Stability and Deformation Analysis of Embankment on Soft Clay

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Abstract

Soft clay generally poses a problem for civil engineers due to its high compressibility and low shear strength. Structures built on soft soil are subjected to excessive settlement and the settlement is attributed to the consolidation process. A large percentage of the settlement is accredited to a consolidation process that could persist for a long period, depending on the ability of the soil to dispel excess pore water. Prediction of the settlement of embankment is important for the design of the highway, to prevent excessive post-construction settlement which can lead to failure. Construction on soft clays has become more important and common in urban areas

In this study, the time-settlement behavior of an embankment on soft clay predicted using analytical methods is compared with those predicted by Geo studio SIGMA/W software. Thus the magnitudes of final settlement using Terzaghi theory differ from the magnitudes of final settlement computed from SIGMA/W software by approximately ±10% difference. The differences could be due to the assumption properties such as Young's modulus (E) and Poisson's ratio (ν) in the SIGMA/W Geo studio, and the coefficient of compressibility considered in the analytical method.

Moreover, this study presented the results of the stability analysis of the embankment on soft clay, and the performances of the barrier were analyzed. The limit equilibrium method (LEM) using Geo studio SLOPE/W software and hand calculations were used to calculate the factor of safety. The results showed that the calculated factors of safety obtained from Fellenius, Bishop, Janbu, and Morgenstern-Price methods using both ways are very similar, with an approximate difference of only ±10%.

Keywords: Embankment, Soft clay, SIGMA/W Software, Terzaghi theory.

1. Introduction

Evaluating the stability of the slopes is the initial difficulty the geotechnical engineers have to confront. The stability of the slope is required to be limited to some degree, usually by the stress and force calculated by the potential surface of the fault. This method of limit equilibrium is commonly used, essentially because of its reliability in most practical cases [7].

A natural theory regarding the limit equilibrium analysis is that soil reinforcement is similar to a rigid plastic material. There is no deformation of the soil, each time the measurement is smaller than the ground resistance. After measuring the immersion intensity, the shortest way improves upon the resistance of the ground, and the deformation of the ground. This technique presumes that the mobilized shearing strain all the way through the potential surface of the fault together simultaneously is not modified for the majority of situations [7].

According to Lerouiel et al (1990), soft clay generally poses a problem for civil engineers due to its high compressibility and low shear strength. The most significant difficulty associated with construction upon soft clay is settlement and deformation which is comparatively hefty and is very time-consuming to treat. This research was executed to predict embankment stability and deformation upon soft clay [15].

To achieve this, a stability analysis is carried out using the limit equilibrium method. Limit equilibrium methods use computer software programs such as SLOPE/W [20] and analytical methods such as the Bishop method. Consequently, there are considerable sponsorships for studying the application of limit equilibrium software for practical problem-solving.

Difficulties with high compressibility and low shear strength of the soft clay often occur and are commonplace in embankment construction. During the construction of embankments, the stress constantly acts in vertical and horizontal directions, and effective stress depends upon stability conditions both throughout and after construction. The regular manner of ground soil beneath the embankment is similar to settlement, lateral movement, pore water pressure, and total stress. As a result, all of these behaviors are interrelated with each other [17].

Construction on soft clay is prone to extreme settlement. A large percentage of the settlement is accredited to a consolidation process that could persist for a long period, depending on the ability of the soil to dispel excess pore water pressure owing to construction load. The correlation between the settlement and time is not linear for this reason a large percentage of settlement more often than take place earlier [15]. The Terzaghi technique and SIGMA/W program are used to calculate the final settlement. Therefore the degree of consolidation can also be calculated at any time using the Terzaghi technique.

Therefore, the study objectives are stated as follows:

- To predict the final settlement based on Terzaghi's method.
- To predict the final settlement based on Geo studio SIGMA/W.
- To compare the magnitude of final settlement based on Terzaghi 1-D consolidation analysis with that based on Geo studio SIGMA/W.
- To calculate the degree of time required to achieve 90% of the consolidation process using Terzaghi's method.
- To calculate the factor of safety of embankment using hand calculations.
- To conduct and compare the slope stability analysis using the limit equilibrium methods based on Geo studio (2007) SLOPE/W with manual methods.

2. Scopes and Limitations of Search

This study focuses on collecting and re-examining data of backfill and foundation soil which also comprises compressibility and strength data. Subsequently, stability analyses will be performed using the limit equilibrium method and consolidation settlement process. Furthermore, stability analyses will be carried out to evaluate the relative significance of the various factors impacting the stability of slopes constructed on soft clay, such as methods of analysis and variations of shear strength restricting factors.

3. Literature Review

In constructing embankments made of soft clay, an associated common failure mechanism occurs in the form of rotation along the cylindrical slip surface. For stability calculations such as the Bishop simplified method, the equilibrium method is extremely used. Due to the extreme reliability required of embankment limit in most practical cases, the bishop method seeks to balance the limit. In the literature presented by Duncan (1996), a summary of various limit equilibrium methods widely used in practice is presented. The simplified method of limit equilibrium can be used in one preliminary assessment, with accurate results of complex analysis provided with the aid of computer programs [7].

Since Soil cannot be deformed as the driving stress force is less than the soil resistance, the limit equilibrium method treats it as a rigid plastic. As the shear stress exceeds the soil strength, the physical features of the soil deform and hence are incorporated into failure. Duncan's (1996) assumption for this method is that the shear stresses along the surface of the soil are potentially mobilized simultaneously [7].

There are limitations associated with the limit equilibrium method amongst which is the underestimation of the foundation stability of the material, which shows the behavior of clay hardening. The fact that it does not take into consideration the possibility of reducing the kinetic force when deformed and flattened, which is particularly important for lands on springs that have the potential to deform the excess strain during the process of construction, which at this point, the possible solution to this is to overestimate the factor of safety. Moreover, the limit equilibrium method usually assumes circular surfaces, whereas other methods such as the Morgenstern and Price methods assume a noncircular failure plane [1].

According to Geo Studio (2007), many software programs such as SLOPE/W have been developed to aid in analyzing the method of the stability of slope based on limit equilibrium, and they are widely available in the market. It is worth noting that evaluations of the deformations of slope using the method of mechanics of continuity are somewhat close to those of the finite difference methods. Therefore, with the advent of computer technologies, slope stability calculation using mechanical methods of continuity is commonly found in place [14 and 18]. Much commercial software such as Plaxis for the finite element method and FLAC for the finite difference method are now widely available and used for the analysis of slop deformation [14 and 18].

Clay particles consist of complex flat and elongated minerals of varying sizes but less than 0.002mm. The particles of clay are plastic-like and coherent. In addition, it can be deformed to take up a new shape when a force is applied to it [23] and this single act of assuming a new shape results from the size and nature of the clay mineral particles. This property of soil is controlled by the nature of the absorbed layer. In a situation where the average specific surface is high, the plasticity of the soil may be extremely high thus impacting the compressibility of the soil [23].

Fine-grain soils when compared with coarse-grain soils display properties that are undesirable for engineering applications as they tend to exhibit lower shear strength when in an unsteady state. They can be plastic and compressible and also expand when wetted and shrink when dried [23].

Plastic deformation of clay soils may occur under constant application of load over time. As the shear stress of the soil approaches the shear strength of the applied load, the clay soil exhibits signs of sliding, thereby making it prone to landslides. Furthermore, the soil particles experience lateral pressure with low permeability, thus leaving it as a poor wall backfill retaining material. However, despite their impermeable characteristics, they are suitable for use as core materials for earthen dams [23].

According to Tsukamoto, Y. (1996), the behavior of soft clay is largely influenced by the following factors, the source of the parent materials that make up the clay, the consolidation in groundwater, erosion, deposition, and re-deposition means. It has a very high moisture content which is greater than its liquid limit, thereby leaving the soil sample strength extremely low. Furthermore, the effective measurable stress of the soil tends to be insignificant when in-situ state. Soft clay soils are generally found in places such as man-made fills, mines, or in recently deposited young clay that is still undergoing self-weight combination [19].
Bo (2004) insinuated that the settlement and pore behavior of soft soils when the load is applied are different from those of natural soil. They undergo settlement without dissipating the pore pressure in them or gaining effective stress upon additional load application [4].
During the precipitation of clay in salt water, its particle tends to stick together thereby giving rise to an edge-face-like arrangement. The flocculation of clay particles causes the clay (silt and fine) particles to settle almost at the same rate, formulating sediment with a very loose structure [13]. Therefore, these sediments can be considered loose-formed with high ratios of void. Bourges, F. (1979) in their permeability study literature opined that the edge-face arrangement behavior of marine clay leads to conditions where fine-grained soils are sensitive to changes in the moisture content and stress system. However, the compressibility and low shear strength of these weak marine deposits pose great challenges to geotechnical structure constructions of various engineering endeavours [21].
Gue (2000) suggested that embankment loading exists in either single or multi-stage as well as can determine the embankment height, slope and fill material properties. A single-stage embankment loading will cause an instantaneous increase in the overall stress and also increase the pore water pressure if the filling is so rapid since water cannot be dissipated from the soil. On the other hand, multi-stage embankment loading has the potential to allow for consolidation as well as dissipating water from the soil. However, it requires longer construction time than that of single-stage embankment loading [19].
For embankment loading, the lowest loading occurs at the toe of the embankment and increases to the central line. Also, its variations depend on embankment type height as well as geometry [12].
Geotechnical design and construction of embankments on soft soils is a very odious task since without sufficient soil reinforcement, the structure is likely to fail during construction or after construction at a later time. The resulting failures that may occur include; failure due to deep-seated sliding wedge, lateral spreading, and bearing capacity failure. Therefore, it is recommended to adopt a design approach for reinforcement embankment to design against failure. The possible failure modes that can occur for a reinforced embankment built on soft soil include bearing capacity failure, sliding displacement failure, and rotational failure [6 and 11].

4. Case Study

This study is based on eight cases of an embankment built on soft clay soil to study the behavior of soft clay under the embankment in terms of stability analyses and settlement behavior. Properties of soft clay layers that have been used for the stability analysis of embankment in terms of the height of embankment, magnitude, and time of consolidation settlement have been collected from a previous thesis [2]. The table below shows the material properties of the embankment and foundation soil that have been used in the modeling of the eight cases.

Table 1: The Material Properties of Embankment and Foundation Soil [2]

parameter	Symbol	Very soft to soft silty CLAY	Dark greenish grey silty CLAY	Dark grey silty CLAY	Whitish grey and firm silty CLAY	Lateritic (fill Material)	Unit
Material model	-	MC	MC	MC	MC	MC	-
Type of behaviour	-	Undrained	Undrained	Undrained	Undrained	Undrained	-
Dry soil unit weight	γdry	12.52	13.48	11.55	14.44	16.6	KN/m^3
Sat. Soil unit weight	γsat	15.78	16.52	14.61	17.27	18	KN/m^3
Horizontal permeability	K_x	$14.291*10^{-4}$	$16.26*10^{-4}$	$14.705*10^{-4}$	$10.109*10^{-4}$	0.04	m/day
Vertical permeability	K_y	$7.145*10^{-4}$	$8.130*10^{-4}$	$7.353*10^{-4}$	$5.054*10^{-4}$	0.04	m/day
Young's modulus	E_{ref}	1286.12	1724.29	1088.44	1465.33	2000	KN/m^2
Poisson's ratio	ν	0.35	0.35	0.35	0.35	0.3	-
Cohesion	C_{ref}	15	17	37.3	20	10	KN/m^2
Friction angle	φ	2	1	4.9	10	23.54	°
Dilatancy angle	ψ	0	0	0	0	0	°

A. Stability and Settlement Analysis

The slope stability analysis has been conducted using the limit equilibrium based on the program SLOPE/W [7]. Methods of limit equilibrium analysis adopted in SLOPE/W have been chosen based on those of Morgenstern-Price, Bishop, Janbu, and Ordinary. Moreover, the settlement has been computed using the SIGMA/W Geo studio (2007) program.

B. The Theoretical Calculations

Theoretical calculations were carried out to determine the performance of the foundation soil and embankment. Hence the prediction of the final settlement and the degree of consolidation at any time has been calculated using Terzaghi's method. In addition, the factor of safety has been calculated using the three manual methods of Bishop, Janbu, and Ordinary. The results of the theoretical calculations will be evaluated with the results obtained from the Geo Studio (2007) program.

5. Results and Discussion of Settlement Behaviour

In this study the settlement behaviour of an embankment on soft clay predicted using analytical method is compared with that predicted by Geo studio SIGMA/W software. The comparison is made in terms of final settlement, coefficient of consolidation and time required to achieve 90% consolidation. By using Terzaghi's theory were done the Hand calculations for consolidation settlement and the time required to reach 90%. Table 2 exhibits the material properties employed in SIGMA/W modelling.

Table 2: The Material Properties Employed in SIGMA/W Modelling [2]

parameter	Symbol	Very soft to soft silty clay	Dark greenish grey silty clay	Dark grey silty clay	Whitish grey and firm silty clay	Lateritic (fill Material)	Unit
Young's modulus	E_{ref}	1286.12	1724.29	1088.44	1465.33	2000	KN/m^2
Poisson's ratio	ν	0.35	0.35	0.35	0.35	0.3	-

Table 3: The Magnitude of Final Settlement using Terzaghi Theory and SIGMA/W Software

Method/Case	Case 0	Case 1	Case2	Case 3	Case 4	Case 5	Case 6	Case 7
Settlement (software) (m)	0.506	0.494	0.531	0.439	0.653	0.735	0.777	0.610
Settlement (hand calculations)(m)	0.551	0.391	0.531	0.476	0.653	0.676	0.692	0.526
Time of 90% consolidation (hand calculations) (Day)	222	588	415	335	193	95	130	56

Based on the results, with Terzaghi's method in case 1 the final settlement is 391 mm of the soft clay is expected to achieve 90% consolidation after 588 days. Hence, the final settlement obtained by Geostudio programme of 484 mm is slightly higher than Terzaghi's value. The differences could be due to the assumption properties such as young's modulus (E) and Poisson's ratio (ν) in the SIGMA/W Geo studio and coefficient of compressibility m_v considered in the analytical method.

Moreover, the differences may be due to the method of analysis that Terzaghi used. Terzaghi (1967) made some assumptions in his theory which are;

- The coefficient of compressibility m_v decreases with decrease in void ratio and increase in effective stress during 1-D consolidation process.
- Compression and flow of pore water pressure are vertical only (1-D consolidation).
- The compressible soil layer is homogenous and completely saturated
- Soil particles and water is incompressible.

The increment of the applied load produces only small strains. Thereby leaving the thickness of the layer unchanged during the consolidation process.

The coefficient of compressibility m_v, the coefficient of permeability k and the coefficient of consolidation C_v, remain constant throughout the consolidation process.

The relationship between void ratio and vertical effective stress is linear and unique. This assumption also implies that there is no secondary compression settlement.
Additionally, case 1 will take the longest time to achieve 90% of consolidation after 588 days. In contrast, case 7 will consolidate in the shortest time, taking only 56 days. This is due to the difference of soil properties in each case. Indeed, it may be that case 1 has softer clay with less permeability and lower hydraulic conductivity than the clay in case 7, thereby requiring longer time for a complete consolidation.

A. Lateral Movement

The lateral movement of the soil beneath the embankment can cause a variety of problems which will waste time and drain financial resources if these are not recognised and the underlying cause is not corrected ahead of endeavouring to make repairs. Lateral movement can be due to lateral moisture movement, underground pressures, adjacent excavations or natural erosions. This usually takes place throughout and following the construction of embankment on soft grounds [8]
According to Asrul and Huat (2003), lateral movement is defined as the horizontal outward flow of soil when subjected to shear stresses. This shear stress increases at
Different rates with vertical settlement because of the anisotropic behaviour of soil. Furthermore, embankments constructed on limited width induce vertical settlement and lateral deformation of the foundation soil, whereas those made on large fill soil experience appreciable vertical settlement. According to Lerouiel et al (1990), the lateral movement of construction foundation soil at the edge of the embankment exhibits similar behaviour to that of the settlement. The table below shows the maximum lateral movement documented for the embankment which took place at an identified depth [8 and 19].

Table 4: The Maximum Lateral Movement at Identified Depth

Case	Maximum lateral movement value at identified depth (m)
Case 0	0.058 m at a depth of 11 m
Case 1	0.058 m at a depth of 17 m
Case 2	0.062 m at a depth of 14 m
Case 3	0.050 m at a depth of 16 m
Case 4	0.185 m at a depth of 6.4 m
Case 5	0.215 m at a depth of 6.6 m
Case 6	0.220 m at a depth of 6.5 m
Case 7	0.176 m at a depth of 4.2 m

B. Total Pressure under the Embankment

An overburden pressure is the perpendicular upright pressure which is produced from the embankment as the weight builds up with the increasing elevation height of the fill. The pressure is monitored by the pressure cells which are placed below the embankments. Theoretically, the total pressure calculated from beneath the embankment ought to be approximately identical to the unit weight of the fill soil multiplied by the fill thickness.
The total induced stress due to the increase in embankment height can be correlatedwith the pore

water pressure generated during embankment construction. During the early stage of construction, the excess pore water pressure, (Δu), is somewhat less than the change in the principal stress, (Δσv). As the construction work progresses, a stage is reached when the generated excess pore pressure increases approximately to an equivalent change in the principal stress. At this stage, the embankment height is the same as that of the critical height of the construction, and begins to react in an undrained manner. Upon further application of load, a localised failure state is attained. At this state, a portion of the foundation reaches an effective stress condition which is equivalent to the failure surface. This failure surface occurs in the foundation at a level that is below the embankment slope and toe [19].

6. Results and Discussion of Stability Analyses

A. Geo Studio SLOPE/W Analyses and Results

The slope stability analyses have also been conducted based upon the data from the findings of the limit equilibrium based program SLOPE/W (Geo studio, 2007) to present the minimum factor of safety. The methods of limit equilibrium analysis implemented in SLOPE/W for this research were those of Morgenstern-Price, Bishop, Janbu and Ordinary. Table 5 presents the material properties employed in SLOPE/W modelling.

Table 5: Material Properties used in SLOPE /W Modelling [2]

Parameter	Symbol	Very soft to soft silty CLAY	Dark greenish grey silty CLAY	Dark grey siltyCLAY	Whitish grey and firm CLAY	Lateritic	Unit
Soil unitweigh	γ	15.78	16.52	14.61	17.27	18	KN/m³
Cohesion	c	15	17	37.3	20	10	KN/m²
Friction angle	φ	2	1	4.9	10	23.54	°

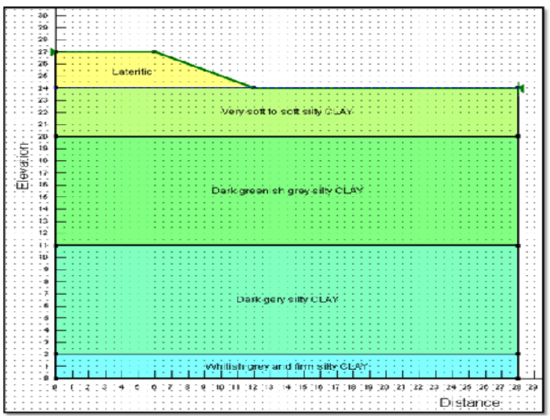

Fig 1: Geometry Model of Embankment Using SLOPE/W Case 0 [2]

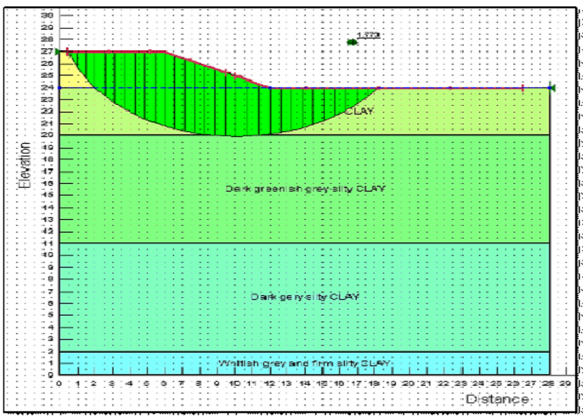

Fig 2: Typical Results of Slope Stability Analysis Using SLOPE/W for Control Embankment Case 0

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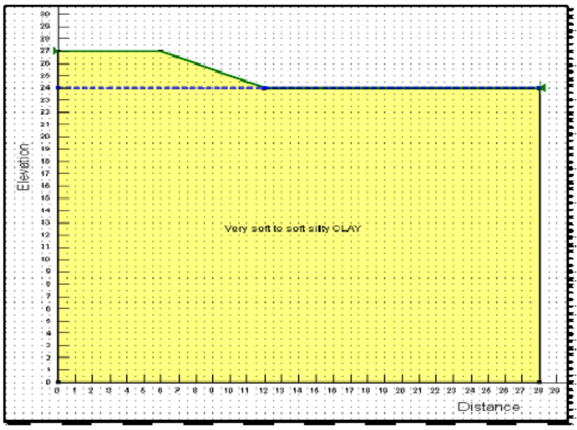

Fig 3: Geometry Model of Embankment Using SLOPE/W Case 1

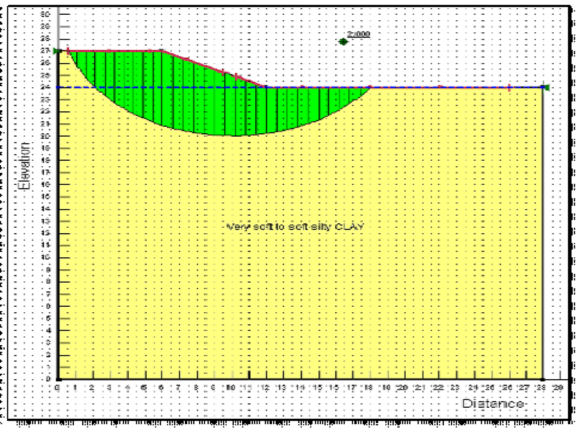

Fig 4: Typical Results of Slope Stability Analysis Using SLOPE/W for Control Embankment Case 1

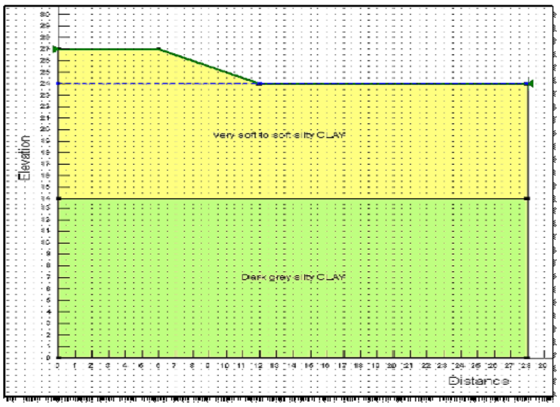

Fig 5: Geometry Model of Embankment Using SLOPE/W Case 2

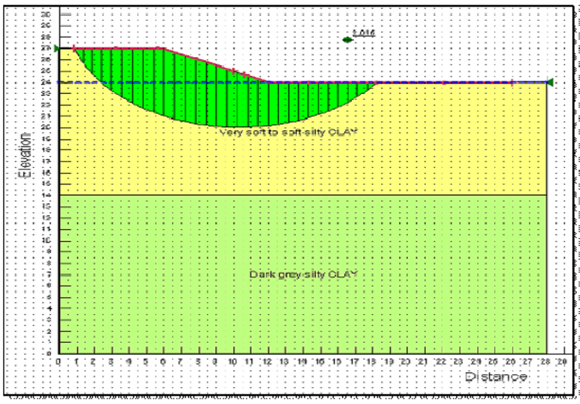

Fig 6: Typical Results of Slope Stability Analysis Using SLOPE/W for Control Embankment Case 2

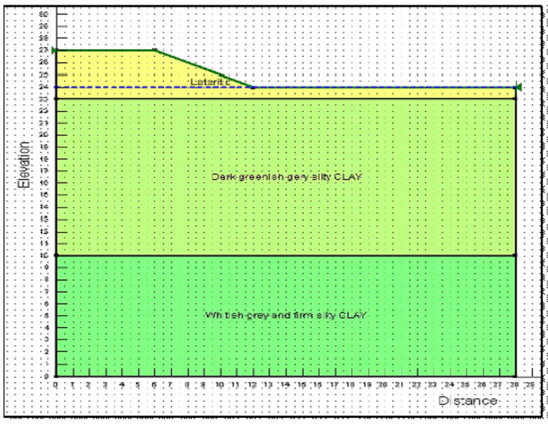

Fig 7: Geometry Model of Embankment Using SLOPE/W Case 3

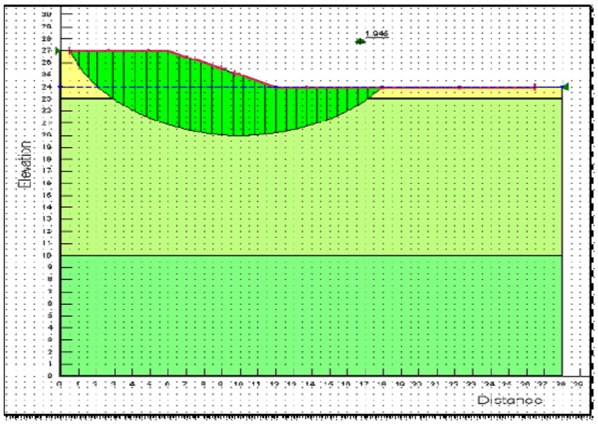

Fig 8: Typical Results of Slope Stability Analysis Using SLOPE/W for Control Embankment Case 3

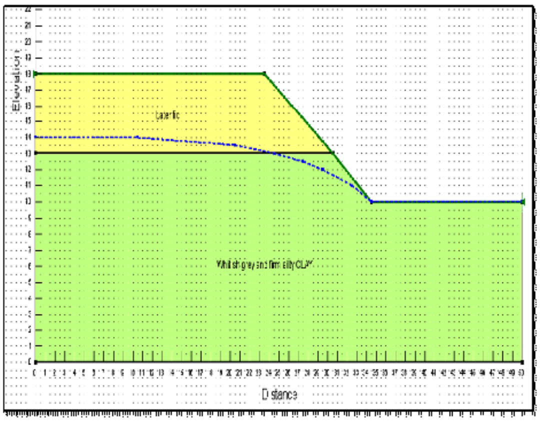

Fig 9: Geometry Model of Embankment Using SLOPE/W Case 4

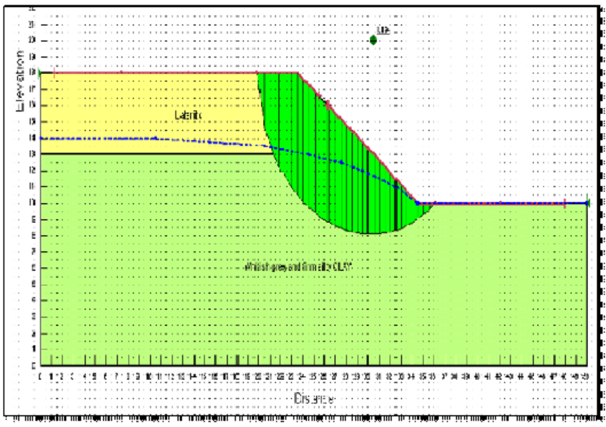

Fig 10: Typical Results of Slope Stability Analysis Using SLOPE/W for Control Embankment Case 4

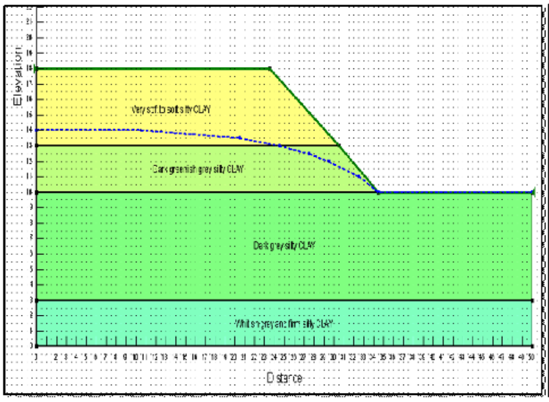

Fig 11: Geometry Model of Embankment Using SLOPE/W Case 5

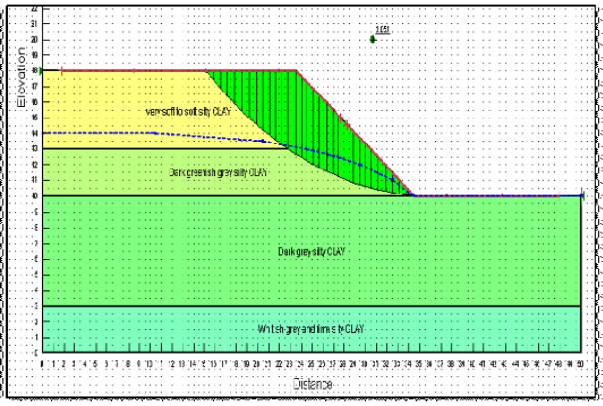

Fig 12: Typical Results of Slope Stability Analysis Using SLOPE/W for Control Embankment Case 5

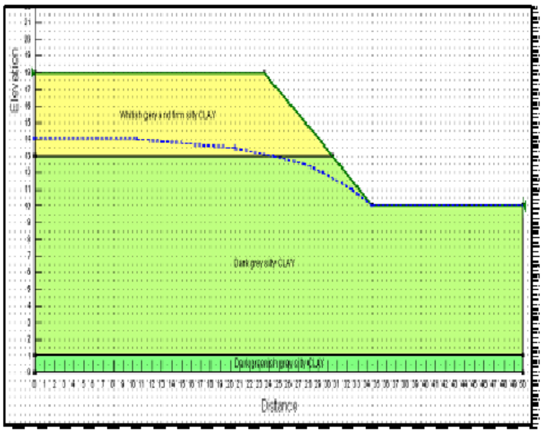

Fig 13: Geometry Model of Embankment Using SLOPE/W Case 6

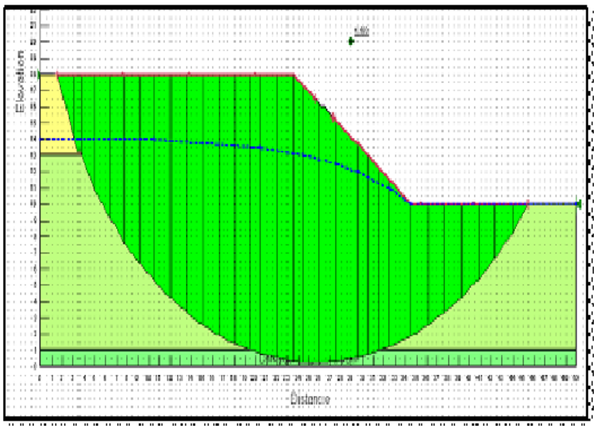

Fig 14: Typical Results of Slope Stability Analysis Using SLOPE/W for Control Embankment Case 6

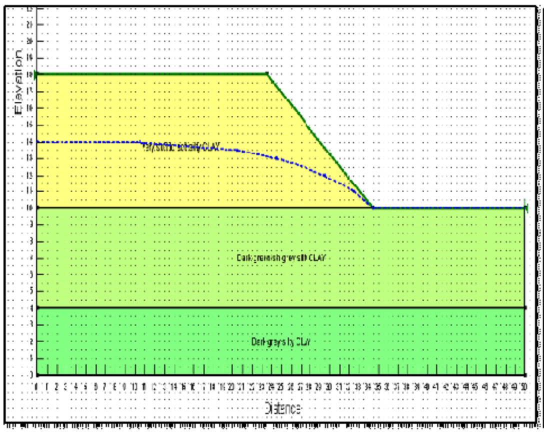

Fig 15: Geometry Model of Embankment Using SLOPE/W Case 7

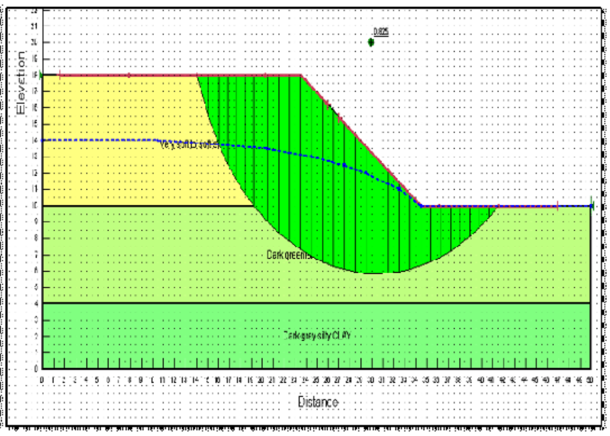

Fig 16: Typical Results of Slope Stability Analysis Using SLOPE/W for Control Embankment Case7

Slope stability was studied by evaluating the factor of safety using hand calculations from Bishop's method, Janbu's method and Fellenius' method. In addition, the factor of safety was calculated using Geo studio Slope/W computer software, by Bishop's method, Morgenstern price's method, Janbu's method and Ordinary's method. The hand calculations of stability analysis of embankment using limit equilibrium method were presented in appendix C.

The main objective of this study is to compare the performance of several methods of slope stability analysis derived from the limit equilibrium concept using Geo studio SLOPE/W and hand calculations. The results of the different cases are compared and analysed as follows:

Table 6: Factor of Safety Using Hand Calculations

Method / Case	Case 0	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7
Bishop's method	1.778	1.998	2.025	1.987	1.202	0.964	1.573	0.837
Janbu's method	1.706	1.827	1.822	1.832	1.065	1.043	1.415	0.723
Ordinary's method	1.641	1.967	1.988	1.853	0.918	0.967	1.414	0.811

Table 7: Factor of Safety Using SLOPE/W

Method / Case	F O S Case0	F O S Case1	F O S Case2	F O S Case3	F O S Case4	F O S Case5	F O S Case6	F O S Case7
Bishop's method	1.774	2.001	2.015	1.949	1.198	1.052	1.585	0.825
Morgenstern – Price's method	1.773	2	2.015	1.945	1.194	1.051	1.583	0.825
Janbu's method	1.637	1.889	1.9	1.811	1.076	1.041	1.479	0.79
Ordinary's method	1.669	1.956	1.971	1.793	1.1	1.05	1.486	0.813

It can be seen from tables 6 and 7 that, the calculated factors of safety obtained from Fellenius, Bishop, Janbu and Morgenstern-Price's methods are very similar, with an approximate difference of only ±10%. In most cases, Fellenius' and Janbu's methods generally result in a lower factor of safety than Bishop's method. The reason for this is that Bishop's method satisfies the moment equilibrium in its equation, considers the normal inter-slice forces and ignores the shear force between the slices. Additionally Bishop's method simplifies the equation by assuming that, the vertical of the inter-slice forces is zero, therefore the effect of the inter-slice force is ignored [24].
The factors of safety of moment equilibrium such as Bishop's method are similar to those computed using the Morgenstern-Price method, which considers both normal and shear inter-slice forces, and satisfies both force and moment equilibrium. As a result, it is recommended to use methods which satisfy both force and moment equilibrium in order to minimise errors when calculating the factor of safety of slope. The reason for this is that, these methods ignore the of shear forces in their equations between the slices. Additionally the assumption is made regarding inclinations of resultants of the forces on side of each slice are calculated in the process of solution [24].
It can be seen from tables 9, 10 and 18 that there is a slight difference in the factors of safety obtained from hand calculations and the Geo studio SLOPE/W program. This difference has occurred due to a number of slices being used in both ways. Indeed, the number of slices used to describe the potential slip surface can affect the results of stability analysis. In hand calculations the slip surface failure has divided to 7 slices in cases 0, 1, 2, 3 and 7 and cases 4 and 5 the number of slices is 8 while case 6 the slip surface failure has divided to 9 slices. In practice, 7, 8 or even 9 slices are not sufficient. Thus, increasing the number of slices from 7 to 30 makes a difference in the factor of safety. The table 15 below shows the influence of number of slices on the value of factor of safety using different methods for case (3).

Table 8: Effect the Number of Slices

Method	7 slices	30 slices
Bishop	1.987	1.949
Janbu	1.832	1.811
Ordinary	1.853	1.793

SLOPE/W is formulated to solve two factors of safety equations; one with respect to moment equilibrium and the other with respect to horizontal force equilibrium. Janbu's method uses only force equilibrium in its calculations; moment equilibrium is not taken into consideration. It is thought that narrow slices should be used in this method [14].

$$FOS = \frac{\Sigma[\mathrm{cb}+(\mathrm{w}-\mathrm{ub})\tan\emptyset\,][\sec 2\alpha/(1+\tan\alpha\tan\Phi/F)]}{\Sigma \mathrm{w}\tan\alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. (1)$$

Where:
h is the middle height of each slice (the intermediate height).
b is the width of each slice.
W is the weight of each slice = $b*(h_1*\gamma_1 + h_2*\gamma_2)$, h_1, h_2 are the intermediate height of top soil and bottom soil respectively of each slice and γ_1, γ_2 are the unit weight of topsoil and bottom soil of each slice.
α is the angle of the base of slice with horizontal lineC is the cohesion of the soil at failure surface.

u is the pore water pressure for each slice = unit weight of water γw * height of the water column (hw) in each slice.
$\gamma w = 10$ kn/m³.
ϕ is the angle of friction at the failure surface.
Based on the results, generally the factor of safety in Janbu's method is smaller when compared to the factor of safety obtained from the other two methods. For example, in case 2, the resulting factor of safety of 1.822 is less than the Bishop's value of 2.025 using manual methods. The main reason for this is that Janbu's method does not consider moments and assumes that it has a maximum active thrust and a minimum passive resistance. Therefore, it gives a conservative estimate for the factor of safety.

Table 9: Factor of Safety Using Janbu's Method

Method	F O S case 0	F O S case1	F O S case 2	F O S case 3	F O S case 4	F O S case 5	F O S case 6	F O S case7
Janbu's method (Hand Calculation)	1.706	1.827	1.822	1.832	1.065	1.051	1.415	0.723
Janbu's method (Software)	1.637	1.889	1.9	1.811	1.076	1.041	1.479	0.79

The Ordinary method, also known as the Fellenius method only satisfies moment equilibrium. The equation is linear and the repetitive is not considered. Therefore, the error percentage which ranges from 5-20% is bigger than Bishop's method [28].

$$FOS = \frac{\Sigma[cl+(w\cos\alpha - ul)\tan\emptyset]}{\Sigma w \sin\alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2)$$

Where:
l is sloping length of each slice.
With Ordinary's method, the factor of safety is 1.853 in case (3), while for Bishop's method, the factor of safety is 1.987. The first point to note is that, the main reason for the difference in factor of safety between these two methods lies in the fact that Ordinary's method ignores the inter-slice normal forces, meaning that the slices are not in force equilibrium that due to the side forces of each slice cancel each other, thus result a low factor of safety. Moreover the side forces inclinations of resultants on each slice are not considered as a part of the solution. Thus, this shows considerable differences in the stability analysis [28].

Table 10: Factor of Safety using Fellenius's Method

Method	F O S case 0	F O S case1	F O S case 2	F O S case 3	F O S case4	F O S case 5	F O S case 6	F O S case7
Fellenius method (handcalculation)	1.641	1.967	1.988	1.853	0.918	0.967	1.414	0.811
Fellenius method (Software)	1.669	1.956	1.971	1.793	1.1	1.05	1.486	0.813

Bishop's method satisfies two equilibrium conditions, namely moment equilibrium and vertical force equilibrium on each slice. Bishop's method includes the inter-slice normal forces between the slices and ignores the inter-slice shear forces. The Bishop factor of safety equation is non-linear, meaning that an iterative technique is required to solve the factor of safety from the equation below. Iterations of successive approximation are performed until value of F converges to within a given tolerance [25].

$$FOS = \frac{\Sigma[\mathrm{cb}+(\mathrm{w}-\mathrm{ub})\tan\emptyset\,][\mathrm{Sec}\,\alpha/(1+\tan\alpha\tan\Phi/F)]}{\Sigma \mathrm{w}\sin\alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3)$$

As shown in the table 11 below, the most calculated factors of safety obtained from Bishop's method SLOPE/W and hand calculations are very similar, with an approximate difference of about ±1%, except both cases 3 and 5 with an approximate difference of about +4% and -9% respectively. Therefore, they mostly concur and yield great results with 1% difference. Indeed, this is due to the fact that moment equilibrium is usually insensitive to inter-slice shear forces [24].

Table 11: Factor of Safety using Bishop's Method

Method	**F O S case 0**	**F O S case 1**	**F O S case2**	**F O S case3**	**F O S case4**	**F O S case 5**	**F O S case 6**	**F O S case 7**
Bishop's method (Hand Calculation)	1.778	1.998	2.025	1.987	1.202	0.964	1.573	0.837
Bishop's method (Software)	1.774	2.001	2.015	1.949	1.198	1.052	1.585	0.825

Based on the results, the factor of safety in cases 4, 5 and 7 is less than 1.5 when compared to factor of safety for other cases. The embankment cannot be constructed exactly at the crest of the slope, as the factor of safety is only around 1.1, 1 and 0.8 respectively whilst a higher factor is required. Therefore, the embankment will be stable but will likely fail due to several reasons which are highlighted below.

The slope angle has a great influence on stability analysis of embankment. Indeed, as shown in cases 4, 5 and 7 the slope angle is 36º, compared to cases 0, 1, 2 and 3 where it is 26.56º. Therefore, the instability of embankment will be considered. As a result, the increase in slope angle will lead to high probability of foundation failure [26].

As a result, the differences between these methods do not present themselves in the form of the slip surfaces but instead in the inter-slice force assumptions and the formula of equilibrium equation which each method satisfies [24].

Water table has been included in all cases, meaning that the resulting factor of safety will be low compared to ignoring the water table. The water table decreases the soil strength and increases pore water pressure as well as shear force, since the shear force has a significant effect on the embankment which is built on soft clay soil. For years now it has been known that soft clay has low shear strength which is a big problem as most solutions are sensitive to shear strength and the parameters. In such conditions, thefactor of safety will be low due to high inter-slice shear forces [1].

In addition, the water table increases soil weight, and therefore most of the loading is caused by soil weight meaning that an increase in soil weight would necessarily lead to collapse. It should be noted that designing with low factor of safety increases the possibility of large vertical and lateral ground deformations, as well as the risk of failure. Moreover, the weight distribution along the slope is crucial, since the loading of the slope peak has an effect on the stability [9].

According to Wong, 2001 properties of the soft clay layer such as angle of faction and cohesion are important for the stability analysis of embankment in terms of height of embankment. That means the weakness of any type of soil refers to the properties of this soil such as cohesion, when this parameter is high the soil is good and strong because cohesion has a great effective on factor of safety [27].

Conclusion

All conclusions have been made on the basisof the comparisons of the results obtained from the analytical methods and those obtained from the SLOPE/W and SIGMA/W analyses. The embankments upon a number of different types of soft clay have been analysed. Hence the following conclusions are made:-

The factor of safety (FOS) measurement for the all cases except case 7 is about 1.0 or more using both ways, hence the embankment is unlikely to fail on soft clay. For these cases the values of factor of safety and of the deformation which have taken place indicate that the embankment is envisaged to be stable and durable for a lengthy period of time if there are no external forces on the embankment, for example, forceful dynamic weights caused by excess traffic, heavy vehicles or ground shifts due to earthquakes. There is little lateral deformation, confirming that there is no indication of the sliding of the slope.

These findings can then assist engineers in choosing a suitable method that almost certainly will decrease the cost of construction and increase the speed of construction. As a result it is anticipated that this study will present an innovative approach to the difficulties surrounding the bearing capacity of soft clay.

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