05,18.66,4.5,24.66,1.5,28.66,1.5
130 REM DATA FOR YT,YB & X
140 DATA "Y" :  REM  SLICE DATA REQUIRED? (Y/N)
150 DATA 10.65,10.65,3.33,10.65,7.65,6.2,7.05,2.5,13.33,4.5,0.45,18.66
160 DATA 1.5,0.10,24.66,1.5,1.5,28.66
180 DIM A(20),C(20),L(20),P(20),U(20),W(20)
190 PI=3.14159265359#
200 READ NS
210 N2=NS+1
220 FOR I=0 TO NS
230 READ YW(I)
240 NEXT I
250 FOR I=1 TO NS
260 READ C(I)
270 NEXT I
280 FOR I=1 TO NS
290 READ PD(I)
300 P(I)=PD(I)*PI/180
310 NEXT I
320 FOR I=1 TO NS
330 READ GA(I)
340 NEXT I
350 READ P1
360 P2=P1-1
370 FOR I=0 TO P2
380 READ EE(I),EN(I)
390 NEXT I
400 READ Y$
410 IF Y$="N" THEN GOTO 460
420 FOR I=0 TO NS
430 READ YT(I),YB(I),X(I)
440 NEXT I
450 GOTO 650
460 PRINT "    DO YOU WANT A GRID -G"
470 PRINT  "              OR A FRAME -F"
480 PRINT "OR TO INPUT COORDS"
490 PRINT "AND RADIUS INDIVIDUALLY-I"
500 INPUT G$
510 IF G$<>"G" THEN GOTO 540
520 PRINT "ENTER CENTRAL COORDS, RADIUS AND RANGE (EO, NO,R,RA)"
530 INPUT EO,NO,R,RA
540 GG=0
550 IF G$="I" THEN INPUT "ENTER COORDS AND RADIUS (EO,NO,R)";EO,NO,R
560 IF G$<>"F" THEN GOTO 650
570 PRINT "ENTER LIMITS OF EAST COORDS (E MIN,E, MAX)
580 INPUT F1,F2
590 PRINT "ENTER LIMITS OF NORTH COORDS (N MIN, N MAX)
```

```
600 INPUT F3,F4
610 INPUT "ENTER STEP";F5
620 INPUT "ENTER RADIUS";R
630 EO=F1
640 NO=F3
650 PRINT "DO YOU WANT TO PRINT SLICE DATA?"
660 PRINT "ALL SLICE DATA =A"
670 PRINT "SUMMARY ONLY   =S"
680 PRINT "NO PRINTOUT    =N"
690 INPUT P$
700 CLS
710 PRINT "       BISHOP'S SIMPLIFIED METHOD"
720 PRINT "       ========================":PRINT""
730 PRINT " EAST    NORTH   RADIUS      F.O.S"
740 PRINT " "
750 IF P$="N" THEN GOTO 820
770 LPRINT CHR$(14)"BISHOP'S SIMPLIFIED METHOD"
780 LPRINT CHR$(14)"========================"
800 IF P$="S" THEN LPRINT  " EAST    NORTH     RADIUS     F.O.S ":LPRINT,""
820 REM START GEOMETRIC CALCULATIONS
830 IF Y$="Y" THEN GOTO 1600
840 Q=0
850 Z=SQR((EE(0)-EO)^2+(EN(0)-NO)^2)
860 IF Z<R THEN Q=1
870 Z=SQR((EE(P2)-EO)^2+(EN(P2)-NO)^2)
880 IF Z<R THEN Q=1
890 IF Q=0 GOTO 920
900 PRINT "RADIUS IS TOO LARGE"
910 GOTO 1910
920 S=1
930 I=0
940 J=I+1
950 IF I>P1 THEN GOTO 1040
960 AE=EE(I)
970 AN=EN(I)
980 IF SQR((EO-AE)^2+(NO-AN)^2)<R THEN GOTO 1090
990 AF=AE: NF=AN
1000 AE=AE+S
1010 IF AE>EE(J) THEN GOTO 1070
1020 AN=(AE-EE(I))*(EN(J)-EN(I))/(EE(J)-EE(I))+EN(I)
1030 GOTO 980
1040 RR=SQR((EO-EE(0))^2+(NO-EN(0))^2)
1050 IF R<RR THEN PRINT "RADIUS IS TOO SMALL"
1060 GOTO 1910
1070 I=I+1
1080 GOTO 940
1090 IF S<.05 THEN GOTO 1130
1100 S=S/10
1110 AE=AE-9*S
1120 GOTO 980
1130 IF (SQR((AE-EO)^2+(NO-AN)^2)-R)<(SQR((AF-EO)^2+(NO-NF)^2)-R) THEN GOTO 1160
1140 AE=AF
1150 AN=NF
1160 REM
1170 S=1
1180 I=P2
1190 J=I-1
1200 BE=EE(I)
1210 BN=EN(I)
1220 IF SQR((EO-BE)^2+(NO-BN)^2)<R THEN GOTO 1300
1230 BF=BE: NF=BN
1240 BE=BE-S
1250 IF BE<EE(J) THEN GOTO 1280
1260 BN=(BE-EE(I))*(EN(J)-EN(I))/(EE(J)-EE(I))+EN(I)
1270 GOTO 1220
```

```
1280 I=I-1
1290 GOTO 1190
1300 IF S<.05 THEN GOTO 1340
1310 S=S/10
1320 BE=BE+9*S
1330 GOTO 1220
1340 IF(SQR((BE-EO)^2+(NO-BN)^2)-R)<(SQR((BF-EO)^2+(NO-NF)^2-R)) THEN GOTO 1370
1350 BE=BF
1360 BN=NF
1370 REM
1380 W=(BE-AE)/NS
1390 N2=NS+1
1400 FOR I=0 TO NS
1410 X(I)=AE+I*W
1420 NEXT I
1430 YT(0)=AN
1440 YT(NS)=BN
1450 FOR I=1 TO NS-1
1460 FOR J=0 TO P2
1470 K=J+1
1480 IF X(I)>EE(K) THEN GOTO 1510
1490 YT(I)=EN(J)+(X(I)-EE(J))*(EN(K)-EN(J))/(EE(K)-EE(J))
1500 GOTO 1520
1510 NEXT J
1520 NEXT I
1530 YB(0)=AN
1540 YB(NS)=BN
1550 FOR I=1 TO NS-1
1560 X=EO-X(I)
1570 Y=SQR(R*R-X*X)
1580 YB(I)=NO-Y
1590 NEXT I
1600 IF P$="A" THEN GOSUB 2190
1610 FOR I=1 TO NS
1620 A(I)=ATN((YB(I)-YB(I-1))/(X(I-1)-X(I)))
1630 L(I)=SQR(((YB(I)-YB(I-1))^2)+((X(I)-X(I-1))^2))
1640 U(I)=9.810001*.5*(YW(I)+YW(I-1)-YB(I)-YB(I-1))
1650 IF U(I)<0 THEN U(I)=0
1660 W(I)=GA(I)*(X(I)-X(I-1))*.5*(YT(I)+YT(I-1)-YB(I)-YB(I-1))
1670 NEXT I
1680 REM
1690 REM CALCULATE FACTOR OF SAFETY, INITIAL GUESS = 1
1700 REM
1710 F=1!
1720 T=0
1730 B=0
1740 FOR I=1 TO NS
1750 G=1 + TAN(A(I))*TAN(P(I))/F
1760 T=T+(C(I)*L(I)+TAN(P(I))*(W(I)*COS(A(I))-U(I)*L(I)))/G
1770 B=B+W(I)*SIN(A(I))
1780 NEXT I
1790 FL=F
1800 F=T/B
1810 IF ABS(FL/F-1)>.0001 GOTO 1720
1820 IF Y$="N" THEN PRINT EO;TAB(9)NO;TAB(18);R;TAB(26)F
1830 IF Y$="Y" THEN PRINT TAB(26);F
1840 IF P$="N" THEN GOTO 1910
1860 IF P$="S" THEN GOTO 1890
1870 IF Y$="N" THEN LPRINT " EAST     NORTH     RADIUS     F.O.S"
1880 IF Y$="Y" THEN LPRINT "FACTOR OF SAFETY=";F
1890 IF Y$="N" THEN LPRINT EO;TAB(12)NO;TAB(23);R;TAB(32)F
1910 REM * ASK FOR NEW RADIUS & COORDS
1920 IF Y$="Y" THEN GOTO 2310
1930 IF G$="I" THEN GOTO 2380
1940 IF G$="F" THEN GOTO 2420
1950 IF G$="G" THEN GOTO 2000
```

```
1960 INPUT "ANOTHER GRID";Y1$
1970 IF Y1$="Y" THEN GOTO 520
1980 IF Y1$="N" THEN GOTO 2500
2000 REM NEW GRID POINT
2010 GG=GG+1
2020 ON GG GOTO 2030,2050,2070,2090,2110,2130,2150,2170,1960
2030 EO=EO-RA
2040 GOTO 820
2050 NO=NO-RA
2060 GOTO 820
2070 EO=EO+RA
2080 GOTO 820
2090 EO=EO+RA
2100 GOTO 820
2110 NO=NO+RA
2120 GOTO 820
2130 NO=NO+RA
2140 GOTO 820
2150 EO=EO-RA
2160 GOTO 820
2170 EO=EO-RA
2180 GOTO 820
2190 REM
2200 LPRINT"":LPRINT""
2210 LPRINT  "COH        PHI    GAMMA    X        YT        YB            YB";
2220 LPRINT"         EE        EN"
2230 FOR I=0 TO NS
2240 LPRINT C(I);TAB(10)PD(I);TAB(16)GA(I);TAB(24)X(I);
2250 LPRINT TAB(32)YT(I);TAB(42)YB(I);TAB(55)YW(I);
2260 LPRINT TAB(63)EE(I);TAB(72)EN(I)
2270 NEXT I
2280 LPRINT ""
2290 REM
2300 RETURN
2310 INPUT "DO YOU WANT ANOTHER SLIP SURFACE";Y1$
2320 IF Y1$="N" THEN GOTO 2500
2330 FOR I=0 TO NS
2340 PRINT "YB(";I;")?
2350 INPUT YB(I)
2360 NEXT I
2370 GOTO 1600
2380 REM NEW INDIVIDUAL POINT
2390 INPUT "ANOTHER POINT";Y1$
2400 IF Y1$="N" THEN GOTO 2500
2410 GOTO 550
2420 REM FRAME
2430 EO=EO+F5
2440 IF EO>F2 THEN EO=F1:  NO=NO+F5
2450  IF NO>F4 THEN GOTO 2470
2460 GOTO 820
2470 INPUT "ANOTHER FRAME (Y/N)";Y1$
2480 IF Y1$="Y" THEN GOTO 570
2500 END
```

DOCUMENTATION OF THE BISHOP'S SIMPLIFIED METHOD PROGRAMME

1. INTRODUCTION

The important parameters in the programme mainly consist of -

C - the cohesion of the soil

Ø - the internal angle of friction of the soil

γ - the density of the soil

Wherever possible these values should be taken from site investigations.

2. PROGRAMME USAGE

Data is accepted by the Programme in two ways.

a) Data statements

b) Conversation mode. (by typing in information when prompted to do so by screen introductions).

One of the prompt questions asks what type of printout is required. There are three choices as follows:

a) No printout. Details and value of factor of safety will appear on the monitor screen only.

b) A summary printout. This is most useful for the multiple runs (grid and frame, discussed in Sections 2.3 & 2.4). The printer will produce a copy of the coordinates, radius and factor of safety.

c) Full printout. For each set of data the components for choesion, angle of friction, unit weight, slice coordinates, water level and embankment coordinates will be given. This printout is useful to verify the parameters for the most critical factor of safety.

There are four different methods of running the programme. Each will be described and illustrated by use of an example. They are as follows :

2.1 METHOD 1

By introducing all parameters into the programme through data statements. This is especially useful if the critical slipe is not circular in section. Fig.C.1 illustrates an example. Note the width of the each slice may vary.

i) Nr. of slices

The number of slices is taken as 5 in the example as this coincides with the section shape. It should be noted that the programme will assume the slip surface between each value of

YB(I) is straight.

Therefore, the larger the number of slices, the more accurate the computation.

The value of the number of slices should be placed in Data statement 50.

ii) Water Levels

For this example, the embankment is assumed to be above water level. The coordinate height at the edge of each slice is therefore taken as 0. As there are 6 edges to the slices, 6 values of zero are required in statement Nr. 60. This parameter is further discussed in Section 2.2.

iii) Cohesion

This example assumes the soil to be silty sandy and cohesive. It also assume the soil to be homogeneous. The programme does cater for differing values of cohesion, angle of friction and unit saturated soil weight. Therefore, should the parameters vary between slices, alteations may be made.

As there are five slices in this example five values of 20 have been placed in statement Nr. 70.

iv) Angle of Fiction

The angle of friction for all slices (taken as 10° in this example) are placed in statement Nr. 80.

v) Unit saturated soil weight

The unit saturated soil weight for all slices (taken as 19.6 kN/m^3) in this example are placed in statement Nr. 90.

vi) Embankment Coords

If, as in this example, values of YT, YB and X are to be placed in data statements, the number and coordinates of the embankment profile are superfluous. However, for the integrity of the programme in integer should be placed in data statement 100, followed by twice that number of coordinate values in statement Nr. 120.

vii) Values for YT,YB & X

If these values are to be placed in the programme (as in this example) then the letter Y (for yes) should be placed in inverted commas in data statement Nr. 140.

This should be followed by the coordinate values of YT(I), YB(I). and X(I), in data statements 150 & 160. An example of the derivation of these values in shown in Fig.C.1 .

```
20 REM ***FOR FACTOR OF SAFETY AGAINST
30 REM *** EMBANKMENT COLLAPSE BY
40 REM *** SLIPPING
50 DATA 5 :REM NR SLICES
60 DATA 0,0,0,0,0,0  :REM WATER LEVELS
70 DATA 20,20,20,20,20 : REM COHESION
80 DATA 10,10,10,10,10 :  REM ANGLE FRICTION
90 DATA 19.6,19.6,19.6,19.6,19.6  : REM UNIT SAT SOIL WT
100 DATA 6 :REM WR OF EMBANKMENT COORDS
110 REM * EMBANKMENT COORDS *
120 DATA 3.33,10.65,6.2,10.65,13.33,7.05,18.66,4.5,24.66,1.5,28.66,1.5
130 REM DATA FOR YT,YB & X
140 DATA "Y" :  REM  SLICE DATA REQUIRED? (Y/N)
150 DATA 10.65,10.65,3.33,10.65,7.65,6.2,7.05,2.5,13.33,4.5,0.45,18.66
160 DATA 1.5,0.10,24.66,1.5,1.5,28.66
```

BISHOP'S SIMPLIFIED METHOD

COH	PHI	GAMMA	X	YT	YB	YB	EE	EN
0	0	0	3.33	10.65	10.65	0	3.33	10.65
20	10	19.6	6.2	10.65	7.65	0	6.2	10.65
20	10	19.6	13.33	7.05	2.5	0	13.33	7.05
20	10	19.6	18.66	4.5	.45	0	18.66	4.5
20	10	19.6	24.66	1.5	.1	0	24.66	1.5
20	10	19.6	28.66	1.5	1.5	0	28.66	1.5

FACTOR OF SAFETY= 1.442308

DATA AT JAFFORGONJ

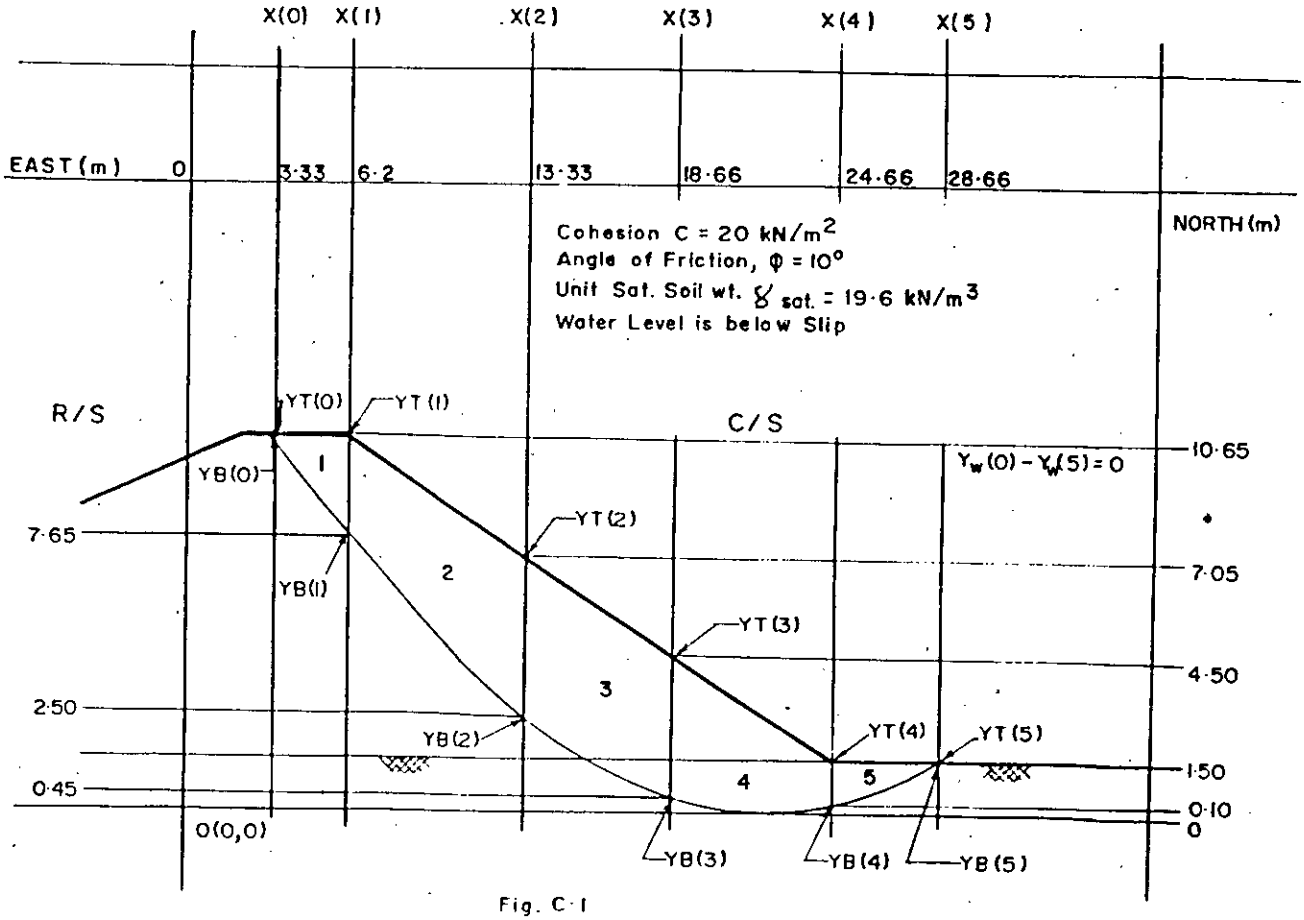

Fig. C·1

Having correctly completed the data statements from 50 to 160, the programme can be run.

The question concerning the printout required will appear on the screen. This has been discussed in section 2. For this example A (for all information) was used. The example printout appears on Fig.C.1 .

The programme will then request if a new slip surface is required. This is a shortcut to coming out of programme mode and replacing values for YB in data statement 150 and (if required) 160. If a new surface is required, the new values for the Y coordinate, from YB(O) to YB(Nr of slices) is then typed in, at the correct prompt. All other parameters are assumed to remain unchanged.

2.2 METHOD 2

The remaining three methods of introducing data into the programme all assume that the slip surface is circular in section. As such, instead of introducing coordinate values for each slide, the embankment coordinates are required, along with the coordinates of the centre of the slip circle and the circle radius.

This method has the big advantage of requiring only 2 coordinates and a value of radius to be input, thus cutting out tedious

calculations for individual slice coordinates. However there are disadvantages, as follows -

The programme, which computes the slice data from embankment and circle coordinates assumes the width of each slice is the same. Also the beginning and end of the slip surface will be at variable locations along the embankment profile. If the soil is homogeneous with water level below slip surface, then there will be no error.

The simple way around the problem is to identify the critical slip surface while assuming no water and homogenous soil and then rerun the programme with amended criteria, suiting the critical slip circle.

i) The data statements 50-90 are completed as described 3a i - v, according to the parameters shown in Fig.C.2 . However, in this example, we have introduced a water profile. The Y coordinate of the edge of each slice (Nr. of slices + 1 values) should be input into data statement 60.

ii) The number of points (including beginning and end) of the embankment section profile should be placed in data statement 100.

```
10 REM ***BISHOP'S SIMPLIFIED METHOD***
20 REM ***FOR FACTOR OF SAFETY AGAINST
30 REM *** EMBANKMENT COLLAPSE BY
40 REM *** SLIPPING
50 DATA 5 :REM NR SLICES
60 DATA 9,7.6,5.5,3.9,2.75,2  :REM WATER LEVELS
70 DATA 10,10,10,10,10 : REM COHESION
80 DATA 20,20,20,20,20 :  REM ANGLE FRICTION
90 DATA 19.6,19.6,19.6,19.6,19.6  : REM UNIT SAT SOIL WT
100 DATA 5 :REM WR OF EMBANKMENT COORDS
110 REM * EMBANKMENT COORDS *
120 DATA 0,10.5,1.33,11.08,5.66,11.08,23.73,2,34.4,2
130 REM DATA FOR YT,YB & X
140 DATA "N" :  REM  SLICE DATA REQUIRED? (Y/N)
150 DATA 11.08,11.08,3.55,9.80,6.50,8.00,8.00,4.40,12.00,5.90,3.00,16.00
160 DATA 4.00,2.25,20.00,2.00,2.00,23.73
```

BISHOP'S SIMPLIFIED METHOD

COH	PHI	GAMMA	X	YT	YB	YW	EE	EN
0	0	0	3.55	11.08	11.08	9	0	10.5
10	20	19.6	8	9.8	6.5	7.6	1.33	11.08
10	20	19.6	12	8	4.4	5.5	5.66	11.08
10	20	19.6	16	5.9	3	3.9	23.73	2
10	20	19.6	20	4	2.25	2.75	34.4	2
10	20	19.6	23.73	2	2	2	0	0

EAST	NORTH	RADIUS	F.O.S
23.8	19.4	17.4	1.414848

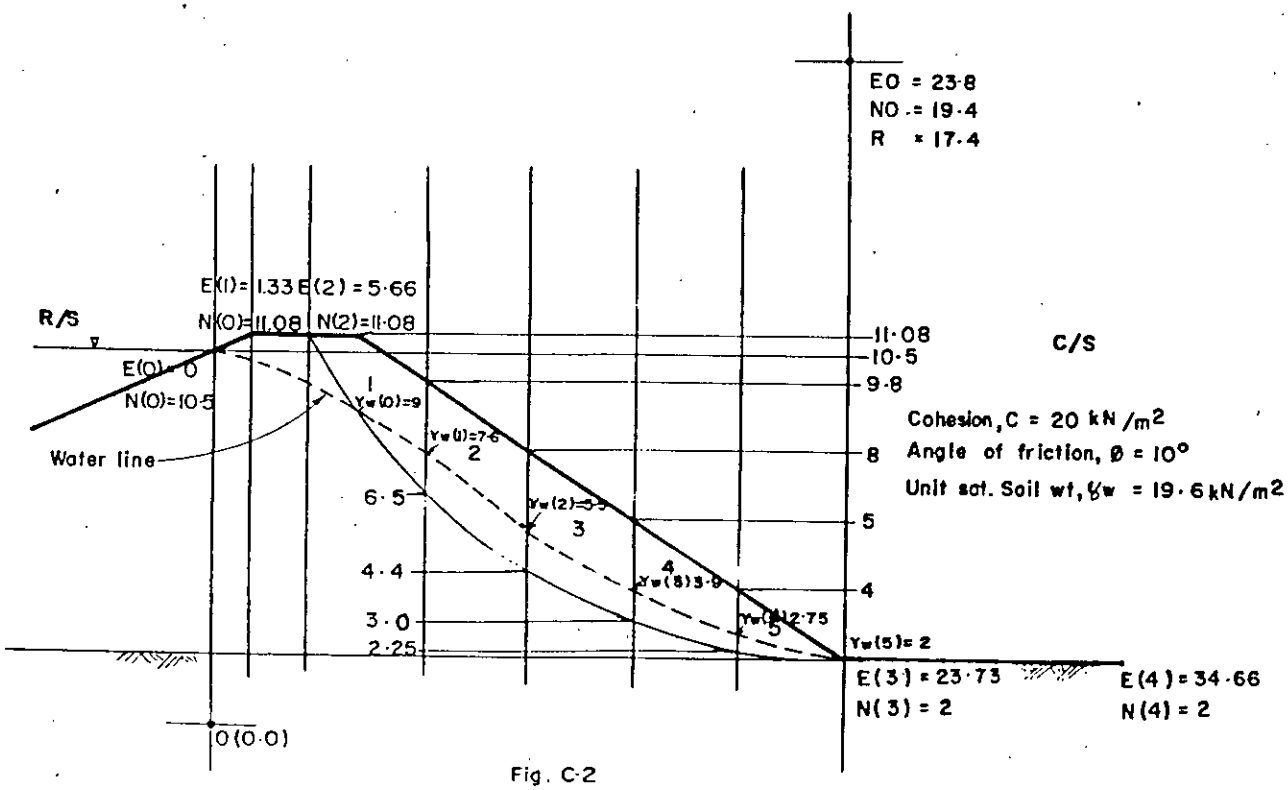

Fig. C-2

iii) The embankment coordinates (Easting then Northing) at each point should be placed in data statement 120.

iv) The answer to the question - is slice data required ? (in this case N for No should be placed in data statement 140.

Having completed the statements, the programme may be run.

v) The programme will ask if we want -

a) a grid (G)

b) a frame (F)

c) or to input coords and radius independently (I)

In this example, we want option (c) so we type in I (ENTER)

vi) The programme will ask for slip circle centre coords Easting (EO)

" " Northing (NO)

" " radius (R)

In this example type in 23.8,19.4,17.4 (ENTER)

vii) The programme will ask which print option is required -

All slice Data (A)

Summary only (S)

No printout (N)

In this example we type in A (ENTER)

The computer will perform the calculations, if required printout the data and also print a summary on the screen.

vii) It will then ask if another point is required. If one is, type Y (ENTER) - the programme will then return to item vi.

If no further runs are required Type N (ENTER) and the programme will finish.

2.3 METHOD 3

Methods 3 is the same in principle as method 2, as the required information is in the form of Embankment coordinates and slip circle coordinates.

The benefit of using grid is that when we input our slip circle coordinates, the programme not only calculates the slice parameters and factor of safety for that location, but also for a further eight slip circle center locations East, S.E,S, S.W,W,N,W,N and N.E. of the initial point (Fig.C.3 refers). The only additional information the programme requires is the range (RA), which is the distance between the center of the grid and side of the grid (In the example shown in Fig.C.3,the range of 1 metre).

```
10 REM ***BISHOP'S SIMPLIFIED METHOD***
20 REM ***FOR FACTOR OF SAFETY AGAINST
30 REM *** EMBANKMENT COLLAPSE BY
40 REM *** SLIPPING
50 DATA 5 :REM NR SLICES
60 DATA 9,7.6,5.5,3.9,2.75,2  :REM WATER LEVELS
70 DATA 10,10,10,10,10 : REM COHESION
80 DATA 20,20,20,20,20 :  REM ANGLE FRICTION
90 DATA 19.6,19.6,19.6,19.6,19.6  : REM UNIT SAT SOIL WT
100 DATA 5 :REM WR OF EMBANKMENT COORDS
110 REM * EMBANKMENT COORDS *
120 DATA 0,10.5,1.33,11.08,5.66,11.08,23.73,2,34.4,2
130 REM DATA FOR YT,YB & X
140 DATA "N" :  REM  SLICE DATA REQUIRED? (Y/N)
150 DATA 11.08,11.08,3.55,9.80,6.50,8.00,8.00,4.40,12.00,5.90,3.00,16.00
160 DATA 4.00,2.25,20.00,2.00,2.00,23.73
```

BISHOP'S SIMPLIFIED METHOD

EAST	NORTH	RADIUS	F.O.S
23.8	19.4	17.4	1.414848
22.8	19.4	17.4	1.312738
22.8	18.4	17.4	1.021448
23.8	18.4	17.4	1.076887
24.8	18.4	17.4	1.14099
24.8	19.4	17.4	1.580626
24.8	20.4	17.4	3.851895
23.8	20.4	17.4	2.433037
22.8	20.4	17.4	1.954301

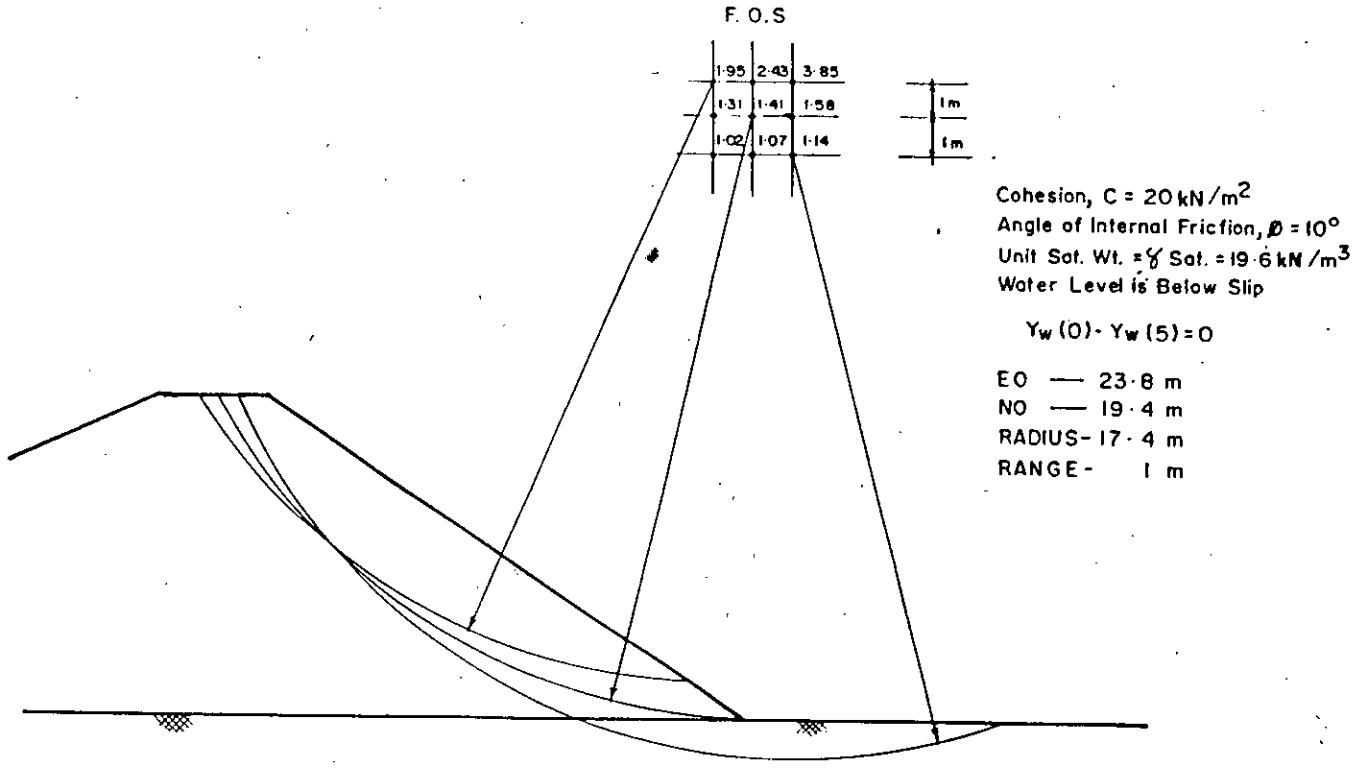

Fig. C·3

The data statements are identical for Fig.C.3 as for Fig.C.2 except that we assume the embankment is dry. The values of water level in statement 60 are therefore reduced to zero.

Having amended satement 60 the programme may be run.

ii) The programme will ask if we want a grid, a frame or individual coordinates. In this example we want to use a grid, so we type in G(ENTER).

iii) The programme will ask for the coordinates EO, NO and Radius R, as in example 2 . However for grid, it will also ask for range (RA).

Thus for this example we type 23.8,19.4,17.4,1 (ENTER)

iv) In reply to the type of printout for this example we want only a summary. Thus we type S (ENTER).

The programme will then run, printing a summary (shown in Fig.B.3) and also giving a summary in the monitor.

v) It will the ask if another central point is required. If so, type Y (ENTER) and the programme will return to step iii. If not, type N (ENTER) and the programme will finish.

2.4 METHOD 4

The example for method 4 assumes all the parameters same as example 3. The only change are the coordinates of the slip centre and the radius.

i) It response Grid,Frame or Individual we type in F (ENTER).

ii) The programme will then ask for Minimum and Maximum East coordinates.

For this example type in 22.8,24.8 (ENTER).

iii) The programme will ask for Minimum and Maximum North coordinates.

For this example type in 18.9,20.4 (ENTER).

iv) The programme will ask for step internal
For this example type in 0.5 (ENTER).

v) The programme will ask for Radius
For this example type in 17.6 (ENTER).

The programme will calculate the factors of safety for each circle within the specified frame.

BISHOP'S SIMPLIFIED METHOD

EAST	NORTH	RADIUS	F.O.S
22.8	18.9	17.6	1.054816
23.3	18.9	17.6	1.084308
23.8	18.9	17.6	1.120483
24.3	18.9	17.6	1.158538
24.8	18.9	17.6	1.204518
22.8	19.4	17.6	1.199795
23.3	19.4	17.6	1.226124
23.8	19.4	17.6	1.259605
24.3	19.4	17.6	1.289717
24.8	19.4	17.6	1.360538
22.8	19.9	17.6	1.431595
23.3	19.9	17.6	1.496713
23.8	19.9	17.6	1.579969
24.3	19.9	17.6	1.690904
24.8	19.9	17.6	1.849508
22.8	20.4	17.6	1.759954
23.3	20.4	17.6	1.894206
23.8	20.4	17.6	2.086142
24.3	20.4	17.6	2.377932
24.8	20.4	17.6	2.876717

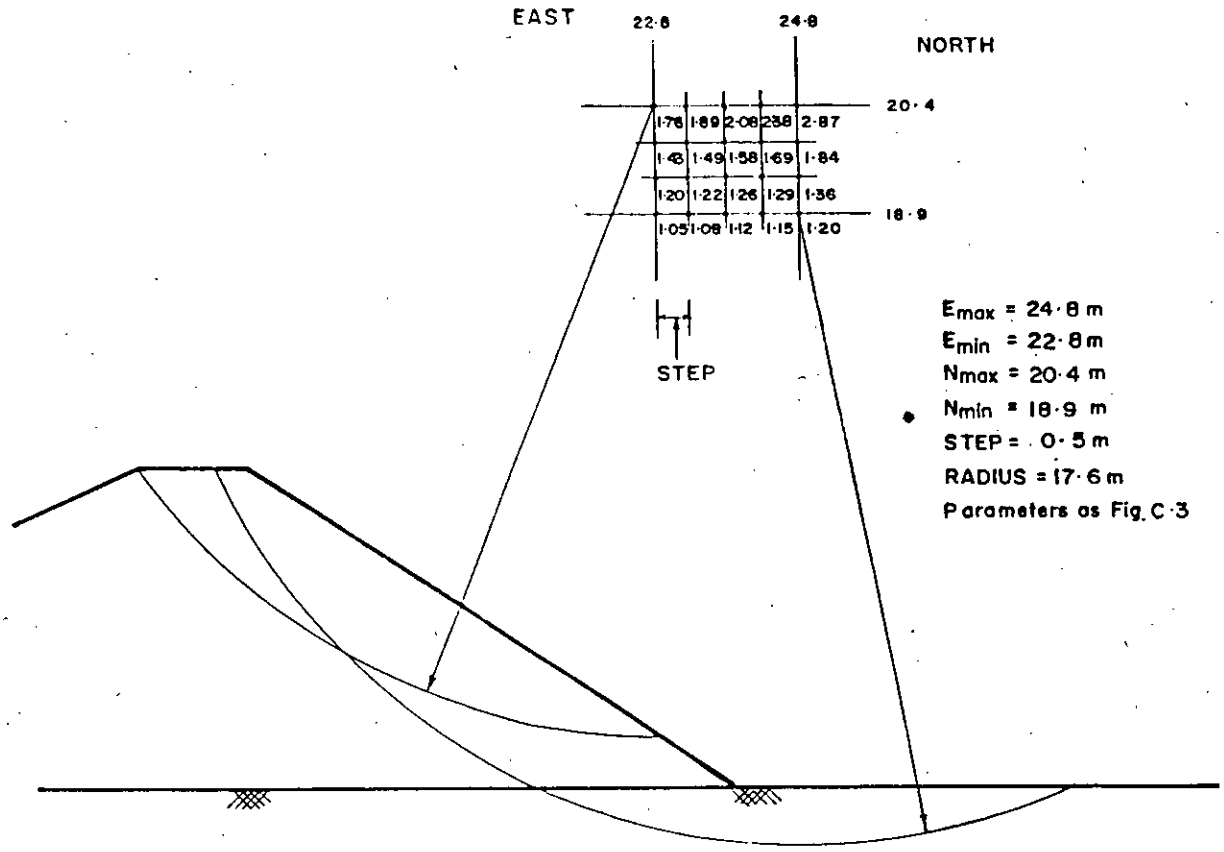

Fig. C·4

However, if (as in the example case) there is a situation where the radius is too large to intercept with the embankment as specified by the coordinates, the message "Radius too large" will appear on the screen. The printed version ignores any circle which is outside embankments limits.

Likewise, if the radius is too small (so the circle does not actually reach the defined surface) the message "Radius too small" will appear on the screen.

vi) Having completed the execution of the programme, for a new frame, simply type Y (ENTER) when prompted to do so.

The user may choose any of the four methods to calculate the factor of safety against embankment collapse. However, in general, the recommended procedure would be to use method 1 for non circular slips or special cases. For circular slips a coarse frame could be used (assuming homogeneous soil conditions and no water pressure) in order to arrive at a general area of the critical slip. This can be refined with grid, using decreasing values of range.

A printout of circle data should be obtained for the most critical slip. If there is a water profile or if the soil has different stratum, then the refined information should be placed in the data statements, and a check carried out using grid to ensure the critical slip circle location has not changed significantly.

SAMPLE CALCULATIONS

SWEDISH SLIP CIRCLE METHOD

JAFFORGONJ

Riverside slope 1:3

Countryside slope 1:2

$H = 9.08$ m

$\gamma_{sub} = 9.8$ kN/m^3

$\gamma_{sat} = 19.8$ kN/m^3

$\emptyset = 10^o$

$C = 20$ kN/m^2

$\alpha_1 = 25^o$

$\alpha_2 = 35^o$

From Fig.D.1

$\theta = 51^o$

$h = 10.78 - 2.23 = 8.55$ m

$Q = 25.75$ m

$H.W.L = 10.78$ m

$R = 28$ m

$$L_{arc} = \frac{2 \times \pi R \times 51}{360} = 24.923 \text{ m}$$

$AB = 0.3Q = 7.725$ m

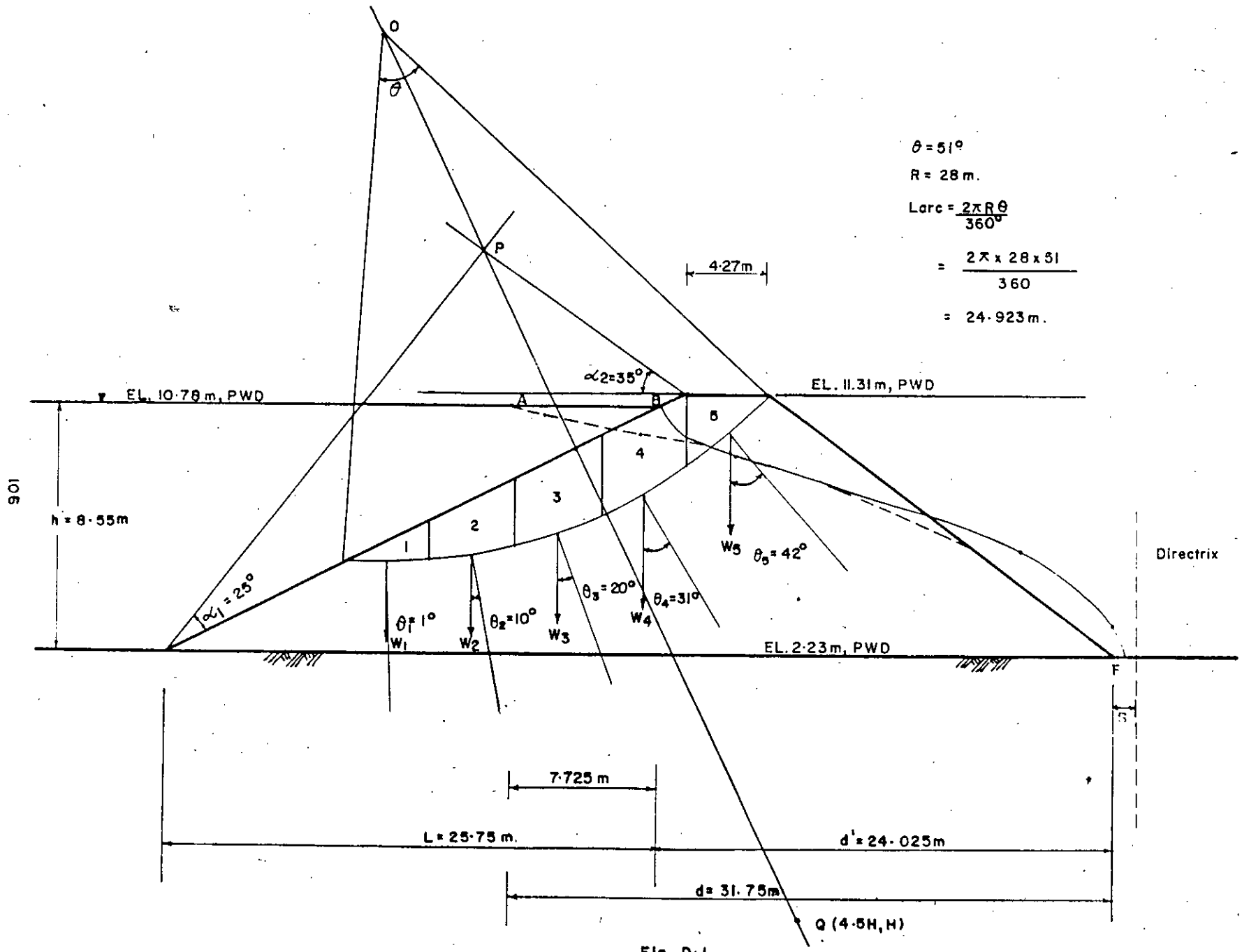
O
θ
P
4·27m
θ = 51°
R = 28 m.
Larc = 2πRθ / 360°
= 2π x 28 x 51 / 360
= 24·923 m.
α2=35°
EL. 10·78 m, PWD
EL. 11.31 m, PWD
A
B
5
4
3
2
1
h = 8·55m
α1 = 25°
θ1 = 1°
W1
W2
θ2=10°
W3
θ3 = 20°
W4
θ4=31°
W5
θ5 = 42°
EL. 2·23 m, PWD
F
Directrix
7·725 m
L = 25·75 m.
d' = 24·025m
d = 31·75m
Q (4·5H, H)

Fig. D·1

$4.5H = 40.86$ m

$Q\ (4.5H, H)$

From equation(3.5)

$$d = \frac{2}{1} \times 8.55 + 4.27 + \left(\frac{2}{1} + \frac{3}{1} \right) \times (11.31 - 10.78) + 0.3\ Q$$

$= 31.745$ m

$= 31.75$ m

$d = 31.75 - 0.3\ Q$

$= 31.75 - 7.725$

$= 24.025$ m

$$\alpha = \tan^{-1}\left(\frac{9.08}{2 \times 9.08} \right)$$

$= 26.56^{o}$

From equation (3.6)

$$a = \frac{24.025}{\cos 26.56^{o}} - \left(\frac{24.025^{2}}{\cos^{2} 26.56^{o}} + \frac{8.55^{2}}{\sin^{2} 26.56^{o}} \right)^{1/2}$$

$= 7.997$

$= 8.0$ m

From equation (3.2)

$$S = (d^{2} + h^{2})^{1/2} - 31.75$$

$$= (31.75^{2} + 8.55^{2})^{1/2} - 31.75$$

$= 1.131$ m

Taking focus F as the origin,

From equation (3.2)

$$(x^{2} + y^{2})^{1/2} = x + S$$

$$x^{2} + y^{2} = x^{2} + 2xS + S^{2}$$

$$y = (2xS + S^{2})^{1/2}$$

$$y = (2.262x + 1.279)^{1/2}$$

Coordinate of the base parabola are as follows-

x	0	5	10	15	20	25	30	31.75
y	1.131	3.548	4.888	5.934	6.821	7.602	8.315	8.55

Slice area and weight (with reference to Fig.D.1)

Slice No.	Area (m^2)	Saturated Wt. $W_{sat.}$ kN	Submerged Wt. $W_{sub.}$ kN
1	1/2x4.5x1.416=3.186	62.44	31.22
2	(1.416+2.25)/2x4.5=8.25	161.67	80.84
3	(2.25+2.833)/2x4.5=11.44	224.16	112.08
4	(2.833+2.5)/2x4.5 =12.00	235.20	117.60
5	1/2x4.5x2.5 =5.625	110.25	55.13

Resulation of Forces (with reference to Fig.D.1)

$N_s = W_{sub}.\cos\theta$ $\qquad$ $T_s = W_{sat}.\sin\theta$

Slice No.	Angle (θ^o)	N_s kN	T_s kN
1	1	31.21	1.09
2	10	79.61	28.07
3	20	105.32	76.66
4	31	100.80	121.14
5	42	40.97	73.77
		$\Sigma N_s = 357.91$	$\Sigma T_s = 300.73$

Uplift Forces

Maximum uplift force, $\Sigma U_s = \gamma_w i_{max} V$ (where $V = 1\ m^3$)

$= 9.8 \times 1/2 \times 1$

$= 4.9$ kN

$$F.S = \frac{CL_{arc} + \tan\emptyset(\Sigma N_s - \Sigma U_s)}{\Sigma T_s}$$

$$= \frac{20 \times 24.923 + \tan 10^o (357.91 - 4.9)}{300.73}$$

$= 1.864 > 1.0$ (O.K)

JAFFARGONJ

$C = 20.0\ kN/m^2$

$\emptyset = 10^o$

$\gamma_{sat} = 19.6$ kN/m (saturated unit weight)

$H = 9.08$ m

From the Fig.A.1 will seen that a slope with $\emptyset = 10^o$ an inclination of $18^o\ 26'$ has a stability number (N) of 0.035. This means that, if the factor of safety for friction was unity, the cohesion which must be developed would be found from the experession:

$$\frac{C_m}{\gamma H} = 0.035$$

$C_m = 0.035 \times 19.6 \times 9.08 = 6.228\ kN/m^2$

Factor of safety with respect to cohesion = $\frac{20.0}{6.228} = 3.21$

This is not the F.S within slope stability, which is

$$F.S = \frac{\text{Shear strength}}{\text{disturbing shear}}$$

$$\text{i.e } F.S = \frac{C + \tan\emptyset}{\tau}$$

The F.S. applies equally to cohesion and to friction.

F.S can be found by successive approximations:

$$\tau = \frac{C}{F.S} + \frac{\tan\emptyset}{F.S}$$

So,

Try F.S = 1.5

$$\frac{\tan\emptyset}{F.S} = \frac{\tan 10^{o}}{1.5} = 0.11755 = \text{tangent of an angle of } 6.704^{o}$$

Using this value of Ø to establish a new N value from Fig.A.1

N = 0.061.

C_m = 0.061x19.6x9.08 = 10.856 kN/m^2

$$F.S(\text{for } C) = \frac{20.0}{10.856} = 1.842$$

Try F.S = 1.6

$$\frac{\tan\emptyset}{F.S} = \frac{\tan 10^{o}}{1.6} = 0.110 \ (\emptyset = 6.288^{o})$$

from Fig.A.1, N = 0.063

C_m= 0.063x19.6x9.08 = 11.21 kN/m^2

$$F.S(\text{for } C) = \frac{20.0}{11.21} \quad 1.783$$

Try F.S= 1.9

$$\frac{\tan\emptyset}{F.S} = \frac{\tan 10^{o}}{1.9} \quad (\emptyset = 5.302^{o})$$

From Fig.A.1, N = 0.081

C_m= 0.081x19.6x9.08 = 14.515 kN/m^2

$$F.S(\text{for } C) = \frac{20.0}{14.415} = 1.387$$

Try F.S = 1.7

$$\frac{\tan\emptyset}{F.S} = \frac{\tan 10^{2}}{1.7} = 0.1037 \quad (\emptyset = 5.92^{o})$$

from Fig.A.1, N = 0.065

C_m= 0.065x19.6x9.08 = 11.567 kN/m^2

$$F.S \text{ (for C)} = \frac{20.0}{11.567} = 1.73 \quad (= F\emptyset)$$

Factor of safety for slope = 1.7 > 1.5 (O.K)

JAFFARGONJ

$C' = 20\ kN/m^2$

$\emptyset = 10^o$

$\gamma_{sat} = 19.6\ kN/m$

From Fig.D.2, $\theta = 74^o$

$R = 15.75\ m$

$$C'\ l = C' \times \frac{2\pi R}{360^o} \times 74^o = \frac{20 \times 2 \times \pi \times 15.75 \times 74^o}{360^o} = 406.83\ kN$$

Determination of r_u as follows:

Slice No.	h (m)	$u = \gamma_w h_w$ (kN/m^2)	Z (m)	$r_u = \frac{u}{\gamma_{sat} Z}$
1	0.90	8.82	1.20	0.375
2	2.25	22.05	3.10	0.363
3	3.10	30.38	4.51	0.343
4	2.95	28.91	4.75	0.310
5	1.0	9.8	2.75	0.181

Slice No.	Z (m)	b (m)	$W=\gamma_{sat}bZ$ kN	α^{o}	$\cos\alpha$	$\sec\alpha$	$\cos\alpha - r_u\sec\alpha$	$W(\cos\alpha - r_u\sec\alpha)\tan\phi'$	$\sin\alpha$	$W\sin\alpha$
1	1.20	4.42	103.96	-4	0.997	1.002	0.621	11.38	-0.069	-7.25
2	3.10	4.42	268.56	10	0.984	1.015	0.615	29.12	0.174	46.63
3	4.51	4.41	390.71	22	0.927	1.078	0.557	38.37	0.375	146.36
4	4.75	4.42	411.50	37	0.798	1.252	0.409	29.68	0.602	247.64
5	2.75	4.42	238.24	53	0.602	1.661	0.301	12.64	0.798	190.27
								Σ 121.19		Σ 623.65

$$F.S = \frac{C'l + \Sigma W(\cos\alpha - r \sec\alpha)\tan\phi'}{\Sigma W \sin\alpha}$$

$$= \frac{406.83+121.19}{623.65}$$

$$= 0.85 < 1.0 \quad (\text{ Not O.K })$$

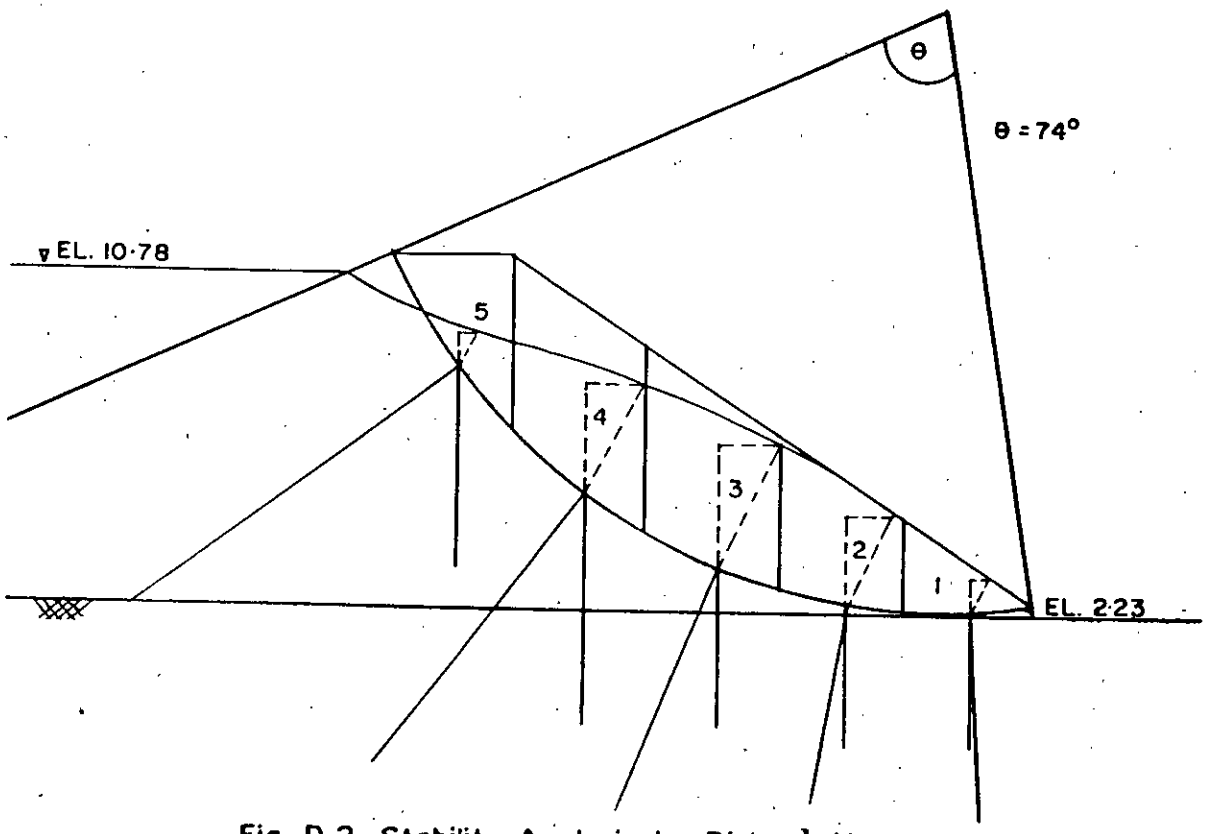

Fig. D·2 Stability Analysis by Bishop's Method.

JAFFARGONJ

With reference to Fig. D.3

No. of significant flow line, $N_f = 3$

No. of potential drops, $N_d = 8$

Seepage quality, $q = K\ h\ \frac{N_f}{N_d}$

where,

h = total head loss

= 10.78-4.66

= 6.12 m

$$q = \frac{5x10x^{-4}}{100} \times 3600x24x6.12x\ \frac{3}{8}$$

= 0.432x6.12x3/8

= 0.9914 m^3/day/m < 1.00 m^3/day/m (Which is critical)

The seepage flow through the embankment is very nominal.

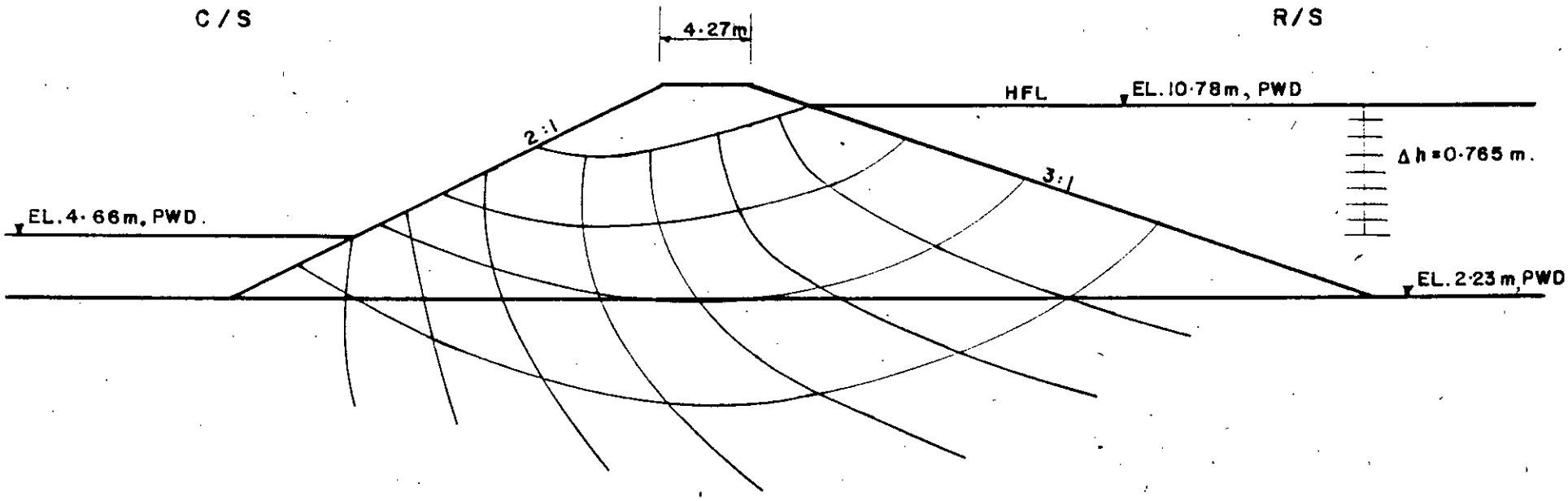

Fig. D-3 Quantity of Seepage analysis of Gumti Phase I embankment at Jaffarganj

UPLIFT ANALYSIS

Maximum upward hydraulic gradient, $i_{max} = \tan\beta$

i.e $i_{max} = \tan\beta = 1/2 = 0.50$

Maximum uplift force, $F_u = \gamma_w \, i_{max} \, V$ (where $V = 1 \, m^3$)

$= 9.8 \times 0.50 \times 1$

$= 4.9$ kN

Buoyant weight of soil, $W' = \gamma_{sat} - \gamma_w$

$= 19.6 - 9.8$

$= 9.8$ kN

Factor of safety, $F.S = \dfrac{W'}{F_u}$

$= \dfrac{9.8}{4.9}$

$= 2.0 > 1.5$ (O.K)

বাংলাদেশ প্রকৌশল বিশ্ববিদ্যালয়
নং 76758
তাং 26/8/90
লাইব্রেরী, ঢাকা