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Influence of the Phreatic Level on the Stability of Earth Embankments

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1. Introduction

Slopes in soils and rocks are common place in nature and man-made structures partly due to the fact that they are generally less expensive than constructing walls. However slope stability problems may arise due to the construction of artificial slopes in cuttings and embankments for roads and railways, or the construction of earth dams and water retaining embankments. Other reasons may include the study of the process of large scale natural slips or the application of remedial measures when such slips have taken place (Capper & Cassie, 1971). Existing slopes that have been stable may experience significant movement due to natural or man-made conditions. Such changes can result from the occurrence of earthquake, subsidence, erosion, the progression of tension or shrinkage cracks coupled with water ingress, changes in groundwater elevation or changes in the slope's subsurface flow which induces new seepage forces (McCarthy, 1998). Further causes may include the removal of earth below the toe of a slope or increased loading close to the crest of the slope. Slips may occur suddenly or gradually, commencing with a crack at the top of an earth embankment and slight upheaval near to the bottom and subsequently developing to a complete slip. All the foregoing actions make slip surface stability analysis of earth embankments complex and very difficult.

Several notable methods of analyzing slip surface stability have been developed over the years. Among the earliest was one that had its slip circle passing through soil materials whose shear strength is based upon internal friction and effective stresses (Fellenius, 1927). In this method an area of unit thickness of the volume tending to slide is divided into vertical strips and it is assumed that for each slice the resultant of the interslice forces is zero. A more significant and certainly more widely used approach utilizing the method of slices assumed a circular failure surface and fulfilled moment equilibrium but did not fully satisfy force equilibrium (Bishop, 1955). Yet another method considered a cylindrical slip surface and assumed that the forces on the sides of the slices are parallel (Spencer, 1967). A generalized approach (Morgenstern & Price, 1965) was developed in which all boundary and equilibrium equations are satisfied and the failure surface could assume any shape. The method involved solving systems of singular simultaneous equations and was unduly long in obtaining approximate answers despite the several assumptions made. An alternative generalized procedure (Bell, 1968) has been advanced which satisfies all conditions of equilibrium and assumed any failure surface. Here a solution is obtained by assuming a distribution of normal stress along the rupture surface. An approach involving the determination of the critical earthquake acceleration required to produce a condition of limiting equilibrium has been developed (Sarma, 1979). Also stability charts have been proposed which were partly based on the work of previous investigators and are applicable to a wide range of practical conditions (Cousins, 1978).

In practically all the afore-mentioned methods, the mass of soil assumed to be associated with the slope slide is divided into vertical slices. A slice is selected, the free body diagram of the forces acting on the slice is drawn and subsequently, based on limit equilibrium methods, an expression is derived for determining the factor of safety of the slope. The stability problem is dealt with by assuming that the tangential interslice forces are equal and opposite (Bishop, 1955). An iterative method for analyzing the stability problem in non-circular slip failures (Janbu, 1973) utilized a full rigorous algorithm and assumed a known line of thrust for the interslice horizontal forces. The method is best suited for computer solution. Similarly analytical solutions have been presented (Morgenstern & Price, 1965) which incorporate all interslice forces but are dependent on several assumptions and are quite lengthy.

1.1 Role of water pressure forces

The action of water is highly significant in slope movement. In clay and shale, softening by rain may lead to slip of a whole layer of material as a mud run. In addition water percolating into fissured clay may result in progressive deterioration and weakening that eventually results in reduction of shear strength so that a rotational or translational slip occurs. Consequently, for the various methods of slices highlighted above, it is important to stress the need to consider the water pressure forces acting not only at the interslice but also at the slice base, for such neglect may produce erroneous results.

More accurate but lower factors of safety are claimed for methods which account for the variation in seepage forces acting on and in the slice (King, 1989). Nevertheless such refinements depend on good estimates of water pressure. The factors of safety obtained for a given slope using different methods of slices are sensitive to the assumptions made in deriving them (Morrison & Greenwood, 1989). Furthermore the interslice forces play an important role in the resulting factors of safety. Hence the present study focuses on the effects of omitting the interslice pore water pressure forces on the overall stability of earth embankments and also endeavours to reduce the complexity features common to the more recent methods of slices outlined earlier.

2. Methodology

In general, any proposed method to evaluate the factor of safety must satisfy several requirements such as fulfilling limit equilibrium laws, account for all forces acting on the slice, adopt few assumptions, which should be easily comprehensible, be applicable to non-homogeneous soils, account for the water pressure distribution at the base of the slice as well as at the interslice, and treat stability problems in terms of both effective and total stresses. Furthermore, it is immaterial whether the horizontal interslice forces are considered as total or effective together with the force due to water pressure. When the force due to water pressure is correctly accounted for at the base as well as on the vertical sides of the slice, then the equilibrium of the body (soil skeleton) and water (hydrostatic) as well as normal and shear forces is maintained.

In order to properly assess the effect of water pressure forces on the stability of earth embankments, two algorithms which utilize both the limit equilibrium approach and method of slices are presented. The algorithms satisfy all the conditions stated above and in addition, the stability problem is treated as a 2-dimensional one in order to arrive at the final solution more quickly. A comprehensive description of the two algorithms and their application to a number of embankments reported in the literature is given elsewhere (Ayininuola & Franklin, 2008). Only the more important features will be considered here. The basic assumptions adopted for the present purpose are as follows: (a) The line of thrust on a slice and the pore water pressure forces are within the slice, preferably at one-third distance from the base or mid-point of the slice vertical sides (b) Factor of safety is defined in terms of the average shear stress developed along the potential failure surface and the average shear strength within the soil (c) Failure occurs simultaneously throughout the soil mass within the assumed rupture boundary.

2.1 Formulation A

Consider an elemental slice in Fig. 1. Due to the many unknown forces acting on the nth slice, the free body diagram in Fig. 2(a) is further divided into two separate bodies (Figs. 2(b) and 2(c)) under the basic requirements for the analysis of earth embankment stability. Since Fig. 2(a) is in equilibrium with all the forces acting on it, consequently Figs. 2(b) and 2(c) are in equilibrium as well.

On examining Fig. 2(a) critically, it is observed that of the twelve forces acting on the slice, only four have known magnitudes. This makes the analysis statically indeterminate of order eight. For the remaining eight forces to be determined there is need to establish logical relationships between the forces. With reference to Fig. 2(c) and letting $\Delta P_{w(n)}$ be the elemental increment of water pressure forces $P_{w(n+1)}$ and $P_{w(n)}$ across the slice, then

$$P_{w(n+1)} - P_{w(n)} = \Delta P_{w(n)} \tag{1}$$

The hydrostatic force U_n is obtained by measuring the free standing height of water in an installed piezometric tube at the slice base. If the piezometric height at the base is $H_{w(n)}$ then

$$U_n = \gamma_w . H_{w(n)} . g \tag{2}$$

where g is the acceleration due to gravity. Alternatively, since the forces acting in Fig. 2(a) are in equilibrium, it implies that the forces in Figs. 2(b) and 2(c) are also in equilibrium. Hence the hydrostatic force U_n at the base of the slice can be determined from the weight of water in the slice. From force equilibrium in the x and y directions the following expressions are obtained:

$$U_n = W_{w(n)} \sec \alpha_n \tag{3}$$

$$\Delta P_{w(n)} = -U_n \sin \alpha_n \tag{4}$$

where $W_{w(n)}$ is the weight of water in the nth slice and α_n is angle at the base of the slice. Substituting the value of U_n in equation (3) into equation (4) yields

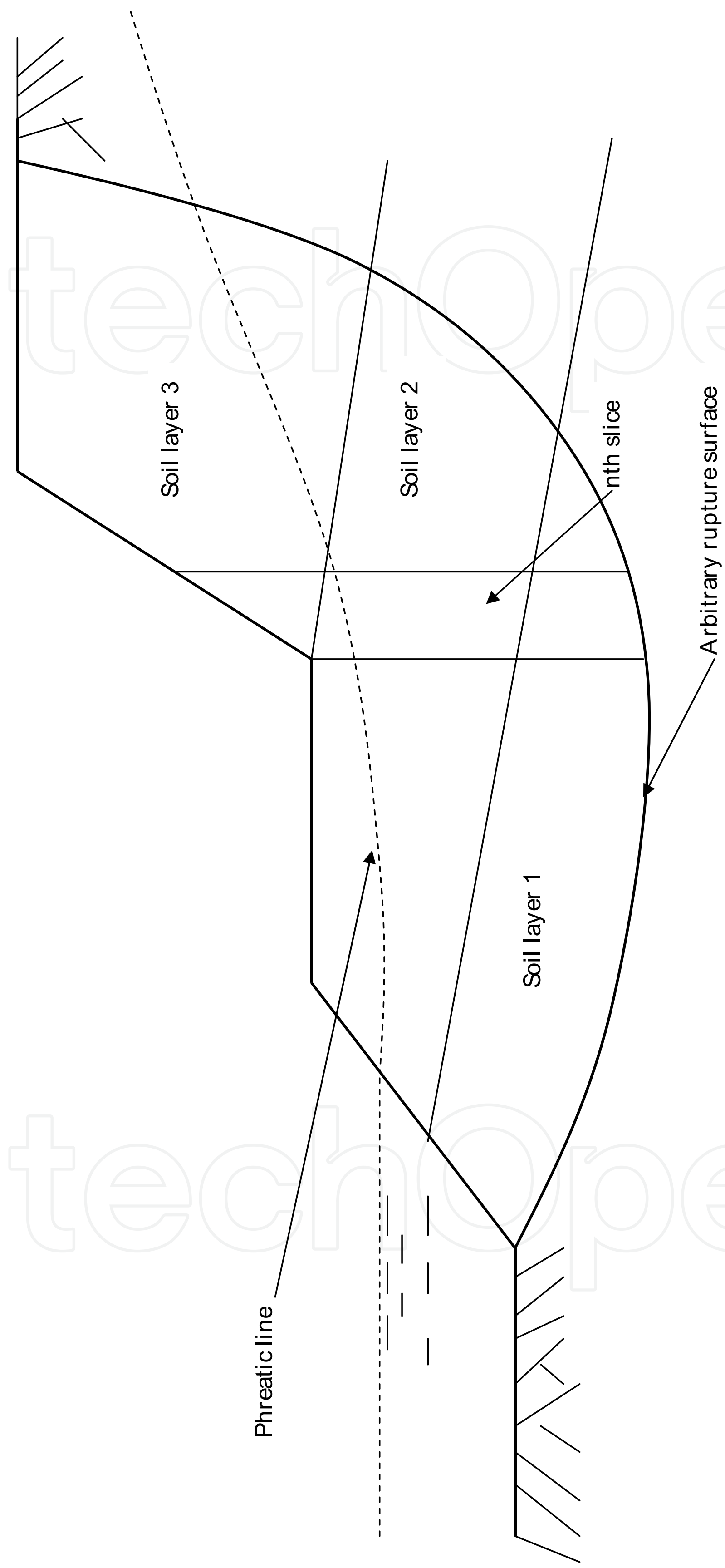

Fig. 1. Cross-section of an earth embankment made of non-homogeneous strata or soil layers

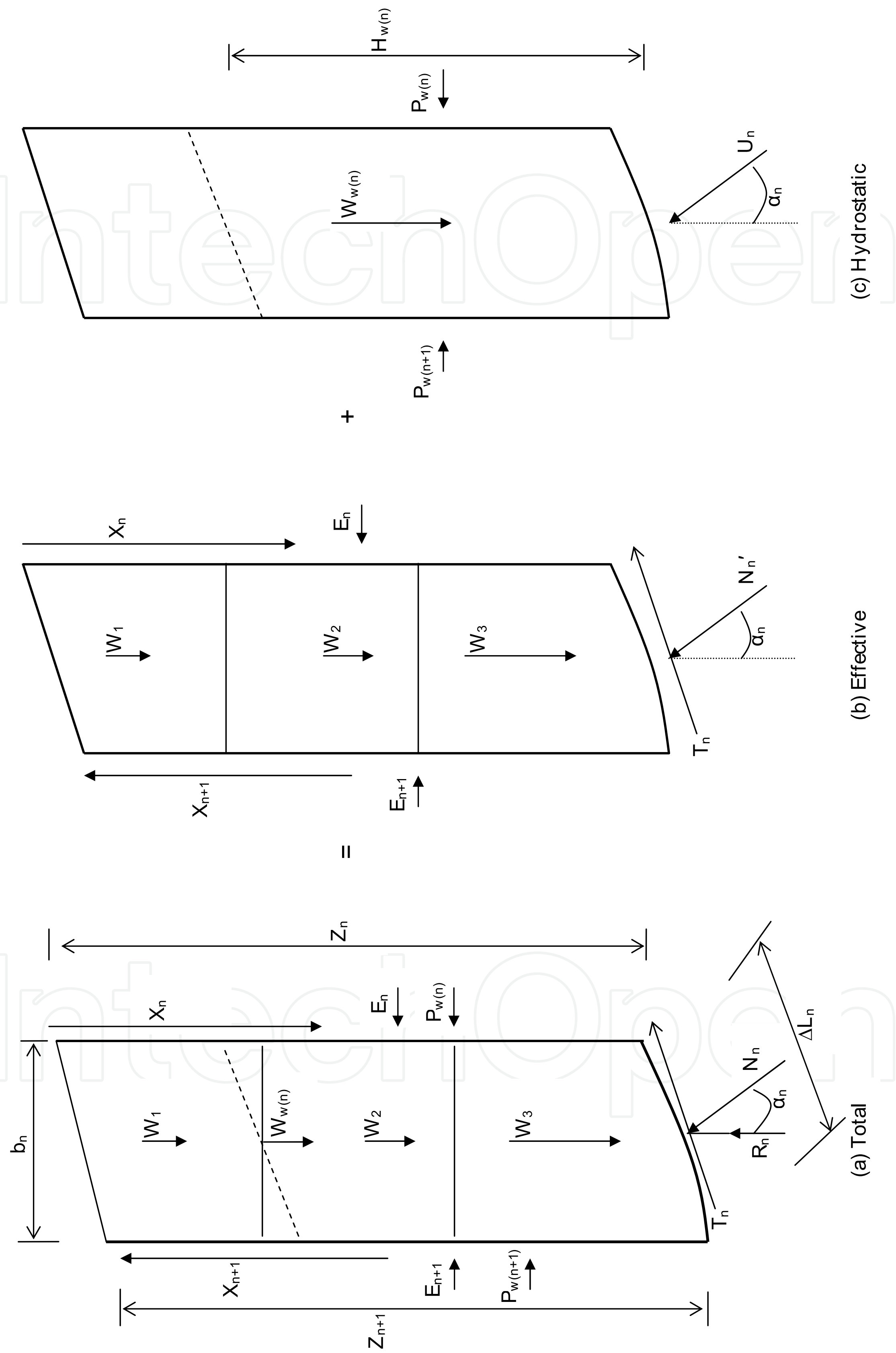

Fig. 2. Forces acting on an nth slice

$$\Delta P_{w(n)} = -W_{w(n)} \tan \alpha_n \tag{5}$$

From Fig. 2(b) let ΔE_n and ΔX_n be the elemental increments of horizontal interslice lateral thrusts E_n, E_{n+1} and vertical interslice shear forces X_n, X_{n+1} respectively across the slice, that is

$$\Delta E_n = E_n - E_{n+1}\,, \quad \Delta X_n = X_n - X_{n+1} \tag{6}$$

In Fig. 2(a) since the total normal force N_n and shear force T_n acting at the base of the slice are orthogonal, they have the same resultant R_n. In order to reduce the number of unknowns at the base of the slice the forces T_n and N_n are expressed in terms of R_n as follows:

$$T_n = R_n \sin \alpha_n\,, \; N_n = R_n \cos \alpha_n \tag{7}$$

It should be noted at this stage that the total and effective normal forces, N_n and N_n' respectively, are related to U_n as follows:

$$N_n' = N_n - U_n \tag{8}$$

Limit equilibrium laws can be applied to the slice in Fig. 2(a). Firstly resolving the forces acting on the slice in the N_n-direction yields

$$N_n - W_n \cos \alpha_n - \Delta X_n \cos \alpha_n + \Delta E_n \sin \alpha_n + \Delta P_{w(n)} \sin \alpha_n - W_{w(n)} \cos \alpha_n = 0 \tag{9}$$

Substituting the values of $\Delta P_{w(n)}$ and N_n from equations (5) and (7) respectively into equation (9) and re-arranging yields

$$R_n \cos \alpha_n + \Delta E_n \sin \alpha_n - \Delta X_n \cos \alpha_n = W_{w(n)} \left(\cos \alpha_n + \tan \alpha_n \sin \alpha_n \right) + W_n \cos \alpha_n \tag{10}$$

Also, resolving the forces acting on the slice in the T_n-direction yields

$$T_n - W_n \sin \alpha_n - \Delta X_n \sin \alpha_n - \Delta E_n \cos \alpha_n - W_{w(n)} \sin \alpha_n - \Delta P_{w(n)} \cos \alpha_n = 0 \tag{11}$$

Substituting the values of $\Delta P_{w(n)}$ and T_n from equations (5) and (7) into equation (11) and simplifying and re-arranging results in

$$R_n \sin \alpha_n - \Delta X_n \sin \alpha_n - \Delta E_n \cos \alpha_n = W_n \sin \alpha_n \tag{12}$$

Examination of equations (10) and (12) reveals that there are three unknowns in the two equations which render them indeterminate; in order to solve for these unknowns it is necessary at this stage to introduce an equilibrium equation based on moments. From Fig. 2(b), taking moments of the resultants of the interslice forces and other forces about the mid-point of the slice base width gives

$$\Delta X_n . b_n/2 - \Delta E_n \left\{ Z/3 + \left(b_n/2 \right) \tan \alpha_n \right\} - \Delta P_{w(n)} \left(H_{w(n)}/3 \right) = 0 \tag{13}$$

where Z is the elevation of one side of the slice, or more correctly, the greater of Z_n and Z_{n+1}, and $H_{w(n)}$ is the height of water in the nth slice. Substituting for the value of $\Delta P_{w(n)}$ from equation (5) into equation (13) and re-arranging results in

$$-\Delta E_n\left\{Z/3+\left(b_n/2\right)\tan\alpha_n\right\}+\Delta X_n.b_n/2=-W_{w(n)}\tan\alpha_n\left(H_{w(n)}/3\right) \tag{14}$$

Equations (10), (12) and (14) can now be assembled together as a set of simultaneous equations in the following form

$$\left.\begin{array}{c} R_n\cos\alpha_n+\Delta E_n\sin\alpha_n-\Delta X_n\cos\alpha_n=W_{w(n)}\left(\cos\alpha_n+\tan\alpha_n\sin\alpha_n\right)+W_n\cos\alpha_n \\ R_n\sin\alpha_n-\Delta X_n\sin\alpha_n-\Delta E_n\cos\alpha_n=W_n\sin\alpha_n \\ -\Delta E_n\left\{Z/3+\left(b_n/2\right)\tan\alpha_n\right\}+\Delta X_n.b_n/2=-W_{w(n)}\tan\alpha_n\left(H_{w(n)}/3\right) \end{array}\right] \tag{15}$$

In matrix format the simultaneous equation (15) becomes

$$\begin{gathered}\begin{bmatrix}\cos\alpha_n & \sin\alpha_n & -\cos\alpha_n \\ \sin\alpha_n & -\cos\alpha_n & -\sin\alpha_n \\ 0 & -\{Z/3+(b_n/2)\tan\alpha_n\} & b_n/2\end{bmatrix}\begin{bmatrix}R_n \\ \Delta E_n \\ \Delta X_n\end{bmatrix} \\ =\begin{bmatrix}W_{w(n)}(\cos\alpha_n+\tan\alpha_n\sin\alpha_n)+W_n\cos\alpha_n \\ W_n\sin\alpha_n \\ -W_{w(n)}\tan\alpha_n(H_{w(n)}/3)\end{bmatrix}\end{gathered} \tag{16}$$

In compact form this can be written as

$$K.D=F \tag{17}$$

$$\text{where } K=\begin{bmatrix}\cos\alpha_n & \sin\alpha_n & -\cos\alpha_n \\ \sin\alpha_n & -\cos\alpha_n & -\sin\alpha_n \\ 0 & -\left\{Z/3+\left(b_n/2\right)\tan\alpha_n\right\} & b_n/2\end{bmatrix}, \qquad D=\left[R_n \quad \Delta E_n \quad \Delta X_n\right]^{-1}$$

$$\text{and } F=\left[W_{w(n)}\left(\cos\alpha_n+\tan\alpha_n\sin\alpha_n\right)+W_n\cos\alpha_n \quad W_n\sin\alpha_n \quad -W_{w(n)}\tan\alpha_n\left(H_{w(n)}/3\right)\right]^{-1}$$

In the above expressions the matrices D and F represent the nodal unknown forces and the nodal applied forces respectively.

Equation (17) above is for the nth slice, and when such sets of equations are assembled for the whole rupture mass this yields

$$K_g.D_g=F_g \tag{18}$$

where K_g is the global stiffness matrix, D_g is the global forces matrix and F_g is the global applied forces matrix. The simultaneous equation (18) can be solved for various values of R_n, ΔE_n and ΔX_n using the Gaussian elimination, Jacobi or Gauss-Siedel iterative techniques. The Gaussian elimination method is, relatively speaking, the simplest and easiest of the three procedures to implement. Once the values of R_n are obtained, the values of N_n and T_n can also be found using equations (7) and (8).

The factor of safety can be defined in terms of the shear strength of the soil and the shear stress developed along the potential failure surface based on the Coulomb-Mohr failure criteria in terms of effective stress as follows:

$$T_n = \left(c_n' \Delta L_n + N_n' \tan\phi_n'\right) \Big/ F_s \tag{19}$$

where F_s is the factor of safety and ϕ_n' is the angle of shearing resistance with respect to effective stress. Substituting for the values of T_n and N_n' from equations (7) and (8) into equation (19) yields

$$R_n \sin\alpha_n = \left[c_n' \Delta L_n + \left(R_n \cos\alpha_n - U_n\right) \tan\phi_n'\right] \Big/ F_s \tag{20}$$

Also substituting the value of U_n from equation (3) into equation (20) and then considering the whole of the rupture mass consisting of the set of slices will give

$$\sum R_n \sin\alpha_n = \sum\left[c_n' \Delta L_n + \left(R_n \cos\alpha_n - W_{w(n)} \sec\alpha_n\right) \tan\phi_n'\right] \Big/ F_s \tag{21}$$

Making F_s the subject of the expression in equation (21) will yield

$$F_s = \frac{\sum\left[c_n' \Delta L_n + \left(R_n \cos\alpha_n - W_{w(n)} \sec\alpha_n\right) \tan\phi_n'\right]}{\sum\left(R_n \sin\alpha_n\right)} \tag{22}$$

Equation (22) can be use to analyze stability problems involving both homogeneous and non-homogeneous soils types.

Irrespective of whether the earth embankment is partially or wholly drained, the equation can be applied because during its formulation both states of stress were taken into account.

2.2 Formulation B

The present study seeks to investigate the effect of hydrostatic pore water pressure forces on the overall stability of earth embankments and as such, in order to establish a basis of comparison with the earlier algorithm presented, an alternative approach is developed. This treats stability problems in terms of effective stresses and assumes that the influence of water pressure forces acting at the interslice can be neglected. The lines of action of $P_{w(n+1)}$ and $P_{w(n)}$ are taken to be coincident and also $\Delta P_{w(n)} = 0$. Proceeding along the same lines as the previous formulation, the following set of simultaneous equations can be arrived at:

$$\left.\begin{array}{c} R_n \cos\alpha_n + \Delta E_n \sin\alpha_n - \Delta X_n \cos\alpha_n = \left(W_{w(n)} + W_n\right) \cos\alpha_n \\ R_n \sin\alpha_n - \Delta X_n \sin\alpha_n - \Delta E_n \cos\alpha_n = \left(W_{w(n)} + W_n\right) \sin\alpha_n \\ -\Delta E_n \left\{Z/3 + \left(b_n/2\right) \tan\alpha_n\right\} + \Delta X_n . b_n/2 = 0 \end{array}\right] \tag{23}$$

For the typical nth slice the above equation in matrix format becomes

$$\begin{bmatrix} \cos\alpha_n & \sin\alpha_n & -\cos\alpha_n \\ \sin\alpha_n & -\cos\alpha_n & -\sin\alpha_n \\ 0 & -\{Z/3+(b_n/2)\tan\alpha_n\} & b_n/2 \end{bmatrix} \begin{bmatrix} R_n \\ \Delta E_n \\ \Delta X_n \end{bmatrix} = \begin{bmatrix} (W_{w(n)}+W_n)\cos\alpha_n \\ (W_{w(n)}+W_n)\sin\alpha_n \\ 0 \end{bmatrix} \tag{24}$$

Again proceeding along the same lines as the previous formulation, an expression very similar to equation (22) can be obtained as follows:

$$F_s = \frac{\sum\left[c_n'\Delta L_n + \left(R_n\cos\alpha_n - W_{w(n)}\sec\alpha_n\right)\tan\phi_n'\right]}{\sum\left(R_n\sin\alpha_n\right)} \tag{25}$$

Although equations (25) and (22) are very similar, the procedures for evaluating the values of R_n in both equations are certainly not the same. Consequently different values of factors of safety will be obtained using both approaches. The methods developed can be used for slip surface stability analysis either manually or with a programmable calculator. However while this may be true for fairly homogeneous slopes, for real or non-homogeneous soils the computation work is quite daunting for practical design. This is on account of the number of rupture surfaces that may need to be analyzed in order to obtain the most critical rupture surface for design purposes as well as the fact that the global stiffness matrix K_g mentioned earlier may be of the order 60 x 60 or more, depending on the number of slices within the rupture mass. Hence comprehensive computer software was developed involving two minimization computer programmes which can handle problems of up to three soil strata; some details of the programmes are given elsewhere (Ayininuola, 1999).

3. Results

In order to assess the effect of the pore water pressure forces, the procedures developed in the present study have been applied to a number of earth embankments some of which are reported in the literature. Firstly, the Lodalen Landslide (Sevaldson, 1956) is examined and then, the case of a non-homogeneous earth dam (Sherard et al, 1978) is investigated. Finally the effect of altering the phreatic level on the Okuku dam in South-Western Nigeria (Ayininuola & Franklin, 2008) is studied.

3.1 Stability analysis of the Lodalen Landslide (Sevaldson, 1956)

Fig. 3 shows a sectional view of the Lodalen Landslide. A stability analysis of the earth embankment prior to the occurrence of the slide will be carried out. Towards this end the initial rupture surface has been divided into 13 slices. The necessary data have been taken from the initial rupture surface and fed into the computer programme mentioned earlier. A total of 100 rupture surfaces have been considered in the analysis. Details of the computer output are not presented here, but a summary of the main findings are shown in Table 1 and these results are discussed at a later stage.

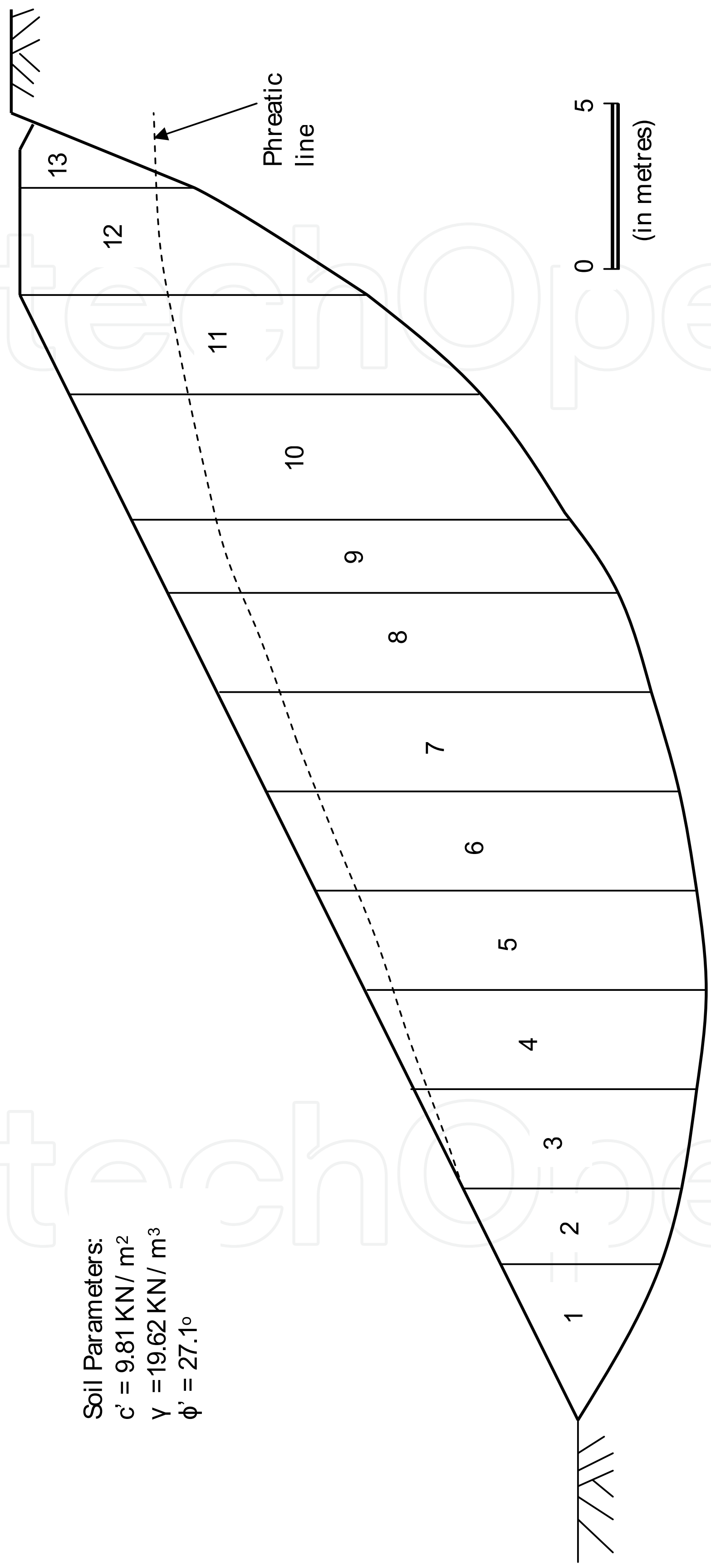

Fig. 3. Re-examination of the Lodalen slide (Modified from Sevaldson, 1956)

3.2 Stability analysis of a non-homogeneous earth dam (Sherard et al, 1978)

A non-homogeneous earth dam is shown in section in Figs. 4 and 5 and it is required to carry out a stability analysis of both the upstream and downstream sides of the dam. The assumed initial rupture surface on both sides of the dam has been divided into 12 slices each and the necessary data taken from the rupture surfaces have been fed into the computer programme referred to earlier. A total of 200 rupture surfaces have been considered at both the upstream and downstream sections. Again details of the computer output are not given in the present study, however a summary of the main findings are shown in Table1.

Earth Embankment	Method	Factor of Safety
Lodalen slide (Sevaldson, 1956)	Authors' Formulation A Authors' Formulation B Bishop's Simplified	0.80 0.90 0.90
Downstream section of non-homogeneous earth dam (Sherard et al, 1978)	Authors' Formulation A Authors' Formulation B Bishop's Simplified	1.49 1.83 1.83
Upstream section of non-homogeneous earth dam (Sherard et al, 1978)	Authors' Formulation A Authors' Formulation B Bishop's Simplified	0.66 0.78 0.78

Table 1. Results of stability analysis of Lodalen slide and a non-homogeneous earth dam

3.3 Stability analysis of Okuku earth dam, Nigeria

On account of the accessibility to data, the Okuku dam has been utilized as a case study in order to investigate and understand the response of the proposed formulations to changes in the phreatic levels in the earth embankment due to variation in water levels in the storage reservoir. The dam was constructed in 1995 at Okuku town located on the 8^o 02'N and 4^o 40'E coordinates and approximately 40 km North-East of Osogbo in Osun State, South-Western Nigeria. The dam axis located across River Anle, a seasonal stream, is about 1.5 km South-East of Okuku town. The dam is a homogeneous earth dam built with poorly graded sand clay mixtures which possess the following soil characteristics, namely, cohesion $c' = 45$ KN/m^2, angle of shearing resistance $\phi' = 12^o$ and additionally, average dry density of dam construction materials, $\gamma = 19.63$ KN/m^3. The height of the crest above the base of the dam is 10 metres and the upstream and downstream sections are sloped at ratios 1:3 and 1:2.5 respectively. In Figs. 6 and 7, diagrams of the dam embankment for both the upstream and downstream sections at different levels of water in the storage reservoir are shown. Additional details in respect of the dam design may be found elsewhere (Ayininuola, 1999). The factors of safety at different phreatic levels for both the upstream and downstream sections of the dam have been estimated using the two formulations developed earlier. A total of 500 rupture surfaces have been examined for each section. A summary of the results is presented in Table 2 and Figs. 8 and 9.

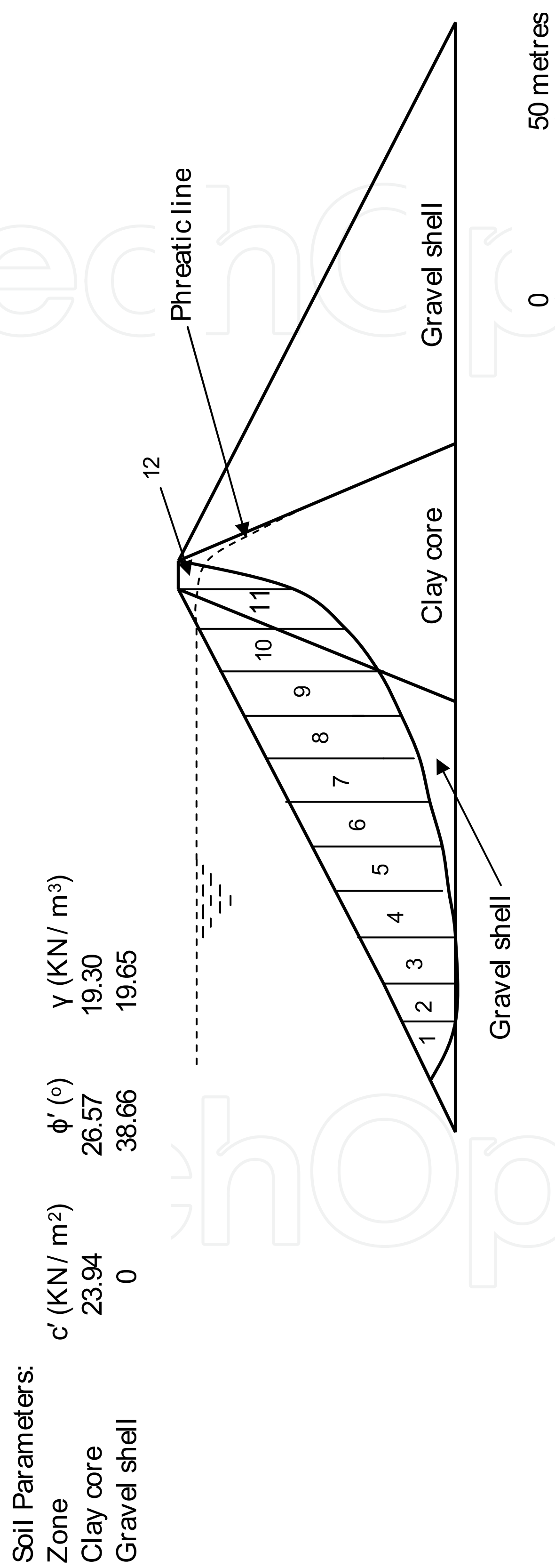

Fig. 4. Upstream section of an earth dam embankment (Modified from Sherard et al, 1978)

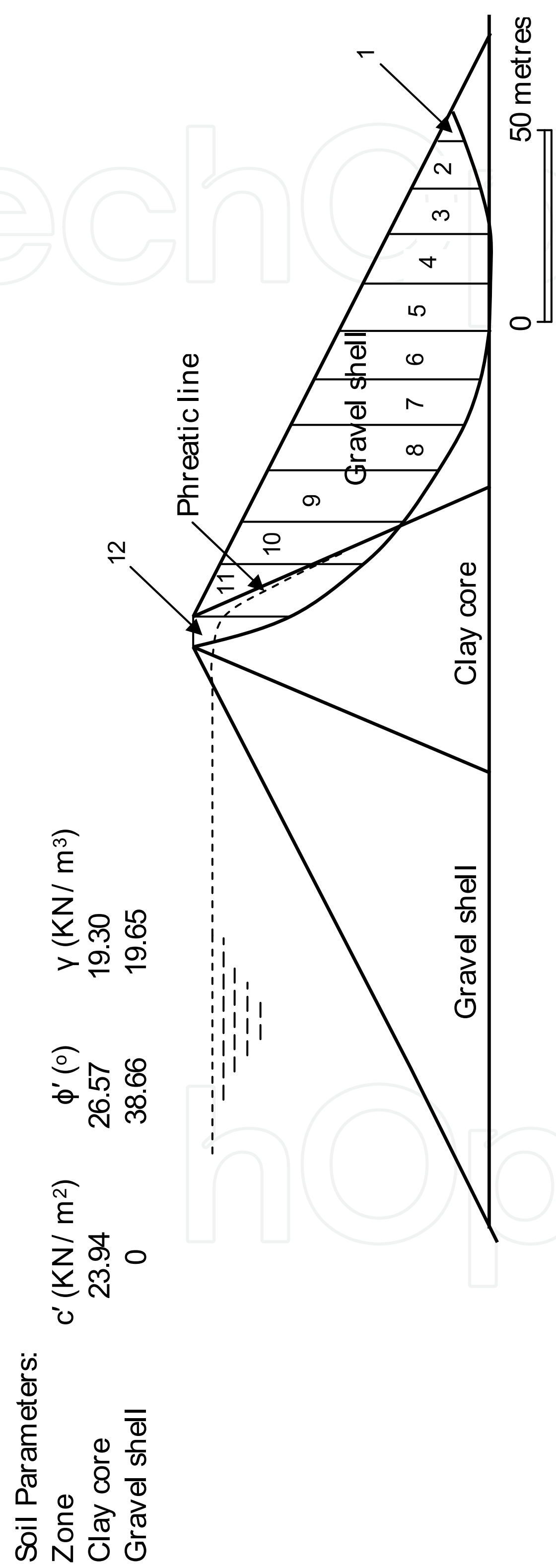

Fig. 5. Downstream section of an earth dam embankment (Modified from Sherard et al, 1978)

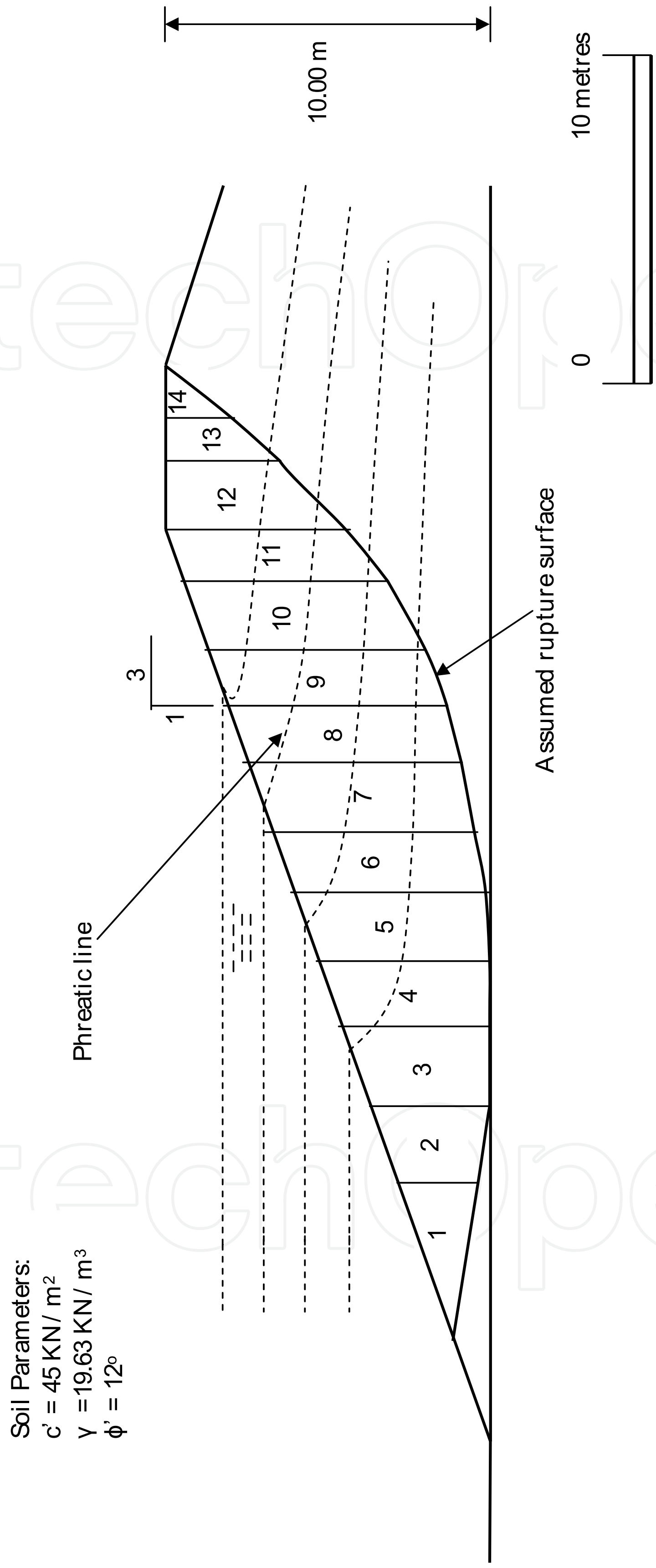

Fig. 6. Upstream section of Okuku earth dam (Courtesy Konsadem Associates Ltd., Nigeria)

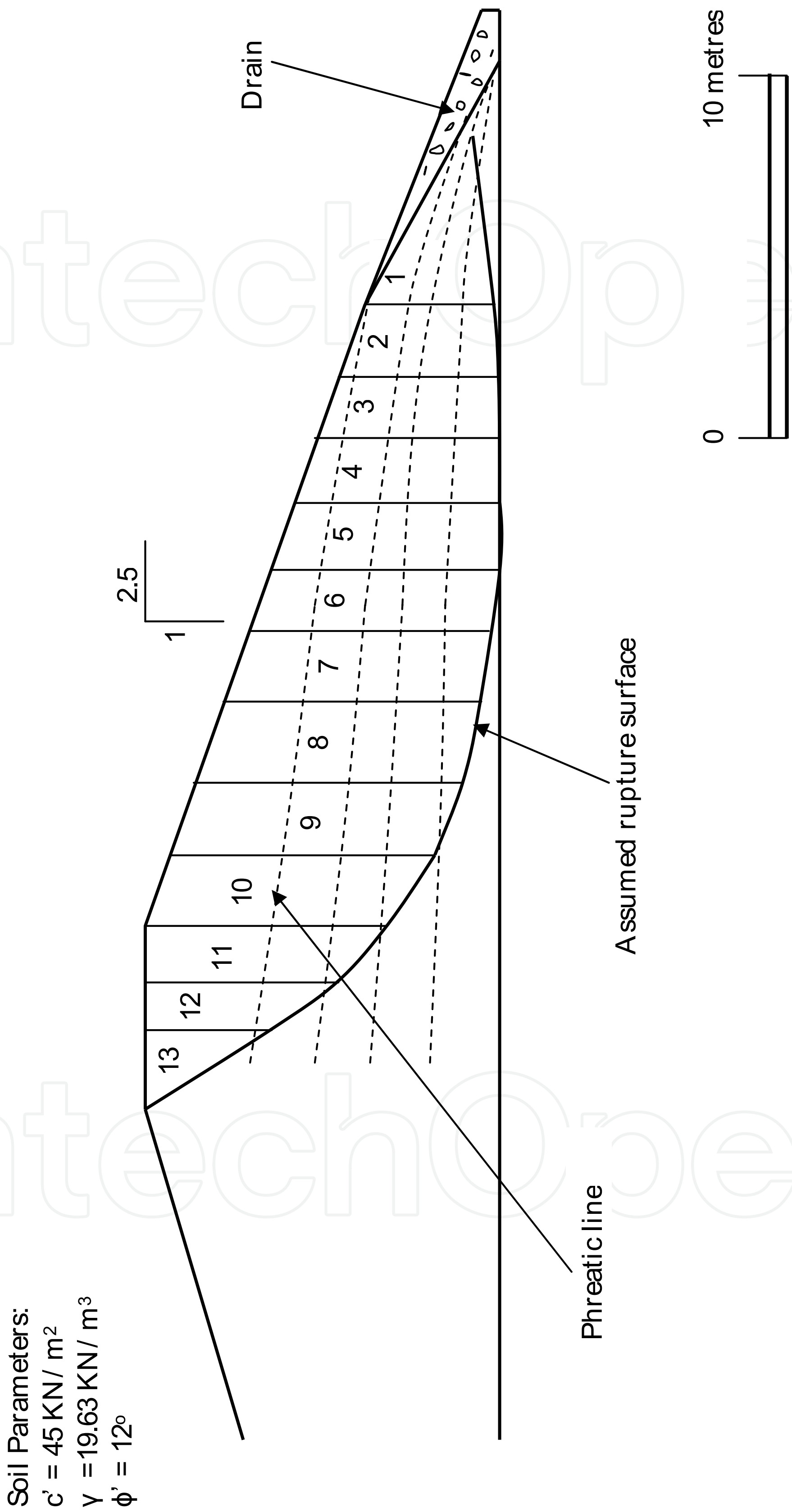

Fig. 7. Downstream section of Okuku earth dam (Courtesy Konsadem Associates Ltd., Nigeria)

Section of dam under consideration	Water level (metres)	Stability values for Formulation A	Stability values using formulation B	Difference (%)
Upstream	9.00* 6.55* 5.25* 3.70*	2.43 2.60 2.81 3.01	2.78 2.85 2.93 3.03	14.40 9.62 4.27 0.66
Downstream	9.00** 6.55** 5.25** 3.70**	2.25 2.53 2.74 2.89	2.67 2.76 2.83 2.90	18.67 9.09 3.28 0.35

Table 2. Results of stability analysis of Okuku Dam using Formulation A and B

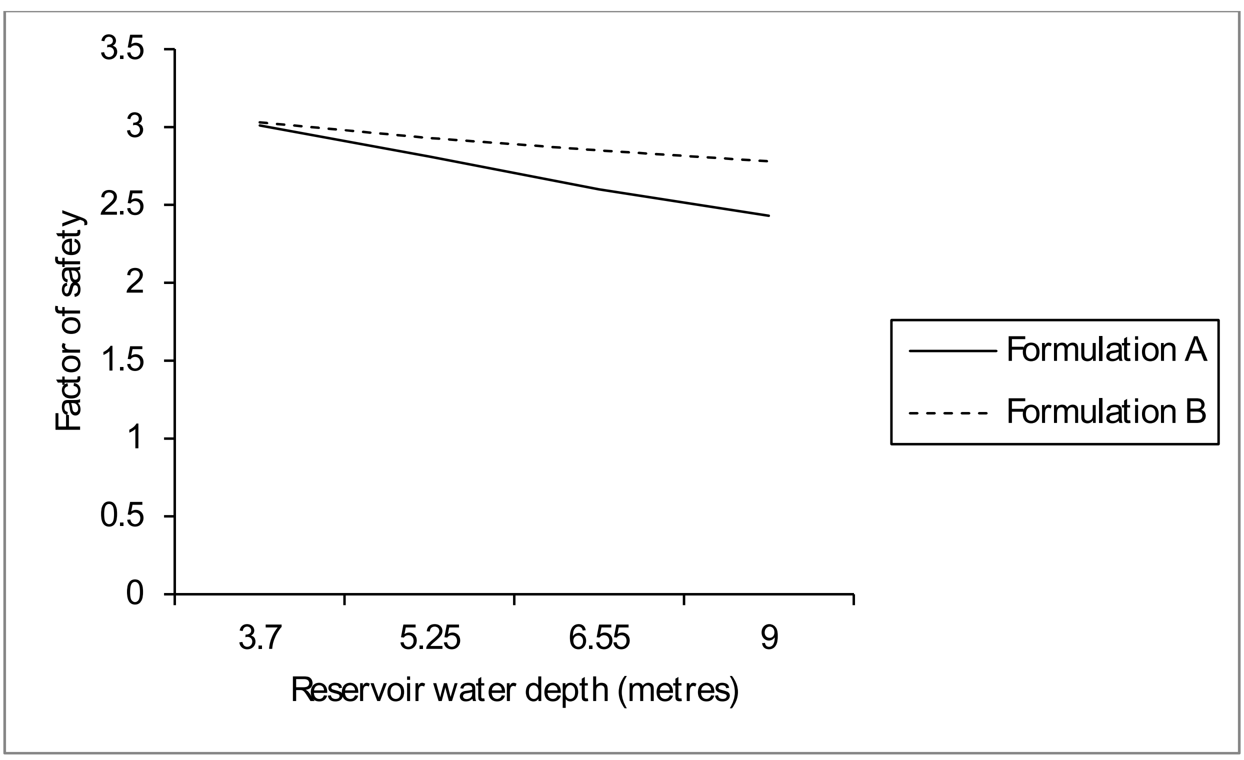

Fig. 8. Variation of factor of safety of dam embankment with reservoir water depth (upstream section)

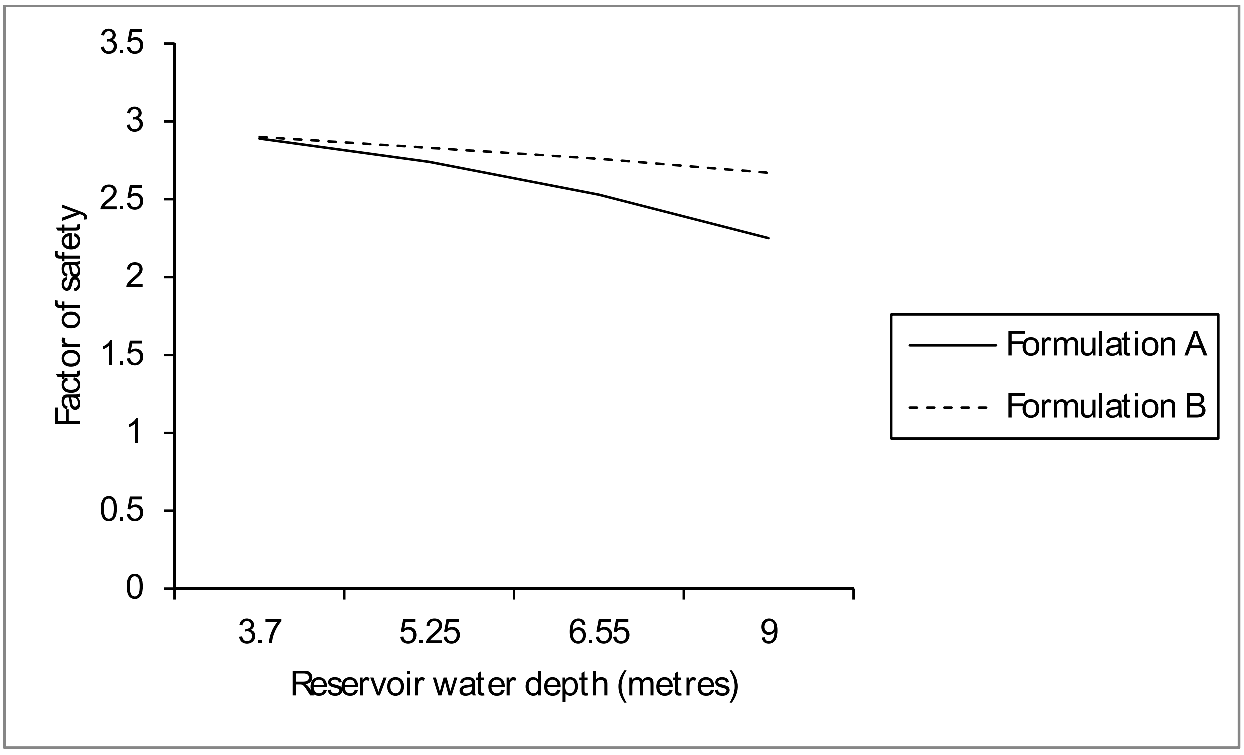

Fig. 9. Variation of factor of safety of dam embankment with reservoir water depth (downstream section)

4. Discussion

The comments outlined here are based primarily on the results presented in Tables 1 and 2, as well as in Figs. 8 and 9. As noted in the preceding section, details of the computer output in respect of the stability analysis carried out are not presented here, but may be obtained elsewhere (Ayininuola & Franklin, 2008; Ayininuola, 1999).

4.1 Comparison between formulations A and B and Bishop's simplified method

The procedures developed in the present study as well as Bishop's have been applied to the Lodalen landslide as well as the non-homogeneous earth dam described in the preceding section. Using Table 1 and the results of the computer generated output for the Lodalen landslide as guide, several points may be noted. Firstly the water pressure forces acting at the interslice have great influence on the elemental horizontal thrusts generated at the interslice. They also have a direct influence on the elemental shear forces at the interslice. In this region when the elemental water pressure forces are assumed to be zero, the values of the elemental horizontal thrusts and shear forces that develop are much smaller than those obtained when water pressure forces are taken into account.

In addition to the above, water pressure forces, elemental horizontal thrusts and elemental shear forces are directly affected by the slice inclination angles. When for example the inclination angle is zero, the effect of all the forces mentioned is practically negligible. At the interslice when the piezometric height $H_{w(n)}$ is zero, the values of elemental horizontal thrust and elemental shear force are zero. Furthermore the factors of safety obtained when the net effect of water pressure forces at the interslice is taken to be zero are higher than the corresponding values when the afore-mentioned forces are considered by the order of 8% – 24% depending on the phreatic level. Also the results given by Formulation B, which ignores the water pressure forces effect at the interslices, are practically identical to those obtained using Bishop's method.

4.2 Influence of the phreatic level on earth dam stability

With reference to Table 2 and Figs. 8 and 9, the results of stability analysis of Okuku earth dam reveal that the higher the phreatic level in the storage reservoir of the earth dam, the greater the variation between the stability values obtained using Formulations A and B. When the water pressure forces are ignored, higher factors of safety of the order of 0.35% – 18.67% are obtained. With regards to the curves in Figs. 8 and 9, and also from a study of the computer generated output, it is observed that at low phreatic levels in the storage reservoir of between 20% and 30% of the dam height, both approaches yield similar stability values.

5. Conclusions

An in-depth study of the effect of the pore water pressure forces acting at the interslice on the stability of earth dams has been carried out. This has been achieved by developing two procedures and applying the formulations to a number of practical cases. Based on the results of the investigation, a number of conclusions can be drawn: Firstly the magnitudes of effective horizontal thrusts and shear forces generated at the interslice when pore water pressure forces induced are taken into consideration are higher than those obtained when these forces are ignored. This demonstrates that the pore water pressures developed have an influence on the values of other interslice forces. Secondly the inclusion of net water pressure forces in the stability analysis of the earth embankments studied clearly show that the action of the water pressure forces serves to promote instability, as would be expected. Thirdly the popular practice amongst geotechnical engineers of resolving the water pressure within a given slice in a direction of normal at the slice base in order to estimate its value, whilst the horizontal effect of the slice base water pressure is taken as zero, constitutes a grave error. This action is not in line with limit equilibrium procedures and yields erroneous results. Fourthly at low phreatic levels the proposed approaches give practically similar factors of safety. This implies that the effect of water pressure forces acting on the interslice can only be neglected when the phreatic line in an embankment is at its lowest stage, a considerable period after drawdown, or preferably between 20% and 30% of the overall height of the embankment. Finally the factors of safety found using Bishop's simplified method and that based on Formulation B, which ignores the pore water pressure forces effect, are in very close agreement. This simply implies that the inclusion of only the effective horizontal thrusts and shear forces acting at the interslice has little influence on the resulting factors of safety.

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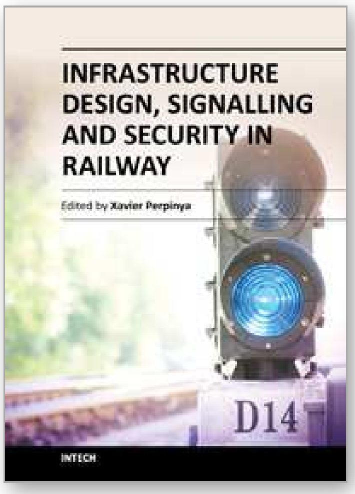

Infrastructure Design, Signalling and Security in Railway
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Railway transportation has become one of the main technological advances of our society. Since the first railway used to carry coal from a mine in Shropshire (England, 1600), a lot of efforts have been made to improve this transportation concept. One of its milestones was the invention and development of the steam locomotive, but commercial rail travels became practical two hundred years later. From these first attempts, railway infrastructures, signalling and security have evolved and become more complex than those performed in its earlier stages. This book will provide readers a comprehensive technical guide, covering these topics and presenting a brief overview of selected railway systems in the world. The objective of the book is to serve as a valuable reference for students, educators, scientists, faculty members, researchers, and engineers.