Hydraulics Research
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THE STABILITY OF COHESIVE DREDGED SLOPES

A review of methods of analysis
and experimental techniques

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CONTRACT

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ABSTRACT

The cost of dredging in order to provide access for ships and other vessels can be a large component of a project. There is an initial capital cost involved in creating a dredged area and the subsequent recurring cost of maintenance.

In creating a dredged area, material is removed to the required depth leaving side walls sloping up to original bed level. With the passage of time material will accumulate in the dredged section, some having been carried in suspension by the water, and possibly some resulting from gradual failure of the side slopes.

In this review we consider the stability of side slopes dredged in cohesive soils. We examine both the choice of slope angle to achieve a safe initial condition and the more gradual movement of material from the slopes. It has been undertaken it has been undertaken in order to provide a background for an experimental study of underwater slope stability.

Mechanisms of failure in both still and moving water are considered and appropriate methods of analysis identified, leading to recommendations on a number of features of the experimental facility and on the analysis of tests.

There are two fields of study which may be used in analysing the behaviour of underwater cohesive slopes - soil mechanics and rheology. Within the field of soil mechanics, currently available computer programmes using various methods of slices are suitable for underwater slopes and can also be made to take account of the effects of moving water. The rheological approach, although less developed, can be used in the form of Morgenstern's work to examine sediment flows.

The physical parameters to be measured in the field, and in experiments, have been listed. Laboratory scale work need not be viewed as an exact replica of a real prototype, but rather as an attempt to establish laws which apply on laboratory slopes and hence, it is assumed, on larger slopes.

Although most of the interest is in slopes of cohesive material many of the phenomena described may be easily studied by using coarser material, ie, sands. These have the advantage of requiring less time to consolidate than clays and silts.

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Fig 3 An approxiate static solution for waves over a dredged slope

APPENDIX A

Outline of classical methods of analysis

1 INTRODUCTION

The cost of dredging in order to provide access for ships and other vessels can be a large component of a project. There is an initial capital cost involved in creating a dredged area and the subsequent recurring cost of maintenance.

In creating a dredged area, material is removed to the required depth leaving side walls sloping up to original bed level. With the passage of time material will accumulate in the dredged section, some having been carried in suspension by the water, and possibly some resulting from gradual failure of the side slopes.

In this review we consider the stability of side slopes dredged in cohesive soils. We examine both the choice of slope angle to achieve a safe initial condition and the more gradual movement of material from the slopes.

The stability of slopes on land (ie, not submerged) has been the subject of a good deal of attention in the past and techniques such as slip circle analysis and the method of slices are well established. These techniques can be applied to submerged or emergent (ie, partly submerged) slopes with little modification, provided they are in still water.

However, dredged slopes are almost invariably found in water which is not still. The effect of currents, waves and tides on the stability of the side-slope may be significant and introduces an extra degree of complexity into the analysis. In this review, we examine the stability of slopes under still and moving water by addressing the following questions:

- what are the most likely mechanisms of failure?

- how can these be analysed?

- therefore, what parameters need to be measured in the field in order to perform the analysis?

We begin by considering slopes in still water and then move on to examine the effect of moving water. Finally, we review previous experimental work and identify the most appropriate directions for future laboratory investigations.

2 SLOPES IN STILL WATER

2.1 Introduction

A review of the literature reveals that a good deal of work has been done on the stability of submarine slopes. However, this work has concentrated almost exclusively on large marine slides on long slopes which can be considered, for the sake of analysis, to be infinite in extent. These slopes lend themselves to a fairly straightforward analysis.

Dredged slopes, however, are relatively short, so that this 'infinite slope' treatment may not be applicable. Nevertheless, it is described here, together with techniques which may be more appropriate to a finite, dredged slope.

A second distinction to make, besides the one between finite and infinite slopes, lies in the choice of which field of study is most appropriate to underwater soil movements. The choice is between the fields of soil mechanics and rheology.

In the field of soil mechanics, the study of slopes on-land is well advanced compared to the study of underwater slopes. To apply soil mechanics techniques to the latter, therefore, would seem to require the translation of above-water methods of analysis to make them suitable for underwater soils. The result of this is that the parameters to be measured in the field would be those which are usual in soil mechanics.

Rheology is the study of flow of matter. In our context, to take a rheological approach is to consider a mass of moving soil and water as a flow in which material is not necessarily moving as one block. Taking this approach, the parameters to measure are those common to rheology.

Both fields can be appropriate; soil mechanics is perhaps most suitable where the soil moves as one rigid mass, rheology where that is not the case.

2.2 Mechanisms of failure

Boehmer et al (1983) carried out full scale slope stability tests in which they identified two main geotechnical problems.

(1) <u>Initial slope stability</u>

(2) <u>Post failure slope stability</u> as a result of sediment 'flow' triggered by an initial failure

Hampton (1970) defines three categories into which movements of underwater sediment can be divided on the basis of the degree of internal deformation within the moving material. Hence, we can use the following definitions:

Slide: large blocks or slabs of material move on a few relatively well-defined slippage planes with only slight internal deformation

Slump: material is broken up into many blocks and is generally internally deformed

Sediment gravity flows: the sediment-water mixture flows in a manner most akin to that of a viscous fluid. There is a high degree of internal deformation. Such flows are distinguished from fluid gravity flows, where sediment is carried along in a moving fluid, by the fact that it is the particles which are driven by gravity and move the interstitial fluid.

Movements which fall into the category of slides can take various forms. On a long slope the slippage plane may be parallel to the slope itself with the material moving as a 'flake'. Failure planes will follow lines of weakness in the material and in short slopes (such as dredged ones) these often take the approximate form of arcs of circles.

In all types of failure in still water, the driving force is gravity acting on the soil particles and the basic reaction to that force arises from the soil strength. The strength of underwater deposits is often rather low, the factor which in still water contributes most to the shallow slope angles observed.

The low strength may be a consequence of the loose packing of particles. An additional factor may be the generation of high pore pressures during consolidation. Morgenstern (1967) reports Terzaghi (1956), who drew attention to the influence of high rates of sedimentation on the development of strength in a consolidating sediment. Excess pore pressures can develop in a stratum, especially if it is undergoing rapid deposition. That excess pressure reduces the effective stress, and hence, the strength of the material.

Another cause of instability of slopes in still water is the possible presence of gas (Hampton et al, 1978). This can contribute to raising pore pressures and so lowering strength.

Finally, earthquakes can also add an extra destabilising force which may lead to failure and may also contribute to high pore pressures (Hampton).

It seems evident that slides, as defined above, lend themselves to analysis using concepts derived from the field of soil mechanics, whereas sediment gravity flows are more suited to a rheological approach. Slumps may be treated as a intermediate phenomenon.

In the sections which follow, the factor of safety, F, is defined in terms of the soil shear strength, ie, as available shear strength divided by the strength at which failure would occur. This is used by most authors in preference to other definitions.

2.3 Infinite slope

The method of assuming the slope is infinite has been used by most of those who have investigated large marine slides (Hampton et al (1978), Almagor and Wieseman (1982), Morgenstern (1967), Karlsrud and Edgers (1982), Olsen et al (1982)). Failure is treated as taking place on a plane parallel to the slope itself.

Morgenstern (1967) uses the techniques for normally consolidated clays with an undrained shear strength, Cu at the plane of failure. The factor of safety, F, against sliding is given by

$$F = \frac{Cu}{\gamma' z} \frac{2}{\sin 2\alpha} \qquad (1)$$

where

γ' = buoyant unit weight of soil
z = depth below surface of sediment
α = slope angle

Morgenstern also introduced a horizontal force due to earthquakes having an acceleration k which is some percentage of gravity. He derived an expression for the value of the factor of safety as

$$F = \frac{Cu}{\gamma' z} \frac{1}{(\frac{1}{2} \sin 2\alpha + 3k\cos^2\alpha)} \qquad (2)$$

The factor of safety against sliding of such a slope is given in terms of effective stress, and ignoring cohesion, by Hampton et al (1978) as

$$F = \left(1 - \frac{\Delta u}{\gamma' z\cos^2\alpha}\right) \frac{\tan\phi'}{\tan\alpha} \qquad (3)$$

where

Δu = excess pore pressure
ϕ' = effective stress friction angle

Whereas the soil mechanics techniques described above are most appropriate for the analysis of initial stability, the rheological approach described by Morgenstern (1967) can be applied to the subsequent gradual failure. This gradual failure is most likely to be due to the influence of moving water. Morgenstern considers the velocities within a sediment flow where viscous forces are included, resulting in a velocity distribution ranging from maximum at the slope surface down to zero at the failure plane as determined by Equation (1).

He considers a non-cohesive soil, and examines the forces on a small element in the flow, resulting in the equation

$$\gamma' \sin\alpha - \frac{\partial\tau}{\partial z} = \frac{\gamma}{g}\frac{\partial V}{\partial t} \qquad (4)$$

where

τ is shear stress on the failure plane (parallel to the slope)
z is measured positive downwards, perpendicular to the slope
V is the velocity parallel to the slope

He also assumes that the excess pore pressure u increases linearly with depth so that

$u = nz$

The shear stress is given by

$$\tau = (\gamma' z \cos\alpha - nz)\tan\phi' - \eta\frac{\partial V}{\partial z} \qquad (5)$$

where there is no apparent cohesion c'.

Morgenstern gives a solution for Equations (4) and (5) combined, which allows the flow velocity throughout a slide to be calculated by means of a graph. This could be used to predict rates of sediment movement.

Although the rheological approach has not been developed to the same degree as various soil mechanics techniques, it appears to have a potential application to the case of the gradual movement of sediment.

2.4 Dredged slope

There are a number of classical soil mechanics techniques for the calculation of the likely factor of safety of a finite slope. Many of these are based on a circular slip with a solution obtained often by dividing the slip zone into a number of slices and calculating the disturbing and restoring forces on each slice. An outline of some of these methods is given in Appendix A.

In application to dredged slopes the shear strength of the mud is most likely to increase with depth (ie effective stress) below the original surface, such that

$$\frac{Cu}{\sigma_v'} = \text{constant}$$

It is reasonable in this instance to use a solution given by Booker and Davis (1972) based on an exact plasticity method. The solution is shown in Figure 1 as a graph of the relationship at failure (ie, F = 1) between the slope angle α and the ratio $\frac{Cu}{\gamma' z}$. It is interesting to note that the solution is independent of the actual height of the dredged slope and depends only on the rate of increase of the strength of the mud with effective stress. Also shown on Figure 1 is the infinite slope solution of Morgenstern (1967) as given in Equation (1). For any given value of the ratio $\frac{Cu}{\gamma' z}$ the critical slope angle is substantially less for the infinite slope than for the dredged (finite) slope.

2.5 Degree of consolidation

It is most likely that muds in estuarine or near coastal regions will be underconsolidated. This means that the mud is still consolidating under its self weight and squeezing out water trapped in the pores of the mud. The pressure of the water in these pores (u) will therefore be higher than the ambient hydrostatic pressure. As a result of this excess pore pressure the effective stress (σ_v') within the soil matrix will be less than the total stress (σ_v) and is given by:

$$\sigma_v' = \sigma_v - u$$

The ratio of undrained shear strength to effective stress $\frac{Cu}{\sigma_v'}$ has been found in practice to be reasonably constant, and therefore, a reduction in effective stress due to an excess pore pressure will accordingly lower the undrained shear strength of the underconsolidated soil.

For example, suppose a mud with an effective weight, γ' of $2.5 kN/m^3$ had a ratio Cu/σ_v' of 0.15. For the normally consolidated situation $\sigma_v' = \gamma' z$, and hence, $Cu/\gamma' z = 0.15$. From Figure 1 this corresponds to a critical finite slope angle of 22°. However, if the mud was underconsolidated and had an excess pore pressure of say 0.5m head of water ($\gamma_w = 10 kN/m^3$) at a depth of 5m then $Cu/\gamma' z = 0.15(5x2.5-0.5x10)/(2.5x5) = 0.09$ and the critical slope angle is reduced to around 10°.

2.6 Typical values of $Cu/\gamma' z$

For normally consolidated clays it is reasonable to assume that the value of Cu/σ_v' is likely to be in the range of 0.15 to 0.40. A typical value would be around 0.25. However, the effects of underconsolidation tend to reduce the rate of increase in effective stress with depth and also to reduce the gain in shear strength with depth. Accordingly, the value of the ratio $Cu/\gamma'z$ will be lower in underconsolidated soils than in normally consolidated soil.

Henkel (1970) quoted values of $Cu/\gamma'z$ in the Mississippi Delta of 0.03-0.05 although no pore water pressure measurements were given. From values given of the density and shear rigidity in Zeebrugge harbour muds it was possible to estimate that $Cu/\gamma'z$ was 0.04.

In the absence of any site specific data then it would be advisable to choose a number of different $Cu/\gamma'z$ values in the range 0.03-0.15 depending on the anticipated degree of excess pore pressure.

3 SLOPES IN MOVING WATER

3.1 Introduction

The stability of an underwater slope will be effected by the movement of the water around the slope. This movement could be in the form of waves, tidal currents, tidal variation of water level or a combination of any of these factors. Their likely influence on the stability and degradation of an underwater slope is considered in this section.

3.2 Waves

3.2.1 Infinite slope

Henkel (1970) discussed the role of waves in causing submarine landslides. He considered an infinite slope and calculated the static effect of an idealised wave above the sea bed. The solution was presented graphically in dimensionless parameters and is shown in Figure 2.

Wright (1976) also used the infinite slope approach. He used a stress-strain law for the soil and developed a finite element model which would predict cyclic soil movements as a result of a series of waves. This he applied successfully to an actual submarine slide in the Gulf of Mexico.

Mallard and Dalrymple (1977) considered the actual variation in water pressure at the bed, given that the bed is likely to be deformable and that, in responding to pressure changes, it will also tend to have a damping effect on those pressures. They assumed that the bed was an elastic medium and derived expressions for the resulting (total) stresses in the soil, and for the soil displacements.

De Groot and Sellmeijer (1979) presented an analytical solution for the variation of pore pressure due to the action of waves over a bed consisting of two layers of soil. They found that in certain cases the pore pressure variation may be out of phase with the corresponding variation in total stress (due to the fluctuating water level over the bed). This may result in a destabilising force when the soil strength is viewed in effective stress terms.

Yamamots et al (1978) also derived equations for the variations of pore pressure and effective stresses in bed material as a result of wave loading. They considered some extreme cases and presented simple expressions for those conditions. In general they found that the pore pressure response is very dependent on the permeability of the material and on the stiffness.

Karlsrud and Edgers (1982) have reviewed further work on this topic, and concluded that "there is room for theoretical improvements ...." but " there are many more uncertainties associated with the determination of the necessary input parameters..... Future work should therefore concentrate on developing better and more suitable sampling and in-situ testing techniques".

3.2.2 Dredged slope

The effect of waves on a finite dredged slope is considerable. The passage of a wave above a slope will have two effects. Firstly, the slope will be subject to a non-uniform hydrostatic pressure and secondly, the surface of the slope will be subject to a shear stress generated by the orbital motion of the water.

Little information on the design of dredged slopes subject to the effects of waves appears in the literature and it has therefore been necessary to explore methods of analysis which may be appropriate in evaluating the effect of waves.

With reference to the variation in hydrostatic head over the slope due to the passage of the wave a simple static calculation of the stability of the slope was undertaken using Bishop's Simplified Method. The shape of the wave was idealised to a triangular form with the peak co-inciding with the top of the slope (Fig 3).

The strength of the soil was assumed to increase linearly with depth from zero at the original surface. The steepness of the wave (h/l_o) was varied between

0 to 0.07 which may be interpreted as indicating the severity of the waves. For example, a wave steepness of 0.02 is typical of swell, whereas, a wave steepness of 0.06 is an average storm. The wave length l_o would typically be in the region of 40m for a water depth of 5m and near to 150m for a water depth of 20m. The results of the static analysis are not valid when the length of the dredged slope l_s is greater than $l_o/2$.

The results of the simple analysis are presented in Figure 3 as the relationship between $Cu/\gamma' z$ and slope angle for different wave steepnesses. The effect of wave steepness on the critical slope angle is considerable even for a wave steepness of 0.01. For example, in still water a slope with $\frac{Cu}{\gamma' z}$ of 0.14 has a critical angle of 20°, whereas, the same material subject to a wave of steepness 0.01 has a critical angle of only 9°. However, the same soil would appear to be unstable under a wave steepness of 0.02 for any slope angle. This conclusion is to some extent verified by Henkel's work for an infinite slope (see Fig 2) which shows that for a zero slope angle the critical value of $\frac{\gamma'_w hz}{Cu\ 2\ l_o}$ for failure is about 0.16 which corresponds to a wave steepness (assuming $Cu/\gamma' z = 0.14$, $\gamma' = 2.5kN/m^3$ and $\gamma_w = 10kN/m^3$) of around 0.01.

It is also interesting to note that the critical slope angle actually decreases with increasing strength for a wave steepness of 0.07. The reason for this apparent anomaly is due to the overriding influence of the length of the slope rather than the strength of the slope on stability at high wave steepnesses. Because the change in hydrostatic pressure between the top and bottom of the slope increases with the overall length of the slope, and is inversely proportional to the angle of the slope, a flatter slope will be effected relatively more by the steep waves than will a steep slope. Hence, the steep slopes are in some circumstances more stable.

The shear stress on the face of the slope induced by the orbital motion of the water during the passage of the wave will have the effect of fluidising the surface layer of the slope face. The depth of the fluid mud layer is dependent on the magnitude of the orbital velocities, the time during which the orbital velocities are applied and the strength characteristics of the soil. Once a fluid mud layer has formed on the surface of the slope material would tend to flow down the slope and accumulate near the toe of the slope. This may well be detrimental to the function of a trench as it would decrease the depth of the trench.

The depth of soil resuspended by the orbital motion of a wave is difficult to calculate, although some estimate may be made from the analysis of Laboratory tests conducted at Hydraulics Research and reported by Soulsby (1986). In broad terms, the typical depths of resuspension for small waves in about 10m of water will be in the range 20-80mm. For storm waves the depth of resuspension may well be approaching 150mm. However, it must be appreciated that because the strength of the soil increases with depth, the amount of material which is fluidised and which may then flow down the slope will considerably reduce from one wave event to the next as the exposed surface becomes more and more resistant.

3.3 Currents

A moving stream of water over or across a slope will cause erosion of the surface of the slope. The amount of erosion is broadly a function of the shear stress exerted on the surface by the flowing water and the characteristics of the soil. An estimate of the likely depth of erosion may be made following the procedures given by Delo and Burt (1986). Nevertheless, the overall depth of erosion due to a current (which may well be tidal) will be limited by the maximum shear stress of the current in relation to the shear strength of the soil through the depth. When the exposed surface of the soil has a shear strength equal to the maximum applied shear stress then no erosion will occur.

In rough figures it is unlikely that direct current erosion would remove in total more than 100mm of soil from the surface of the slope. As with waves, the effects of the current would be substantially greater at the top of the slope where the material is naturally weaker. At the toe of the slope, which may well be more than 2m below the original surface, the resistance of the soil to erosion will be greater, perhaps to the extent that no erosion would take place at the toe of the slope.

3.4 Varying water depth

Long period changes in the water depth (ie, tidal) will effect only those slopes which are to some degree emergent. In this case the stable angle of the slope will be less for the emergent saturated part of the slope than for the submerged part of the slope. Provided the changes in the pore pressures in the soil follow the changes in water depth the slope may be analysed by considering it to be in still water.

For long shallow slopes which become emergent it may be necessary to consider the effects of a lag in pore pressure response by analysing the slope for the passage of a long length wave.

4 EXPERIMENTAL FACILITY

Attempts to model soil mechanics problems on a laboratory scale run up against difficulties with the scaling laws involved. The only technique, which has found a reasonable degree of acceptance is the use of centrifuges.

However, a laboratory facility need not claim to be an exact replica of a full-scale prototype. Work can be done on a laboratory scale to establish general laws for soil behaviour. In that case, no scaling is involved, only the assumption that what has been observed on a 0.5m slope in respect of general laws is also valid on a 20m slope. That approach appears to offer the best prospects for our work.

4.1 Review of previous work

Migniot's work (1968) has been described earlier. His observations of the angle of repose of mud slopes were all based on experiments using slopes 0.4m high. Measurements of the rheological properties (ie, viscosity, initial rigidity) were made using a viscometer with rotating cylinders.

Yamamoto et al (1978) report experiments that were performed in order to evaluate the theory which they had developed for wave induced pore pressures. They used a sand bed 0.5m deep with a water depth of 0.9m. The wave period in their tests was between 1 and 2.6 seconds and two types of sand were used, one coarse

(~ 1.2mm) and one fine (~ 0.2mm). They found that their theory was successful in predicting the observed pore pressure response, which showed a definite phase lag in the lower permeability fine sand.

Kroezen et al (1982) report similar experiments carried out at Delft Hydraulics Laboratory to test the theory developed by de Groot and Sellmeijer for pore pressure response in a two layer system. They used a 2.5m high layer of fine, medium dense sand, loaded by standing water waves up to 2.5m high and with periods up to 5 seconds. The average water depth was 5m. The concrete bottom of the flume was taken to represent the second, underlying layer with very stiff and impermeable properties. From the study it was concluded that the analytical calculation method may yield reliable results for relatively dense sands.

Kroezen et al go on to describe experiments to observe flow slides in loose sands. The results were qualitative, rather than providing governing equations for the flow. A 30m long, 3m wide and 2.5m high body of sand was built in a flume. At one end a gate held the sand at a slope angle of 2:1. The gate was suddenly removed, triggering a flow slide. Various measurements, such as pore pressure variation, were taken. The phenomenon was identified as being a hybrid of soil mechanics and hydraulics and a joint effort in those two fields was recommended.

On a larger scale still, Boehmer et al (1983) described full scale dredging tests carried out with cooperation between all parties concerned, to determine feasible slopes either side of a new shipping channel.

4.2 Important parameters

From the work discussed above it is possible to identify the parameters which are necessary for the analysis of a slope:

- soil strength: either undrained, Cu, as measured for example by a miniature vane; or effective stress parameters, c' and ϕ', either measured (eg, in a triaxial cell) or estimated from empirical formulae

- soil density.

Or alternatively

- soil rigidity: analogous to undrained strength.

- soil viscosity: measured by a suitable viscometer.

Also

- particle concentration: can be used as a fundamental parameter and, with various correlations (such as Migniot's), to estimate soil strength, density, etc.

- pore pressures: in a dynamic environment, measured by fast reacting electrical transducers.

- external loading.

- slope geometry.

- currents, wave dimensions, etc.

5 CONCLUSIONS

1. Underwater slope stability involves the consideration of both initial slope stability and also post-construction gradual failure.

2. Soil mechanics techniques are most applicable where soil fails as a moving block. The rheological approach is most applicable where failure is in the form of a flow of material.

3. Within the field of soil mechanics, currently available computer programmes using various methods of slices are suitable for underwater slopes and can also be made to take account of the effects of moving water.

4. The rheological approach, although less developed, can be used, in the form of Morgenstern's work, to examine sediment flows.

5. The physical parameters to be measured in the field, and in experiments, have been listed.

6. Laboratory scale work need not be viewed as an exact replica of a real prototype, but rather as an attempt to establish laws which apply on laboratory slopes and hence, it is assumed, on larger slopes.

7. Although most of the intenest is in slopes of cohesive material, many of the phenomena described may be most easily studied by using coarser material, ie, sands. These have the advantage of requiring less time to consolidate than clays and silts.

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FIGURES.

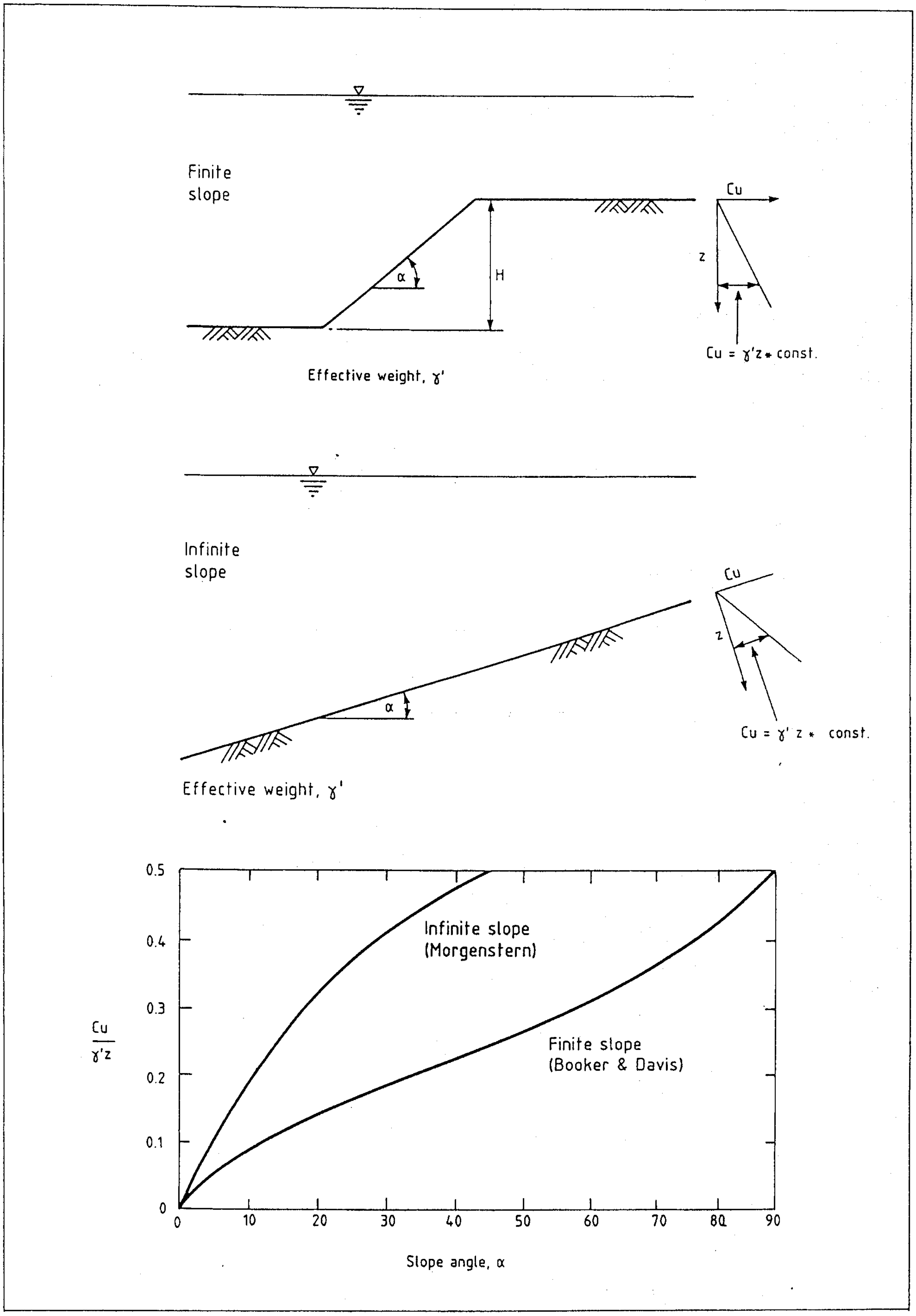


Fig 1 Slope angle and rate of increase in cohesion for finite and infinite slopes

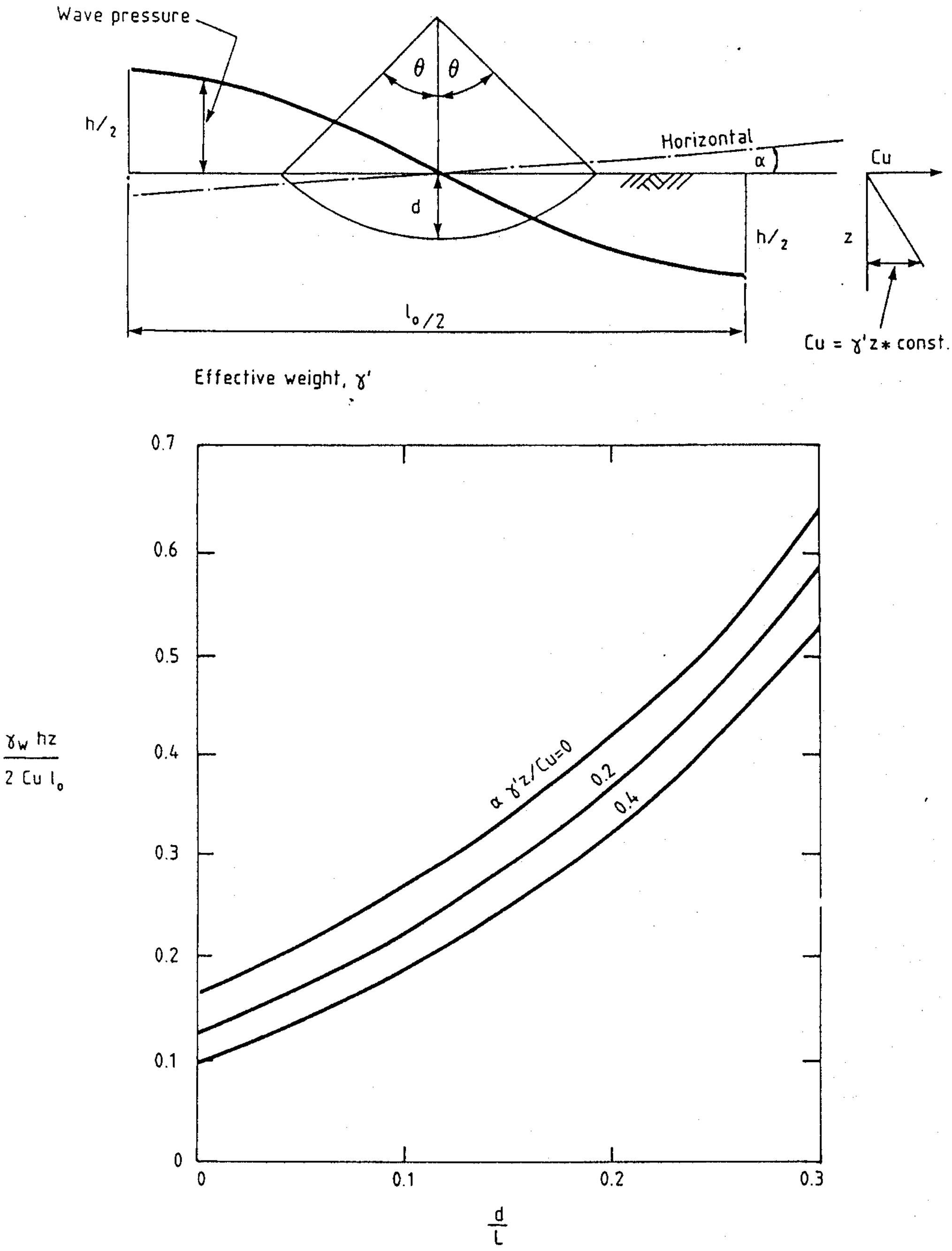
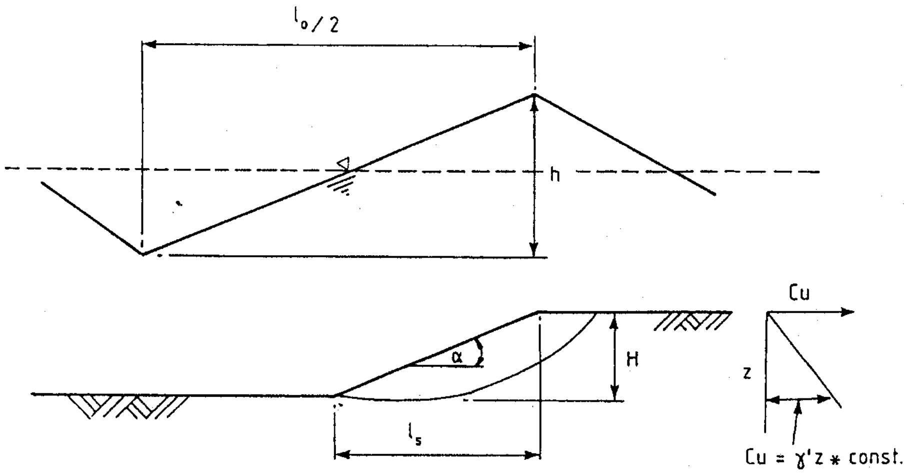


Fig 2 Solution for waves over an infinite slope (Henkel)



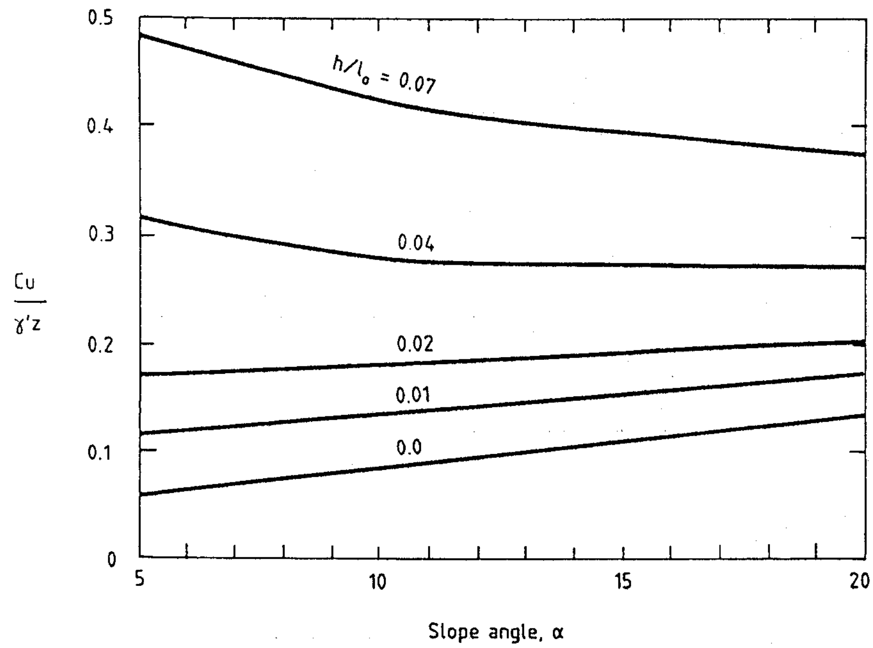


Fig 3 An approximate static solution for waves over a dredged slope

APPENDIX.

APPENDIX A

OUTLINE OF CLASSICAL METHODS OF ANALYSIS

A number of the techniques described below are well established and are described in standard texts, which should be referred to for more details. Capper and Cassie (1954), for example, describe the use of the slip circle and the method of slices for stability analysis. Such techniques can be, and often are, applied to underwater slopes, and they are well suited to the purpose, provided that the water is still.

The choice of a method of analysis should take account of the distribution of measuring soil parameters, especially underwater. A sophisticated technique which uses unreliable data may lead to unmerited confidence.

Slip Circles

This technique is suitable for finite slopes. It is assumed that failure will take place by means of a rigid body of soil sliding on a failure plane which is an arc of a circle. This assumption is justified to a certain extent by the observation that this mode of failure is often the case. It is also, however, a means of simplifying the calculations.

The actual failure plane may be any one of an infinite number of arcs, and the technique involves examining a large number of these in order to find the one with the lowest factor of safety.

For saturated clay soils, where short-term stability is of interest, the undrained shear strength Cu can be used. In this case, the safety factor is found by considering the moment equilibrium of the failing soil under the influence of just two forces - its own self-weight and the shear resistance along the plane of failure. This is known as the $\phi = 0$ method.

For homogeneous, isotropic soils, where an undrained analysis is appropriate, Taylor (1948) has produced charts which enable the user to identify the "least favourable" slip-circle in a straightforward manner.

For granular soils, or in cases where an effective stress treatment is more suitable, the analysis becomes more complicated. The ϕ-circle method can be used for a soil where the effective stress strength parameters, c' and ϕ', are constant. Previously, in the case where undrained strength was used, the normal reaction between the two masses of soil on the failure plane could be overlooked simply by choosing a suitable point about which to take moments. For an effective stress analysis, however, the direction of that reaction is not known.

In the ϕ-circle method the reaction is assumed to act in a certain direction which depends on the magnitude of ϕ'. This assumption allows the problem to be solved. The method is explained at greater length in Capper and Cassie (1954).

Both the $\phi = 0$ method (for undrained strength analysis) and the ϕ-circle (for effective stress) may be suitable for underwater slopes if they consist of homogeneous material. However, this is unlikely to be the case.

Method of Slices

In this method the moving body of soil is divided into many vertical slices and the equilibrium of each slice is considered. The technique can also be applied to non-circular slip surfaces so that judgement can be used in choosing slip planes that, for instance, pass through weak soil. It is still necessary to examine a large number of potential planes of failure in order to identify the most critical one; computer programs are necessary and many exist.

Early versions of the method of slices are suitable for circular slips. The Swedish Circle Method (or Fellenius Method) is an example, but, it has been shown to be over-conservative at times and is not very satisfactory for use on submerged slopes (Whitman and Bailey, 1967).

The Simplified Bishop Method also applies to circular slip circles. The most awkward aspect of the method of slices is the determination of the inter-slice forces and the Simplified Bishop Method overcomes that difficulty by assuming that those forces are horizontal. This can lead to errors in the calculated factors of safety, but these are usually insignificant and are on the safe side (Spencer, 1967). The limitations of the method have been investigated by Whitman and Bailey (1967) who conclude that it can occasionally give misleading answers. An important case is that of a deep slip with a low factor of safety where the toe of the slip surface passes through a frictional material. Computer programmes using this method usually give a warning when a problem of this sort is likely to occur.

Spencer's method is a refinement of Bishop's method. Inter-slice forces are assumed to be parallel rather than simply horizontal. Spencer (1967) has shown that the results usually differ little from those obtained by Bishop, except for steep slopes, where Spencer's method is to be preferred.

Janbu's method (Janbu et al, 1956) is applicable to circular and non-circular slip surfaces. The assumed inter-slice force distribution satisfies overall horizontal and vertical equilibrium, but not moment equilibrium. Errors can be up to 15% (on the safe side) but are small for shallow slopes. Errors increase according to the depth to length ratio of the sliding mass. A correction factor can be applied to obtain an improved factor of safety.

In another form, Janbu's method is similar to Spencer's method but, unlike Spencer's method, it is applicable to non-circular slip surfaces.

A method proposed by Morgenstern and Price (1965) is described by Whitman and Bailey (1967). Judgement is used to choose a distribution of the directions in which interslice forces act. In addition to the factor of safety, the solution also yields the magnitude and the line of thrust of the interslice forces throughout the body. These can be checked for 'reasonableness'.

Whitman and Bailey (1967) investigated this method. They concluded that it involves considerable effort on the part of the engineer but that it can be useful in practical work to study non-circular slips or to check solutions obtained by simpler methods. In research it can be used as a more refined technique.

Techniques of this sort have been used to study underwater slopes. Almagor and Wiseman (1982) have used the Morgenstern and Price method to investigate slumped scar slopes on the continental margin of Israel. Slip circle analyses were used to assess the initial stability of a dredged slope in the full-scale tests described by Boehmer et al (1983).

Wedge Method

When the failure surface can be satisfactorily approximated by two or three straight lines, the Wedge Method can be used, as described in standard texts (eg, Bolton, 1979). The soil is divided into a few wedges and the forces acting on between each of these are used to examine the stability. The technique again needs to be used thoroughly in order to be sure that the most critical failure plane has been identified.

Earthquakes

Almogor and Wiseman (1982) and Hampton et al (1978) have considered the effect of earthquakes on slopes and demonstrate how their effect can be taken into account in the infinite slope model. Earthquakes can also be incorporated into computer programmes for slip circle/method of slices analyses.

More recent methods

The methods described above are relatively well established. More recent work has produced techniques which are more sophisticated.

Booker and Davis (1972) described a plasticity solution to a problem tackled by Gibson and Morgenstern (1962) using a slip circle method; a cutting in clay where the undrained strength increases with depth.

Booker and Davis found that the slip circle method gave results which could be in error by 50% for very shallow slopes (approaching zero degrees), but which erred by no more than 10% for slopes over 5%.

Baker and Garver (1978) investigated the shape of the critical failure surface, which has to be assumed in the standard techniques described so far. They derived some geometrical rules which characterise the failure plane, but their work has not been developed into a usable analysis technique yet.

Leshchinsky et al (1985) described a 3-D mathematical approach to slope stability, based on limiting equilibrium and variational analysis. The technique was applied to some simple cases of homogeneous slopes.