STABILITY ANALYSIS OF EARTH SLOPES USING QIMSTAR SOFTWARE

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ABSTRACT This study helps us understand slope stability, it analysis and method through literature review and some work examples. In this study, you get to know some of the methods used and to calculate them. Methods like the Ordinary is also known as the Swedish circle method, Simplified Bishop method, Spencer's method, Morgenstern-Price method, and many others. Throughout this study, you see some worked examples and their results. Some results are questionable not because it is wrong but because it falls in between the safe and unsafe factor of safety region. In conclusion, you find out that some of the work examples are not safe to be used some are questionable whilst others are safe. We get to know why they are safe, questionable, or unsafe.

Keywords; Stability analysis; Swedish method; Bishop method; shear stress; shear strength; slope stability; mass movement; failure; mechanics of limit equilibrium procedures

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1 INTRODUCTION

1.1 General

A slope is an inclined boundary surface between air and the body of an earthwork such as a dam, highway cut, or fill. Soil with sloping (non-horizontal) surfaces may be because of natural or artificial (man - made) causes. Natural slope occurs out of natural instances such as, earthquakes, volcanic eruption, falling of rocks whilst artificial causes are due to earthworks such as the construction of a dam, highway cuts or fills.

Evaluating the stability of slopes in soil is an important, interesting, and challenging aspect of civil engineering. Concerns with slope stability have driven some of the most important advances in our under-standing of the complex behavior of soils. Extensive engineering and research studies performed over the past 70 years provide a sound set of soil mechanics principles with which to attack practical problems of slope stability. Over the past decades, experience with the behavior of slopes and often with their failure, has led to development of improved understanding of the changes in soil properties that can occur over time. Recognition of the requirements and the limitations of laboratory and in situation testing for evaluating soil strengths, development of new and more effective types of instrumentation to observe the behavior of slopes. Improved understanding of the principles of soil mechanics that connect soil behavior to slope stability, and improved analytical procedures augmented by extensive examination of the mechanics of slope stability analyses, detailed comparisons with field behavior, and use of computers to perform thorough analyses. Through these advances, the art of slope stability evaluation has entered a more mature phase, where experience and judgment, which continue to be of prime importance, have been combined with improved understanding and rational methods to improve the level of confidence that is achievable through systematic observation, testing, and analysis. This seems an appropriate stage in the development of the state of the art to summarize some of these experiences and advances in a form that will be useful for students learning about the subject and for geotechnical engineers putting these techniques into practice.

1.2 Slope Stability

Slope stability is the potential of soil-covered slopes to withstand and undergo movement. The balance of shear stress and shear strength determines stability. A previously stable slope may be initially affected by preparatory factors, making the slope conditionally unstable. Triggering factors of a slope failure can be climatic events, which can then make a slope actively unstable, leading to mass movements. Mass movements can be caused by increase in shear stress, such as loading, lateral pressure, and transient forces. Alternatively, shear strength may be decreased by weathering, changes in pore water pressure, and organic material.

Slope stability involves analyzing whether a slope is safe or not, and how to possibly enhance the stability of the slope. This is indeterminate concept because no slopes made in or of soil can be regarded as fully guaranteed for their stability during their performance over a period of many years. The balance of shear stress and shear strength determines stability. The field of slope stability encompasses static and dynamic of slopes of earth and rock-fill dams, slopes of other types of embankments, excavated slopes, and natural slopes in soil and soft rock. Climatic and hydrologic conditions, and man's activities in the immediate and or adjacent area of the dam or other earthworks, may bring about years later, changes affecting the stability of man – made and natural slope.

The stability of slopes depends on these factors:

1. The type of soil used in making the slope
2. The geometry of the cross section of the slope
3. Weights and load or load distribution
4. Increase in moisture content of the soil material
5. Decrease in shear strength of soil for reasons, other than water

1.3 Types of Mass Movement

Gravity causes sections of cliffs and hills to move down slope. These are called mass movements. Mass movements can be very dangerous because they can happen very

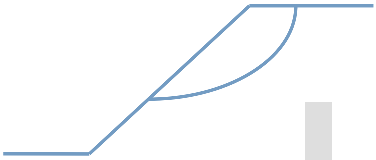
quickly and involve entire hillsides. Mass movement also known as slope rupture or slope

sliding may take place as the result of a shear failure along a given internal surface. Wide varieties of types of movement have been observed. As to the mode of rupture, the slope in different soils results in different soils result in various rupture surface, and the most important types are rotational slip, translational slip and compound slip. The slip surface of slope in homogeneous cohesive soil is in general a continued curved, assumed as circular arc. This type of slope may fail in two ways:

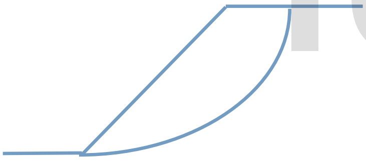
- ✓ The rupture surface sets in above the top of the slope (Fig. 1-1a) and the rupture surface passes through the toe of the slope (Fig. 1-1b). Such failures are termed slope failure.
- ✓ The rupture surface is deep – seated and passes through the embankment supporting soil below the toe of the slop (Fig. 1-1c).

This mode of the slope failure is known as the base failure. The latter mode of failure takes place particularly when the soil beneath the embankment is softer and more plastic than the slope – forming soil itself.

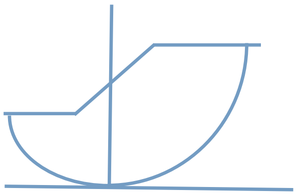
Plane translational slip often takes place in a slope of soil with less cohesion as shown in (Fig. 1-1d). Compound slip occurs where the form of the failure surface is influenced by the presence of an adjacent stratum of significantly different strength (Fig. 1-1e)



a) Slope failure above toe

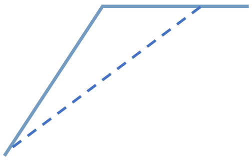


b) Slope failure through toe



c) Base failure

Deep-seated rupture surface below toe



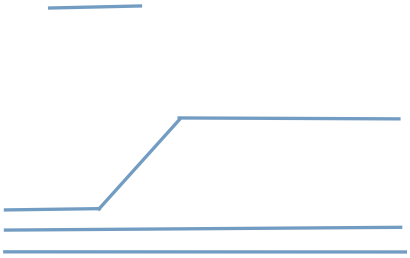


Fig 1.1 (a, b, c, d, e) Slope failures and their movement

1.4 Slope Failure

A slope failure is a phenomenon that a slope collapses abruptly due to weakened self-retain ability of the earth under the influence of a rainfall or an earthquake. Because of collapse of slope, many people fail to escape from it if it occurs near a residential area, thus resulting in a higher rate of fatalities. Slope failures are major natural hazards that occur in many areas throughout the world. Slopes expose two or more free surfaces because of geometry. Plane, wedge, toppling, rock fall and rotational (circular/non-circular) types of failure are common in slopes (Figure 1). The first four are more predominant in rock slopes and are primarily controlled by the orientation and the spacing of discontinuities planes with respect to the slope face. The pattern of the discontinuities may be comprised of a single discontinuity, or a pair of discontinuities that intersect each other, or a combination of multiple discontinuities that are linked together to form a failure mode. Circular and non-circular failure occurs in soil; mine dump, heavily jointed or fractured rock mass and very weak rock. The types of slope failure are primarily controlled by material properties, water content and foundation strength.

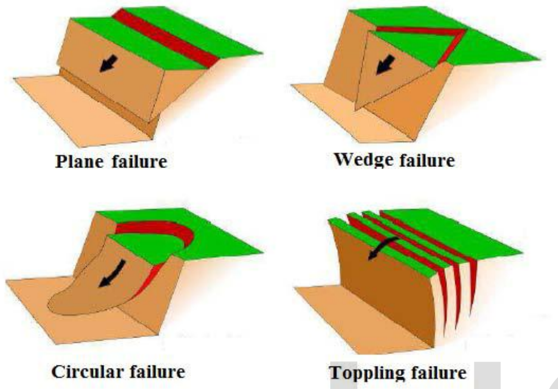


Fig 1.2 Common types of slope failure

1.4.1 Plane failure

A rock slope undergoes this mode of failure when combinations of discontinuities in the rock mass form blocks or wedges within the rock, which are free to move. The pattern of the discontinuities may be comprised of a single discontinuity or a pair of discontinuities that intersect each other, or a combination of multiple discontinuities that are linked together to form a failure mode.

A planar failure of rock slope occurs when a mass of rock in a slope slides down along a relatively planar failure surface. The failure surfaces are usually structural discontinuities such as bedding planes, faults, joints or the interface between bedrock and an overlying layer of weathered rock. Block sliding along a single plane represents the simplest sliding mechanism. The favorable conditions of plane failure are as follows:

- ✓ The dip direction of the planar discontinuity must be within ($\pm 20^{o}$) of the dip direction of the slope face.
- ✓ The dip of the planar discontinuity must be less than the dip of the slope face (Daylight)
- ✓ The dip of the planar discontinuity must be greater than the angle of friction of the surface.

The study of planar failure mechanism provides insight knowledge of the behavior of rock slopes, and is particularly valuable for investigating the sensitivity of slope behavior to variations in parameters such as shear strength of failure surfaces and groundwater conditions.

1.4.2 Wedge Failure

Wedge failure of rock slope results when rock mass slides along two intersecting discontinuities, both of which dip out of the cut slope at an oblique angle to the cut face, thus forming a wedge-shaped block. Wedge failure can occur in rock mass with two or more sets of discontinuities whose lines of intersection are approximately perpendicular to the strike of the slope and dip towards the plane of the slope.

1.4.3 Toppling failure

Toppling failures occur when columns of rock, formed by steeply dipping discontinuities in the rock rotates about an essentially fixed point at or near the base of the slope followed by slippage between the layers. The center of gravity of the column or slab must fall outside the dimension of its base in toppling failure. Jointed rock very closely spaced and steeply dipping discontinuity sets that dip away from the slope surface are prerequisites for toppling failure.

1.4.4 Rock Falls

In rock falls, a rock mass of any size is detached from a steep slope or cliff along a surface on which little or no shear displacement takes place, and descends mostly through the air by free fall, leaping, bouncing, or rolling. It is generally initiated by some climatic or biological event that causes a change in the forces acting on a rock. These events may include pore pressure increase due to rainfall infiltration, erosion of surrounding material during heavy rainstorms, freeze-thaw processes in cold climates, chemical degradation or weathering of the rock, root growth or advantage by roots moving in high winds etc.

1.4.5 Rotational Failure

In rotational slips, the shape of the failure surface in section may be a circular arc or a non-circular curve. In general, circular slips are associated with homogeneous soil conditions and non-circular slips with non-homogeneous conditions. Translational and compound slips occur where the form of the failure surface is influenced by the presence of an adjacent stratum of significantly different strength. Translational slips tend to occur where the adjacent stratum is at a relatively shallow depth below the surface of the slope: the failure surface tends to be plane and roughly parallel to the slope. Compound slips usually occurs where the adjacent stratum is at greater depth the failure surface consisting of curved and plane sections. Circular shear failures are influenced by the size and the mechanical properties of the particles in the soil or the rock mass. Fig. 1-1a, 1-1b, and 1-1c illustrates a few typical modes of circular shear failure. This failure can occur in rock structures that exhibit no plane of weakness, and may not be associated with any underlying critical discontinuity.

Below are some damages caused as results of slope failure

(a)

(b)

Fig 1.3 Damages caused by rock fall (a, b)

2 Slope Stability Analysis

Slope stability analysis is performed to assess the safe design of human-made or natural slopes (e.g. embankments, road cuts, open-pit mining, excavations, landfills etc.) and the equilibrium conditions. Before the age of computers, stability analysis was performed graphically or by using a hand-held calculator. Today engineers have a lot of possibilities to use analysis software, ranges from simple *limit equilibrium* techniques through to computational limit analysis approaches (e.g. Finite element limit analysis, Discontinuity layout optimization) to complex and sophisticated *numerical solutions* (finite-/distinct-element codes). The engineer must fully understand limitations of each technique. For example, limit equilibrium is most commonly used and simple solution method, but it can become inadequate if the slope fails by complex mechanisms (e.g. internal deformation and brittle fracture, progressive creep, liquefaction of weaker soil layers, etc.). In these cases, more sophisticated numerical modelling techniques should be utilized. In addition, even for very simple slopes, the results obtained with typical limit equilibrium methods currently in use (Bishop, Spencer, etc.) may differ considerably. In addition, the use of the risk assessment concept is increasing today. Risk assessment is concerned with both the consequence of slope failure and the probability of failure (both require an understanding of the failure mechanism). Within the last decade, (2003) Slope Stability Radar has been developed to remotely scan a rock slope to monitor the spatial deformation of the face. The methods of slices have become the most common methods due to their ability to accommodate complex geometrics and variable soil and water pressure conditions (Terzaghi and Peck 1967). During the past three decades approximately one dozen methods of slices have been developed (Wright 1969). They differ in (i) the statics employed in deriving the factor of safety equation and (ii) the assumption used to render the problem determinate (Fredlund 1975). Below are six of the most commonly used methods:

- ✓ Ordinary or Fellenius method (some-times referred to as the Swedish circle method or the conventional method)
- ✓ Simplified Bishop method
- ✓ Spencer's method
- ✓ Janbu's simplified method
- ✓ Janbu's rigorous method
- ✓ Morgenstern-Price method

2.1 Mechanics of Limit Equilibrium Procedures

Once appropriate shear strength properties pore water pressures, slope geometry and other soil and slope properties are established, slope stability calculations need to be performed to ensure that the resisting forces are sufficiently greater than the forces tending to cause a slope to fail. Calculations usually consist of computing a factor of safety using one of several limit equilibrium procedures of analysis. All of these procedures of analysis employ the same definition of the factor of safety and compute the factor of safety using the equations of static equilibrium.

2.2 FACTOR OF SAFETY (FOS)

In slope design, and in fact generally in the area of geotechnical engineering, the factor, which is very often in doubt, is the shear strength of the soil. The loading is known more accurately because usually it merely consists of the self-weight of the slope. The FOS is therefore chosen as a ratio of the available shear strength to that required to keep the slope stable.

Table 2.1 Guidelines for limit equilibrium of a slope

FACTOR OF SAFETY	DETAILS OF SLOPE
<1.0	Unsafe
1.0-1.25	Questionable safety
1.25-1.4	Satisfactory for routine cuts and fills, Questionable for dams, or where failure would be catastrophic
>1.4	Satisfactory for dams

For highly unlikely loading conditions, factors of safety can be as low as 1.2-1.25, even for dams. Example; situations based on seismic effects or a rapid drawdown of the water level in a reservoir.

2.3 DEFINITION OF THE FACTOR OF SAFETY

The *factor of safety, F,* is defined with respect to the shear strength of the soil as $F = \frac{s}{t}, t = \frac{s}{F}, \; s = Ft$

Expressed by the Mohr– Coulomb equation, the total stresses will be

$$t = \frac{s}{F} \qquad (2.1)$$

$$t = \frac{cs \tan f}{F} \quad (2.2)$$

$$t = \frac{c}{f} \frac{s \tan f}{F} \quad (2.3)$$

The equilibrium shear stress is equal to the available shear strength divided (*factored*) by the factor of safety. The factor of safety represents the factor by which the shear strength must be reduced so that the reduced strength is just in equilibrium with the shear stress (t) (i.e., the slope is in a state of just-stable *limiting equilibrium*. The procedures used to perform the quantities c_d and f_d represent the developed (or *mobilized*) cohesion and friction angle, respectively.

To calculate the factor of safety, a slip surface is assumed and one or more equations of static equilibrium are used to calculate the stresses and factor of safety for each surface assumed. The term *slip surface* is used here to refer to an assumed surface along which sliding or rupture might occur. However, it is the intent of slope stability calculations that sliding and rupture not occur along such surfaces if the slope is designed adequately.

The factor of safety is assumed to be the same at all points along the slip surface. Thus, the value represents an average or overall value for the assumed slip sur-face. If failure were to occur, the shear stress would be equal to the shear strength at all points along the failure surface and the assumption that the factor of safety is constant would be valid. If, instead, the slope is stable, the factor of safety probably varies along the slip surface (e.g., Wright et al., 1973). However, this should not be of significant consequence as long as the overall factor of safety is suitably greater than 1 and the assumed shear strengths can be fully mobilized along the entire slip surface.

A number of slip surfaces must be assumed to find the slip surface that produces a minimum factor of safety. The surface with the minimum factor of safety is termed the *critical slip surface*. Such a critical surface and the corresponding minimum factor of safety represent the most likely sliding surface, presuming that all of the shear strengths have been determined in a comparable way and with comparable degrees of certainty. Although the slip surface with the minimum factor of safety may not represent a failure mechanism with a significant consequence, the minimum factor of safety is unique for a given problem and should be calculated as part of any analysis of stability.

2.3.1 Recapitulation

- The factor of safety is defined with respect to shear strength.
- The factor of safety is applied to both cohesion (c, c9) and friction (tan f, tan f9).
- The factor of safety is computed for an assumed slip surface.

- The factor of safety is assumed to be constant along the slip surface.
- A number of different slip surfaces must be assumed and the factor of safety computed for each to determine a critical slip surface with a mini-mum factor of safety.

2.4 SWEDISH METHOD

If the cross-section of a slope-forming body of soil is composed of cohesive soil layers, each layer of them has different shear strength properties and stresses along the trail slip surface vary. If a homogeneous slope is partially submerged or through a homogenous dam, a seepage takes place, then stability calculations of slopes over circular rupture surfaces can be more conveniently perform by the method of slices as originally shown by Peterson. Ordinary or Fellenius Method (Swedish method).

The ordinary method of slices also referred to as the Swedish method is considered the simplest of the methods of slices since it is the only procedure that results in a linear factor of safety equation. It is generally stated that the inter-slice forces can be neglected be-cause they are parallel to the base of each slice (Fellenius 1936). However, Newton's principle of 'action equals reaction' is not satisfied between slices (Fig. 2) . The indiscriminate change in direction of the resultant inter-slice force from one slice to the next results in factor of safety errors that may be as much as 60% (Whitman and Bailey 1967). The normal force on the base of each slice is derived either from summation of forces perpendicular to the base or from the summation of forces in the vertical and horizontal directions.

$$T_{ri} = \frac{\tau_f \Delta l_i}{F_s} = \frac{(C+\sigma_n \tan\theta)\Delta l_i}{F_s} \quad (2.4)$$

$$= \frac{c\Delta l_i + N_n \tan\theta}{F_s} \quad (2.5)$$

$$= \frac{c\Delta l_i + W_i \cos\alpha_i \tan\theta}{F_s} \quad (2.6)$$

$$\sum_{i=1}^{m}(W_i R \sin\alpha_i) = \sum_{i=1}^{m}(T_{ri} R) \quad (2.7)$$

$$F_s = \frac{\sum_{i=1}^{m}(c\Delta l_i + W_i \cos\alpha_i \tan\phi)}{\sum_{i=1}^{m}(W_i \sin\alpha_i)} \quad (2.8)$$

2.5 BISHOP METHOD

Different from the Swedish method, this method has a vertical side force difference as follows:

$$\Delta T_i = T_i - T_{i-1} \quad (2.9)$$

$$W_i + \Delta T_i = N_n \cos\alpha_i + \left(\frac{c\Delta l_i + N_n \cos\alpha_i \tan\theta}{F_s}\right)\sin\alpha_i \quad (2.10)$$

$$N_n = \frac{W_i + \Delta T - c\Delta l^{\sin\alpha}/F_s}{\cos\alpha_i + \tan\theta^{\sin\alpha_i}/F_s} \quad (2.11)$$

$$F_s = \frac{\sum_{i=1}^{m}\left[(cb_i + W_i\tan\phi + \Delta T\tan\theta)\right]/m_{ai}}{\sum_{i=1}^{m}(W_i\sin\alpha_i)} \quad (2.12)$$

$$m_{ai} = \cos\alpha_i + \frac{\tan\theta\sin\alpha_i}{F_s} \quad (2.13)$$

$$\Delta T = 0 \quad (2.14)$$

$$F_s = \frac{\sum_{i=1}^{m}\left[(cb_i + W_i\tan\theta)\right]/m_{ai}}{\sum_{i=1}^{m}(W_i\sin\alpha_i)} \quad (2.15)$$

3 STABILITY ANALYSIS OF EARTH SLOPES

3.1 Stability Of Natural Earth Slopes And Constructed Embankments

Modern farmers are being forced to increase the productivity of their farms in order to satisfy expanding economic and consumer requirements. Additional capital works such as new roads, canals and dams, or the cultivation of steeper slopes of farming land may become necessary. The stability of earth slopes, embankments and hillsides is therefore a factor which is important to the fanner, both for economic reasons and because the safety of human lives may be affected by badly designed dams and embankments. The object of this paper is to provide the farmer and the scientist with a summary of some of the factors, which should be considered. This paper will be sub-divided into three categories, namely: (a) The stability of earth dams;
(b) The stability of road embankments; and
(c) The stability of natural hillsides.

3.1.1 The Design and Stability of Earth Dams

In the case of farms, dams built to a height of less than 10 feet it is usually sufficient to provide an adequate spillway and earth side slopes of approximately 1 vertical to **3** horizontal units. However, special precautions should be taken for larger dams, and may prevent financial losses in the smaller dams. Because the stability of a dam is also affected by the over-topping of the dam and matters other than Soil Mechanics theory, the author will also mention these other problems.

3.1.2 Location of the Dam

The location of the dam is chosen based on the following factors:
(i) The narrowness of the valley;
(ii) The size of the watershed;
(iii) The proximity of the irrigated lands and canals;
(iv) The available volume of storage in the proposed dam;
(v) The possibility of additional canals for leading the water into the dam. These five factors may be assessed from contour maps of the area (see Ref. 1, Ref. 2).
(vi) In addition, the geology of the dam site must be

considered. In dolomite or limestone areas, underground seepage channels and sinkholes can cause large water losses from the dam. Fissured shales and sand lenses also cause water to be lost and may cause springs or otherwise adversely affect the stability of the downstream slope of the dam. Fissures in rocks can often be sealed by pumping concrete grout down drill holes into the rock. In certain areas of South Africa, "collapsing sands" soften and settle upon being wetted if a load exists on the sand. Stiff fissured clays can also soften with time. If water is poured into a saucer on which stands a mound of sugar, it will be noticed that the sugar slumps and slides downwards on the soft layer of syrup which is formed under the mound. In a similar manner an earth dam built on a soft horizontal layer of clay may slump and slide outwards in both the upstream and downstream directions. It may be necessary to remove a soft foundation clay, even though clay is the best soil for preventing seepage losses

3.2 Water Run-off

When assessing the quantity of water, which will run into the dam, two separate calculations, should be made:

(i) The first calculation should be a conservative estimate of the available volume of water, which is available for storage. The object of this calculation is to determine whether the dam is an economical proposition. For large dams the "hydraulic mass diagram" is used (Ref. 3,4). In this diagram, the Accumulated Flow (gallons) in the stream is plotted against time. To plot this diagram, it is necessary to gauge the stream flow for a number of years by using measuring weirs. Alternatively, a rough calculation may be made by using the average monthly rainfalls if a correction is made for evaporation and the absorption of water into the soil.

(ii) The second calculation is to determine the required spillway capacity. It is obvious that this must depend upon the worst flood conditions. Unfortunately, many hydraulics textbooks quote overseas rainstorm figures, which are inadequate for South African design purposes. In May 1905, a rainfall of 17.65 inches was recorded during 24 hours in Durban, and 15.65 inches fell at Marian hill in 15 hours. These values exceed most overseas figures.

3.3 The Stability of the Dam

Two main factors influence the stability of the dam. These are firstly the types of soil chosen and secondly the relative geometrical positions of the different soil types in the dam. Coarse sandy materials usually possess good strength characteristics, but allow the water to percolate freely through the dam. Clayey soils are almost impermeable, but they give unreliable strengths unless compacted under engineering supervision. For this reason, clayey soils are usually used for an impermeable center core wall, which extends downwards into the dam sub-base. The sandy soils are used for the upstream and downstream banks on both sides of this core wall (see Fig. 1). Difficulty is experienced in convincing farmers that an earth dam requires flatter upstream and downstream slopes than the angle of repose of the freshly deposited soil. It can be proved by theory and practice that a steep dry slope, which is stable, may become unstable when water seeps out of the slope. This instability is caused mainly by the water pressures in the soil and by the fact that the moisture increases the weight of the soil. The *downstream* slope of a dam must therefore be flatter than the angle of repose of the soil. Whenever the water level in the dam is lowered, water will also flow backwards into the dam from the voids in the dam embankment. (This is known as the 'Draw-down condition'.) For this reason, the *upstream* slope of the dam must also be flatter than the angle of repose of the soil. Water must be diverted before it seeps out of the downstream slope in the form of springs. Graded stone filters below the toe of the downstream slope will effectively gather the permeating water before it appears on the downstream slope. These filters will also lower the water table and water pressures in the embankment. On no account should an impermeable blanket be laid on the *downstream* slope to "prevent" water seepage, because water pressures will build up behind the blanket. Graded stone filters should be provided in all large dams, and they will also increase the stability of smaller dams. The grading of these stone sizes should be done after a sieve analysis has been made on the available materials. A grading theory has been developed in order that the pores of a coarse aggregate may not be blocked by smaller particles from the adjacent finer soil. If the filters are blocked, portions of the dam may become unstable. A few permanent vertical standpipes (1$ ins. diameter) will allow periodic checks on the level of the water table in the embankment (see Fig. 3). If the filters become ineffective, the water table will rise. An alternative dam construction is shown in Fig. 2a. This is suitable for an area where there is a shortage of sandy material. A cut-off wall may be built at a site where there is little clay (Fig. 5). Alternatively, polythene sheets, or a clay blanket, may be used (Fig. 4). However, the clay blanket shown in Fig. 4 may be unstable in dams in which the draw-down condition occurs (e.g. dams used for irrigation purposes).

3.4 Soil Tests

The following tests should be performed before a large dam is designed:

(a) Atterberg Limit tests, and linear shrinkage tests.

This is to classify the soils.

(b) Permeability tests to determine the ease with which water can permeate through the soils.

(c) Optimum water content tests to assist with the compaction control of large dams.

(d) Sieve analysis of the filter material to determine the particle size grading curve. Sieve analysis (and hydrometer grading tests) can also be used to estimate the permeability of the soil.

(e) In large dam's, sheer strength tests should be performed both before and during construction especially when dealing with clayey material. These tests indicate the strength of the material and the permissible slopes for the banks. (f) Inspection test pits should be made in the proposed sub-base for the embankment.

3.4.1 Shear strength tests

A shear box consists of a metal box which is divided horizontally into an upper and a lower portion (see Fig. 8a). A bearing plate in the top portion transmits the force N to the soil. A horizontal force F is applied to the upper portion to cause the soil to shear at the level X - X. Obviously in the case of a sand an increase in the value of force N will require an increase in the failure value of force F. The failure combinations of F and N are plotted in Fig. 8b. The slope of this line is known as the angle of friction + of the soil. In the case of a sand only a slight value of F is required to cause failure if there is no force N. However, in the case of clays the cohesion between the soil grains is such that even when the force N is zero an appreciable value of F is required for failure. If this particular value of F is divided by the cross-sectional area **A** of the shear box, the resulting value F/A is known as the "cohesion c" of the clay (see Fig. 8b).

The shear box is not used to find the cohesion c and the angle of friction + of clays. Instead, the usual test for a clay is the "saturated undrained triaxial test" performed on soaked samples which are compacted at approximately "optimum water content". The values of c and + for the soil are used to calculate a suitable slope for the dam. This suitable slope is also dependent on the weight of the soil *y* (lb/ft3), and the working height Hw of the slope. The author with aid of an electronic digital computer derived the chart in Fig. 10. In this chart Hw is measured in feet. The weight *y* of the soil is usually 125 (lb/ft3). The cohesion c is expressed as pounds per square foot. This chart has been calculated for a slope of 1 vertical unit in **3** horizontal units. Charts for other slopes have also been obtained. The chart should be entered with the known values of Hw, c, + and *y* in order to find the factor of safety F. If F is found to be 1.0, or less, the bank will fail. The value of F should be greater than 2.5, unless engineers, in which case a lower value of F can be used, supervise the dam. The value of F will be increased if flatter slopes are used.

3.5 Reinforced Slopes and Embankments

Reinforcement can be used to improve the stability of slopes and embankments, making it possible to construct slopes and embankments steeper and higher than would otherwise be possible. Reinforcement has been used in four distinct types of applications:

1. Reinforced slopes.
2. Multiple layers of reinforcement at various elevations within fill slopes have been used to increase the factor of safety for slip surfaces that cut through the reinforcement, making it possible to construct slopes steeper than would be possible without reinforcement.
3. Reinforced embankments on weak foundations. Reinforcement at the bottom of an embankment on a weak foundation can increase the factor of safety for slip surfaces passing through the embankment, making it possible to construct the embankment higher than would be possible with-out reinforcement.
4. Reinforced soil walls or mechanically stabilized earth walls. Several different proprietary systems have been developed for reinforced soil walls, which are used as alternatives to conventional retaining walls. Most of the companies that market MSE walls have developed proprietary design procedures. The stability of MSE walls can also be evaluated using the methods described in this chapter.
5. Anchored walls. Vertical soldier pile walls or slurry trench concrete walls can be ‘‘tied back’’ or anchored at one or more levels to provide vertical support for excavations or fills. Anchored walls have been used in both temporary and permanent applications. The methods described in this chapter can be used to evaluate the stability of anchored walls.

3.6LIMIT EQUILIBRIUM ANALYSES WITH REINFORCING FORCES

Reinforced slopes can be analyzed using the procedures described in Chapter 6 by including the reinforcement forces in the analyses as known forces. Zornberg et al. (1998 *a*, *b*) have shown through centrifuge tests that limit equilibrium analyses provide valid indications of factor of safety and failure mechanisms for reinforced slopes. Their analyses, which agreed well with the results of their tests, were performed using peak values of *f*9 rather than the lower critical-state friction angle of the backfill soil. The amount of force required to achieve a target value of factor of safety can be determined using repeated trials, varying the magnitude of the force until the factor of safety computed is the one desired. Some computer programs can perform this operation automatically—the input is the desired factor of safety, and the output is the required reinforcement force. This type of program is better adapted to design of rein-forced slopes, since there is no need for repeated analyses.

3.6.1 FACTORS OF SAFETY FOR REINFORCING FORCES AND SOIL STRENGTHS

Two methods have been used for limit equilibrium analyses of reinforced slopes.

- Method A. The reinforcement forces used in the analysis are allowable forces and are not divided by the factor of safety calculated during the slope stability analysis. Only the soil strength is divided by the factor of safety calculated in the slope stability analysis.
- Method B. The reinforcement forces used in the analysis are ultimate forces, and are divided by the factor of safety calculated in the slope stability analysis. Both the reinforcing force and the soil strength are divided by the factor of safety calculated in the slope stability analysis.

Method A is preferable, because the soil strength and the reinforcement forces have different sources of uncertainty, and they therefore involve different amounts of uncertainty. Factoring them separately makes it possible to reflect these differences. When a computer program is used to analyze rein-forced slopes, it is essential to understand which of these methods is being used within the program, so that the appropriate measure of reinforcing force (allowable force or ultimate force) can be specified in the input for the analysis.

If the documentation of a computer program does not specify whether the reinforcement force should be allowable or ultimate, this can be deduced from the equations employed to compute the factor of safety.

Method A Equations

If the factor of safety for circular slip surfaces is defined by an equation of the form

$$F = \frac{\text{soil resisting moment}}{\text{overturning moment} - \text{reinforcement moment}} \tag{3.1}$$

or, more generally, if the factor of safety is defined by an equation of the form

$$F = \frac{\text{shear strength}}{\text{shear stress required for equilibrium} - \text{reinforcement resistance}} \tag{3.2}$$

the program uses method A, and the reinforcement forces specified in the input should be allowable forces, denoted here as P_{all}.

.

Method B Equations

If the factor of safety for circular slip surfaces is defined by an equation of the form

$$F = \frac{\text{soil resisting moment}+\text{reinforcement moment}}{\text{overturning moment}} \qquad (3.3)$$

4 TONGJI QIMSTAR SOFTWARE SLOPE V.1.0

4.1.1 Software Function Introduction

Tongji Qimstar Software Slope V. 1.0 helps in easily calculating the various types of retaining wall, and the safety factor overall stability of the slope, not just that, this software aslo provides many kinds of calculation methods and models, to help you according to the actual situation analysis.

- ✓ Method to choose: Swedish method and Bishop method.
- ✓ Slope shape: Arbitrary shape
- ✓ Sliding surface type: Can calculate the circular arc sliding surface and arbitrary sliding surface, the circular arc sliding surface can help you search out the most dangerous sliding surface.
- ✓ The stress pattern choice: Provide total stress model and effective stress; can also according to the seepage situation considering or not considering seepage force.

  Slope load: can be anywhere in the slope surface and a line distribution load.

4.1.2 Basic Work flow of Software

Use of this software, the basic working process as shown in the figure below:

6. Start ⇨Open or build new file ⇨Enter or modify data ⇨Calculating ⇨Calculation error Output result Modify the data to calculate Save file Finish.

4.2 Operation Steps

This section is to help you understand the user interface on the meaning of each data item, will also tell you step by step how to input data to get the final result , and finally form the data file.

4.2.1 Start Slope

Double-click the icon for the slope under the windows working area. You can start the software, showing the cover of the software Fig 4.1

Click or press anywhere on the screen or the cover to the software.

Fig 4.1 The cover of the software

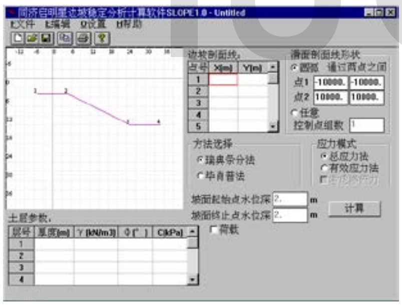


Fig 4.2 Working interface

4.2.2 Data input of the slope line

The slope line is composed of a series of points. The data of these points are saved in “slope line” form (Fig 4.3). From a line in a point, where the first as an X coordinates of points, and they are based on the user coordinates system, grid lines in the drawing area is the scale line user coordinate system, is at the top of the painting area and on the left with the scale live scale value. They are “m” for unit and the direction of the user’s coordinate system are stipulated as “x” is positive to the right and down “y” is also positive. It’s important to note that these points from the top to the bottom in the form of order into line, so you must pay attention to when fill in point and the relative position between the points.

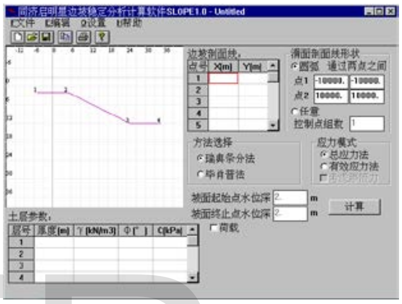
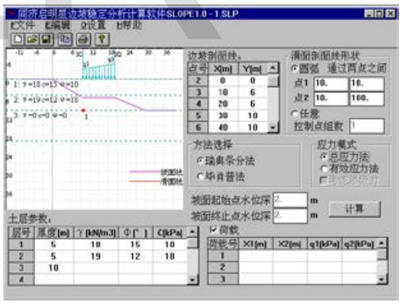


Fig. 4.3 Slope line form



点号	X[m]	Y[m]
2	0	0
3	10	6
4	20	6
5	30	10
6	40	10

层号	厚度[m]	γ[kN/m3]	Φ[°]	C[kPa]
1	5	18	15	10
2	5	19	12	18
3	10			
4				

Fig. 4.4 Slope line

Note:

1. Dot big point of the value of x is greater than or equal to the dot small point x, that is, the cant appear in a chamfering phenomenon;
2. The starting point and end point is limited in slide slope surface starting line (the first) and suspension line (last line). If the starting line and the stop line is too short, it will affect the

calculation result of circular arc sliding surface and if its too long, it will increase the computation time;

3. The slope of the sliding direction is stipulated as from left to right. For circular arc sliding surface, sliding the beginning is always higher than the end of the arc, and slope stop line at least part of below slope starting line. To use the mouse to drag the arbitrary sliding surface.
4. Slope surface starting line and stop line not vertical

4.2.3 Entering Of the Soil Data

Soil data table is located in the “soil parameters” under the two-dimensional table, it’s a line said a layer of earth, you can enter up to 20 layers of soil, measuring each column parameters are as follows as in the example c=5kPa, φ=20°, γ=20kN/m^3:

“m” is the unit for the thickness of the soil layer

“kN/m$^{3:}$ is the unit for γ

“°” is the unit for the internal friction of the soil angle

“kPa” is the unit for “c” which is the soil cohesion

When you lose every line of the first column of the data, the bank said the soil with a dotted line is drawn out in the drawing area, then the column parameters are initialized to been written as the data changes as shown if Fig 4.4.

Note:

1. At the top of the first layer of the soil highest point located on the slope surface, and only when there is slope, soil interface can be mapped in the graphics area.
2. The sum of each soil layer thickness must be large enough so the system could determine whether the soil thickness is large enough. If the soil layer isn’t thick enough, the system won’t be able to calculate the already inputs and also an error message will be sent.

4.2.4 Input Load Data

If there is no load side, this one does not have to fill in data, but for the following conditions, can make the following treatment:

1. Complex natural slope of the slope shape, with a fold line in place of the missing portion inevitably lead to lumps, which partially deleted load can be used to replace the mud;
2. To have a slope retaining wall, the wall will be greater than severe soil, this difference can be used to make up the load, which is equal to the size of (the weight of the wall - wall weight within the range of the soil) □ wall

thick;

3. Load acts on the slope (e.g., human, mechanical activity, etc. contained in the stack) can also be used to simulate the load.

When you want to fill in the data load, check "Load" Check button, the data table appears below the load, while the load pattern drawing area appears tips, such as **Error! Reference source not found.**.4. It represents a load of a trapezoidal distribution line, x1 column starting point representative of load, x2 representative of the end of the column load, q1 column load represents the starting point of the set, q2 set of columns indicates the end of the load. After the fill line, which represents the load is plotted. When you do not load, clear "Load" Check button, and load data tables and graphics load disappears. Note that only when there is a slope line, the load can be drawn.

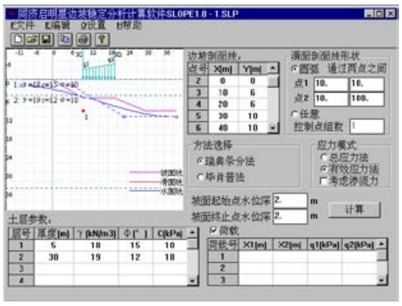
Notice: When the load is very narrow range (| x1-x2 | <0.01), it will not be counted.

4.2.5 Selection Method

The software provides two methods namely ***“Swedish”*** and ***“Bishop”*** method. Click on the "*Swedish slice method*" Radio button for the Swedish slice method, if not you can also click on “*Bishop Method*" Radio button for the Bishop method. The default on this software is the Swedish method.

4.3 The Stress Patterns

This software provides a choice of two stress modes, if the soil is clayey, select "Total stress method" Suitable more; such as sandy soil, is selected "Effective stress method" Reasonable; The presence of groundwater flow, need to consider seepage force. Note that only when the stress mode is effective stress model in order to consider the seepage force, so that when "Effective stress method" When the radio button is selected, "Considering seepage force "Check button was not prohibited, see **Error! Reference source not found.** , otherwise" Considering seepage force “Check button is prohibited, see **Error! Reference source not found.**5”.



边坡剖面线:

点号	X[m]	Y[m]
2	0	0
3	10	6
4	20	6
5	30	10
6	40	10

滑面剖面线形状
● 圆弧 通过两点之间
点1 10. 10.
点2 10. 100.
○ 任意
控制点组数 1

方法选择
● 瑞典条分法
○ 毕肖普法

应力模式
○ 总应力法
● 有效应力法
☐ 考虑渗流力

坡面起始点水位深 2. m
坡面终止点水位深 2. m

土层参数:

层号	厚度[m]	γ[kN/m3]	Φ[°]	C[kPa]
1	5	18	15	10
2	30	19	12	18
3				
4				

☑ 荷载

荷载号	X1[m]	X2[m]	q1[kPa]	q2[kPa]
1				
2				
3				

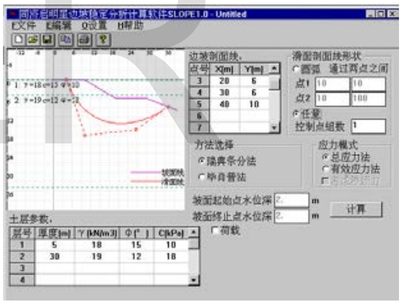
Fig 4.5 Checking button is prohibited

4.4The input of groundwater data

Only stress mode is effective stress method, the water table makes sense. Select "Effective stress method" After the radio button, "Slope starting point for deep water" with "End point water level deep slope". After the edit box becomes active, while a blue line appears in the groundwater level slope. This line consists of four Control points, four points with a blue dashed connection, known as the control polygon. The four points of the first and last are fixed, by "Slope starting point for deep water" with "End point water level deep slope "To control the edit box, use the mouse to intermediate points "drag" (See mouse "drag" Definitions), thus obtained the water line you need. When the slope is higher than the groundwater level, and a slope surface line conventions line coincide. Note that only when there is a slope surface line, water lines to appear.

4.5 Selection of slide surface shape

There are two options sliding surface shape: circular sliding surface and the sliding surface of any shape, respectively "Arc" Radio button and "Arbitrarily" Radio button. If you select "Arc" Radio button, which becomes effective at the edit box and four "The number of control points set" After the edit box is invalid. These four-edit boxes represent two-points-Arc control point (shown as two solid red dot in the drawing area. For controlling the position of the substantially circular slip surface Arc passing between these two points, two edit box above the point x 1 and y coordinates, the following two points x and y coordinates input 2, which are relative to the user coordinate system. If you select "Arbitrarily" Radio button that controls the number of points set in the edit box becomes active control point and an arc four edit box becomes inactive. While the control point of the arc disappears, a red line appears on which a number (group of control points□3 + 1) points, which can be "drag". To control the shape of the sliding surface wherein the first and last, respectively, can move only in the first and last line of slope, see **Error! Reference source not found.**4.6. Only when there is a slope surface line, smooth surface lines to appear.



边坡剖面线:

点号	X[m]	Y[m]
3	20	6
4	30	6
5	40	10
6		
7		

滑面剖面线形状
○圆弧 通过两点之间
点1 10 10
点2 10 100
●任意
控制点组数 1

方法选择
●瑞典条分法
○毕肖普法

应力模式
●总应力法
○有效应力法
☐

坡面起始点水位深 2. m
坡面终止点水位深 2. m
☐荷载

土层参数:

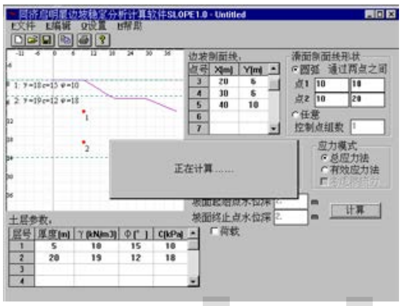
层号	厚度[m]	γ[kN/m3]	φ[°]	C[kPa]
1	5	18	15	10
2	30	19	12	18
3				
4				

Error! Reference source not found.Fig 4.6 Slide surface shape

Note: two arc control points are valid only in the range of positive slope below, otherwise they will not be bound by the position of the arc.

4.6 Computing

Good data is lost after the above procedure can be calculated, and clicks "Compute" Button. If you enter data or unreasonable lack of data, an error message will pop-up dialog box, or pop-up "Counting" Message boxes. Wait a moment, the message box disappears, the calculation result is displayed below the drawing area, see Fig 4.7. If the user data have been adjusted, click "Compute" Data button, the software will be recalculated and updated results.



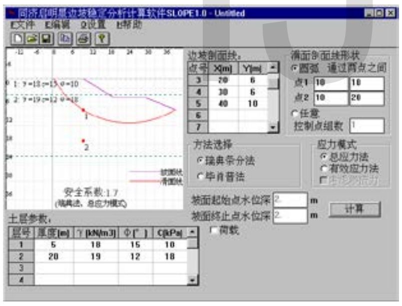
正在计算......

点号	X[m]	Y[m]
3	20	6
4	30	6
5	40	10
6		
7		

点1 10 18
点2 10 20

层号	厚度[m]	γ[kN/m3]	Φ[°]	C[kPa]
1	5	18	15	10
2	20	19	12	18
3				
4				

Fig 4.7 Computing



安全系数:1.7
(瑞典法、总应力模式)

点号	X[m]	Y[m]
3	20	6
4	30	6
5	40	10
6		
7		

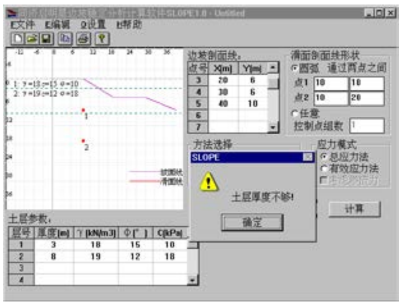
●圆弧 通过两点之间
点1 10 10
点2 10 20
控制点组数 1
●瑞典条分法
●总应力法
坡面起始点水位深 2 m
坡面终止点水位深 2 m

层号	厚度[m]	γ[kN/m3]	Φ[°]	C[kPa]
1	5	18	15	10
2	20	19	12	18
3				
4				

Fig 4.8 Computed results

4.6.1 Error message

When your data is unreasonable, execution "Compute" after the command, an error message will pop up a message box, as Fig 4.9 Prompt error message box with an orientation effect on your errors in the data. Press "Enter" key or click "determine" button may return the modified data.



SLOPE
土层厚度不够!
确定

点号	X[m]	Y[m]
3	20	6
4	30	6
5	40	10
6		
7		

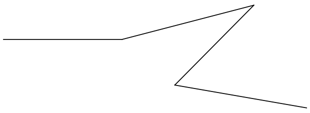
●圆弧 通过两点之间
点1 10 10
点2 10 20
控制点组数 1
●总应力法

层号	厚度[m]	γ[kN/m3]	Φ[°]	C[kPa]
1	3	18	15	10
2	8	19	12	18
3				
4				

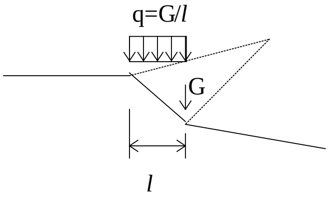
Fig 4.9 Error message

Error message boxes, the meaning of the text is as follows:

1. At least two slope section line regulations are required at least two lines in order to constitute the slope.
2. A first slope section line cannot be vertical or last. A first cross-sectional line of a predetermined slope or last not vertical. The first vertical impossible, such as the user needs to determine a final vertical, can be handled as follows; add in the final surface as short horizontal lines of termination. The following reasons; presumably, if the termination of the vertical line, the most dangerous slip surface will through the end. Before or after adding the vertical line ensures short horizontal sliding surface approximated by the endpoint.
3. The slope section line cannot appear in a "Chamfer phenomenon" such as Fig 4.10. The slope cannot be calculated so that the weight of the strips of soil, and therefore is not allowed. The existence of this kind of slopes, can be "Chamfer" Resection, on behalf of the load, see Fig 4.11.
4. The thickness of the soil is not enough.



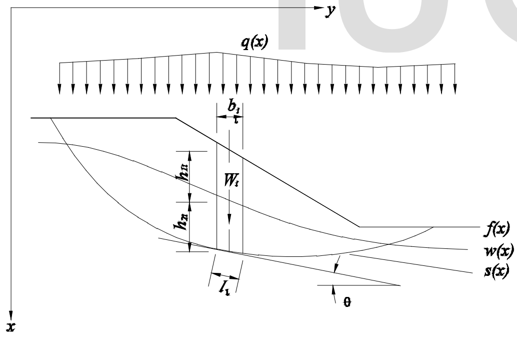
(4.10)



(4.11)

4.6.2 Calculation for the theoretical foundation

Swedish Method (Total Stress Method)



(4.12)

$$F_s=\frac{\sum_{i=1}^{n}\left[c_il_i+(q_ib_i+W_i)\cos\theta_i tg\varphi_i\right]}{\sum_{i=1}^{n}(q_ib_i+W_i)\sin\theta_i} \quad (4.13)$$

In the formula, W_i −−the natural weight of the soil in article i, qi −−the average load on the soil of article i, c_i,φ_i −− cohesive force and internal friction angle. Transformation formula;

$$F_s=\frac{\sum_{i=1}^{n}\left[c_il_i+(q_ib_i+W_i)\cos\theta_i tg\varphi_i\right]}{\sum_{i=1}^{n}(q_ib_i+W_i)\sin\theta_i} \quad (4.14)$$

$$\underline{\underline{n\longrightarrow\infty}}\frac{\int_{\overline{ABC}}(c+\sigma_c\cos^2\theta tg\varphi)ds}{\int_{x_A}^{x_C}\sigma_c\sin\theta dx_c} \quad (4.15)$$

$$=\frac{\int_{x_A}^{x_C}\left(c+\frac{\sigma_C tg\varphi}{1+s'^2(x)}\right)\sqrt{1+s'^2(x)}dx}{\int_{x_A}^{x_C}\frac{\sigma_C s'(x)dx}{\sqrt{1+s'^2(x)}}} \quad (4.16)$$

Effective Stress Method

$$F_s=\frac{\sum_{i=1}^{n}\left[c_il_i+(qb_i+W_i')\cos\theta_i tg\varphi_i'\right]}{\sum_{i=1}^{n}(qb_i+W_i')\sin\theta_i} \quad (4.17)$$

$$\underline{\underline{n\longrightarrow\infty}}\frac{\int_{x_A}^{x_C}\left(c'+\sigma' tg\varphi'\big/(1+s'^2(x))\right)\sqrt{1+s'^2(x)}dx}{\int_{x_A}^{x_C}\sigma'_C s'(x)dx\big/\sqrt{1+s'^2(x)}} \quad (4.18)$$

Effective Stress Method Considering The Seepage Force

$$F_s=\frac{\sum_{i=1}^{n}\left[c'_il_i+\left[(qb_i+W_i)\cos\theta_i-u_il_i\right]tg\varphi'_i\right]}{\sum_{i=1}^{n}(qb_i+W_i)\sin\theta_i} \quad (4.19)$$

$$=\frac{\sum_{i=1}^{n}\left[c'_il_i+b_i(q+\gamma h_{1i}+\gamma_m h_{2i}-\gamma_w\frac{h_{wi}}{\cos^2\theta_i})tg\varphi'_i\right]}{\sum_{i=1}^{n}(q+\gamma h_{1i}+\gamma_m h_{2i})\sin\theta_i} \quad (4.20)$$

In the formula, u_i is the pore water pressure at the bottom of soil i, $u_i = \gamma_w h_{wi,}$ γ_w is bulk density of water, make

$$F_s = \frac{\sum_{i=1}^{n}\left[c_i l_i + (qb_i + W_i^{'})\cos\theta g\varphi_i^{'}\right]}{\sum_{i=1}^{n}(qb_i + W_i)\sin\theta_i} \qquad (4.21)$$

$$\underline{\underline{n \longrightarrow \infty}}\ \frac{\int_{x_A}^{x_c}\left(c^{'} + \sigma^{'}\Big/(1+s'^2(x))^{tg\varphi^{'}}\right)\sqrt{1+s'^2(x)}dx}{\int_{x_A}^{x_c}\sigma^{'}{}_c s'(x)dx\Big/\sqrt{1+s'^2(x)}} \qquad (4.22)$$

Bishop Method (Total Stress Method)

$$F_s = \frac{\dfrac{\sum_{i=1}^{n}\left[c_i l_i + (qb_i + W_i)\cos\theta tg\varphi_i^{'}\right]}{\cos\theta_i + \dfrac{\sin\theta_i tg\varphi_i}{F_s}}}{\sum_{i=1}^{n}(qb_i + W_i)\sin\theta_i} \qquad (4.23)$$

$$\underline{\underline{n \longrightarrow \infty}}\ \frac{\int_{x_A}^{x_c}(c^{'} + \sigma_c tg\varphi)\sqrt{1+s'^2(x)}dx\Big/(1+\frac{s^{i}(x)tg\varphi}{F_s})}{\int_{x_A}^{x_c}\sigma_c s'(x)\Big/\sqrt{1+s'^2(x)}^{dx}} \qquad (4.24)$$

Effective Stress Method

$$F_s = \frac{\sum_{i=1}^{n}\dfrac{\left[c_i^{'} l_i + (qb_i + W_i^{'})\cos\theta tg\varphi_i^{'}\right]}{\cos\theta_i + \dfrac{\sin\theta_i tg\varphi_i}{F_s}}}{\sum_{i=1}^{n}(qb_i + W_i)\sin\theta_i} \qquad (4.25)$$

$$\underline{\underline{n \longrightarrow \infty}}\ \frac{\int_{x_A}^{x_c}(c^{'} + \sigma_c^{'} tg\varphi^{'})\sqrt{1+s'^2(x)}dx\Big/(1+\frac{s^{'}(x)tg\varphi^{'}}{F_s})}{\int_{x_A}^{x_c}\sigma_c^{'} s'(x)dx\Big/\sqrt{1+s'^2(x)}} \qquad (4.26)$$

Effective Stress Method Considering The Seepage Force

$$F_s = \frac{\sum_{i=1}^{n}\left[c_i^{'} l_i + (qb_i + W_i - u_i b_i)\cos\theta tg\varphi_i^{'}\right]\Big/\left(\cos\theta_i + \frac{\sin\theta_i tg\varphi_i}{F_s}\right)}{\sum_{i=1}^{n}(qb_i + W_i)\sin\theta_i}$$

(4.27)

$$\underline{\underline{n \longrightarrow \infty}}\ \frac{\int_{x_A}^{x_c}(c^{'} + \sigma_c^{'} tg\varphi^{'})\sqrt{1+s'^2(x)}dx\Big/(1+\frac{s^{'}(x)tg\varphi^{'}}{F_s})}{\int_{x_A}^{x_c}\sigma_c^{'} s'(x)dx\Big/\sqrt{1+s'^2(x)}}$$

(4.28)

5 WORKED EXAMPLES

5.1 Introduction

Slope in engineering must be stable; the slope stability factor of safety in engineering is an important indicator that needs to be checked always. However, calculation of slope stability factor of safety generally by Swedish Method and or Bishop Method. Hand calculation is very tiring, time consuming and prone to errors due to tiredness. I used Tongji Qimstar Slope V.1.0 to do all the factor of safety calculations.

5.2 Stability Analysis of earth slopes

This study provides guidance for analyzing the static stability of slopes of earth and rock-fill dams, slopes of other types of embankments, excavated slopes, and natural slopes in soil and soft rock. Methods for analysis of slope stability are described and are illustrated by examples. Criteria are presented for strength tests, analysis conditions, and factors of safety. The criteria in this study are to be used with methods of stability analysis that satisfy all conditions of equilibrium. Methods that do not satisfy all conditions of equilibrium may involve significant inaccuracies and should be used only under the restricted conditions described herein. This manual is intended to guide design and construction engineers, rather than to specify rigid procedures to be followed in connection with a particular project. The stability of dams and slopes must be evaluated utilizing pertinent geologic information and information regarding in situ engineering properties of soil and rock materials. The geologic information and site characteristics that should be considered include:

- Groundwater and seepage conditions.
- Lithology, stratigraphy, and geologic details disclosed by borings and geologic interpretations.
- Maximum past overburden at the site as deduced from geological evidence.
- Structure, including bedding, folding, and faulting.
- Alteration of materials by faulting.
- Joints and joint systems.
- Weathering.
- Cementation.
- Slickensides.
- Field evidence relating to slides, earthquake activity, movement along existing faults, and tension jointing

During my study, I came across some of following and here are the meanings and their units;

Table 5.1 units and there meanings

Symbol	Meaning	units
γ	Weights	K/Nm^3
φ	cohesion	o
c	Friction	kPa

My work examples where in two form. They were use the software to find the effect of varied slope angles with the same slope height on factors of safety and the effect of varied slope angles with the same slope height on factors of safety.

Steps involved

I had to critically understand the use of the software and its command since it is programmed in Chinese language. I had to understand the diagrams and the readings of the diagrams. I also had to know what happens if the thickness is too very little or if the height was slightly adjusted. The damaged it could cause or how helpful it could be to the project being worked on.

Example

Question 1;

a. For slopes with the same slope heights.

The effect of varied slope angles with the same slope height on factors of safety

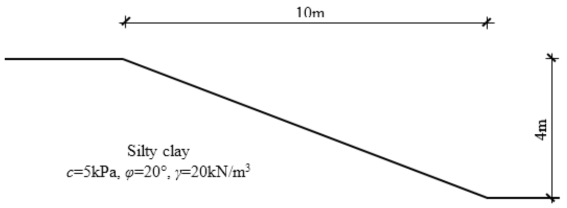


Figure 4-1 Worked example No. 1.1 of soil slope stability analysis

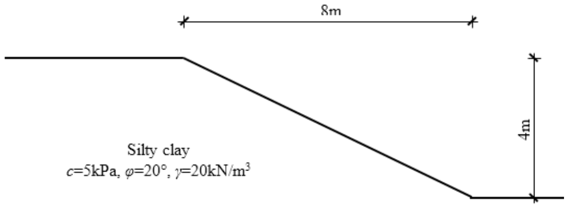


Fig 5.1 working examples with the same slope heights

b. For slopes with the same slope angles

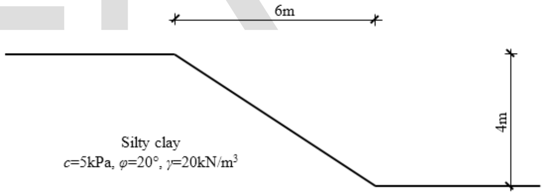


Figure 4-4 Worked example No. 1.4 of soil slope stability analysis

4.1.2 The effect of varied slope heights with the same slope angle on factors of safety

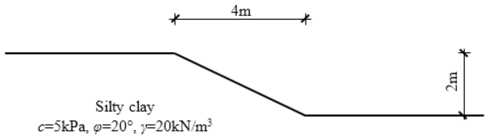
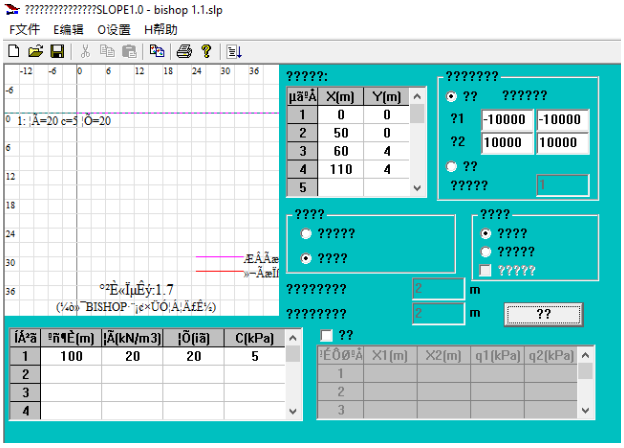


Fig 5.2 working examples with the same slope angle

Swedish and Bishop method calculation



µã ºÅ	X(m)	Y(m)
1	0	0
2	50	0
3	60	4
4	110	4
5		

● ?? ??????
?1 -10000 -10000
?2 10000 10000
○ ??
????? 1

○ ?????
● ????

● ????
○ ?????
☐ ?????

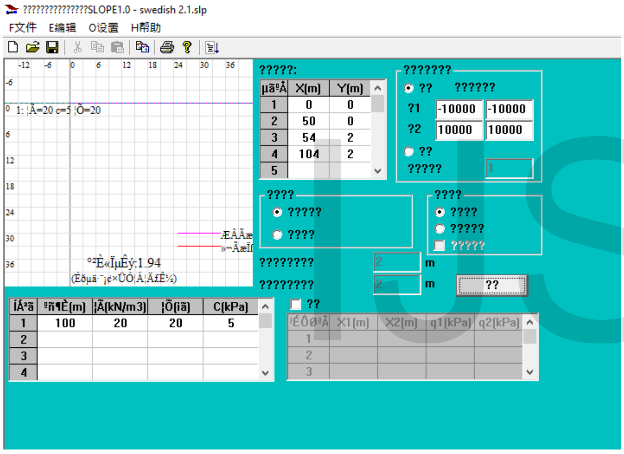
???????? 2 m
???????? 2 m

☐ ??

ÍÁ²ã	ºñ¶È(m)	¦Ã(kN/m3)	¦Õ(ïã)	C(kPa)
1	100	20	20	5
2				
3				
4				

ºÉÔØºÅ	X1(m)	X2(m)	q1(kPa)	q2(kPa)
1				
2				
3				

FIG 5.3 Bishop method calculation example 1a



µã ºÅ	X(m)	Y(m)
1	0	0
2	50	0
3	54	2
4	104	2
5		

● ?? ??????
?1 -10000 -10000
?2 10000 10000
○ ??
????? 1

● ?????
○ ????

● ????
○ ?????
☐ ?????

???????? 2 m
???????? 2 m

☐ ??

ÍÁ²ã	ºñ¶È(m)	¦Ã(kN/m3)	¦Õ(ïã)	C(kPa)
1	100	20	20	5
2				
3				
4				

ºÉÔØºÅ	X1(m)	X2(m)	q1(kPa)	q2(kPa)
1				
2				
3				

Fig 5.4 Swedish method calculation example 1b

Table 5.2 values from working examples 1a

SLOPE ANGLE(°)	HEIGHT(m)	SWEDISH	BISHOP
21.8	4	1.59	1.7
26.5	4	1.4	1.49
29.7	4	1.31	1.39
33.6	4	1.22	1.28

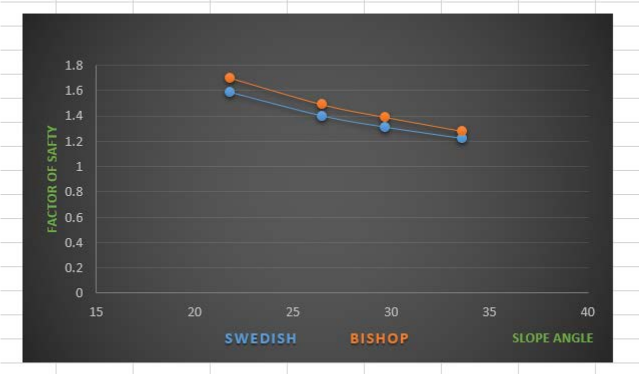


Fig 5.5 Example 1a

The slope angle increases from 20° to 35° as the factor of safety decreases from 1.8 to 1.2

Table 5.3 values from working example 1b

SLOPE ANGLE (°)	HEIGHTS (m)	SWEDISH	BISHOP
26.5	2	1.94	2.05
26.5	3	1.58	1.68
26.5	4	1.4	1.49
26.5	5	1.29	1.37

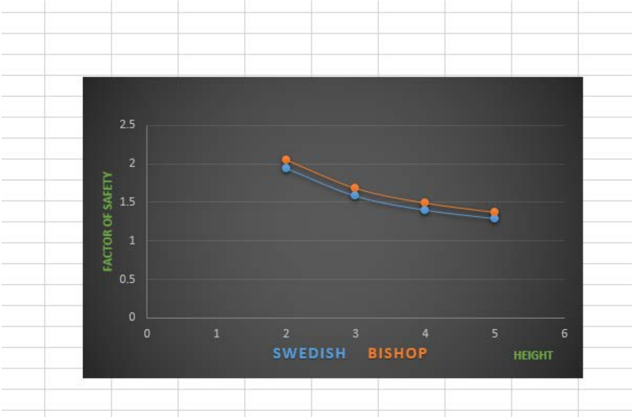


Fig 5.6 Example 1b

The slope height increases from 2m to 5m as the factor of safety decreases from 2.05 to 1.29

Question 2

a. For slopes with the same slope heights in the presence of groundwater

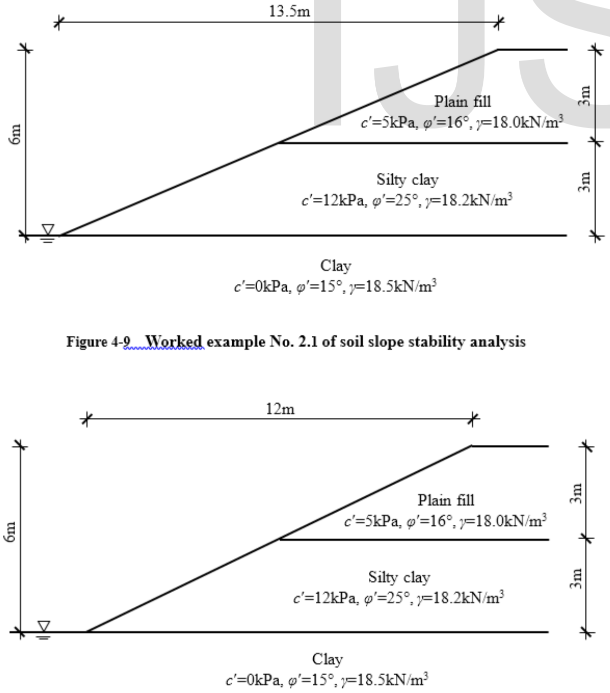


Fig 5.7 working examples with the same slope heights

b. For slopes with the same slope angles in the presence of ground water

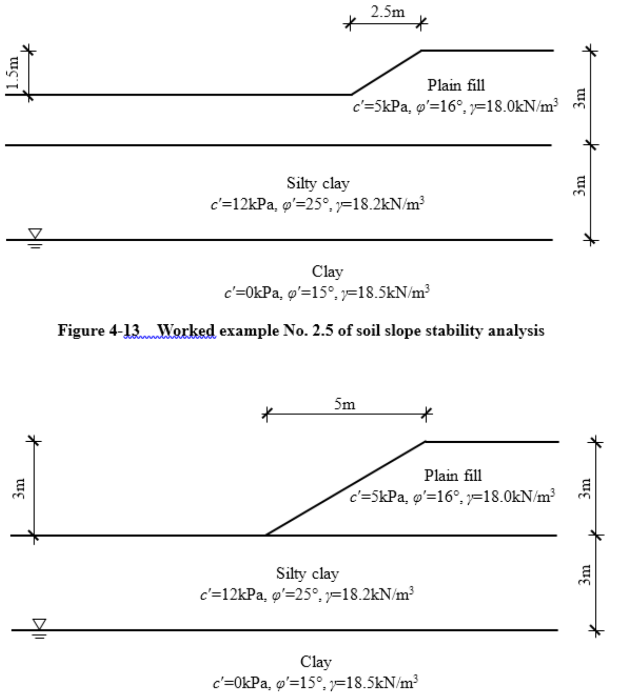
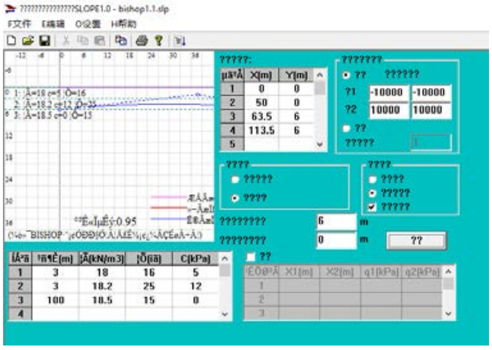


Fig 5.8 working examples with the same slope angle

Swedish and Bishop method calculation

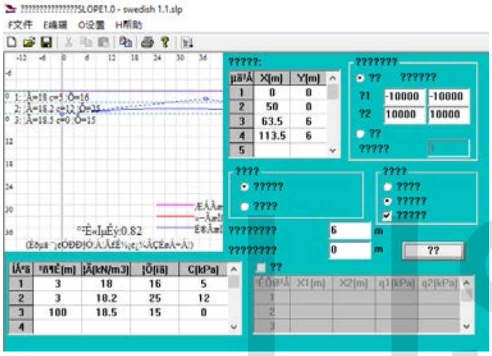


µã³Å	X(m)	Y(m)
1	0	0
2	50	0
3	63.5	6
4	113.5	6
5		

ÍÁ²ã	²ñ¶È(m)	¦Ã(kN/m3)	¦Õ(¶ã)	C(kPa)
1	3	18	16	5
2	3	18.2	25	12
3	100	18.5	15	0
4				

°²È«ÏµÊý:0.95

FIG 5.9 **Bishop method calculation** example 2a

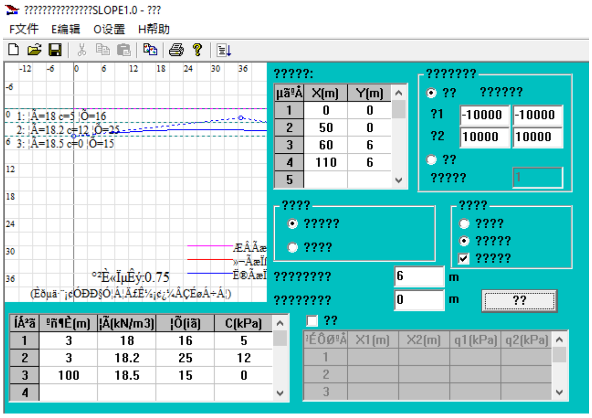


µã³Å	X(m)	Y(m)
1	0	0
2	50	0
3	63.5	6
4	113.5	6
5		

ÍÁ²ã	²ñ¶È(m)	¦Ã(kN/m3)	¦Õ(¶ã)	C(kPa)
1	3	18	16	5
2	3	18.2	25	12
3	100	18.5	15	0
4				

°²È«ÏµÊý:0.82

Fig 5.10 Swedish method calculation example 2a

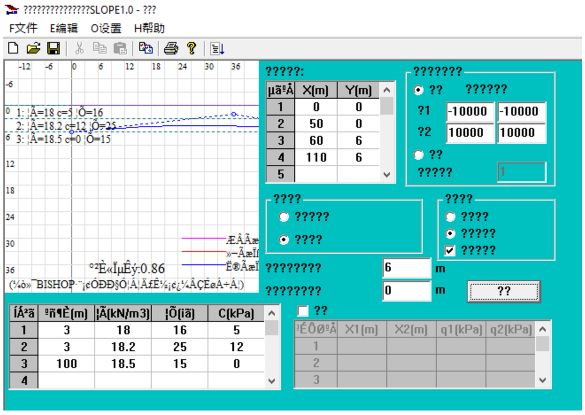


µã³Å	X(m)	Y(m)
1	0	0
2	50	0
3	60	6
4	110	6
5		

ÍÁ²ã	²ñ¶È(m)	¦Ã(kN/m3)	¦Õ(¶ã)	C(kPa)
1	3	18	16	5
2	3	18.2	25	12
3	100	18.5	15	0
4				

°²È«ÏµÊý:0.75

FIG 5.11 **Swedish method calculation** example 2b



µã³Å	X(m)	Y(m)
1	0	0
2	50	0
3	60	6
4	110	6
5		

ÍÁ²ã	²ñ¶È(m)	¦Ã(kN/m3)	¦Õ(¶ã)	C(kPa)
1	3	18	16	5
2	3	18.2	25	12
3	100	18.5	15	0
4				

°²È«ÏµÊý:0.86

FIG 5.12 **Bishop method calculation** example 2b

Table 5.4 values from working examples 2a

SLOPE ANGLE (º)	HEIGHTS (m)	SWEDISH	BISHOP
23.9	6	0.82	0.95
26.5	6	0.79	0.91
30.9	6	0.75	0.86
33.6	6	0.73	0.83

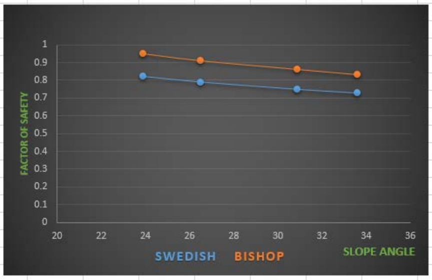


Fig 5.12 Example 2a

The slope angle increases from 24º to 34º as the factor of safety decreases from 0.95 to 0.73

Table 5.5 values from working examples 2b

SLOPE ANGLE (º)	**HEIGHTS (m)**	**SWEDISH**	**BISHOP**
30.9	1.5	2.1	2.19
30.9	3	1.52	1.55
30.9	4.5	1.09	1.4
30.9	6	0.75	0.86

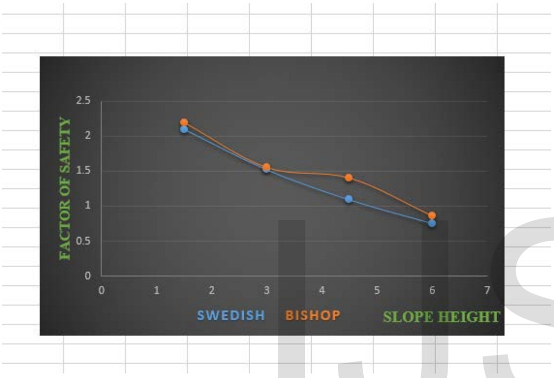


Fig 5.13 Example 2b

The slope height increases from 1.5m to 6m as the factor of safety decreases from 2.19 to 0.75

5.3 Summury

From the above, all the work examples with the same slope angles increases whilst their factor of safety decreases but at a lower margin of less than one. The work examples with the same slope height increases whilst it factor of safety is decreases but at a higher margin of more than one. In this case, the ones with the factor of safety decreasing more than one may not are is not safe for construction.

CONCLUSIONS

According to the study conducted in thesis, some conclusions can be drawn;

1. Evaluation has entered a more mature phase, where experience and judgment, which continue to be of prime importance, have been combined with improved understanding and rational methods
2. Factor of safety of a slope height or angle can easily be derived the use of the methods found in this study, especially Swedish method and Bishop method. Mostly, calculations usually consist of computing a factor of safety using one of several limit equilibrium procedures of analysis
3. Capital works or the construction of new roads, canals and dams, or the cultivation of steeper slopes of farming land may become necessary in the case of stabilizing natural slopes. Some other factors such as farmers increasing the products of their farmlands to satisfy the expanding economic and consumer requirements is given to avoid slope failure.
4. Calculating for the various types of retaining wall, and the overall factor of safety stability of the slope. It has other functions that helps you to calculate in other methods and models according to the actual situation analysis.
5. From the case study, it is found that;
    1) In the absence of groundwater, when the slope height is stable, the slope angle increases whilst the factor of safety decreases. From this analysis and study, we can say the factor is safety is **questionable** because it neither safe nor unsafe to be used.
    2) In the absence of groundwater, when the slope angle is stable, the slope height increases whilst the factor of safety decreases. From this analysis and study, we can the factor of safety is **safe** to be used.
    3) In the presence of groundwater, when the slope height is stable, the slope angle increases whilst the factor of safety decreases. From this analysis and study, we can say the factor is safety nor unsafe to be used is **not safe** to be used.
    4) In the presence of groundwater, when the slope angle is stable, the slope height increases whilst the factor of safety decreases. From this analysis and study, we can the factor of safety is also **not safe** to be used.

REFERENCES

[1] Abrams, T. G., and Wright, S. G. (1972). A Survey of Earth Slope Failures and Remedial Measures in Texas, Research Report 161-1, Center for Highway Research, University of Texas, Austin, TX, December.

[2] Abramson, L. W., Lee, T. S., Sharma, S., and Boyce, G. M. (2002). Slope Stability and Stabilization Methods, 2nd ed., Wiley, Hoboken, NJ.

[3] ACI Committee 318. ACI 318-05 Building code requirements for structural concrete[S]. Farmington Hills: American Concrete Institute, 2005.

[4] Bishop, A. W. (1954). The use of pore pressure coefficients in practice, Geotechnique, 4(4), 148–152.

[5] Bishop, A. W. (1955). The use of slip circles in the stability analysis of earth slopes, Geotechnique, 5(1), 7–17.

[6] Bishop, A. W., and Bjerrum, L. (1960). The relevance of the triaxial test to the solution of stability problems, Proceedings of the ASCE Research Conference on the Shear Strength of Cohesive Soils, Boulder, CO.

[7] Bishop, A. W., and Morgenstern, N. (1960). Stability coefficients for earth slopes, Geotechnique, 10(4), 129–150.

[8] Chen Z Y. Soil slope stability analysis: theory, methods and programs [M]. Beijing: China Water & Power Press, 2003. (in Chinese)

[9] Collins Civil Engineers Diary - obtainable each year from most bookstalls - cost approximately

[10] Deep foundation[DB/OL]. April 5, 2016. https://en.wikipedia.org/wiki/Deep_ foundation.

[11] Duncan, J. M. (1974). Finite element analyses of slopes and excavations, State-of-the-Art Report, Proceedings of the First Brazilian Seminar on the Application of the Finite Element Method in Soil Mechanics, COPPE, Rio de Janeiro, Brazil, pp. 195–208.

[12] Duncan, J. M. (1986). ‘‘Methods of analyzing the stability of natural slopes, notes for a lecture to be presented at the 17th Annual Ohio River Valley Soil Seminar, Louisville, KY.

[13] Duncan, J. M. (1988). Prediction, design and performance in geotechnical engineering, keynote paper or the 5th Australia–New Zealand Conference on Geomechanics, Sydney, Australia.

[14] Duncan, J. M. (1992). State-of-the-art static stability and deformation analysis, Stability and Performance of Slopes and Embankments: II, Geotechnical Special Publication 31, ASCE, Reston, VA, pp. 222–266.

[15] Duncan, J. M. (1993). Limitations of conventional analysis of consolidation

settlement, ASCE, Journal of Geotechnical Engineering, 119(9), 1333–1359.

[16] Duncan, J. M. (1996b). State of the art: limit equilibrium and finite element analysis of slopes, ASCE, Journal of Geotechnical Engineering, 122(7), 577–59.

[17] Holtz, R. D., Lancellotta, R., Jamiolkowski, M., and Pedrona, S. (1991). Prefabricated Vertical Drains: Design and Performance, Construction Industry Research and Information Association, Butterworth-Heinemann.

[18] Hong, W. P., and Han, J. G. (1996). The behavior of stabilizing piles installed in slopes, Proceedings of the 7th International Symposium on Landslides, Rotterdam, pp. 1709–1714.

[19] Hong, W. P., Han, J. G., and Nam, J. M. (1997). Stability of a cut slope reinforced by stabilizing piles, Proceedings of the International Conference on Soil Mechanics and Foundation Engineering, pp. 1319–1322.

[20] Hull, T. S., and Poulos, H. G. (1999). Design method for stabilization of slopes with piles (discussion), ASCE, Journal of Geotechnical and Geoenvironmental Engineering, 125(10), 911–913.

[21] Hull, T. S., Lee, C. Y., and Poulos, H. G. (1991). Mechanics of Pile Reinforcement for Unstable Slopes, Report 636, School of Civil and Mining Engineering, University of Sydney, Sydney, Australia.

[22] Hunter, J. H., and Schuster, R. L. (1968). Stability of simple cuttings in normally consolidated clays, Geotechnique, 18(3), 372–378.

[23] Lee W Y. A freeze-thaw test on Halton till treated with cement kiln dust[D]. Toronto: The University of Toronto, 1999.

[24] Ministry of Housing and Urban-Rural Development of PRC. GB 50007-2011 Code for design of building foundation[S]. Beijing: China Architecture & Building Press, 2012. (in Chinese)

[25] Qian J H, Yin Z Z. Theory and calculation of soil engineering[M]. 2nd edition. Beijing: China Water & Power Press, 1996. (in Chinese)

[26] Rutledge P C. Review of the cooperative triaxial shear research program of the corps of engineers[R]. Mississippi: U.S. Waterways Experiment Station, 1947

[27] Shi J Y. Soil Mechanics[M].Beijing: China Communications Press, 2004.

[28] Taylor D W. Fundamentals of soil mechanics[M]. New York: John Wiley & Sons, 1948.

[29] Terzaghi K, Peck R B. Soil mechanics in engineering practice[M]. 2nd edition. New York: John Wiley & Sons, 1967.

[30] Uriel A O, Sagaseta C. Selection of design parameters for underground construction[C]//Proceedings of the 12th International Congress on Soil Mechanics, Río de Janeiro, Rotterdam: A.A. Balkema, 1989, 9: 2521–2551.

[31] User's Guide of Tongji Qimstar software SLOPE V1.0[R]. Shanghai: Department of Geotechnical Engineering, 1997. (in Chinese)

[32] Verruijt A, Booker J R. Surface settlements due to deformation of a tunnel in an elastic half plane[J]. Géotechnique, 1996, 46(4): 753–757.

[33] Wei R L, Zhang L. Strength parameters in stability analysis[J]. Chinese Journal of Geotechnical Engineering, 1993, 15(5): 24–30. (in Chinese)

[34] Zhu J. The shear strength increase evaluation of soft soils during consolidation[D]. Hangzhou: Zhejiang University, 2010. (in Chinese)