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SLOPE STABILITY ANALYSIS USING COMPUTER SOFTWARE FOR BLACK COTTON SOIL OF NORTH - EASTERN NIGERIA

Ashiru M. Ashiru, Jacob E. Sanni and Sule Mohammed
Department of Civil Engineering Technology,
The Federal Polytechnic, Nasarawa, Nasarawa state, Nigeria.
E-mail: ashiruashiru@yahoo.com

Abstract: Black cotton soils found across the globe have been described by researchers as problematic soils due to the present of montmorillonite and kaolinite minerals in large quantities. Its swell-shrink movement has been reported to pose serious problems to engineering structures and lives from landslides failures. This research work focuses on stability analysis of slope using limit equilibrium method as a technological solution to the adverse effect of these routine slope failures. The study utilises three limit equilibrium methods: stability charts, SLOPE/W software and traditional methods. There is no difference in the application of these analyses because they are all formulated based on the static of equilibrium and slices discretisation. The results obtained under dry condition from the entire methods shows that, the slope is safe; although, at wet situations, all methods indicated instability. Comparing the results between techniques shows that Bishop's simplified method is similar with Morgenstern Price's method. Furthermore, the results from Janbu and Ordinary method in some situations are similar. It has been demonstrated that loads, pressures and shear strength parameters significantly influenced the factor of safety for critical slip surfaces and the stability state of a slope.

Keywords: Black Cotton Soils, Swell, Shrink, Problematic, Montmorillonite, Kaolinite, Minerals, Slope, Stability, Limit Equilibrium, Slice, Factor of Safety, and Software.

INTRODUCTION

The vulnerability and the problems to the repeated loss of lives and properties asserted by land-slip failure due to the swell – shrink behaviour of Black Cotton soil over decades can never be over emphasised. The spacious extent of issues arises from landslide failure has over years, been a problem to the field of geology and geotechnical engineering. Although the damages caused by landslides in Nigeria and some other countries have not been recorded, in the United State of America these are estimated to cause $ 1-2 billion worth of damage and over 25 fatalities each year (USGS, 2009).

As the name indicates, black cotton soil (BCS) derived its name from the fact that the cotton plants thrives well in it (Ijimdiya, Ashimiyu and Abubakar, 2012). Black cotton soil occurs principally in hot temperate environments, in arid and semi arid areas with remarkable alternating wet and dry seasons, it is a product of igneous and sedimentary rocks. The soil is an expansive type of clay that exhibit swell-shrink characteristics, with cracks at the surface during the dry period due to high volume of montmorillonite in its mineralogical content, and hence makes it unsuitable for construction of engineering structures such as embankment, roads and buildings (Bowles, 1979 and Das, 1998). Black cotton soil is found in the North - Eastern parts of Nigeria, Lake Chad Basin, Cameroon, Ethiopia, Sudan, Southern Zimbabwe, Kenya, Zimbabwe and other Eastern African countries, India, Australia, South Western U.S.A., South Africa and Israel (Ola, 1978 and Osinubi, 2007).

In Nigeria, road network constructed over black cotton soil in the North-Eastern areas poses a difficult problem due to expansive characteristics of the material and lack of drainage (Ola, 1978). The devastating behaviour of the troublesome material such as collapsing behaviour, dispersive characteristics, remarkable swell potential, low bearing values and unnecessary cracks necessitate slope stability or soil stabilisation.

Slope stability is often evaluated in order to make the slope safe or to reduce the rate of its failure. Methods of analyses based on limit equilibrium principles such as SLOPE/W software, traditional and preliminary analysis by stability charts were employed. The outcome of these analyses would be the safety factor for the stability of a slope.

BLACK COTTON SOIL OF NIGERIA (BCS)

The Black cotton soil of north-eastern Nigeria is specifically originated as a result of weathering of shaly, clayey sediments and basaltic rock (Carter *et al.,* 1963), (NBRRI, 1983), and (Ijimdiya *et al.,* 2012). Ola (1978 and 1983a) and Klinkenberg and Higgins (1968) reported that Black cotton soil of Nigeria contains more of the montmorillonite clay mineral with defined swell properties and expensive tendencies. These lacustrine sediments of north-eastern Nigeria cover an extensive area of 10.4 x 10^4km^2 (40, 000 square miles).

SLOPE STABILITY

Stability analysis for slope is usually carried out to establish the safety factor of natural slopes, embankments, retaining walls and landfills. Various stability techniques have been developed over years by researchers ranging from tedious hand calculations and charts methods to computer software. Limit equilibrium and finite element analysis are generally common solutions in practice to slope stability issues in geotechnical engineering. Slope stability methods used to determine the safety factors are briefly discussed as follows:

Limit Equilibrium Analysis

Limit equilibrium method compare forces / or moments tending to set stresses in the soil mass to forces / or moments opposing the action. Among the basic reasons the limit equilibrium method has being widely in practice, is that the solution could be estimated by hand – calculations (Krahn, 2004). Currently, the method is most used based on its simplicity and little number of parameters needed such as geometry of slope, geotechnical parameters, detailed mapping of the area, loading conditions, geology and underground water conditions. Nevertheless, the method does not take into account ground behaviour (Khadija *et al.,* 2012)

Assumption and Fundamentals for Limit Equilibrium Method (LEM)

The following assumptions were made in the implementation of limit equilibrium method:

1. The mass of the soil is assumed to be at a point of failure along sliding surface in question. Conventionally, the sliding surface is also assumed to be circular for clays and logarithmic spiral for sand and gravels (Mohammed, 2010).

2. Total stress analysis in form of short term condition or undrained slope is applicable to embankments and multistage loading problems (Duncan, 1996). In the application of total stress, ground water pressure are neglected and the shear strength is represented by undrained shear strength s_u (or $ø_u$ = 0). Hence, Mohr's coulomb failure described the shear strength of the soil as:

$$S_u = C^1 + (\sigma_n - U)tan\phi \quad (1)$$

Where;

C^1	=	Soil effective cohesion;
σ_n	=	Stress acting normal to the soil;
u	=	Pore water pressure; and
$\acute{\phi}$	=	Effective angle of friction.

3. Effective Stress Analysis in form of long-term condition or drained slope is applicable to excavation problem (Duncan, 1996). In the application of effective stress analysis, the shear strength of soil is described by Mohr-Coulomb criterion as:

$$S_u = C^1 + (\sigma_n - U)tan\acute{\phi} + (U_a - U)tan\acute{\phi}^b \quad (2)$$

Where; u_a = pore air pressure; ϕ^b = angle of friction of the soil based upon changes in u_a - u when u_n - u is held constant and u_a - u = matric suction and taken as the growth in the value of soil cohesion (Fredlund and Rahardjo, 1993).

4. Janbu (1973) described limit equilibrium condition as a situation when the shear stress (τ) mobilised is defined as a portion of the shear strength. Nash (1987) reported that prior to failure, the shear stress is considered to be absolutely mobilised through the failure surface when the precarious state conditions exists.

Shear stress mobilised is expressed as:

$$= \frac{\tau f}{F} = \frac{c + \sigma\ tan\phi}{F} \quad (3)$$

Where; F = Factor of safety (assumed)

Limit Equilibrium Methods

In the application of limit equilibrium principles, it involves solving equilibrium problem by assuming forces and / or moment equilibrium (Zhou, 2006). Several methods of limit equilibrium in practice for slope stability analysis include: Fellenius method of slices (1936), Modified Bishop's method (1955), Janbu's generalised method (1968), Morgenstern and Price's method (1965) and Spencer (1967). Slope stability charts have been developed also based on limit equilibrium technique by investigators such as: Taylor (1937 and 1948), Bishop and Morgenstern (1960), Spencer (1967), Janbu (1968) among others. They are useful for preliminary analysis and rapid estimation of stability of a slope (Smith, 2006). To simplify the laborious processes of slope stability analysis, researchers developed computer programs and most of which are based upon limit equilibrium approach. The end result of all types of limit equilibrium methods is usually presented as safety factor (**FS**).

Factor of Safety (FS)

At the incipient failure, safety factor for slope stability can be expressed as the proportional relation between the ultimate shear strength and the mobilised shear stress (Tran the Viet, 2011). There are many equations developed by researchers for determination of factor of safety. The common formulation is defined in relation to the force / moment equilibrium which assumed the safety factor to be uniform over the slip surface (Cheng and Lau, 2008).

Theoretically, for stable slope the value of factor of safety is taken as one (1.0), although, imminent failure and unstable conditions are the outcome if the value is one or less than one respectively. According to United State Army Corps of Engineers (1997) the factor of safety is set to 1.5 pertaining slope stability for geotechnical engineering projects.

Considering the rational landslide;

$$FS = \frac{Summation\ of\ resisting\ forces/moments}{Summation\ of\ driving\ forces/moments}$$

In order to meet up with the economic consideration in estimating the safety factor for slope stability in engineering design, Len and Evett (2005) gave some significant values of safety factor **(FS)** for design (table 1).

Table 1: Significant Factors of Safety for Design (Liu and Evett, 2005)

Factor of Safety	**Importance**
Less than 1.0	Failure
1.0 – 1.2	Uncertainty safety
1.3 – 1.4	Acceptable for cuts, fills, uncertainty for dams
1.5 – 1.75	Acceptable for dams

United State National Highway Institute (2006) reported relative worth of minimum design safety factor for slopes as reported by (Zhou, 2006); for side slope of regular roadway embankments a minimum of 1.25, for slopes which failure is disastrous such as bridge abutments end slopes, retaining structures, and highways connecting regions, interstates, and so on, the design factor of safety should be between 1.30 to 1.50. He further suggested a safety factor of 1.50 for cut slopes in fine-grained cohesive material which can easily lose shear strength. According to Smith (2006) safety factor not less than 1.25 is satisfactory for an earth embankment, and it should not be above this value for economic reasons.

METHOD OF SLICES

Basically, three methods of limit equilibrium in practice exist; include method of Slices, Wedge method and the Infinite slope method. In this work method of slices has been considered in relation to its application to SLOPE/W and traditional methods of analysis.

According to Abramson, *et al.* (2002) slices method is widely used by much computer software, as it can accommodate geometry of complex slope, different soil conditions and influence of external boundary loads. Conventionally, the weight of the soil lying at a particular point should influence the stress acting normal to the point on sliding surface. Theoretically, the basic principle of the slices method is the potential slide mass is subdivided into several vertical slices (fig. 1) and the equilibrium of individual slice is then evaluated in terms of forces and moments. This would allow easy estimation of the allowable safety factor of a slide mass.

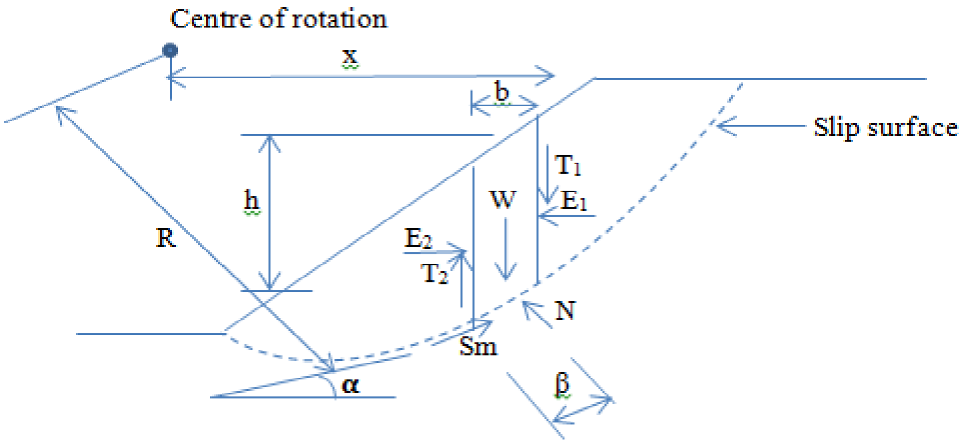

Figure 1: Circular Slip Surface with Underlying Soil Mass Divided Into Vertical Slice (Fredlund & Rahardjo, 1993)

Where,

W	=	Total weight of the slice
N	=	The total normal forces on the base of slice
S_m	=	The shear force mobilised on the base of each slice
E	=	The horizontal interslice normal forces (the "L" and "R" subscript designates the left and right sides of the slice respectively
X	=	The vertical interslice shear forces (the "L" and "R" subscript designates the left and right sides of the slice respectively
f	=	The perpendicular offset of the normal forces from the centre of rotation
R	=	The radius for a circular slip surface or moment arm associated with the mobilised shear force, Sm for any shape of slip surface
X	=	The horizontal distance from the centreline of each slice to the centre of rotation
H	=	The vertical distance from the base of each slice to the uppermost line in the geometry (i.e. generally ground surface)
α	=	The angle between the tangent to the centre of the base of each slice and the horizontal
B	=	Slopping distance across the base of a slice.

Ordinary Method of Slices (OM)

Ordinary method of slices (1936) disregards both the inter-slice normal and shear forces, but satisfies the moment equilibrium. However, the method assumes the slip surface to be circular and also considered as one of the simplest technique of slope stability analysis on the basis of slices method. It is also called Fellenius' method or Swedish circular method of analysis.

The safety factor can be computed using the following expressions (Nash, 1987 and Abramson *et al.,* 2002):

$$FS = \frac{\Sigma(Cl + Ntan\emptyset)}{\Sigma Wsin\emptyset} \quad (4)$$

$$N = \ cos\alpha - ul \quad (5)$$

Where,

u	=	Pore water pressure
l	=	Base length of slice
α	=	Angle of inclination of slip surface at the centre of slice

The figure 2 shows the forces acting on each slice based on Ordinary method, and is defined as T_1 and T_2, the vertical inter-slice forces and E_1 and E_2, the horizontal inter-slice forces. Other parameters on the figure are defined in figure 1.

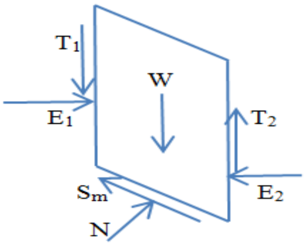

Figure 2: Forces Acting on a Slice in OM

Bishop's Simplified Method of Slices (BSM)

Bishop simplified method of slices (1955) is one of the commonly used slope stability analysis method. This method regards the forces between slices and disregards shear forces (Abramson *et al.,* 2002). However, the method satisfies only equilibrium of moment and assumed the resultant force to be horizontal ($X_L = X_R = 0$; fig. 1) (Krahn, 2004b). Assuming a circular slip surface, normal force at the slice base can be determined by adding forces in the vertical direction and then combine with the failure criteria.

$$FS = \frac{1}{\Sigma W \sin\alpha} \Sigma[cl + W(\cos\alpha - ru\sec\alpha)tan\o] \ (Smith, 2006) \tag{6}$$

Where; r_u = steady seepage

$$r_u = \frac{hw\gamma w}{\gamma Z} \tag{7}$$

Bishop's Rigorous Method (Bishop's Modified Method)

Smith (2006) reported that errors of about 15 per cent in the value of calculated factor of safety is obtained using equation 6 for an existing tip, although the slope would be at a safer side. He further reported that in the case of new embankment and earth dams, this error can lead to unnecessarily high costs. However, Bishop modified equation 6 in order to overcome these errors, and the new technique is known as Bishop's rigorous method or Bishop's modified method of analysis (see equation 8).

$$FS = \frac{1}{\Sigma W\ sin\ \alpha}\ \Sigma\left[(cb\ + W(1 - ru)tan\o)\frac{\sec\alpha}{\left(1+\frac{tan\o\ tan\alpha}{F}\right)}\right] \tag{8}$$

Figure 3 shows the forces acting on each slice according to the Bishop's methods, and is defined as: T_1 and T_2 are vertical inter-slice forces and E_1 and E_2 are horizontal inter-slice forces.

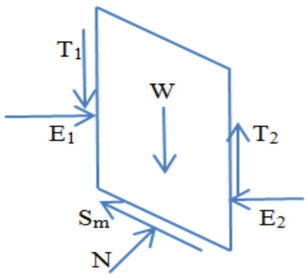

Figure3: Forces Acting on a Slice in BSM.

Janbu's Simplified Method (1968)

This method considers inter-slice normal forces and disregard shear forces. The factor of safety is estimated using horizontal force equilibrium. In order to overcome the deficiency of zero shear forces, Janbu introduced a correction factor, F_o (Abraham *et al.,* 2002). However, this correction factor, F_o, is in relation to cohesion, angle of friction and assumed appearance of the failure surface (Janbu, 1954).

Safety factor can therefore be calculated using;

$$F_f = \frac{\Sigma(cl+(N-ul)tan\emptyset)\sec\alpha}{\Sigma W\tan\alpha + \Sigma\Delta E} \tag{9}$$

ΣE = E_2 – E_1 = net inter-slice internal forces (zero if there is no horizontal force).
In terms of stress, the factor of safety can be calculated using Janbu's original equation (equation 2.11).

$$F_o = \frac{\Sigma\{\frac{b(c+(\rho-u)\tan\emptyset)}{na}\}}{\Sigma\rho b\ tan\emptyset} \tag{10}$$

$$n_a = cos^2\alpha\left(1 + \ tan\alpha\frac{tan\emptyset}{F}\right) \tag{11}$$

where,

$$\rho = \frac{W}{b} = total\ vertical\ stress\ and\ b = width\ of\ slice.$$

By modification of original equation, the new equation becomes;

$$F_f = f_o . F_o \tag{12}$$

The forces that are considered to be acting within the body of each slice mass according to Swedish method are shown in figure 4. The definition of parameters on the figure is similar to previous methods.

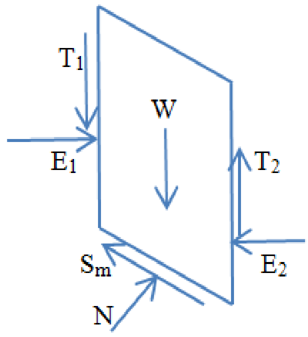

Figure 4: Forces Acting on a Slice in JSM

Morgenstern-Price's Method (M-PM)

Similar to other method, Morgenstern-Price's technique (1965) satisfies force and moment equilibrium. However, it also assumed inter-slice force function. With an arbitrary function (f(x)), inter-slice force inclination can also vary. According to Mohammed (1970), arbitrary function is expressed as:

$$T = f(x).\lambda.E \tag{13}$$

Where,

F(x)	=	Inter-slice force function that varies continuously along the slip surface.
λ	=	Scale factor of the assumed function.

For a given function, the factor of safety, f_r can be computed by iteration procedure when it is equal to f_m (Nash, 1987), and is expressed by the following equations;

$$F_f \frac{\Sigma[\{cl+(N-ul)n\tan ø\}\sec\alpha]}{\Sigma\{W-(T2-T1)\}\tan\alpha+\Sigma(E2-E1)1!} \tag{14}$$

$$F_m = \frac{\Sigma(cl+(N-ul)\tan ø}{\Sigma W\sin\alpha} \tag{15}$$

The forces acting on each slice is shown in figure 5.

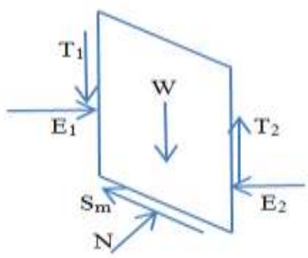

Figure 5: Forces Acting on a Slice in M-PM

COMPUTER PROGRAMME FOR SLOPE STABILITY

Stability analyses of slopes can be carried out today using different geotechnical computer software developed by Geo-slope International, Canada. Generally, both limit equilibrium and finite element computer programs are available in the market for the computation. These powerful computer programs include: SLOPE/W for stability analysis of slopes, SEEP/W for pore water pressure distribution, SIGMA/W for stress-deformation analysis, TEMP/W for geothermal, CTRAN/ W for contaminant transport, Quake/W for earthquake analysis and VADOSE/W for Vadose zone and covers (Geo-slope International, 2008c). SLOPE/W for stability analysis was selected for computation in this study.

SLOPE/W

Over years various versions of SLOPE/W software were developed by Geo-Studio International, for stability analysis of slopes. In the application of SLOPE/W software to determine the safety factor for slope stability, GEO-STUDIO 2007 is used based on limit equilibrium principles. Various myriad of options available in SLOPE/W is more than to just look at safety factor. Other issues include: was the input data specify correctly? Was the input data utilise by the software correctly? Why are there differences between the safety factors obtained from the various methods selected? (Krahn, 2004). SLOPE/W has various tools for inspecting the input data and evaluating the results by displaying a list of different variables along the slip surfaces and also detail forces on each slice. Methods selected for the analyses include; Bishop's simplified method, Janbu's method, Morgenstern-price's method and Ordinary or Swedish method. Geometry of a slope is one of the vital stuff uses by the software to establish a minimum safety factor. Experiment has shown that the steeper the slope, the smaller the value of safety factor, and the smaller the angle of inclination the bigger and acceptable the value.

Shear strength parameters are usually specify in SLOPE/W by Mohr-coulomb model. Depending on the ground condition, by selecting this model, the software will automatically define the friction angle between zero and the actual.

Mohr coulomb general equation can be described as (Krahn, 2004):

$$\tau = c + \sigma_n tan\, ø \tag{16}$$

Where,

τ	=	Failure shear strength
c	=	Cohesion
σ_n	=	Shear plane normal stress
$ø$	=	Internal friction angle.

Graphical representation of this equation (figure 6) gives a straight line indicating that, cohesion (c) is the intercept on the shear strength axis and the internal friction angle is the slope of the line.

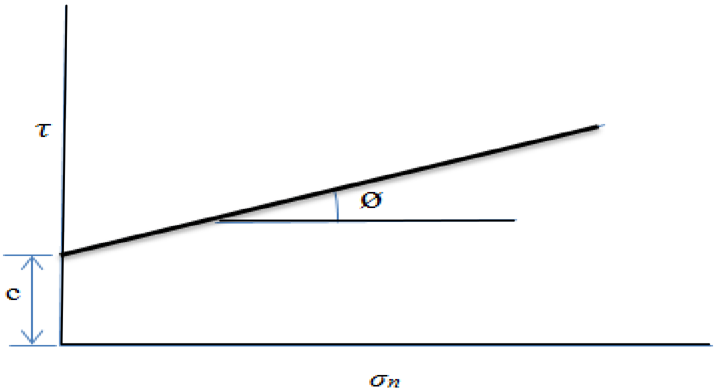

Figure 6: Graphical Representation of Coulomb Shear Strength Equation

Generally, the Mohr-coulomb failure envelope is determined from laboratory test, and the results can be presented as shown in figure 7.

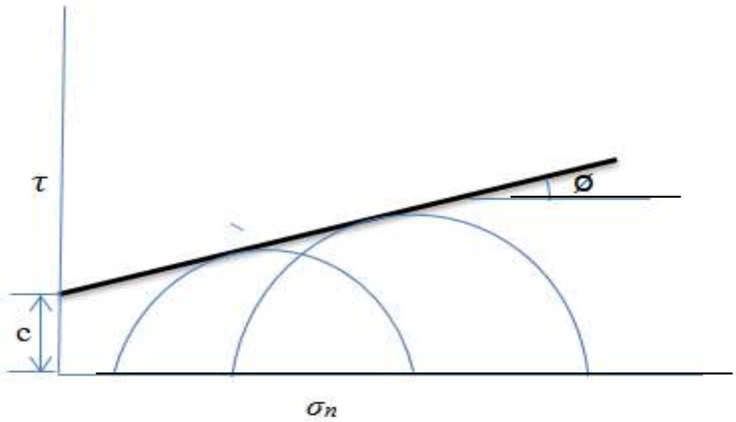

Figure 7: Mohr- Coulomb failure envelop

For saturated (undrained) situations when the internal friction angle ($\varnothing$) is zero, the failure envelope appears as shown in figure 8.

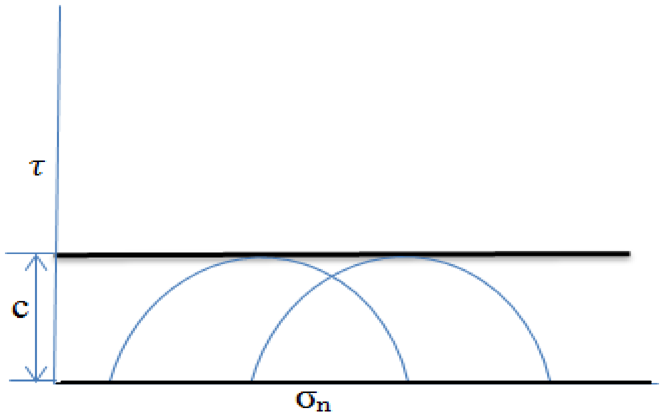

Figure 8: Undrained Strength Envelope

The input data needed for the assessment can be categorised into two: the geotechnical engineering properties of the material obtained from laboratory and field tests and geometry of the slope.

FIELD AND LABORATORY TEST RESULTS

Before any assessment of slope stability of either natural or man-made slope can be carried out, essential soil parameters must be obtained from field and laboratory tests. As shown in table 2, the live data was collected from 'the geology and geotechnical investigation of black cotton soil of north-eastern Nigeria' presented by Ola (Ola, 1978) in the engineering geology journal volume 12. Residual values for shearing resistance are selected for the analysis to give an acceptable and reliable result under worst condition.

Table 2: Input Data for the Analysis (Ola, 1978)

Test	**Shearing Resistance (KNm^2)**		**Density(KN/m^3)**
	Maximum moisture content	Liquid limit	
Unconfined compressive test	124.1	-	
Vane test	120.0	3.2	
Simple direct shear test			
Effective peak cohesion C^1_p (KN/m^2)		6.9	
Effective peak angle of shearing resistance $ø_p$		16^0	
Residual cohesion C^1_r (KN/m^2)		3.9	
Residual angle of shearing resistance $ø_r$		12^0	
Standard protocol density			14.13

GEOMETRY OF SLOPE

Krahn (2004) defined the geometry of a slope as the descriptive of the stratigraphy of shapes of potential slip surfaces. In sketching the slope profile, SLOPE/W uses the notion of regions to describe the geometry, thereby drawing lines around the soil mass in question to form a closed polygon. It should be noted that, the bigger the slope angle the smaller the safety factor and vice versa. Figure 9 below is a typical geometry of slope showing regions, grid lines (green colour), radius of rotation (green horizontal lines), piezometric line (blue dotted line) and angle of slope (angle 341).

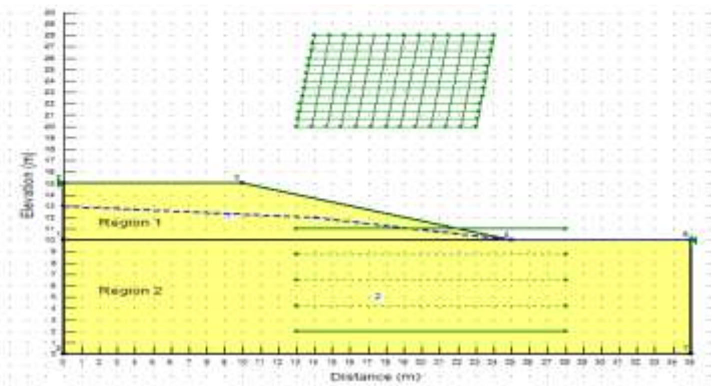

Figure 9: Typical Slope Geometry

Step-by-Step Procedure for the Construction of Slope Geometry:

1. Select a suitable scale for both horizontal and vertical axes.
2. Draw profile of the problem area as shown in figure 9.
3. Divide the polygon into two regions (region 1 and 2).
4. Draw piezometric line (dotted line) to describe the ground water table (GWT) within the slope.
5. Draw grid and radius of rotation.

Analyses Options

Case 1: Dry Slope (Drained Condition)

In this scenario, the slope was assumed to be dry, means no pore water pressures within the soil mass.

Case 2: Fully Saturated Slope (Undrained Condition)

In case 2, the slope was assumed to be completely saturated; meaning the ground water surface rise up to the slope surface and the pore water pressure line was drawn on the surface.

Case 3: Partially Saturated Slope

In this slope condition, the ground water level is at 2m from the slope surface (i.e. H_w = 3), hence piezometric line is drawn at 3m above the toe of the slope.

In SLOPE/W, the shear strength parameters are defined by the model of Mohr –Coulomb. On selecting this model SLOPE/W eventually set friction angle to zero for undrained condition of slope and describe the cohesion as the only shear strength value. Hence, the submergence has no effect to the shear strength of the material. With full strength (drained or dry condition), simply means both cohesion and angle of friction are not equal to zero, SLOPE/W will automatically set the exact values of the strength parameters.

SLOPE STABILITY ASSESSMENT

Required Data for the Analyses

Discussions on the data required for the analyses have been presented.
Table 3 is the summary of the information needed for the analyses.

Table 3: Summary of Data Needed for the Analyses

Description	**Symbol**	**Unit**	**Value**
Unit weight	γ	KN/m^3	14.13
Density of water	γ_w	KN/m^3	10
Cohesion	C	KN/m^2	3.9
Friction angle	ø	Degree	12^0
Angle of slope	β or $\boldsymbol{\alpha}$	Degree	20^0
Height of fill	H	meter	5
Height of water within the slope	H^1_w	meter	3
Height of water outside the slope	H_w	meter	3

Calculations

In order to determine the minimum factor of safety, the geometry of the slope was created as outlined above and the input data needed for Mohr- Coulomb model were assigned. A slope angle of 20^0 ($\beta = 20^0$) was chosen in order to obtain an acceptable factor of safety. Experience has shown that, the flatter the slope the lower the driving force and hence, the higher the safety factor. It is also evident that the geotechnical parameters of the soil are low. The graphical views of the computed details and the force polygon were displayed. This method of analysis is called entry and exit method or grid and radius technique. This is because the entry and exit locations where the slip surface enters and leaves the slope are specified, and the grid and radius of rotation are defined (see figure 9).

Conventionally, the procedure for calculating the stability factor is the same for the entire ground situations, the difference being in-terms of imposed load above or within the slip mass as described.

RESULTS AND DISCUSSION

Introduction

The stability analysis of a typical black cotton soil was analysed for dry, fully saturated and partially saturated slope conditions. It should be noted that the strength of the soil declined under saturation conditions; meaning that the values of safety factors also reduces. Furthermore, it can be seen from results that surcharge load on top of slope and tension cracks influenced the value of safety factor. The general probability failure mode is also investigated. Detailed results from each of the methods used are presented in tabular forms.

Results

Various safety factors for slope stability were determined and the results are displayed in tables and in graphical forms.

Table 4: Summary of Results from SLOPE/W

Method	Factor of Safety (FS)		
	Case 1: Dry Slope	Case 2: Fully Saturated Slope	Partially Saturated Slope
BSM	1.30	0.68	0.92
JSM	1.20	0.62	0.82
M-PM	1.30	0.68	0.92
OM	1.20	0.59	0.76

Case 1: Dry slope

Fig. **10a** and **10b** shows the graphical views of the computed detail and the force polygon of the critical slice in a potential sliding mass respectively.

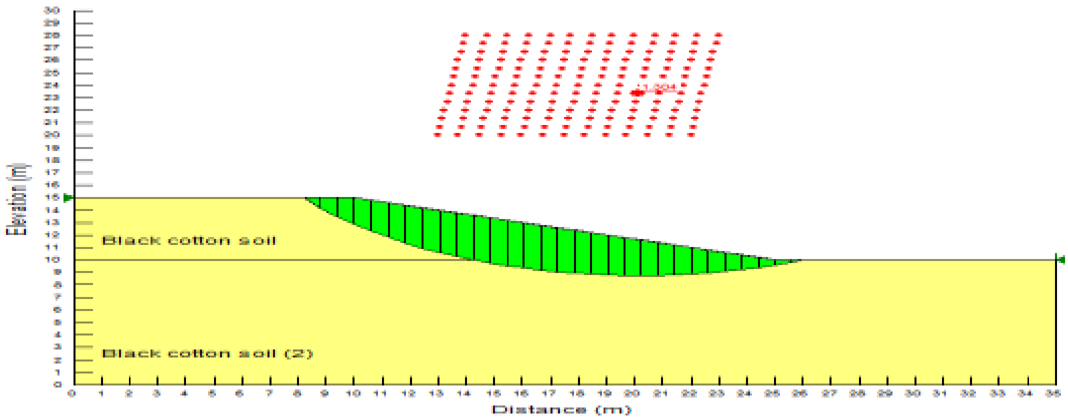

Figure 10a: Graphical View of the Computed Detail for Dry Slope

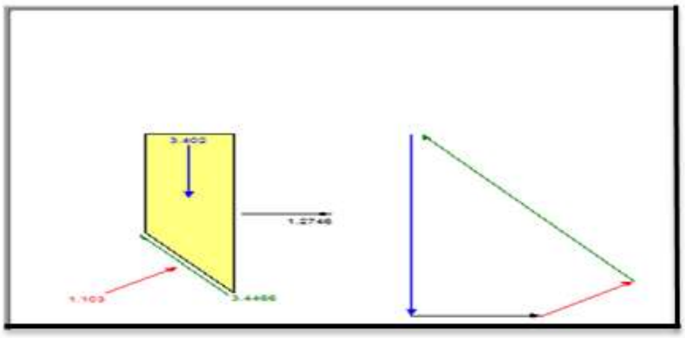

Figure 10b: Force Polygon on the Critical Slice Mass

Case 2: Fully Saturated Slope

Fig. **11a** and **11b** are the graphical views of the computed detail and the force polygon of the critical slice in a potential sliding mass.

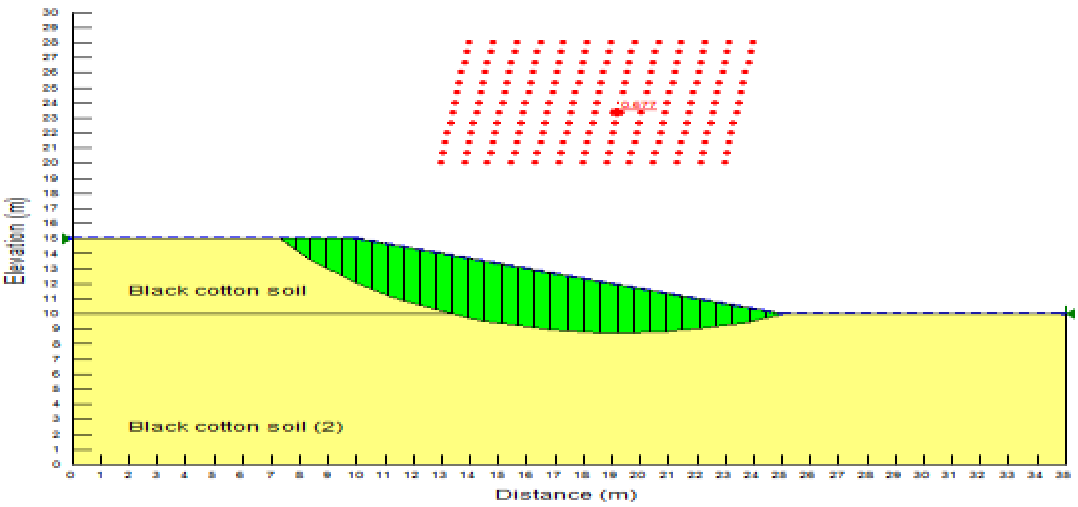

Figure 11a: Graphical View of the Computed Detail for Fully Saturated Slope

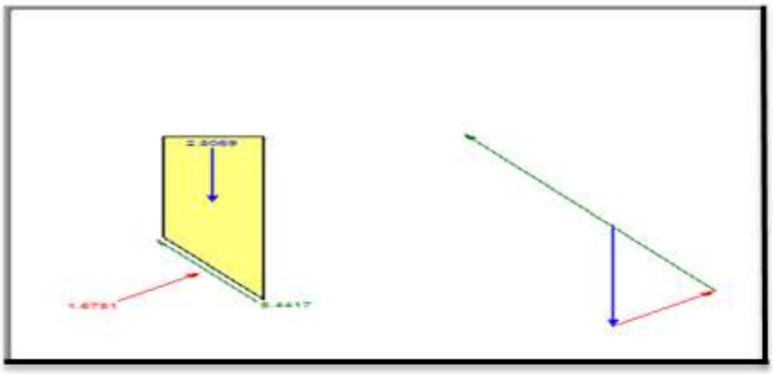

Figure 11b: Force Polygon on the Critical Slice Mass

Case 3: Partially Saturated Slope

The graphical views of the slope and the force polygon of the critical slice in a potential sliding mass are shown in figures **12a** and **12b** respectively.

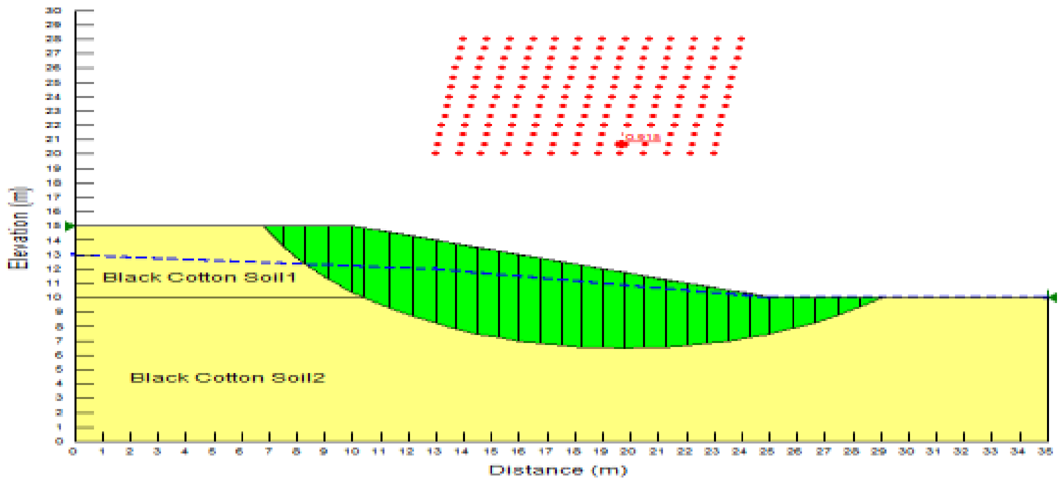

Figure 12a: Graphical View of the Computed Detail for Partially Saturated Slope

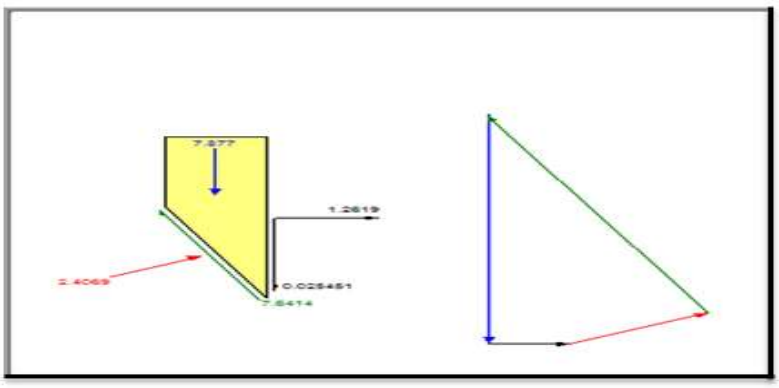

Figure 12b: Force Polygon on the Critical Slice Mass

Discussion of Results

The slope stability analyses went in accordance with the principles of limit equilibrium and the results are presented. Also included in results section are the graphical views of the details and the force polygon of the potential sliding mass. These views lead to the development of the assumed slip failure circle and show the forces that acted on the slices respectively. Furthermore, various factors of safety determined described the state of stability of the black cotton soil (slope) at various ground conditions under different loadings. According to the United State National Highway Institute (2006) slope failure occur when the safety factor is less than 1.0 and is on a safer side when the value is higher than 1.0. Hence, the following discussions from the results were based on the above assertion in relation to Len and Evett suggestions.

Slope Condition 1: Dry Slope

The pore water pressure is assumed to have no effect in this situation. The results obtained from the entire selected stability methods using SLOPE/W falls within the designated acceptable bench mark (table 1) for stable state of sliding mass.

Slope Condition 2: Fully Saturated Slope

The results from the analysis under fully saturated condition show that the slope is not stable from the entire analyses methods. The presence of pore water pressure convincingly influenced the low value of safety factor. Hence, it can be deduced from the analysis that the slope will inevitably fail since the critical minimum factor of safety is less than one.

Slope Condition 3: Partially Saturated Slope

It is obvious from the analysis that the slope is not at the safer side. This is because the ground water table has risen above the toe of the slope and consequently reduces the shear strength of the material.

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