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STABILITY ANALYSIS OF EARTH DAMS: HADITHA DAM AS A CASE STUDY

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By

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تحليل أستقرارية السدود الترايبية: سد حديثة كحالة تطبيقية

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مقدمة الى كلية الهندسة جامعة بابل كجزء من
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عدي عبد الصاحب الطرقي

(بكالوريوس هندسة مدني – 2004)

آيار - 2009

CERTIFICATION

We certify that we have read this thesis entitled “**Stability Analysis of Earth Dams (Haditha Dam as a Case Study)**”, and as examining committee examined the student “**Uday Abdul-Sahib Al-Turrfy**” in its contents and in what is connected with it, and that in our opinion it meets the standard of a thesis for the Degree of Master of Science in **Water Resources Engineering**.

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ABSTRACT

In this research, slope stability analysis of earth dams under different load conditions is investigated using two approaches; **Deterministic** and **probabilistic**. The limit equilibrium method according to Bishop and Morgenstern-Price presented by the computer program **SLIDE** is applied to define the potential slip surface and calculate the factor of safety of the dam slopes. The effects of each design parameter and load conditions on this factor of safety are studied. Design parameters include soil properties (shear strength parameters), geometric properties, and load condition (rapid drawdown and seismic force). Sensitivity analysis is made to investigate the effect of each design parameter and shows the parameters which have sharp effects on the factor of safety of side slopes. Stability of upstream side decreases gradually from the beginning of drawdown to a ratio of drawdown equal to $2/3$ at which the most dangerous state may occur, but not until the emptying.

In case of zoned earth dams, taking Haditha Dam as a case study, the finite element method performed by **SLIDE** program is used to analyze the seepage problem, total head, pressure head distribution, and position of phreatic surface for three major cases, namely, maximum, normal, and minimum water level. Moreover, side slope stability is investigated under different load conditions. The results of analyses showed that a critical condition would happen in the upstream side in case of rapid drawdown condition and seismic load. However, the Dam would be closely safe under the other two cases considered, resulting in a minimum safety factor of (1.09).

APPENDIX- A

SLIDE V.5.0 Computer Program

A-1 General

SLIDE V.5.0 is the most comprehensive slope stability analysis available, complete with sensitivity, probabilistic and back analysis capabilities. It is the only slope stability software to include built-in steady state unsaturated groundwater analysis capabilities using the finite element method. This program is product of **Rocscience** Inc., company.

SLIDE analyzes the stability of slip surfaces using vertical slice limit equilibrium methods (e.g. Bishop, Janbu, Spencer, etc). Individual slip surfaces can be analyzed, or search methods can be applied to locate the critical slip surface for a given slope. Deterministic (safety factor) or probabilistic (probability of failure) analyses can be carried out.

Features include:

- Critical surface search methods for circular or non-circular slip surfaces.
- Analysis methods include Bishop, Janbu, Spencer, GLE / Morgenstern-Price.
- Multiple materials.
- Anisotropic, non-linear Mohr-Coulomb materials.
- Probabilistic analysis – calculate probability of failure, reliability index.
- Sensitivity analysis.
- Groundwater – piezo surfaces, Ru factors, pore pressure grids, finite element groundwater analysis (see below), B-bar factor (excess pore pressure).
- Tension crack (dry or water filled).
- External loading – line, distributed or seismic.
- Support – soil nails, tiebacks, geotextiles, piles. Infinite strength (slip surface exclusion) zones.

- Back analysis of required support force for a given safety factor.
- View any or all surfaces generated by search.
- Detailed analysis results can be plotted for individual slip surfaces.

A-2 SLIDE Model

The **SLIDE** program consists of 3 program modules:

- **MODEL**
- **COMPUTE**
- **INTERPRET**

MODEL is the pre-processing program used for entering and editing the model boundaries, loads, material properties, groundwater conditions, slip surface definition, and saving the input file.

MODEL, **COMPUTE** and **INTERPRET** will each run as standalone programs. They also interact with each other as illustrated in figure (A-1) below:

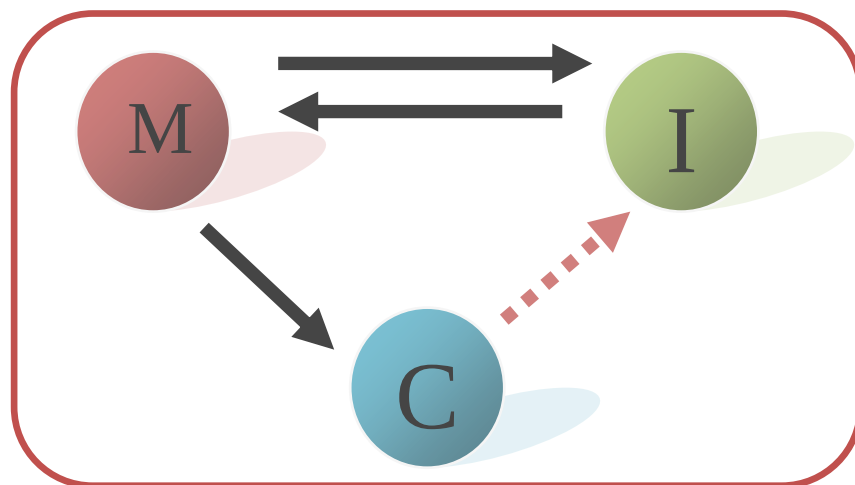


Figure (A-1) The interact between (Model, Compute and Interpret) with each other in SLIDE program.

The structure block of the input data is shown in Figure (A-2). This diagram describes the main features of the input data, and describes the model of groundwater and slope stability by using **SLIDE** program.

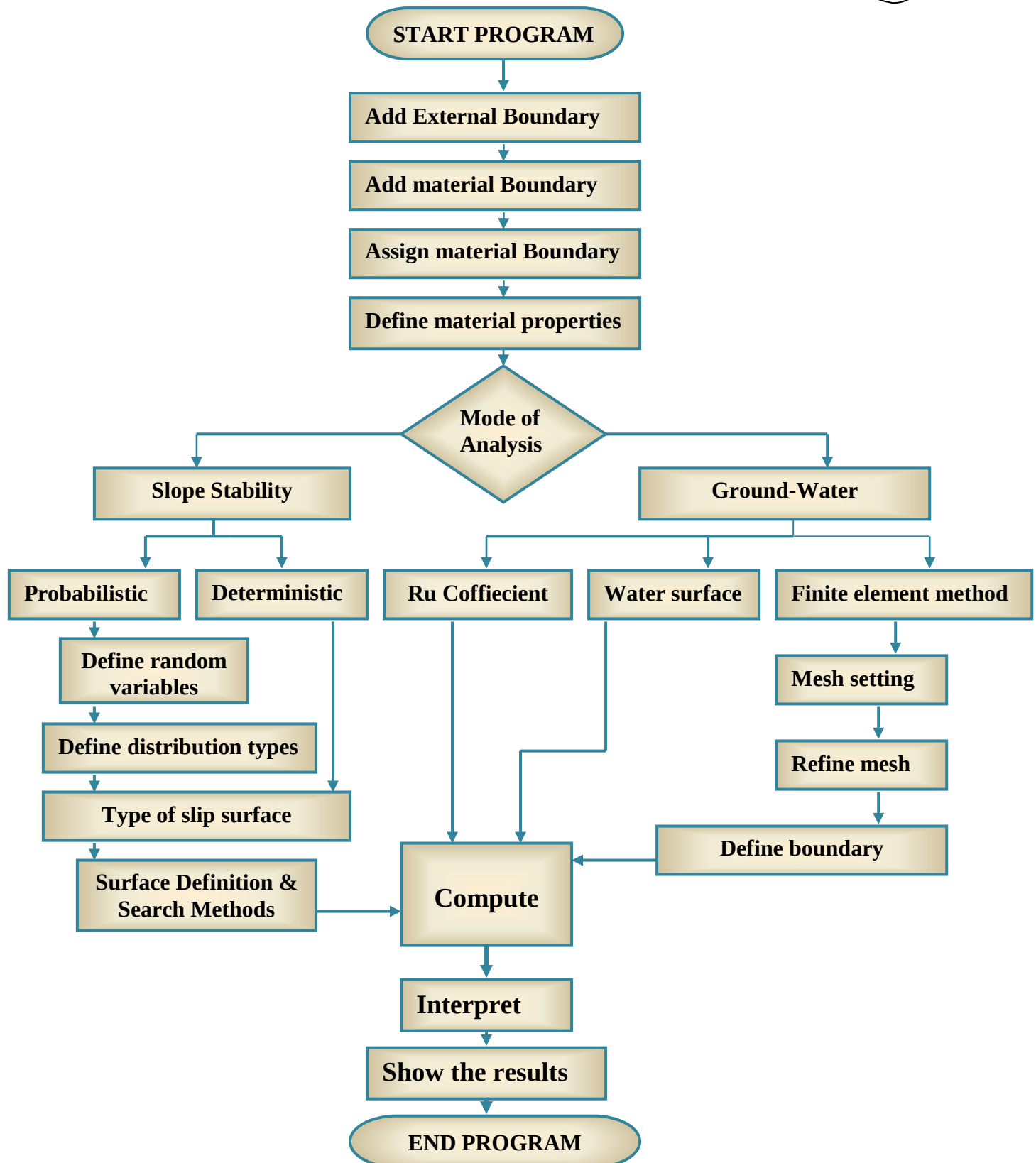


Figure (A-2) Structure blocks of the modeling by **SLIDE** program for both, seepage analysis and slope stability analysis.

APPENDIX- B

Step by Step Solution by SLIDE

B.1 General.

The following paragraphs present step by step solution of examples problems that prementioned in Chapter Three in part of verification of slope stability analysis using **SLIDE** program.

- The first step in creating a model is to draw the boundaries defining the model. Boundaries are added using the add external boundary, add material boundary , add water table , add piezometric line, and add tension crack options. Boundaries can also be added using the import DXF option; it can be defined external boundaries by adding a table coordinates or using graphical user interface (mouse).
- Define the material properties; cohesion, soil unit weight, and angle of internal friction.
- Define the water condition and pore water pressure (the R_u coefficient). In the illustration example, pore water pressure is presented by two ways, the R_u coefficient (0. and 0.25), and position of phreatic surface (by adding a water table in the domain).
- Define the search method and shape of slip surface.

B-2 Solution of Example No.1 (Al-Jorany, 1996).

The most critical slip surfaces and slope profiles for this example problem for different water conditions are presented in the following figures.

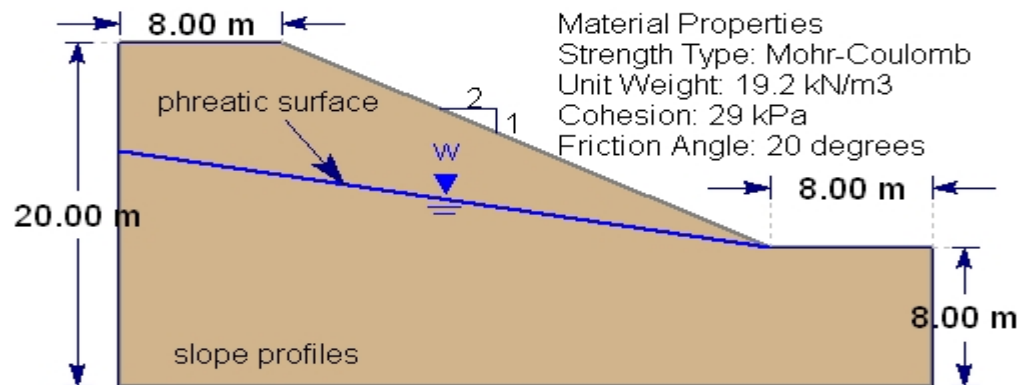


Figure (B.1) Slope profiles for example problem No.1.

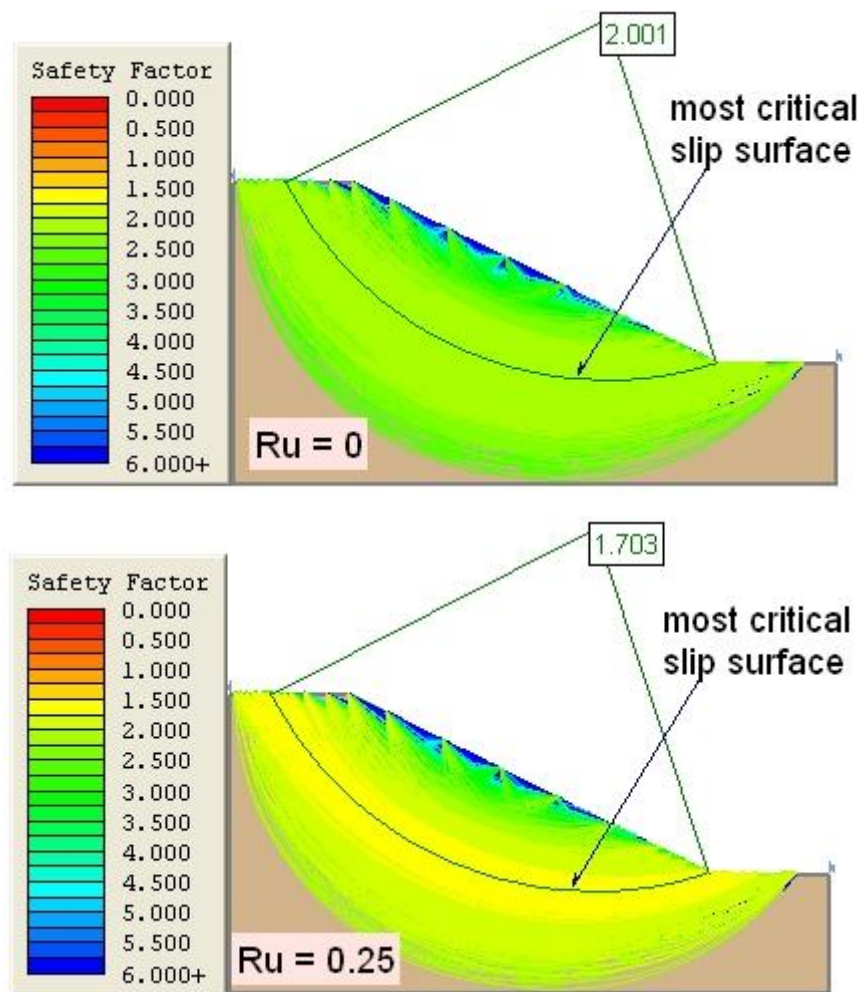


Figure (B.2) Most critical slip surface, example No.1 $R_u = 0$ and, 0.25.

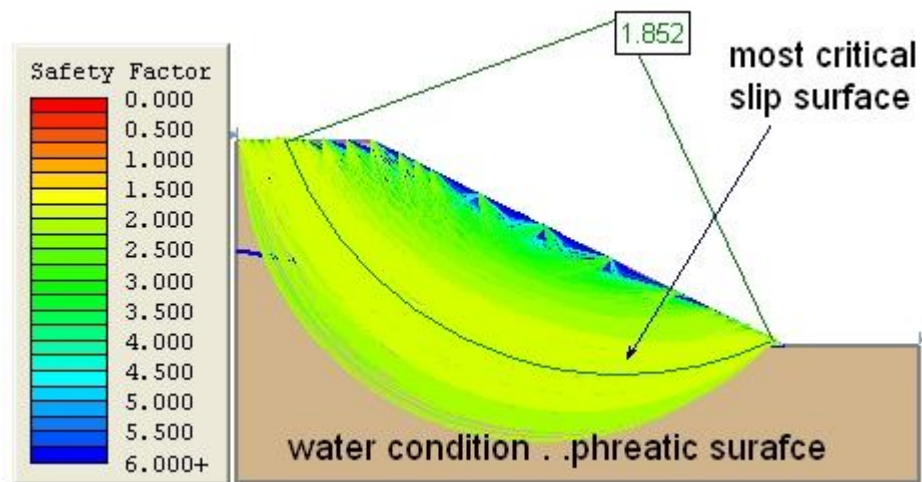


Figure (B.3) Most critical slip surface example problem No.1, water condition, phreatic surface.

B-3 Solution of Example No.2, (Al-Jorany, 1996)

The most critical slip surfaces and slope profiles for this example problem for different water conditions are presented in the following figures.

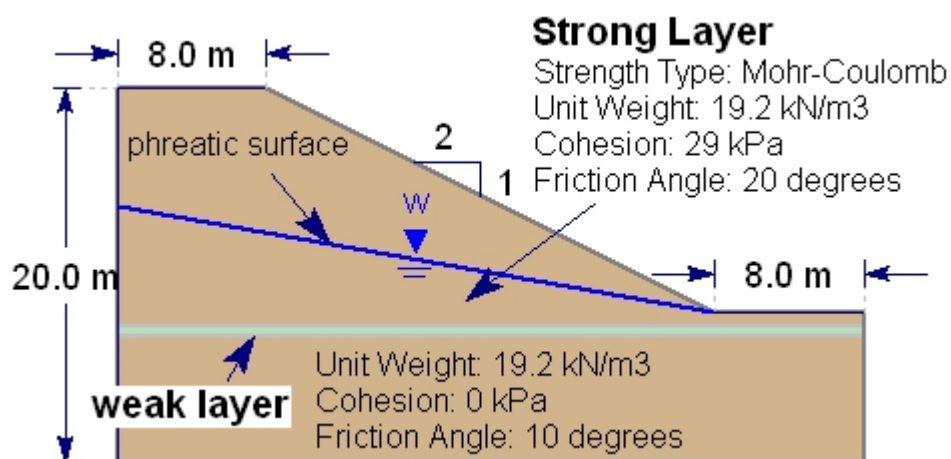


Figure (B.4) Slope profiles for example problem No.2.

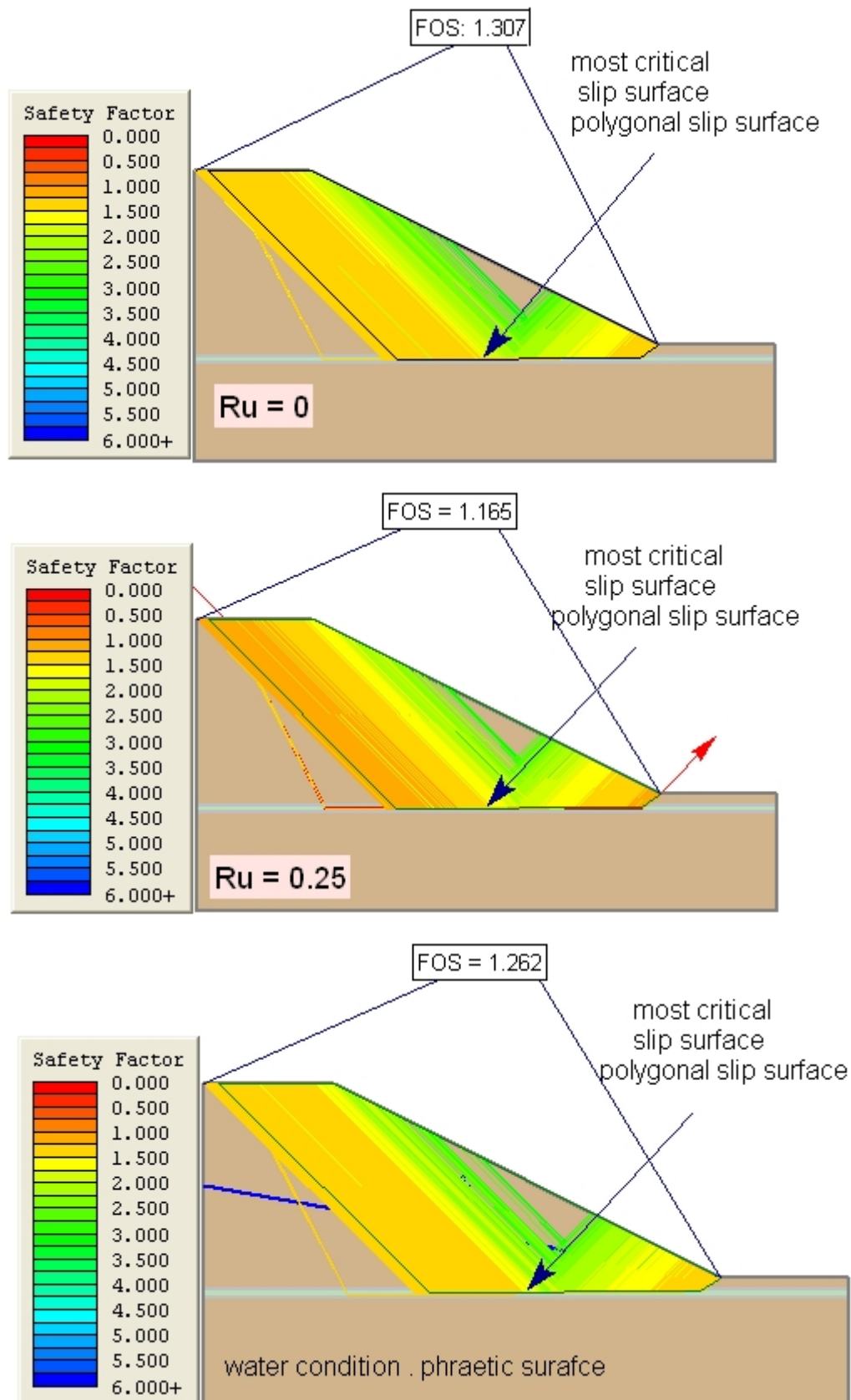
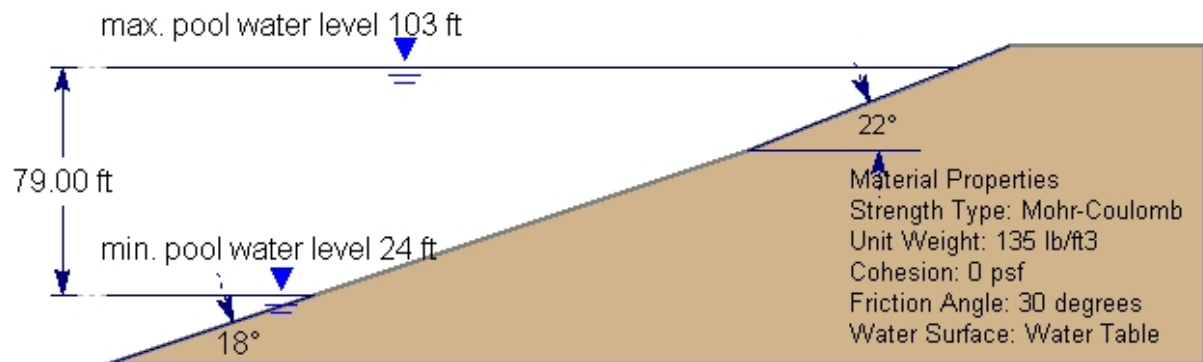


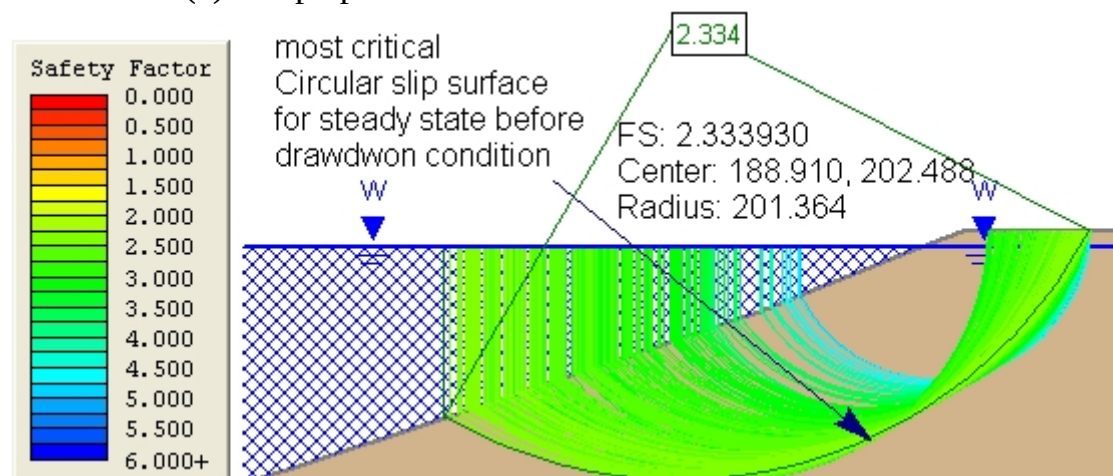
Figure (B.5) Most critical slip surface for example problem no.2 for different water condition, $R_u = 0$, 0.25, and phreatic surface.

B-4 Solution for Example of Rapid Drawdown Condition

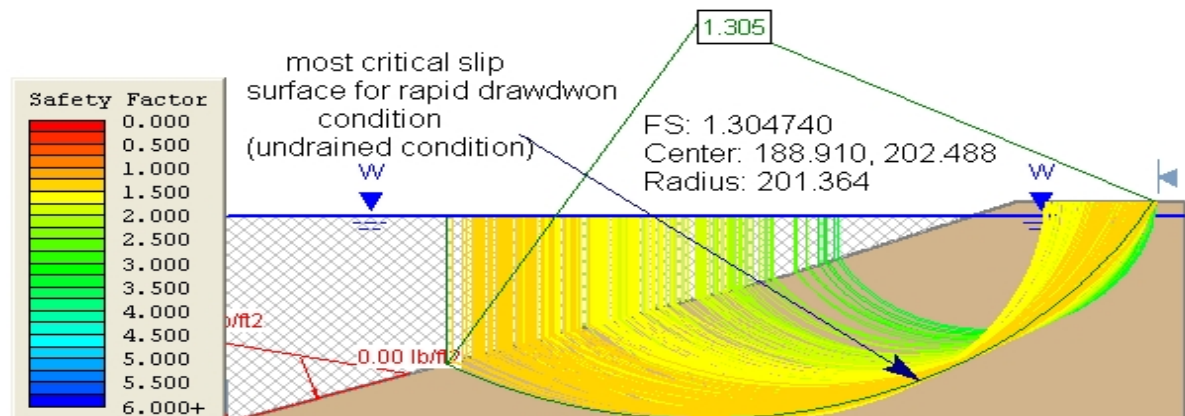
The analysis of the results of the example of rapid drawdown condition in Chapter Three for two conditions, steady state condition and rapid drawdown condition (undrained condition) are presented in the following figures.



(a): Slope profiles



(b): Most critical slip surface, steady state condition.



(c) Most critical slip surface, rapid drawdown condition.

Figure (B.6) Slope profiles, most critical slip surface for steady state and rapid drawdown condition.

تحليل استقرارية السدود الترابية (سد حديثة كحالة تطبيقية)

الخلاصة

في هذا البحث, تم تحري استقرارية الميل الجانبي للسدود الترابية تحت تأثير الأحمال المختلفة باستخدام نظريتي التحليل: التحليل الحتمي والاحتمالي. طريقة الاتزان المحدد تبعا ل Bishop (and Morgenstern-Price) مثلت باستخدام برنامج حاسوب (SLIDE), واستخدم هذا البرنامج لتعريف وتحديد سطح الفشل وحساب عامل الأمان للميول الجانبية في السد. وجرى دراسة تأثيرات عناصر التصميم الأساسية والأحمال المؤثرة على قيمة معامل الأمان, وهذه العناصر تتضمن خواص التربة (معاملات قوة القص), خواص هندسية (الميل الجانبي, وارتفاع السد, وموقع السطح الحر للماء), والأحمال الدينامية المؤثرة (الهبوط المفاجئ وقوة الهزة الأرضية). تم تحري تأثير معاملات التصميم بتحليل الحساسية لكل المتغيرات التي لها تأثيرات ملموسة على استقرارية السد. وجد ان استقرارية الميل الجانبي للسد في المقدمة تتناقص تدريجيا في حالة انخفاض مستوى الماء في مقدمة السد من بدء الانخفاض وحتى وصوله الى نسبة (2 \ 3) من الارتفاع, التي عندها تحصل اخطر حالات عدم الاستقرار, وليس عند افراغ الخزان .

في حالة السد الممنطق (أخذين سد حديثة كحالة تطبيقية), استخدمت نظرية العناصر المحددة لحل مسألة الترشح وإيجاد موقع السطح الحر للجريان وتوزيع ضغط الماء المسامي داخل جسم السد لثلاث حالات مختلفة للتحميل, وهي مستوى ماء المقدمة الاعظم والاعتيادي والخفض . كذلك تمت دراسة استقرارية الميل الجانبي تحت ظروف مختلفة للتحميل . لقد بينت النتائج المستحصلة من التحليل بان الحالة الحرجة ستحصل في مقدمة السد في حالة تزامن الخفض السريع لماء المقدم مع الهزة الأرضية , لكن السد سيكون قريبا من حالة السلامة في ظل الحالتين الاخرين المعتمدة , حيث بلغت اقل قيمة لمعامل الامان (1,09).

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List of Symbols

Symbol	Description
b	Slice width ,(L)
C	Cohesion of soil,(FL ⁻²)
C_v	Coefficient of variation of the safety factor, ($= \sigma / \mu$)
$\bar{c}$	Effective cohesion of soil,(FL ⁻²)
D	Average diameter of soil particles,(L)
E	Interslice normal forces (F)
EL	Horizontal interslice force (from left side of slice),(F)
ER	Horizontal interslice force (from right side of slice),(F)
F_f	Factor of safety with respect to force equilibrium
F_m	Factor of safety with respect to moment equilibrium
FOS	Factor of safety
$FOS_{min.}$	Minimum factor of safety
H	Height of dam ,(L)
HCV	Highest conceivable value of the random variable
hw	Height of water table, (L)
i	Hydraulic gradient
K	Hydraulic conductivity. ,(LT ⁻¹)
K_x	Hydraulic conductivity in x direction .(LT ⁻¹)
K_y	Hydraulic conductivity in y direction .(LT ⁻¹)
L	Drawdown of reservoir level. (L)
L/H	Drawdown ratio in the reservoir,(dimensionless)
LCV	Lowest conceivable value of the random variable
l	The inclined width of slice, (L)
<i>NUM.FAILED</i>	Number of Analyses with Safety Factor < 1.
<i>NUM.TOTAL</i>	Total number of analyses (samples)
$\bar{N}$	Effective normal force, (F)
n	Number of slices
P	Pressure (FL ⁻²)
P	Normal force on the base of the slice,(F)
PF	Probability of failure
R	Radius of slip surface ,(L)

Symbol	Description
Re	Reynolds number.
r_u	Pore pressure coefficient
S	Shear strength, (FL ⁻²)
U	Velocity component in x-direction. (LT ⁻¹)
u	Pore water pressure, (FL ⁻²)
u^-	Initial pore water pressure before drawdown, (FL ⁻²)
V	Velocity component in y-direction. (LT ⁻¹)
V _s	Seepage velocity through porous media. (LT ⁻¹)
W_i	Weight of the slice,(F)
X	Interslice shear forces,(F)
X_L	Vertical component on interslice forces (from left side of slice)
X_R	Vertical component on interslice forces (from right side of slice)
Z	Elevation head (L)
Δu	Change in pore water pressure due to rapid drawdown
γ	Unit weight of soil
λ_1	Constant related to the direction of inter slice forces
θ	Slope angle of slice
β	Reliability index
γ_w	Unit weight of water. (F/L ³)
σ	Normal effective stress
τ	Shear stress
f(x)	Function
λ	Scaling factor
ν	Coefficient of kinematic viscosity of water (L ² /T).
ϕ	Angle of internal friction of soil
μ_{FOS}	Mean safety factor
σ_{FOS}	Standard deviation of safety factor.
β_{LN}	Lognormal Reliability Index
$\bar{\phi}$	Effective angle of internal friction of soil
α	Slope angle of embankment
Ψ	Stream function
θ_i	Slope of slice.

Chapter One

Introduction

1.1 General

With the exception of the Great Wall of China, dams are the largest structures ever built. Throughout history, dams provide tremendous benefits, including water supply for drinking, irrigation and industrial uses, flood control, hydroelectric power generation, recreation, and navigation. However, dams also represent one of the greatest risks to public safety.

The earth dam was the first type of dams built by man to control and manage the water resources. The significance of earth dams stems from their importance to mankind in providing one of the cheapest means for managing large volumes of water. Currently, earthen dams are the most common type, in spite of the rivalry of other types of dams such as concrete dams. The recent development in geotechnical science and in the methods of construction and equipments may be the main reasons behind that [Al-Jorany, 1996].

With respect to earth dams, the term “failure” is defined herein as an occurrence of excessive erosion or deformation of the embankment that may result in an uncontrolled release of reservoir water or damage to appurtenant structures. To assess the safety of a dam and the possibility of failure, the different potential failure mechanisms must be recognized. Failure mechanisms are grouped into four general categories: slope stability, piping, overtopping, and foundation failures, as shown in figure (1.1)

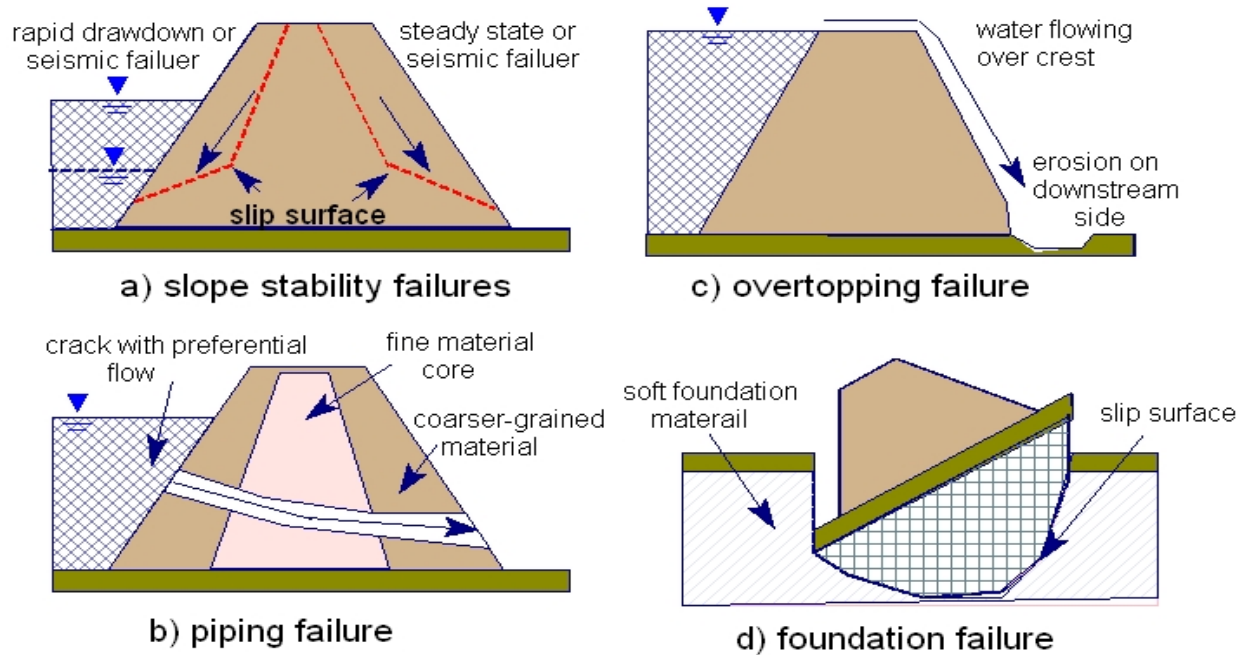


Figure (1.1) Failure mechanisms for earth dams [USBR, 2001].

In the present work, side slope stability against failure will be analyzed under different loads and water conditions.

1.2 Seepage through earth dams:

Seepage of water through the embankment or foundations has been responsible for more than one third of earth dams failures [Varshney and Gupta, (1972), quoted from Subuh, 2002].

The seepage study through the body of earth dams is important during the dam design stage to calculate the losses from reservoir, pore water pressure distribution, and position of the free surface which is used as boundary condition in analysis of the side slope stability. So, in the present work the finite element numerical model will be used to investigate the unconfined seepage problem and position of phreatic surface in a zoned earth dam.

1.3 Stability analysis of earth dams

For earth dams, there are three types of slope stability failures: steady-state, seismic, and rapid-drawdown. For the steady-state case, failure occurs on the downstream side of the dam under conditions of steady-state seepage. This type of failure may occur as a result of an increase in pore water pressure in the dam. For the rapid-drawdown case, failure occurs on the upstream side of the embankment as a result of a sudden lowering of the reservoir level. For the seismic case, the driving force on the soil mass increases due to horizontal earthquake force, while the resisting force may be reduced if portions of the embankment or foundation liquefy. Liquefaction can occur during an earthquake in loose, saturated, sandy soils. During liquefaction, the soil particles are rearranged into a denser configuration, which tends to displace pore water. Since the pore water cannot vacate the pore spaces immediately, the pore water pressure temporarily increases. If this increase is sufficient, the soil particles become supported by the pore water, which has no shear strength. As a result, the shear strength of the soil approaches zero. When performing a seismic slope stability analysis, it may be found that at times during the earthquake when ground shaking is at a maximum, the Factor of Safety falls below 1.0 and some deformation occurs [USBR, (2001)].

For slope stability analysis of earth dams, two types of analysis are used in the present work:

- Deterministic analysis: where all parameters (C, ϕ, γ , and seismic force) are accurately known, hence there will be unique output. Deterministic problems are easier to deal with and are referred to as a certainty design.
- Probabilistic analysis: where many of the involved parameters cannot be evaluated with high certainty. Therefore there will be several outputs and the engineer should choose the right output. These can be solved using probabilistic approach and are referred to as uncertainty problems.

1.4 Slope stability analysis using computer program (SLIDE V.5.0)

In recent years, there has been a considerable development in techniques of analyzing seepage through earth dams and slope stability analysis by using computer programs. One of these programs is **SLIDE V.5.0**. **SLIDE V.5.0** is a large collection of modular engineering programs covering, slope stability analysis, seepage analysis, geotechnical analysis and other general calculations including probabilistic analysis and back analysis. **SLIDE V.5.0** is the slope stability software to include built-in steady state unsaturated groundwater analysis capabilities using finite element methods.

1.5 Aims of the Present Work:

The main targets of the present work can be summarized as follows:

1. To verify of the program (**SLIDE V.5.0**). Some example problems that had been analyzed previously by other researchers are taken for comparison with **SLIDE** computer program results.
2. To carry out a parametric study to demonstrate the effect of dam geometry (height of embankment, slope angle), material properties (cohesion of soil, angle of internal friction, unit weight of soil), loading condition (phreatic surface location, rapid drawdown, seismic load) on the stability of an earthen dam.
3. Analysis of a zoned earth dam [**Haditha Dam**] including seepage analysis under different water conditions, slope stability analysis for steady state condition, rapid drawdown condition, and seismic load condition.
4. Probabilistic analysis to find the probability of failure and reliability index of a zoned earth dam.
5. Investigating the increase of stability of an earth dam by using reinforcement for side slope coupled with back analysis approach.

1.6 Layout of the Present Work

The structure of the present work is as follows:

Chapter two: reviews literature concerning the subject which involves a summary about the theories to solve problems of seepage analysis through earth dams and slope stability analysis of earth dams.

Chapter three: Contains comparing the results of **SLIDE V.5.0** (computer program) with other programs and techniques to check the reliability of this program.

Chapter four: a parametric study is carried out to demonstrate the effect of some design parameters (geometry, material properties, load parameters, and dynamic load parameters). Moreover, a probabilistic analysis is carried out with sensitivity analysis and presents the proposed equations to determine the minimum factor of safety.

Chapter five: Re-analysis of the stability of one Iraqi zoned earth dam under different load condition.

Chapter six: gives the main conclusions obtained from the present study and the recommendations for future works.

Chapter Two

Review on Seepage and Stability Theories

This chapter presents the necessary background and theory concerning stability and seepage. Consideration must, therefore, be given to a number of different topics. First, descriptions of seepage theories in soil mechanics are presented. Later on, brief descriptions of the theories of stability are described. Most of the previous studies on the seepage through earth dams and slope stability analysis under static and pseudo static will be reviewed in this chapter.

2.1 Reviews on Calculation of Seepage Pressure in Embankments

2.1.1 Stability of Earth Dams Subjected to Seepage Forces

The stability of earth dams subjected to the effects of changing reservoir levels is dependent on, among other factors, the pore pressures induced within the earth mass due to seepage. The pore pressures are generally estimated from flow net analysis obtained under steady state conditions. The expected distribution of seepage pressures and the position of the free surface are required so that filters and drains can be designed and positioned correctly to give acceptable flow distribution and seepage quantities. Civil engineering has traditionally relied upon the graphical method of flow nets to determine seepage pressures for steady state conditions in the earth dams. However, the process of flow net construction requires skill and time and in many practical problems simplifying assumptions have to be made. Analytical methods are available but they are mostly limited to simple cases. Numerical methods based on finite difference and finite elements approaches now removed the many limitations associated with anisotropy and non-homogeneity [Obaid, 2002].

2.1.2 Steady-State Seepage through Earth Dams

The most critical conditions are likely to occur with the reservoir full under steady state seepage and those resulting, during and after rapid drawdown of the reservoir water level. In order to assess the factor of safety of a potential slip surface, the distribution of pore pressure in the dam must be known. Failures of earth or rockfill dams can be resulted from excessive leakage from piping at the toe, or from slope failures on the dam face in upstream and downstream side. All the three cases can be analyzed with the aid of steady-state flow net. For rare situations where an earth dam is constructed on an impervious formation, the flow net can be limited to the dam itself. Where the foundation materials are also permeable then the flow net must include the entire dam foundation system.

2.1.3 Transient Seepage through Earth Dams

Slope failures on the upstream face of a dam are usually the result of rapid drawdown in the reservoir level. At full supply levels the effects of high pore pressures in the face are offset by the weight overlying reservoir water. Following rapid drawdown, the high pore pressure remains, but the support has been removed. Unless the transient dissipation of this pore pressure is rapid, that is, unless the transient drainage of the dam face is rapid, instabilities may develop on the critical slip surface and slope failures can occur.

2.2 The Equation of Flow through a Porous Medium

The flow of water through saturated porous medium is generally governed by Darcy's Law [**Harr**, (1962)];

$$V_s = ki \quad \dots (2-1)$$

where:

V_s : Seepage velocity through porous media. (L/T)

k : Hydraulic conductivity. (L/T)

$$i : \text{Hydraulic gradient} = -\frac{\partial H}{\partial s}$$

H : Piezometric head. (L)

S : Distance along the flow line. (L)

Darcy's Law is applicable to flow through porous media of two and three dimensions, that's why this is considered of great importance in studying great number of practical problems.

Equation (2-1) is valid when the flow is laminar, i.e. , Reynolds number is taken equal to or less than one [**Harr**, (1962)], or.

$$Re = \frac{Vs D}{\nu} \leq 1 \quad \dots (2-2)$$

where:

Re: Reynolds number.

Vs: Flow velocity (L/T).

D: average diameter of soil particles (L)

ν : Coefficient of kinematic viscosity of water (L²/T).

2.3 The General Equation of Steady Seepage Flow

The two-dimensional anisotropic components of seepage through porous media according to the general Darcy's Law form are [**Freeze and cherry**, (1979)].

$$U = k_x i_x = -k_x \frac{\partial H}{\partial x} \quad \dots (2-3a)$$

$$V = k_y i_y = -k_y \frac{\partial H}{\partial y} \quad \dots (2-3b)$$

where

U: velocity component in x-direction. (L/T)

V: velocity components in y-direction. (L/T)

k_x, k_y : Hydraulic conductivity in x, y directions respectively. (L/T)

H: Piezometric head ($H = \frac{P}{\gamma_w} + z$). (L)

P: pressure (F/L^2).

γ_w : Unit weight of water. (F/L^3) and.

z: elevation head (L).

The continuity equation for two-dimensional and incompressible, irrotational flow is:

$$\frac{\partial U}{\partial x} + \frac{\partial V}{\partial y} = 0 \quad \dots (2-4)$$

Substituting Darcy's (2-3) in equation (2-4) results in:

$$\frac{\partial}{\partial x} (k_x \frac{\partial H}{\partial x}) + \frac{\partial}{\partial y} (k_y \frac{\partial H}{\partial y}) = 0 \quad \dots (2-5)$$

For homogenous and isotropic soil, the hydraulic conductivity is equal every where in all directions: $k_x = k_y$

Then equation (2-5) would be:

$$\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial y^2} = 0 \quad \dots (2-6a)$$

Equation (2-6a) is called Laplace Equation and it is similar to Laplace Equation of velocity potential for ideal fluid flow or non-viscous, irrotational flow.

Laplace equation represents the condition of steady-state laminar flow and different methods to find the piezometric head of the flow domain can be used.

For homogenous and anisotropic soil the hydraulic conductivity is equal every where and not equal in all directions, i.e.: $k_x \neq k_y$

Then Laplace equation becomes:

$$k_x \frac{\partial^2 H}{\partial x^2} + k_y \frac{\partial^2 H}{\partial y^2} = 0 \quad \dots (2-6b)$$

where:

k_x : Hydraulic conductivity in the horizontal direction.

k_y : Hydraulic conductivity in the vertical direction.

There is another benefit function for the flow through porous media, which is called stream function ψ (x,y).

$$U = \frac{\partial \psi}{\partial y}, V = -\frac{\partial \psi}{\partial x} \quad \dots (2-7)$$

U: velocity component in x-direction.

V: velocity component in y-direction.

ψ : Stream function.

From the similarity between the flow through porous media and the ideal flow, where the flow is irrotational and the vorticity equal to zero, the expression for this relation is:

$$\frac{\partial V}{\partial x} - \frac{\partial U}{\partial y} = 0 \quad \dots (2-8)$$

Substituting equation (2-7) into equation (2-8) results in:

$$\frac{\partial^2 \psi}{\partial x^2} + \frac{\partial^2 \psi}{\partial y^2} = 0 \quad \dots (2-9)$$

This represents Laplace equation for stream function.

2.4 Methods for Solution of Seepage Equations

In order to do seepage analysis, a general model describing the phenomena of seepage must be available. Supplied with specific boundary conditions and soil properties, this model can be used to determine heads and flow distribution

and seepage quantities. The Laplace's equation is a mathematical basis for several methods that have been developed to solve exactly or approximately Laplace's equation for various cases of steady seepage. The following methods are the most widely used methods according to [Lambe and Whitman, 1969].

2.4.1 Graphical Method-The Flow Net Sketching

The flow net sketching was first suggested by Forchheimer and further developed by A.Casagrande (1937), [Lambe and Whitman, 1969]. The solution of seepage equation (2.6a) in two dimensions may be presented by two families of curves intersecting one another orthogonally and forming a pattern of curvilinear squares. The two families of lines are known as equipotential lines, or lines joining points of equal total head, and streamlines. A streamline is the locus of the path of flow of an individual particle of water. The mesh formed by the intersection of these two sets of curves is known as a flow net. A unique solution of Laplace's equation may therefore be obtained graphically by trial-and-error sketching of a flow net, which satisfies the boundary conditions, preserve right angle intersections, and consists of curvilinear squares. The method may be extended to include simple anisotropic conditions by transforming the section before the flow net is constructed so that the dimensions of the cross section are shrink in the direction of greater permeability [Lambe and Whitman, 1969].

If a section consists of two materials having different permeability, the net may be drawn so that the length to width ratio of the squares is equal to the ratio of permeability. Confined and unconfined flow solutions to transient advance problems are also possible. **Cedergren** (1967) comprehensively describes the method and many examples on the flow nets are given. The undesirable feature of the flow net methods lies in the difficulty of sketching the net. An accurate net for even a simple two-dimensional flow situation can require many hours of skilful and experienced labourer.

2.4.2 Analytical Method

Exact solutions to the Laplace equation may be obtained by various analytical methods.

Harr (1962) explained the use of transformations and mapping to transfer the geometry of the seepage problem from one complex plane to another. In this manner, the geometry of a problem may be taken from a plane where the solution is unknown to a plane where the solution is known. While this method has been used to obtain solution to general problems it is not frequently used for solutions to site-specific seepage problems since it requires the use of complex variable theory and proper choice of transformation functions.

The simplest theoretical solution are suggested by kozeny (1931) who considered the problem of seepage throughout an earth dam with a parabolic upstream face, resting on an impervious base and having a horizontal toe drain located on the down stream portion of the dam. The flow net for the section consists of a system of confocal parabolas.

Casagrande (1940)[Quoted from **Al-Qaisi, (1999)**], extended Kozeny's solution to include dams with trapezoidal toe drains and slope drains and suggested an adjustment to account for a straight upstream face.

Unconfined flow in dams was also studied by Numerov [reported by **Harr 1962**] who obtained solution in graphical form but, unfortunately, its application is still not straightforward.

Al-Assaf (1998) has derived analytical solution for aquifer response to the stage variation in a surface-water such as a stream. The effect of the stream fluctuation on aquifer or aquiclude that is bounded laterally by a medium of different hydraulic properties has been investigated. Analytical expressions for the change in groundwater flow rate of groundwater flow and volume of accumulated water in response to stream fluctuation have been derived. Two different approaches of approximating the stream hydrograph, i.e., the sudden

changes and the linear variations were investigated and given nearly the same result [Obaid, 2002].

2.4.3 Models

Models which scale or simulate the flow of water in porous media can provide a good view for what is occurring during seepage and allow a physical view for the reaction of the flow system to changes in head, design geometry, and other assumptions. These models include the following:

(a) Analogy Methods

Processes which involve movement of energy due to differences in energy potential operate by the same principle as movement of groundwater. These processes include electricity and heat flow, which have been used as seepage analogies. Electrical analogues have proven particularly useful in the study of two and three-dimensional problems and in problems where geometric complexities do not allow adequate simplifying assumptions for analytical methods. [U.S. Army Corps of Engineers, 1986].

(b) Sand Models

Sand models which may use prototype materials can provide information about flow path and head at particular points in the aquifer. The sand or porous material may be placed under water to provide a homogenous condition, or layers of different sand sizes may be used to study the effect of internal boundaries or layers.

Flow can be traced by dye injection and heads can be determined by small piezometers. Disadvantages include effects of layering when the porous material is placed, difficulty in modeling prototype permeability, and boundary effects.

(c) Viscous flow Models

These types of models have been used to study transient flow (e.g., sudden drawdown) and effects of drains. These models depend on the flow of a viscous fluid such as oil or glycerine between two parallel plates and are normally used to study two-dimensional flow. As with sand models, dye can be used to trace flow lines. Construction is normally complicated and operation requires care since temperature and capillary forces affect the flow [Obaid, 2002].

2.4.4 Numerical methods

As a result of many difficulties that appear through analytical methods, approximate and experimental methods, it was resorted to numerical methods in order to get the required results with good accuracy. These results could be compared with analytical solutions by applying various boundary conditions for simple problems in addition to the development of computer systems. These methods are: finite difference, finite volume, finite elements, boundary elements, and analytical elements method. Each method has special features which may be desirable for a particular application.

1- Finite – Difference Method

The finite difference method solves the flow equations by approximating them with a set of linear algebraic equations. The flow region is divided into a discrete rectangular grid with nodal points, which are assigned values of head. Using Darcy's law and the assumption that the head at a given node is the average of the surrounding nodes, a set of N linear algebraic equations with N unknown values of head are developed [U.S. Army Corps. of Engineers, 1986]. A numerical finite difference solution to steady state flow situations was first presented by Show and Southwell in (1941) but the development of the method accelerated with the advance of digital computers [Al-Qaisi, 1999].

Pinder and **Bredehoeft** (1968), [Quoted from **Obaid, 2002**] presented a numerical solution to a mathematical problem of unsteady state condition, using finite difference approximation. The results of this model were compared with those obtained from the analytical approach for problems in homogenous media of simple geometry and with electric analogue results for realistic aquifer. The results of this comparison indicated that the numerical solution gives good results.

Desai and **Sherman** (1971) presented a numerical solution for the problem of unconfined transient flow with a free surface in heterogeneous soils involving both rise and fall (drawdown) of external water levels for river banks and embankments by finite difference technique. Both non-linear form and linearized form of the flow equation were solved by employing a numerical method.

Stephenson (1978) studied the recession of the phreatic surface in an embankment following sudden drawdown of a reservoir. The non-equilibrium equation for flow in porous media was simplified and solved by using finite difference indicating the variation of water surface with time, for both battered cut and triangular embankments. The location of point of emergence of the phreatic line was investigated and the relationship between the storage coefficient and coefficient of consolidation was explained.

2- Finite- Element Method

This method is also based on grid pattern (not necessarily rectangular) which divides the flow region into discrete elements and provides N equations with N unknowns. Material properties, such as permeability, are specified for each element and boundary conditions (heads and flow rates) are set. A system of equations is solved to compute heads at nodes and flows in the elements. The finite element has several advantages over the finite difference method for more

complex seepage problems. These include [**U.S. Army Corps. of Engineers, 1986**] :

- Complex geometry including sloping layers of material can be easily accommodated.
- By varying the size of elements, zones where seepage gradients or velocity are high can be accurately modelled.
- Pockets of material in a layer can be modelled

The finite element method was first applied to boundary value field problems by Zienkiewicz and Cheung (1965) and their method was later extended to obtain a solution for steady state seepage in an anisotropic foundation under a concrete dam [**Zienkiewics et al., 1966**].

Various investigators (**Taylor and Brown, 1967; Freeze, 1971**) developed finite element models for describing water flow and seepage in soils. Papagianakis and Fredlund (1984) and Lam et al. (1987)[Quoted from **Al-Qaisi, 1999**], developed a finite element package for performing saturated/unsaturated seepage modelling.

France et al., (1971) developed and presented a numerical analysis of free surface seepage problems by using finite element methods based on the principle of minimum potential energy. Essentially the transient problem was solved by considering the solution to be a series of steady state solutions at small intervals of time apart. Four examples of transient state were analyzed. The conclusions reached, despite their inherent complexity, were free surface groundwater flow problems could be treated by using the finite elements method, which has proved to be extremely versatile, and is completely general enough to solve most practical seepage problems.

Desai (1972) investigated the problem of unsteady unconfined seepage in porous media by using finite elements procedure. The changing free surface was located by computing movements of the nodes on the free surface and by using

iterative scheme. The formulation presented in this study was applicable also to axisymmetric idealizations. Moreover, the procedure could be conveniently extended for three-dimensional analysis. In this study isoparametric element was found to facilitate formulation and computational efforts. An important answer adopted by this study for finite representation of infinite media such as riverbanks depending upon results of numerical solutions. The results obtained by this study were compared with laboratory tests and with various boundary assumptions possible in long porous banks. Attention was directed toward developing a general computer procedure for design and analysis based upon such study.

Guvanasen and Volker (1980)) [Quoted from **Obaid, 2002**] presented one finite difference and three finite element schemes for solving unsteady flow problems in unconfined aquifer. Results from various numerical schemes were compared with experimental results from a sandbox model, which simulated the problem of unsteady free surface seepage. The conclusions showed that the finite element schemes were bound to be more computationally economical for this particular problem. The particular finite element scheme preferred was one that incorporates the prescribed flux and head conditions on the free surface in an iterative solution for the new free surface at the end of each time step. The increased stability of this scheme allows a large time increment to be used without significantly affecting accuracy, so that solutions can be obtained with minimum computational effort.

Li and Desai (1983) developed a finite elements procedure for seepage and stability analysis of dams and earth banks. The seepage analysis adopted in this study was based on a residual flow scheme involving saturated and unsaturated zones in which the original mesh remains invariant during transient flow and iterations. The proposed procedure was applied for solution of typical problems involving seepage and stability analysis. The procedure has been found to provide satisfactory correlation with analytical solutions and field observations for a number of problems. It could be useful and appropriate for seepage and stability analysis of dams and earth banks.

Al-Qaisi (1995) presented a numerical analysis of the elastic – inelastic finite element solutions to investigate the behavior of the core of the Adhaim Dam. The effects of core geometry (shape and dimensions) and the relative core stiffness to shell have been investigated. He found that the upstream inclination of the core reduces the generation of pore pressure.

Obaid (2002) presented a finite element procedure for determination of the time dependent locations of free surface, seepage heads, and pore pressures within the dam body. The seepage analysis was based on **Galerkin** method involving saturated and unsaturated zones in which the original mesh remains invariant during unsteady flow and iterations. The cases of the sudden and gradual drawdown conditions were studied by this method. He found that the method was stable but requires a large number of time steps to reach the steady state. Comparison with analytical results indicated that the maximum percentages of absolute differences are insignificant (less than 2%).

Subuh (2002) presented a mathematical model and applied it for analyzing two-dimensional steady state seepage through stratified and isotropic earth dams. A numerical solution using finite elements method (Galerkin method) was employed to predict the piezometric head distribution, seepage quantity, pore water pressure, and locating the free surface profile.

3- Finite- Volume Method

The finite volume method was originally developed as a special finite difference formulation. Finite volume method is a numerical approach for solving partial differential equations instead of using an analytical method. The domain over which the dependent variable is to be evaluated is broken into a finite number of discrete volumes and algebraic equations can then be written for each volume.

The first step in the finite volume method is to divide the domain into discrete control volumes. A number of nodal points placed in the space, shown

in figure (2.1). The boundaries (or faces) of control volumes are positioned mid-way between adjacent nodes. Thus each node is surrounded by a control volume or cell. It is a common practice to set up control volumes near the edge of the domain in such a way that the physical boundaries coincide with the control volume boundaries [Versteeg and Malalasekera, (1995)].

The finite volume approach is especially useful for problems where interfaces exist between regions having different physical properties. The control volume approach is based on flow balances on a finite control volume instead of the governing partial differential equations [Hoffman, (2001)].

The finite volume method (FVM) finds application in a large number of fields of science and technology, where it is used to solve a variety of partial differential equations that model given physical phenomena. Most notably the method has been used with great success in the fields of computational fluid dynamics (CFD), numerical heat transfer and flow in porous media (drying, groundwater flow, petroleum reservoir modelling, and dispersion of contaminants)

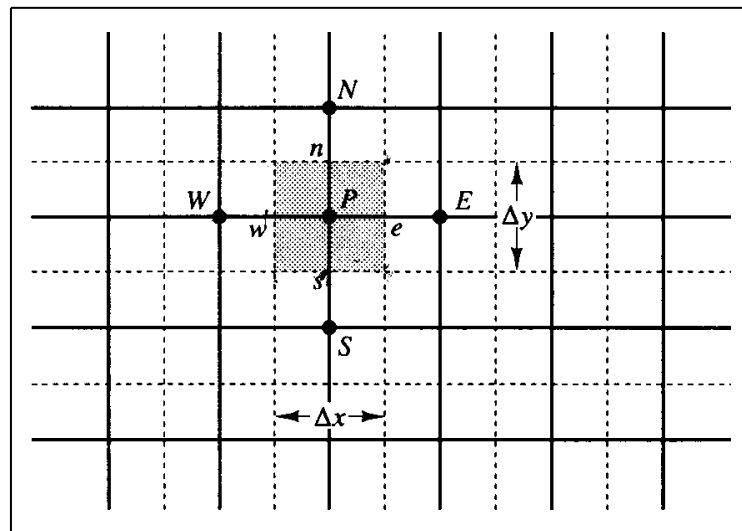


Figure (2.1) Domain discretization of the finite volume method.

The method has been introduced into the field of CFD independently by McDonald (1971) and MacCormack and Paullay (1972) to resolve two-dimensional, time-dependent Euler equations. It was extended to three dimensions by Rizzi (1973) [**Turner, (2005)**].

Applications of this method in the field of seepage analysis and groundwater flow through history are very limited as compared with other numerical analyses, finite element and finite difference.

Darbandi et al., (2007) developed a novel moving mesh finite volume method capable of solving the seepage problem in domains with arbitrary geometry. One major difficulty in analysing the seepage problem is the position of phreatic boundary which is unknown at the beginning of solution. In this study they first chose an arbitrary solution domain with a hypothetical phreatic boundary and distributed the finite volumes method therein. Then, they derived the conservative statement on a curvilinear coordinate system for each cell and implemented the known boundary conditions all over the solution domain. Subsequently, they found that the preceding mesh was suitably deformed so that its upper boundary matches the new location of the phreatic surface. This tactic results in a moving mesh procedure which was contained until the nonlinear boundary conditions were fully satisfied at the phreatic boundary. In that study they found that the moving grid finite volume method was highly robust and it provides sufficient accuracy and reliability.

2.5 Reviews on Stability Analysis Theories

Slope stability analysis of earthen dams by using computer programs is an easy task for engineers when the slope configuration and the soil parameters are known. However, the selection of the slope stability analysis method is not an easy task and effort should be made to collect the field conditions and the failure observations in order to understand the failure mechanism, which determines the

slope stability method that should be used in the analysis. Therefore, the theoretical background of each slope stability method should be investigated in order to properly analyze the slope failure and assess the reliability of the analysis results.

A stability analysis of earth dams and banks requires consideration of the coupled effects of:

- Loads such as body weight, surcharge, and forces caused by sequential construction.
- Seepage forces due to steady or transient flow of water. Often, for simplicity, the effects of external and seepage forces are uncoupled and superimposed [Li and, Desai, 1983].

2.5.1 Two Dimensional Slope Stability Analyses

Two dimensional slope stability methods are the most common used methods among engineers due to their simplicity. However, these methods are based on simplifying assumptions to reduce the three-dimensional problem to a two-dimensional problem and therefore the accuracy of the analysis results vary between the different analysis methods.

2.5.2 Conventional Methods of Slope Stability Analysis

Most of the methods currently utilized in slope stability analysis are based on the limit equilibrium approach. The essential assumption of this approach is the validity of well known Mohr-Coulomb failure criterion which defines the shear strength of soil as follows:

$$S = c + \sigma \tan \phi \quad \dots\dots\dots(2.10)$$

where (c) and (ϕ) are cohesion intercept and effective angle of internal friction, respectively, and (σ) is the normal effective stress. The method of limit equilibrium assumes that the shear strength of the soil is partially mobilized

along an assumed failure surface which may be a straight line, circular arc, logarithmic spiral curve or any other irregular surface. The method, however, defines the factor of safety (FOS) as the ratio of available shear strength (S) and the developed shear stress (τ):

$$\text{FOS} = \frac{S}{\tau} \quad \dots\dots\dots(2.11)$$

Equation (2.11) is a form of definition introduced by Bishop (1955) which has gained fairly wide acceptance. The factor of safety (FOS) is taken as the ratio of the total shear strength available on the slip surface to total shear strength mobilized (τ) in order to maintain equilibrium [**Spencer**, 1967]. The interest lies in materials that are saturated with groundwater; in such a case, equation (2.10) takes the form:

$$S = c + (\sigma - u) \tan \phi \quad \dots\dots\dots(2.12)$$

in which (u) is the pore water pressure. For the mentioned definition, the method of slices appears as a good approach for obtaining an accurate solution for any shape of failure [**Whitman**, and **Bailey**, 1967].

2.5.3 Method of Slices

In the method of slices, the soil mass above the slip surface is divided into a number of vertical slices and the equilibrium of each of these slices is considered. The actual number of the slices depends on the slope geometry and soil profile. However, breaking the mass up into a series of vertical slices does not make the problem statically determinate. In order to get the factor of safety by using method of slices, it is necessary to make assumptions to remove the extra unknowns and these assumptions are the key roles of distinguishing the methods.

Most computer programs use the methods of slices, as they can handle complex slope geometries, variable soil and water conditions and the influence of external boundary loads. Therefore, they are the most commonly used

methods in slope stability analysis [**Al-Bataineh**, 2006]. Some of the most popular and significant methods are described hereinafter.

2.5.3.1 Ordinary method of slices

This method is also referred to as "**Fellenius**' method"; it is the simplest method of slices to use. The method assumes that the resultant of the interslice forces acting on any slice is parallel to its base; therefore the interslice forces are neglected (Fellenius, 1936). Only moment equilibrium is satisfied. In this respect, factors of safety calculated by this method are typically conservative. Factors of safety calculated for flat slopes and/or slopes with high pore pressures can be on the conservative by as much as 60 percent, when compared with values from more exact solutions (**Whitman** and **Baily**, 1967). For this reason this method is not used much nowadays [**Al-Bataineh**, 2006].

2.5.3.2 Simplified Bishop Method

As mentioned in **Lambe** and **Whitman**, (1969), this method was first described by Bishop (1955); a simplified version of the method was developed further by Janbu et al. (1956). The simplified Bishop method neglects the interslice shear force since it assumes that the resultant of the inter-slice forces acting on each slice has a horizontal line of action. This method, associated with circular slip surfaces, violates the equilibrium equation of horizontal force so it is an approximate method.

Whitman and **Bailey** (1967) indicate that error in the values of factor of safety obtained by this method of analysis is usually less than 5%. The value of safety factor by using the simplified method, FOS, can be calculated from:

$$FOS = \frac{\sum_{i=1}^{i=n} \left[\frac{c}{\Delta X_i} + (W_i - U_i) \tan \bar{\phi} \right] \left[1 / M_i(\theta) \right]}{\sum_{i=1}^{i=n} W_i \sin \theta_i} \quad \dots (2-13)$$

where:

$$M_i(\theta) = \cos \theta_i \left(1 + \frac{\tan \theta_i \tan \bar{\phi}}{FOS} \right) \quad \dots (2-14)$$

$\bar{c}$ and $\bar{\phi}$ are effective shear strength parameters for the soil at the base of the slice

n : number of slices

W_i : weight of the slice.

θ_i : slope of slice.

u_i : the average pore water pressure at the bottom of the slice is equal to $u_i = h_i * \gamma_w$

h_i : height of water in the piezometer placed at the bottom of the slice.

Equation (2-13) is to be solved by trial and error method since FOS appears on both sides of the equation. However the convergence of trial is very rapid. $M_i(\theta)$ can be found from Figure (2.2).

The two methods above [ordinary method and simplified Bishop method] are presented in Figure (2.3) which shows the differences between the two methods.

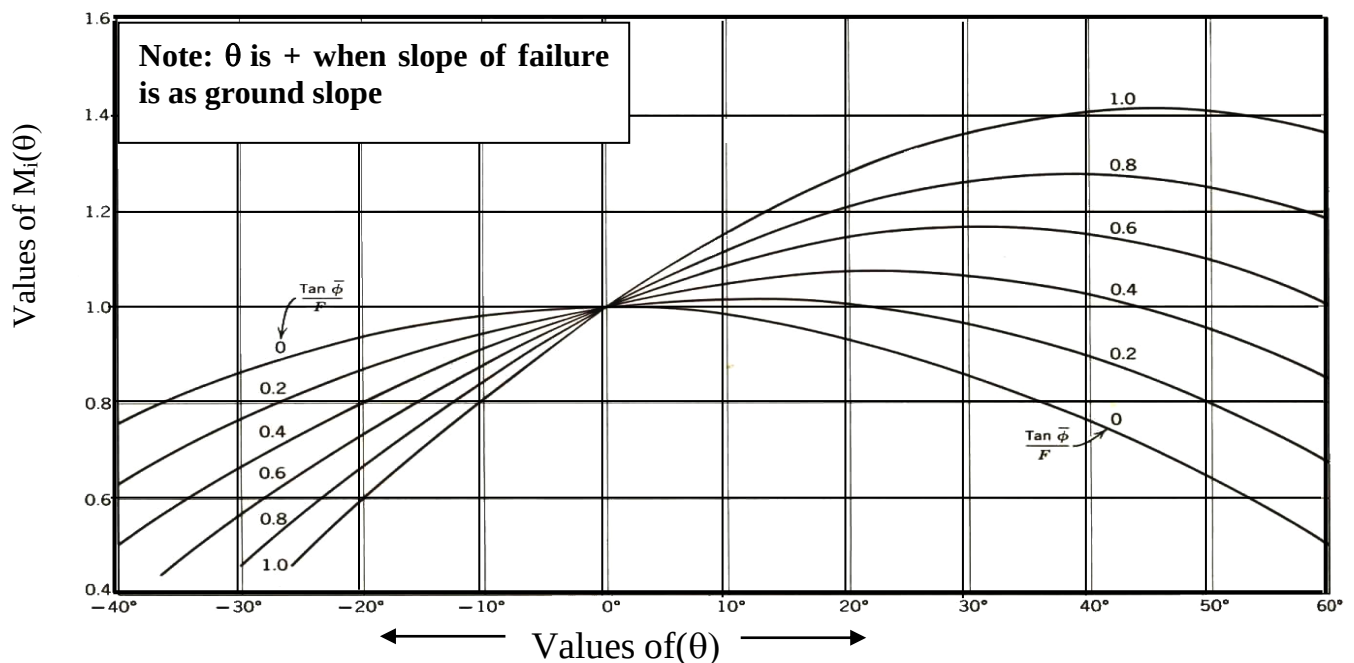


Figure (2.2): Values of $M_i(\theta)$ (after Janbu, 1973)

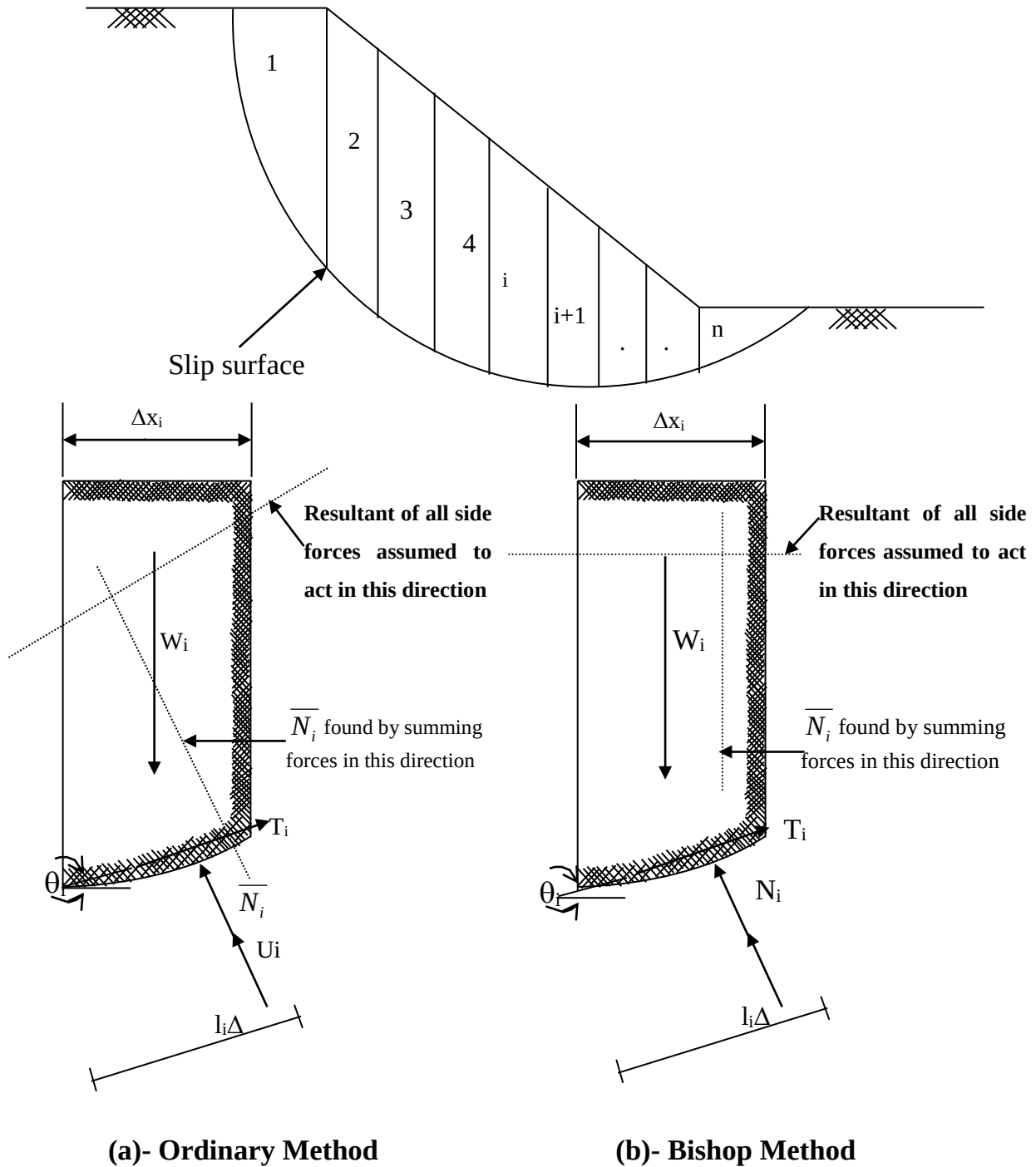


Figure (2.3): General slip surface and forces acting on typical slice according to [Lambe and Whitman, 1969].

2.5.3.3 Morgenstern and Price's Method

This method was developed by **Morgenstern** and **Price** (1965), which considers not only the normal and tangential equilibrium but also the moment equilibrium for each slice in circular and non-circular slip surfaces. In this method, a simplifying assumption is made regarding the relationship between the interslice shear forces (**X**) and the interslice normal forces (**E**) as:

$$X = \lambda \cdot f(x) \cdot E \quad \dots\dots\dots (2.15)$$

where, $f(x)$ is an assumed function that varies continuously across the slip, and (λ) is an unknown scaling factor that is solved for as part of the unknowns.

The unknowns that are solved for in the Morgenstern and Price method are the factor of safety (FOS), the scaling factor (λ) , the normal forces on the base of the slice (P), the horizontal interslice force (E), and the location of the interslice forces (line of thrust). Once the above unknowns are calculated using the equilibrium equations, the vertical component of the resultant force on the interslice forces (X) is calculated from equation (2.15).

An alternative derivation for the Morgenstern-Price method was proposed by **Fredlund** and **Krahn** (1977). They had shown that almost identical results may be obtained using their general formulation of the equations of equilibrium (GLE) together with Morgenstern and price's assumption about the interslice shear forces (equation 2.15). The solution satisfies the same elements of the static case but the derivation is more consistent with that used in the other methods of slices and also presents a complete description of the variation of the factor of safety with respect to (λ) .

According to **Fredlund** and **krahn** (1977) the normal force is derived from the vertical force equilibrium equation as shown in figure (2.4).

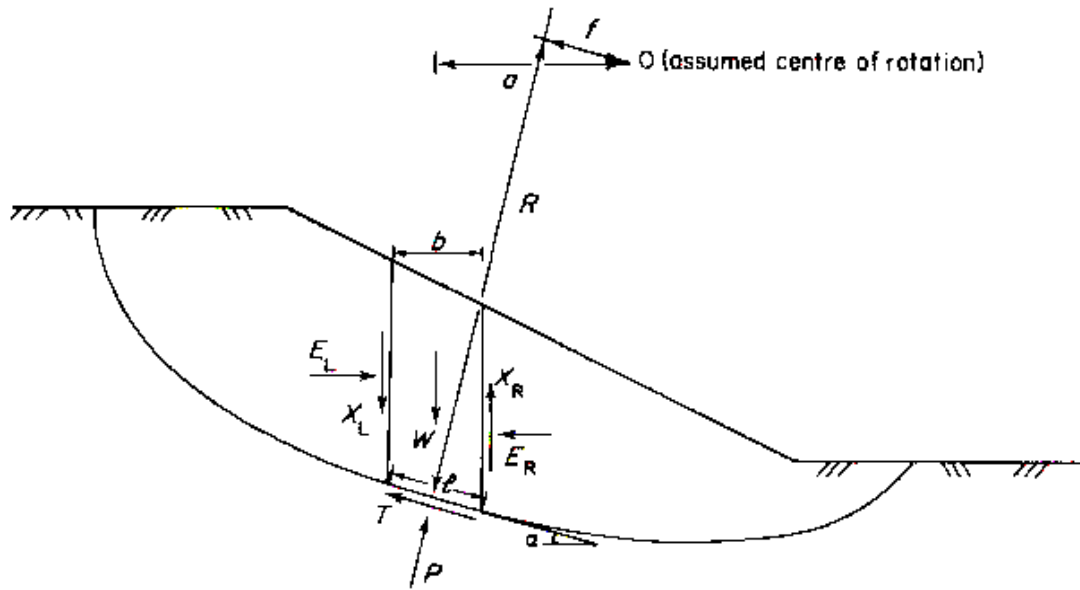


Figure (2.4), General method of slices (**Fredlund and Krahn, 1977**)

$$P = \frac{\left[W - (X_R - X_L) - \frac{1}{f} (c^- l \sin \alpha - ul \tan \phi^- \sin \alpha) \right]}{m_\alpha} \dots\dots\dots (2.16)$$

where:

P : Normal force on the base of the slice

W : Weight of slice

X_R : Vertical component of the resultant force on the interslice (from right side of slice)

X_L : Vertical component of the resultant force on the interslice (from left side of slice)

Two factor of safety equations are computed, one with respect to moment equilibrium (F_m) and the other with respect to force equilibrium (F_f). The moment equilibrium equation is taken with respect to a common point as:

$$F_m = \frac{\sum [c^{-}l + (P - ul)\tan\phi^{-}]R}{\sum (Wd - P_f)} \dots\dots\dots (2.17)$$

For circular slip surfaces: $f = 0$, $d = R\sin\alpha$ and $R = \text{constant}$;

$$F_m = \frac{\sum [c^{-}l + (P - ul)\tan\phi^{-}]}{\sum W \sin\alpha} \dots\dots\dots (2.18)$$

The factor of safety with respect to force equilibrium is:

$$F_f = \frac{\sum [c^{-}l + (P - ul)\tan\phi^{-}]\cos\alpha}{\sum P \sin\alpha} \dots\dots\dots (2.19)$$

On the first iteration, the vertical shear forces (X_R and X_L) are set to zero. On subsequent iterations, the horizontal interslice forces are first computed from:

$$(ER - EL) = P \sin\alpha - \frac{1}{f} [c^{-}l + (P - ul)\tan\phi^{-}]\cos\alpha \dots\dots\dots (2.20)$$

where:

ER : Horizontal interslice force (from right side of slice)

EL : Horizontal interslice force (from left side of slice)

l : The inclined width of slice

u : Pore water pressure

P : Normal force on the base of the slice

Then the vertical shear forces are computed using an assumed (λ) value and $f(x)$. Once X_R and X_L are determined, the normal force P on the base of each slice is then calculated and the value of λ for which $F_m = F_f$ can then be found iteratively as shown in figure (2.5).

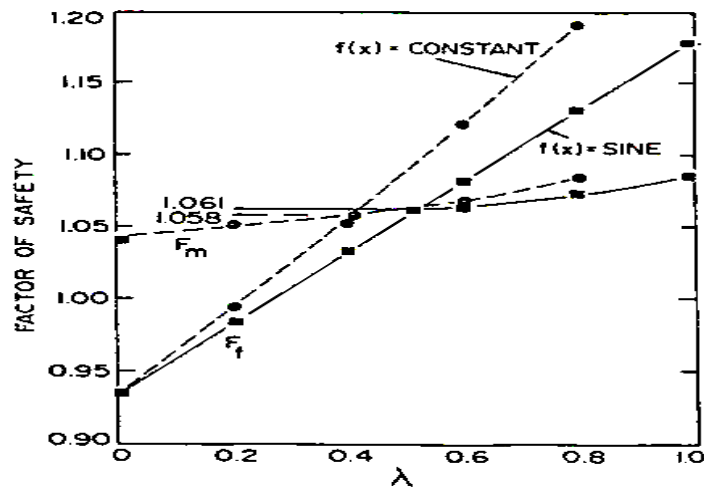


Figure (2.5) Variation the FOS with respect to moment and force equilibrium vs. λ for the Morgenstern-Price method [Fredlund and Krahn, 1977],.

This method of slope stability analysis, which is valid for slip surfaces of any arbitrary shape, is considered as the more general rigorous method [Baker, 1980]. It stems its generality from the fact that no stringent restriction is imposed neither on the direction or location of the interslice forces nor on the shape of the slip surface analyzed [Al-Jorany, 1996].

For these reasons, the Morgenstern – Price method is chosen among all the other methods to be used in the limit equilibrium computation procedure presented in this work.

In addition to the methods mentioned previously for analysis of slope stability by using slices method, there are other methods available, e.g., Spencer's Method, Janbu's Simplified Method, Janbu's Generalized Procedure of Slices (GPS), and Sarma's Method.

The various methods of two-dimensional limit equilibrium analysis differ from each other in two regards [Al-Bataineh, 2006]:

- Different methods use different assumptions to make up the balance between the number of equations of equilibrium and the number of unknowns.
- Different methods use different assumptions regarding the location and orientation of the internal forces between the assumed slices.

In the following paragraphs, will be review on the some previous studies that

presents by different researches in field of stability of earth dam under rapid drawdown condition and seismic load (earthquake).

Sarma (1973) developed a simple but accurate method of stability analysis of embankments and slopes to determine the critical earthquake acceleration that was required to bring a mass of soil, bounded by a slip line of any shape and free surface, to state of limiting equilibrium. It was based on the principle of limiting equilibrium and the method of slices. He found that in any solution, the physical acceptability of the complete solution must be checked before accepting the result. It was suggested to use the critical acceleration as a measure of the FOS.

Sarma (1975) presented a model of rigid block on a plane surface which was used to analyse the effect of inertia forces and pore-pressures on the stability of an earth dam or embankment during strong earthquakes. The study was based on limit equilibrium principles and material obeys the Mohr-coulomb failure criterion with effective stresses. Several interesting conclusions were drawn by this study; the excess pore pressures generated during an earthquake can be accounted for in the effective stresses analysis and displacements during an earthquake can be easily calculated by using sample pulses.

Desai (1977) applied a numerical finite element scheme for development of guidelines and charts for analysis and design of slopes subjected to transient drawdown conditions. The physical problem considered concerns analysis of slopes at certain sections of the banks of the Mississippi River. An important answer adopted by this study as a part of the investigation, the factor of safety at the end of drawdown computed from the numerical procedure was compared with the factor of safety from conventional sudden drawdown analysis. A series of charts were obtained for various combinations of problem parameters, and comments and suggestions concerning the potential of the procedure for analysis of slopes were presented in this study.

Hoe, et al. (1997) discusses the stability and permanent displacement of a slope subjected to combined horizontal and vertical accelerations. A long-spiral

failure mechanism was used. During this study, they found that the seismic force has a significant effect on stability and permanent displacement of slopes, and the parametric study reveals that vertical acceleration may play an important role on stability and permanent displacement if the corresponding horizontal acceleration was large.

Obaid (2002) used a computation procedure by which the most critical slip surface and its relevant minimum factor of safety was obtained. He used this computation procedure to re-analyze the stability of upstream side slope of a zoned earth dam (Al-Qadisiya Dam, nowadays it is called Haditha dam) and found that the dam was safe under the different water level condition (minimum, maximum, and normal water levels).

Tran (2008) developed a numerical model to analyze the stability of the Tieng main dam in rapid drawdown condition for two cases before and after rehabilitation, using limit equilibrium and finite element methods. Changes of stress-strain behaviours and pore pressure, failure mechanism, and factor of safety of the upstream slope were investigated. In this study he found that the stability of the upstream slope is dramatically decreased but still being stable during rapid drawdown condition.

Sung (2009) adopted a numerical procedure for integrating a commercial finite difference method into a probabilistic analysis of slope stability. Given that the limit state function cannot be expressed in an explicit form, an artificial neural network (ANN)-based response surface was adopted to approximate the limit state function, thereby reducing the number of stability analysis calculations. A trained ANN model was used to calculate the probability of failure through the first- and second-order reliability methods and a Monte Carlo simulation technique. Probabilistic stability assessments for a hypothetical two-layer slope as well as for the Cannon Dam in Missouri, USA were performed to verify the application potential of the proposed method.

CHAPTER THREE

Verification of the Computer Program (SLIDE V .5.0) and Comparison

SLIDE V.5.0 is slope stability software to include built-in steady state unsaturated groundwater analysis capabilities using finite element method, full details of this program are shown in appendix-A. Comparison of the results of computer program (**SLIDE V.5.0**) with other results of programs for both seepage of water through earth dams and slope stability analysis are very important, thus it should be done before using the program. The main aims of the comparison are to check the reliability of the theoretical aspects utilized in this program, and then to examine the proper working of this program with previous work.

In this respect the comparisons are of two types of problems analyses, groundwater analysis (seepage through earth dams), and slope stability analysis. It is performed by applying this program to some problems that had been analyzed previously by other researchers.

For the seepage analysis problems, a laboratory-scale embankments is analyzed by two different ways, manometers recorded field data, and 3D numerical analysis presented by **Megan, (2002)**.

For slope stability analysis problems, two different slope profiles are considered; they had been analyzed by **Fredlund and Krahn (1977)** and then reanalyzed by **Baker (1980)**. These examples are taken from **Al Jorany (1996)**. For each of the two slope profiles, three different water conditions are investigated, totalling six possible combinations of geometry- soil properties, and water conditions. For rapid drawdown condition analysis, example problem is taken from **U.S. Army Corps of Engineers (2003b)**. This example is used to verify the procedure used in the rapid drawdown condition by **SLIDE** program in the present work.

3.1 Seepage Analysis Problems

3.1.1 Laboratory-Scale Embankment

Two laboratory-scale embankments were designed and constructed by Powertech Labs Incorporated to study the influence of a pervious zone on the measured self potential response to steady state seepage flow. The embankments were constructed of pure quartz silica sand (Ottawa sand) graded according to ASTM C33-93 and compacted in layers at approximately 5% moisture content within a Plexiglas tank, with dimensions as shown in Figure (3.1). Seepage was induced by means of an upstream reservoir maintained at constant levels of 22.5 cm and 18.0 cm. Two embankment configurations were tested at these reservoir levels: a homogeneous dam and a dam containing an upstream defect. A pervious zone extending the full 15 cm crest length of the embankment was introduced using a 3.75 cm layer of concrete sand placed 4 cm above the base. The defect extended from the upstream face of the dam to a distance of 60 cm from the upstream toe, as indicated in Figure (3.1). The hydraulic conductivity for Ottawa sand and defect zone are $(4.5 \times 10^{-4} \text{ cm/s})$, and $(5.5 \times 10^{-4} \text{ cm/s})$, respectively.

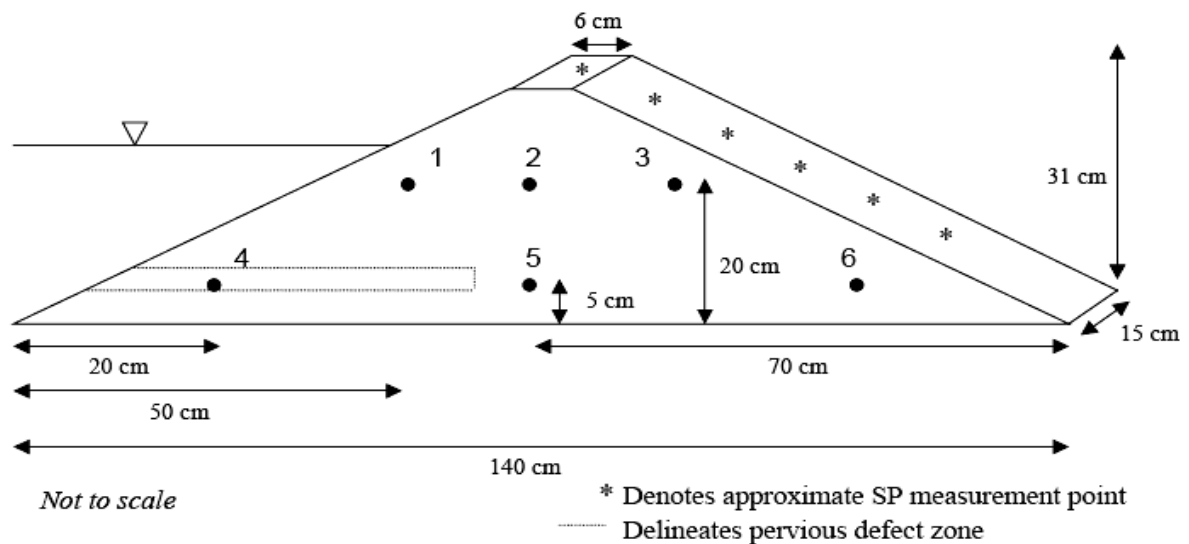


Figure (3.1) Laboratory embankment dimensions with manometer and electrode measurement locations, after (**Megan, 2002**).

The hydraulic head distribution within each embankment was monitored by means of six manometers connected to ports located at the side of the tank, as indicated in Figure (3.1). Electrical self potential (SP) measurements were recorded automatically using a series of gel-filled electrodes placed in contact with the soil along the upstream and downstream faces of the dam. Hydraulic and electrical readings were monitored until the system is stabilized, and were then recorded for a period of five days under steady-state flow conditions.

3.1.2 Experimental and Numerical Results

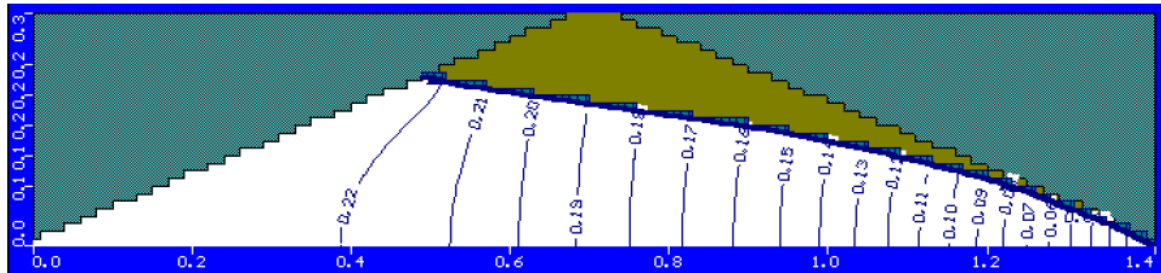
A $141 \times 5 \times 31$ cell mesh had been made by **Megan (2002)** to model the laboratory dams under steady-state flow conditions using three dimensional self potential (3DSP). Measured and modelled hydraulic head data were compared at each port location, as shown in Table (3.1) for the two embankment configurations at both reservoir levels. Figure (3.2) displays the results of the 3-D numerical seepage analyses taken through a central cross-section of each embankment. These cross-sections are representative of the numerical solution in all cells parallel to the crest.

Two laboratory scale embankments are modelled by **SLIDE** program in the present work; finite element analysis approach is used to model the seepage through the embankment. The number of elements used is 942 triangular elements with 522 nodes. The results of the finite element analysis to the total head distribution for two different cases of water conditions (reservoir level, 22.5cm and 18.0cm) and for two different embankments (homogenous embankment, non homogenous with defect zone) are shown in figure(3.3). Measured and modelled hydraulic head data were compared at each port location, as shown in Table (3.1) for the two embankment configurations at both reservoir levels.

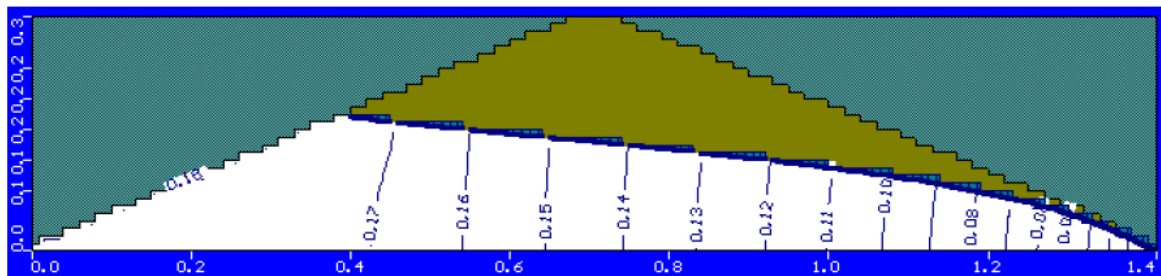
Table (3.1) Comparison of measured data of laboratory work after (Megan, 2002) and the results of **SLIDE** program

Measure points	22.5 cm reservoir level						18.0 cm reservoir level					
	Total head cm of water						Total head cm of water					
	Homogeneous			Upstream defect			Homogeneous			Upstream defect		
	Measured	Model 3D	SLIDE results	Measured	Model 3D	SLIDE results	Measured	Model 3D	SLIDE results	Measured	Model 3D	SLIDE results
1	22.5	22.1	21.7	22.2	22.2	21.9	-	-	-	-	-	-
2	18.5	19.0	18.6	20.2	19.7	19.5	-	-	-	-	-	-
3	-	-	-	-	-	-	-	-	-	-	-	-
4	22.8	22.5	22.4	22.1	22.4	22.3	18.2	18.0	18.0	17.5	18.0	18.0
5	18.6	18.7	18.6	20.2	19.4	19.5	15.1	14.4	14.4	15.0	15.6	15.6
6	10.1	8.8	9.5	9.8	9.2	9.5	8.6	7.7	8.4	7.8	7.9	8.0

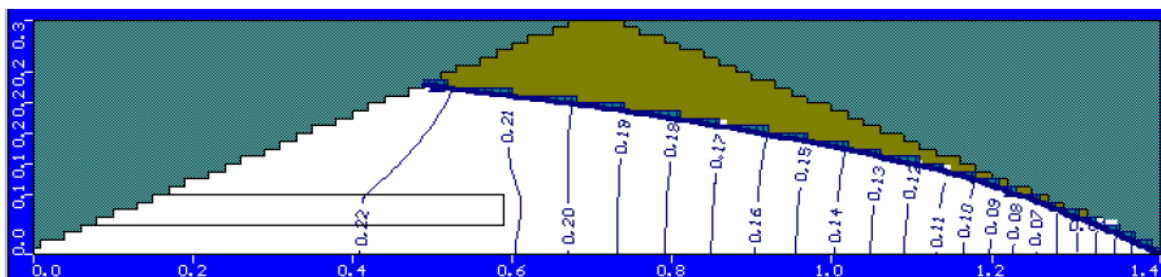
Table (3.1) reveals strong agreement between the measured data and the results of the **SLIDE** program model, and excellent agreement between the results of **SLIDE** program and the results of 3D numerical analysis of **Megan**, (2002).



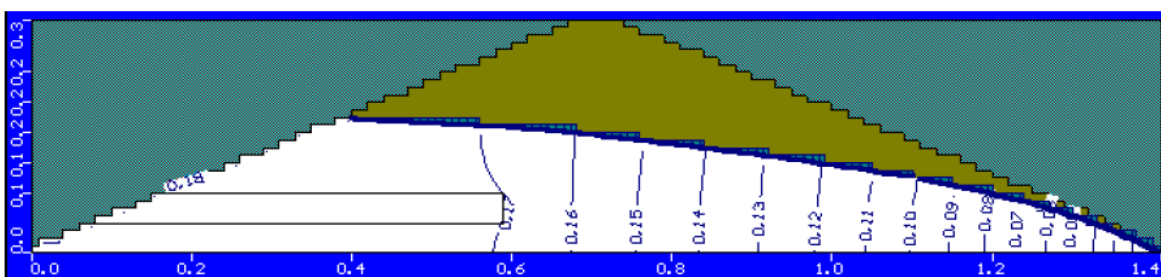
a) homogeneous dam; 22.5 cm reservoir level



b) homogeneous dam; 18.0 cm reservoir level

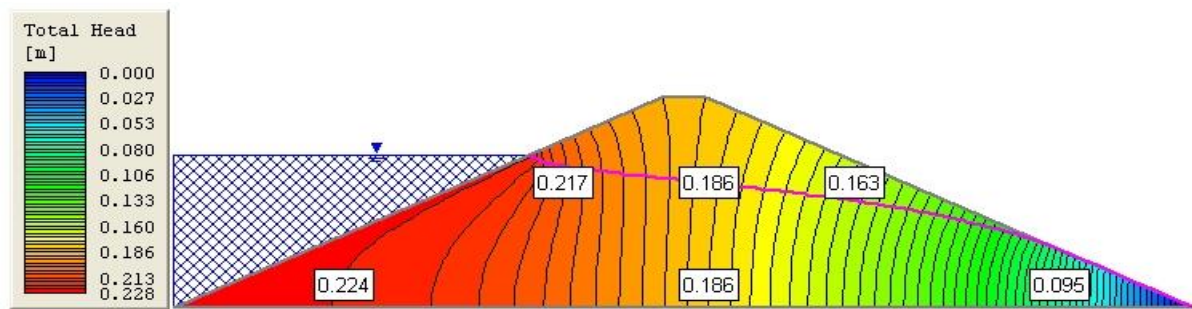


c) dam with upstream defect; 22.5 cm reservoir level

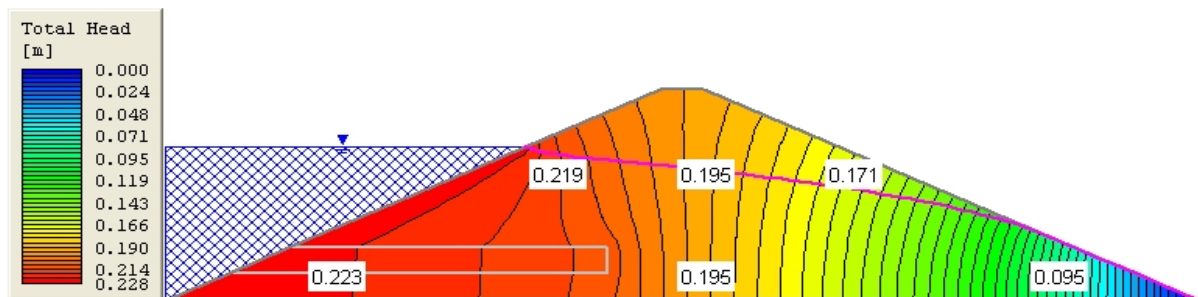


d) dam with upstream defect; 18.0 cm reservoir level

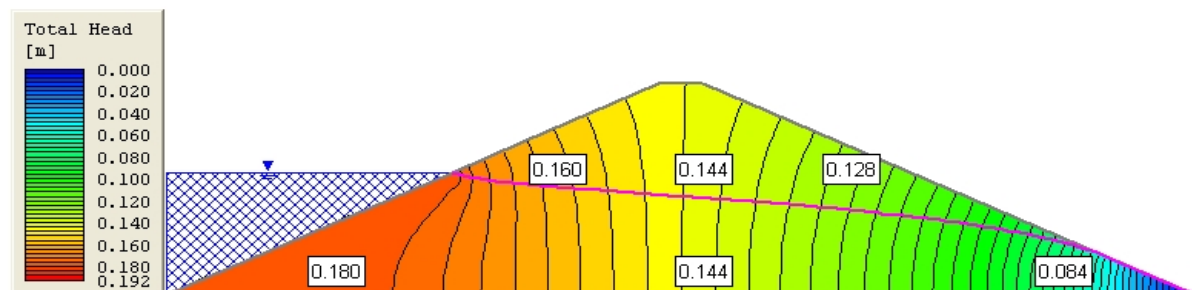
Figure (3.2): The results of the 3-D numerical seepage analysis taken through a central cross-section of each embankment presented by **Megan**, (2002).



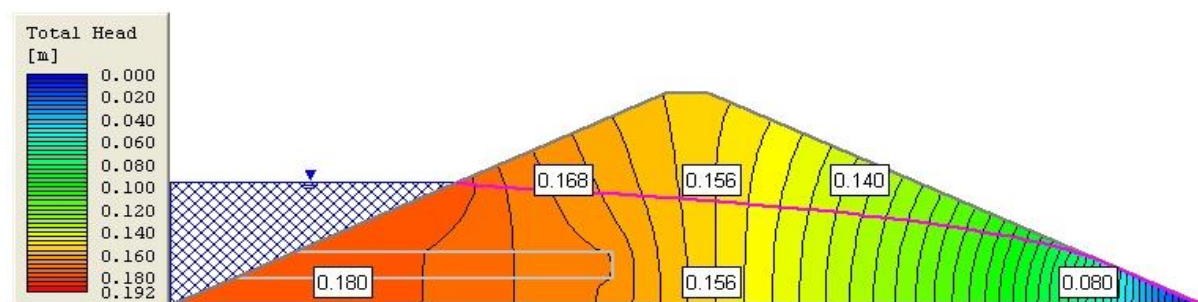
a) Reservoir level 22.5 cm



b) Reservoir level 22.5 cm with defect zone in upstream side.



c) Reservoir level 18.0 cm.



d) Reservoir level 18.0 cm with defect zone in upstream side

Figure (3.3) Total head distribution as obtained in the present work by **SLIDE** program, selected values in the figures represent comparison points.

3.2 Slope Stability Analysis (Case Study Examples)

3.2.1 Example No.1 (Al-Jorany 1996)

In this problem a homogeneous 2:1 slope profile of about 12 m. (30 ft) height is analyzed. The soil properties are: the unit weight $\gamma = 19.2 \text{ kN/m}^3$, $c = 29 \text{ kN/m}^2$ and $\phi=20^\circ$. The water condition is represented by the pore pressure coefficient (r_u) in the first two cases where r_u is taken as 0 and 0.25 respectively, while it is represented by a phreatic surface in the third condition.

The slope profile and the position of the phreatic surface are shown in Figure (3.4). The figure also shows the location of the circular slip surface which is considered by **Fredlund and Krahn (1977)** as the most critical slip surface for all the cases studied.

Table (3.2) displays a comparison of the values of factor of safety obtained by different methods of slope stability analysis as given by **Fredlund and Krahn (1977)**, and reanalyzed by **Usama (2007)** by (SLOPBG) program. It is noted that the values of FOS obtained by the methods which are based on the moment's equilibrium equation are almost similar except that obtained by the ordinary method. This may be attributed to the unreasonable assumptions imposed by this method where Newton's second law of action and reaction is violated at the slice interfaces. The analysis of the three conditions of this slope profile is performed by two computer programs and comparison with **SLIDE** program individually. The computer program **DPSSA** which is prepared at the University of Baghdad according to a developed limit equilibrium approach is used to analyze the same conditions of this profile. The results of these programs are displayed in Table (3.3) for comparison. Three different functions $f(i)$ (which describe the variation of the angle of inclination of the interslice forces) are considered. The differences in values of FOS for the same slip surface are insignificant (less than 0.5%). It is also noted that the effect of the function $f(i)$ on the value of factor of safety is almost insignificant also.

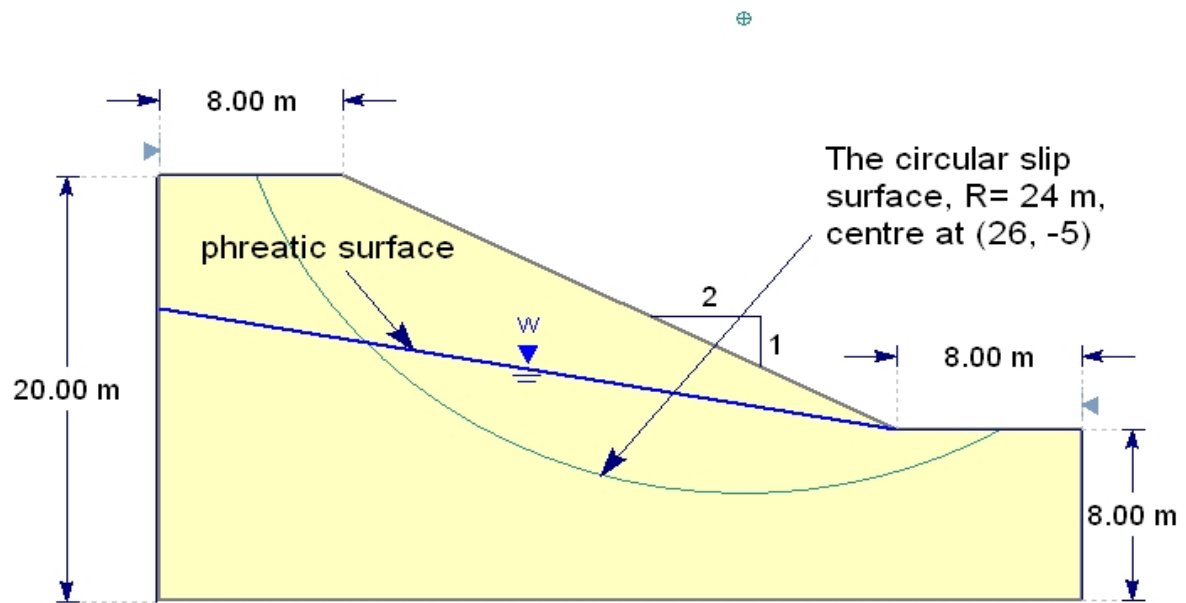


Figure (3.4): Slope profile of example problem No.1 after Al-Jorany, (1996).

Table (3.2) Comparison of factor of safety values obtained by different methods of analysis, example problem. No. 1 (after **Fredlund and Krahn** ,(1977)).

METHOD	ru = 0		ru = 0.25		Phreatic surface.	
	FOS	$\lambda 1$	FOS	$\lambda 1$	FOS	$\lambda 1$
Ordinary Method	1.928		1.607		1.693	
Simplified Bishop Method.	2.080	0	1.766	0	1.834	0
Janbu's Method(Rigorous)	2.008		1.708		1.776	
Janbu's Method(simplified)	2.041		1.735		1.827	
Spencer's Method	2.073	0.237	1.761	0.255	1.830	0.247
Morgenstern-Price-Method. (f(i)=Const.)	2.076	0.254	1.765	0.244	1.833	0.234
SLOPBG (Prokon) Prog.	2.048		1.726		1.789	
SLIDE program	F(i)= half sine	2.001	1.703		1.854	
	F(i)=constant	2.002	1.704		1.853	

$\lambda 1$: A constant related to the direction of inter slice forces.

Table (3.3) Comparison of different solutions to Morgenstern –price method, example No.1

Comp. Program	f(i)	ru = 0		ru = 0.25		Phreatic surface.	
		FOS	$\lambda 1$	FOS	$\lambda 1$	FOS	$\lambda 1$
Alberta Prog.*	Constant	2.085	0.257	1.772	0.351	1.838	0.270
	Half sine	2.085	0.314	1.770	0.432	1.837	0.331
Saskatch slope Prog. *	Constant	2.076	0.254	1.765	0.244	1.833	0.234
	Half sine	2.076	0.381	1.764	0.304	1.832	0.29
	Clip. sine	2.083	0.390	1.779	0.417	1.832	0.300
Baghdad DPSSA Prog. *	Constant	2.065	0.261	1.751	0.304	1.827	0.230
	Half sine	2.066	0.315	1.750	0.327	1.825	0.313
	Clip. sine	2.067	0.378	1.756	0.395	1.824	0.324
DAMSAF Prog. Finite element*		2.07		1.78		1.80	
SLOPBG Prog. *		2.048		1.726		1.789	
SLIDE prog.	Constant	2.002		1.704		1.853	
	Half sine	2.001		1.703		1.854	
	Clip.sine	2.001		1.703		1.854	

$\lambda 1$: A constant related to the direction of inter slice forces.

* Results quoted from (Usama, 2007)

Baker (1980) presented a computation procedure based on the Spencer method of slope stability analysis. He used the dynamic programming to minimize the Spencer safety functional, and then to determine the most critical slip surface and the corresponding factor of safety (**Usama, 2007**). The results obtained by Baker are illustrated in Figure (3.5) for the same conditions of this slope profile. For each case, he analyzed also the same circular slip surface proposed by **Fredlund and Krahn (1977)**.

The program **DPSSA** which utilizes the Morgenstern-Price method of analysis is used to determine the most critical slip surface for each of the same cases of **Al Jorany (1996)**. The results are presented in Figure (3.6).

In addition to the previous methods, to make a comparison of the results of the discussed example, **Al Jorany (1996)** used a computer program that depends

on the finite element method, called **DAMSAF**, which is used to make a comparison with **SLIDE** program. The finite element mesh of the slope profile in this example is shown in figure (3.7). It consists of 86 eight-nodded elements and 299 nodes. The results of the finite element program (**DAMSAF**) are: for $r_u=0$, $FOS = 2.07$, for $r_u = 0.25$, $FOS = 1.78$ and for the case of phreatic surface $FOS = 1.80$. The differences between the results of the two programs are less than 3%.

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Usama (2007) used the computer program package (**SLOPBG**) as a slope stability analysis program to analyze the same example, and then made a comparison with **SLIDE** program.

3.2.2 Results of SLIDE program (Example No.1)

The results of **SLIDE** program for example No.1 are given in tables (3.2),(3.3). The position of the most critical slip surface for each case of water condition and slope profiles are presented in Appendix B. The values of the FOS obtained in the present analysis are according to general limit equilibrium (**Morgenstern-Price-Method**), although other methods of analysis are available. The main reasons which led to choose this method of analysis are discussed in the previous chapter.

In spite of the different approaches that are utilized in this analysis, the results indicate a good correspondence with the results of **SLIDE** program and good agreement to use this program in the present work.

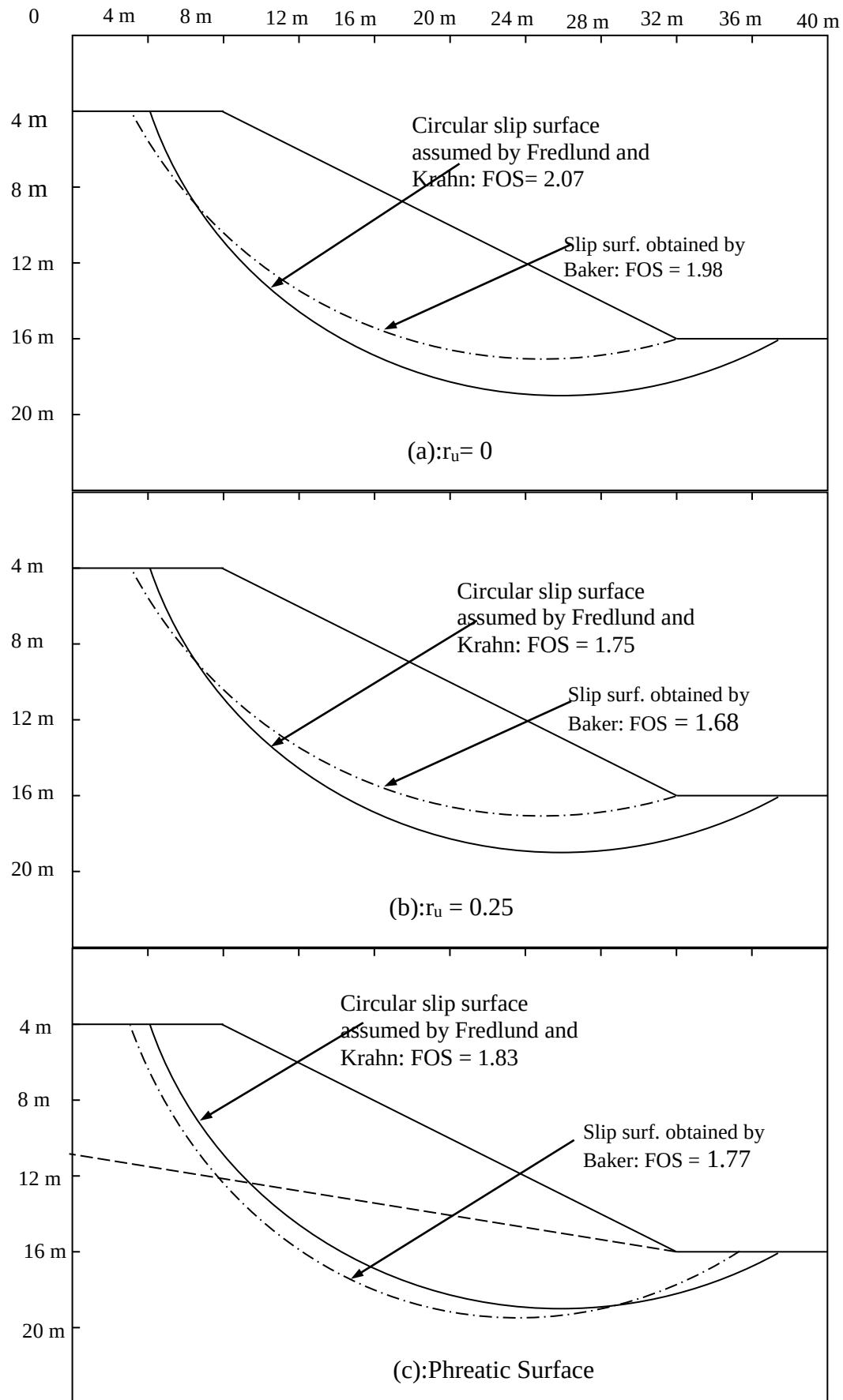


Figure (3.5) Slope stability analysis performed by **Baker, (1980).**

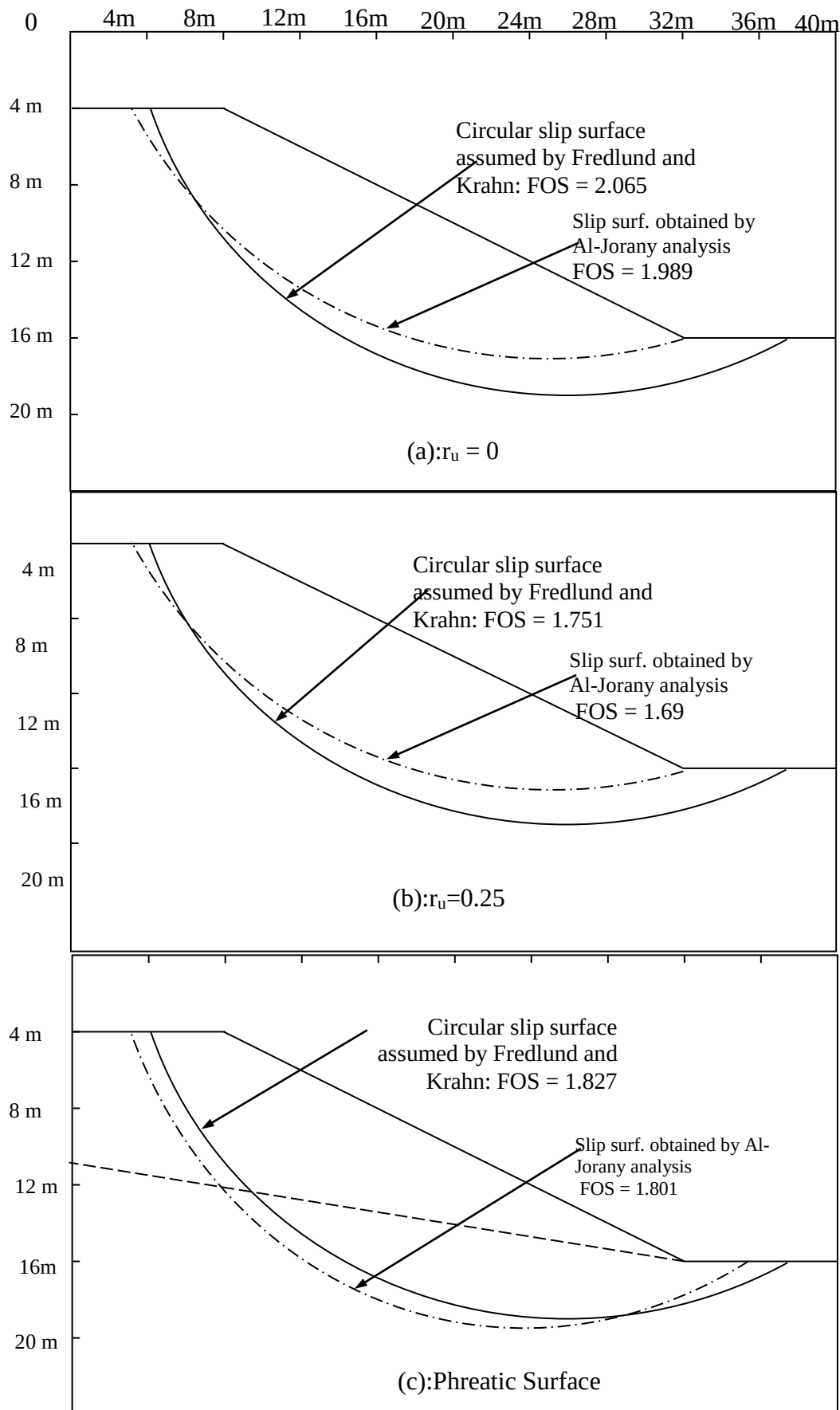


Figure (3.6): Slope stability analysis performed by **Al Jorany (1996)**, with **DPSSA** program, example No. 1.

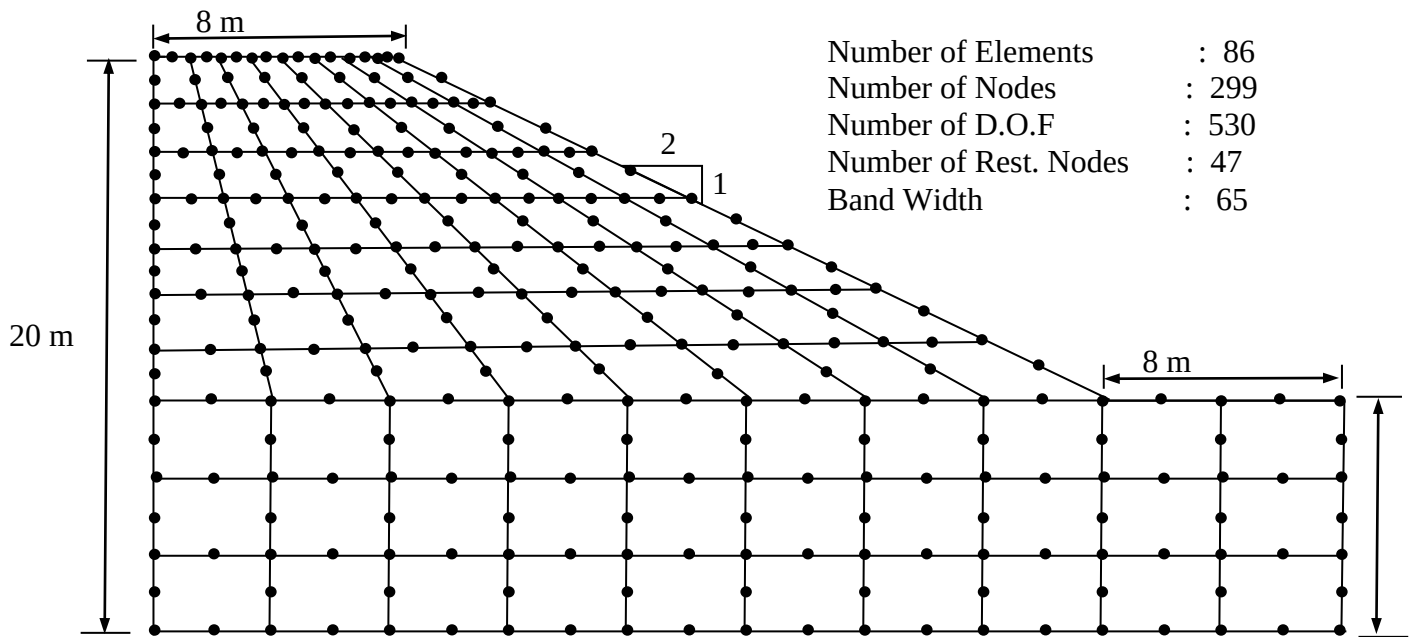


Figure (3.7): The finite element mesh of example No. 1 [Al-Jorany, (1996)].

3.2.3 Example No.2 (Al Jorany 1996)

In this problem, the same slope of the previous example is analyzed, but a thin (0.5 m thick) weak layer is assumed to exist at 0.75m depth below the slope toe. The shear strength parameters of this layer are; $c=0$ and $\phi= 10^\circ$. The same three water conditions are studied ($r_u = 0$, $r_u = 0.25$, and phreatic surface condition). Figure (3.8) displays the slope geometry and the assumed slip surface as given by **Fredlund and Krahn (1977)**. The initial part of this composite surface is a circular arc, while the other is a straight line passes adjacent to the weak layer interface [as shown in figure (3.9)].

Table (3.4) reviews the values of factor of safety obtained by different methods of slope stability analysis. The ordinary method gives under-estimated values of factor of safety for each of the three conditions. When the same slip surface is analyzed by the limit equilibrium approach, the computer program (**DPSSA**) yields the results which are shown in table (3.5), beside those obtained by the prmentioned computer programs, for comparison.

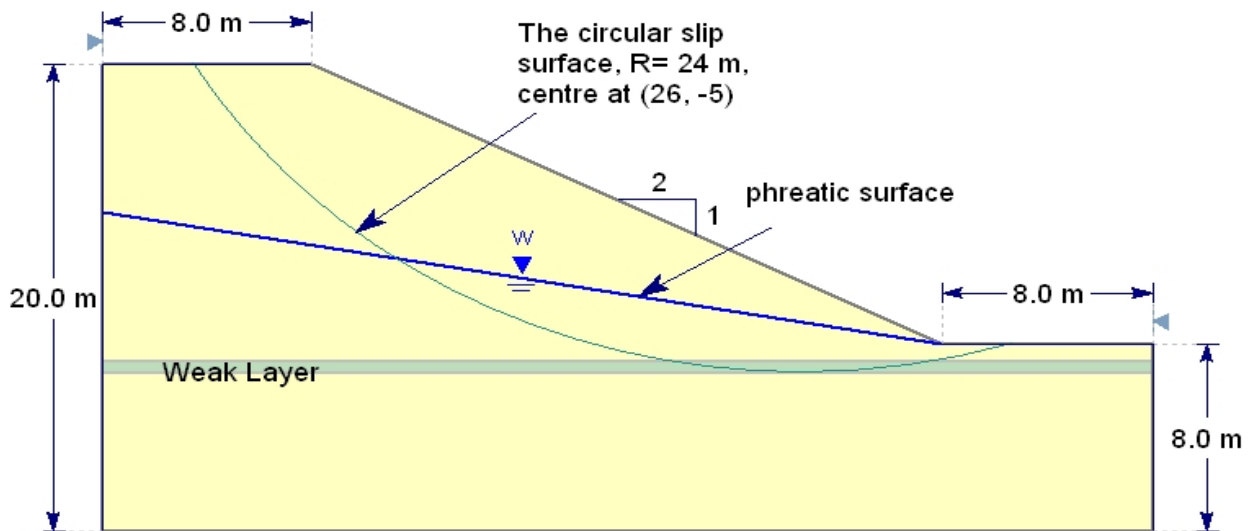


Figure (3.8): Slope profile of example problem No. 2 , Al-Jorany (1996).

Table (3.4) Comparison of factor of safety values obtained by different methods of analysis,[example. No. 2] (after **Fredlund &Krahn (1977)**)

METHOD		ru = 0		ru = 0.25		Phreatic surface.	
		FOS	$\lambda 1$	FOS	$\lambda 1$	FOS	$\lambda 1$
Ordinary Method		1.288		1.029		1.171	
Simplified Bishop Method.		1.377	0	1.124	0	1.248	0
Janbu's Method(Rigorous)		1.432		1.162		1.298	
Janbu's Method (simplified)		1.448		1.191		1.333	
Spencer's Method		1.373	0.185	1.118	0.139	1.245	0.121
Morgenstern-Price-Method. (f(i)=Const.)		1.378	0.159	1.124	0.116	1.298	0.097
SLOPBG (Prokon) Prog.		1.385		1.128		1.267	
SLIDE Prog.	F(i)=constant	1.309		1.165		1.262	
	F(i)= half sine	1.296		1.137		1.242	

$\lambda 1$:- A constant related to the direction of inter slice forces

Table (3.5) Comparison of factor of safety values obtained by different solutions to Morgenstern –price method example No.2.

Computer Program	f(i)	ru = 0		ru = 0.25		Phreatic surface.	
		FOS	$\lambda 1$	FOS	$\lambda 1$	FOS	$\lambda 1$
Alberta Prog.	Constant	1.394	0.182	1.137	0.334	1.265	0.159
	Half sine	1.386	0.218	1.117	0.441	Not Converg	
Saskatch slope Prog.	Constant	1.378	0.159	1.124	0.116	1.250	0.097
	Half sine	1.370	0.187	1.118	0.130	1.245	0.101
	Clip. sine	1.364	0.203	1.113	0.138	1.242	0.104
Baghdad DPSSA Prog.	Constant	1.325	0.166	1.092	0.185	1.228	0.112
	Half sine	1.320	0.202	1.075	0.154	1.224	0.133
	Clip. sine	1.318	0.223	1.073	0.156	1.222	0.117
Finite Element Analysis		1.34		1.165		1.19	
SLOPBG.Prog.		1.385		1.128		1.267	
SLIDE. Prog.	Constant	1.309		1.165		1.262	
	Half sine	1.296		1.137		1.242	
	Clip.sine	1.302		1.143		1.246	

As can be noted, the differences in the values of factor of safety for the considered cases are almost insignificant but, in general, the values obtained by (DPSSA) are slightly less than those obtained by the other two programs (Alberta and Saskatch slope). The comparisons of the results of SLIDE program with the different methods as analyzed in the present work are shown in table (3.5).

Baker (1980) analyzed the same slope profile for the same water conditions. The results of his analysis are shown in Figure (3.9). The composite slip surface of **Fredlund and Krahn (1977)** is investigated in his analysis which is based on the Spencer method. The dashed lines in Figure (3.9) represent the most critical slip surfaces as determined by Baker's computation scheme. The values of factor of safety obtained by Baker are in general less than the corresponding ones obtained by (**Fredlund and Krahn, 1977**) even for the

same slip surface. The same conditions of the slope profile are analyzed by the computer program (**DPSSA**) by **Al Jorany (1996)** and the results are displayed in Figure (3.10). The slip surface presumed by **Fredlund and Krahn (1977)** yields values of factor of safety which are slightly more than those obtained by **Al-Jorany (1996)** and almost similar to those given by **Baker(1980)**.

The most critical slip surface as determined by using **SLIDE** computer program for each of the three conditions consists mainly of a polygonal slip surface part and a straight line passing adjacent to the weak layer interface. The general shape of this slip surface is similar to that obtained by Baker. The values of factor of safety corresponding to this surface are almost consistent with those obtained by Baker analysis. In addition to the previous methods, to make a comparison for the case involved in this example, **Al Jorany (1996)** used **DAMSAF** to make a comparison with **SLIDE** program. The finite element mesh of the slope profile in this example is shown in Figure (3.11). It consists of 80 elements and 279 nodes. The total number of degrees of freedom is 490 with a band width of 65. The critical values of FOS can then be extracted. The results of the finite element program (**DAMSAF**) are: for the case of $r_u = 0$, FOS=1.34, for the case of $r_u = 0.25$ FOS=1.165, and for the case of phreatic surface, FOS =1.19. The differences between these two programs are less than 6%. However, the values of factor of safety obtained by the different computation procedures for each water condition show a fair correspondence.

3.3.4 Results of SLIDE Program (Example No.2)

The results of analysis of this example are shown in tables (3.4) and (3.5). The shape and position of the most critical slip surface for each water condition are presented in Appendix B. In spite of the different approaches that are utilized in this analysis, the results indicate a fair correspondence with **SLIDE** program and the results of this program were identical with those in Tables (3.4) and (3.5).

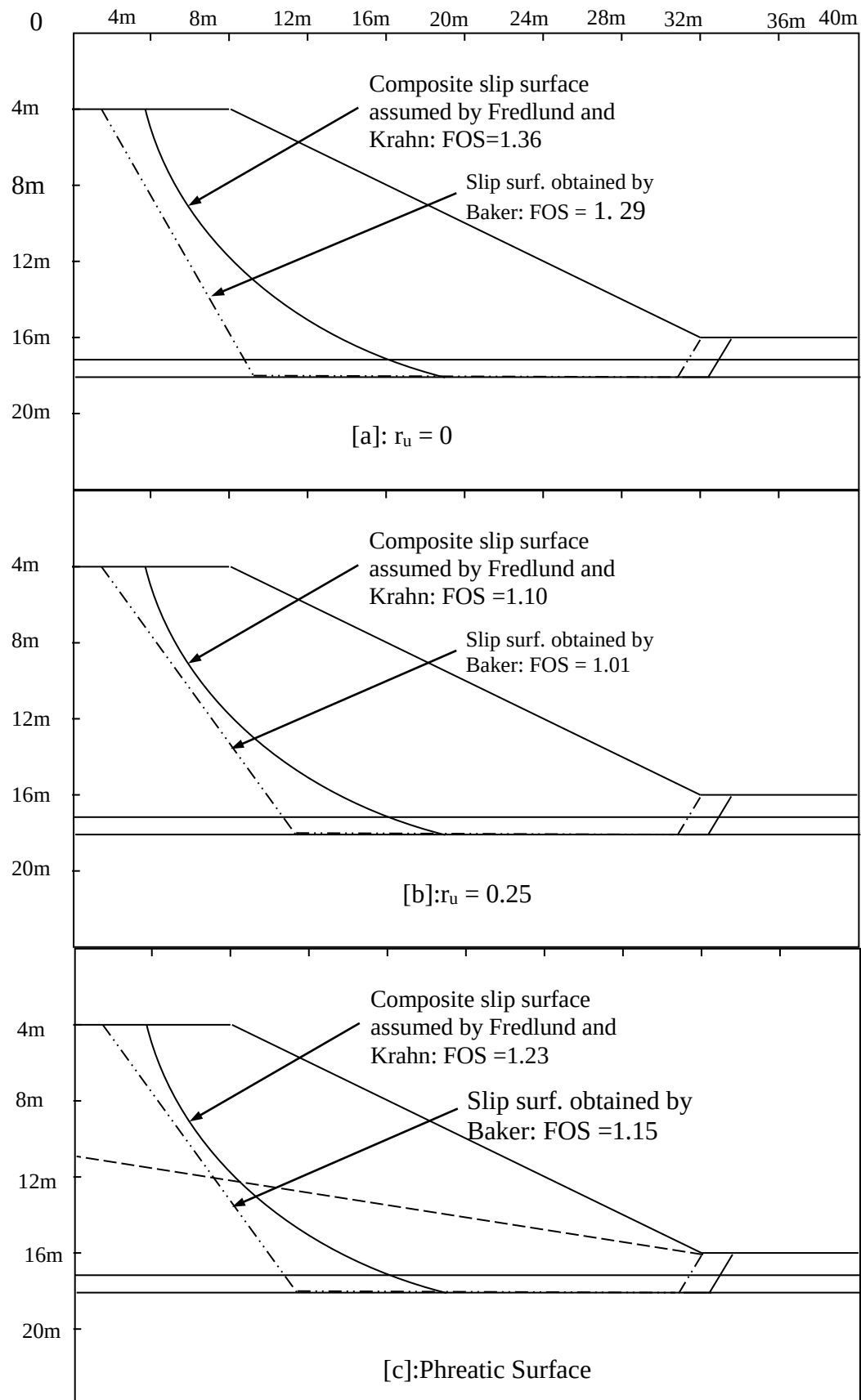


Figure (3.9) Slope stability analysis performed by **Baker ,(1980)** example No. 2

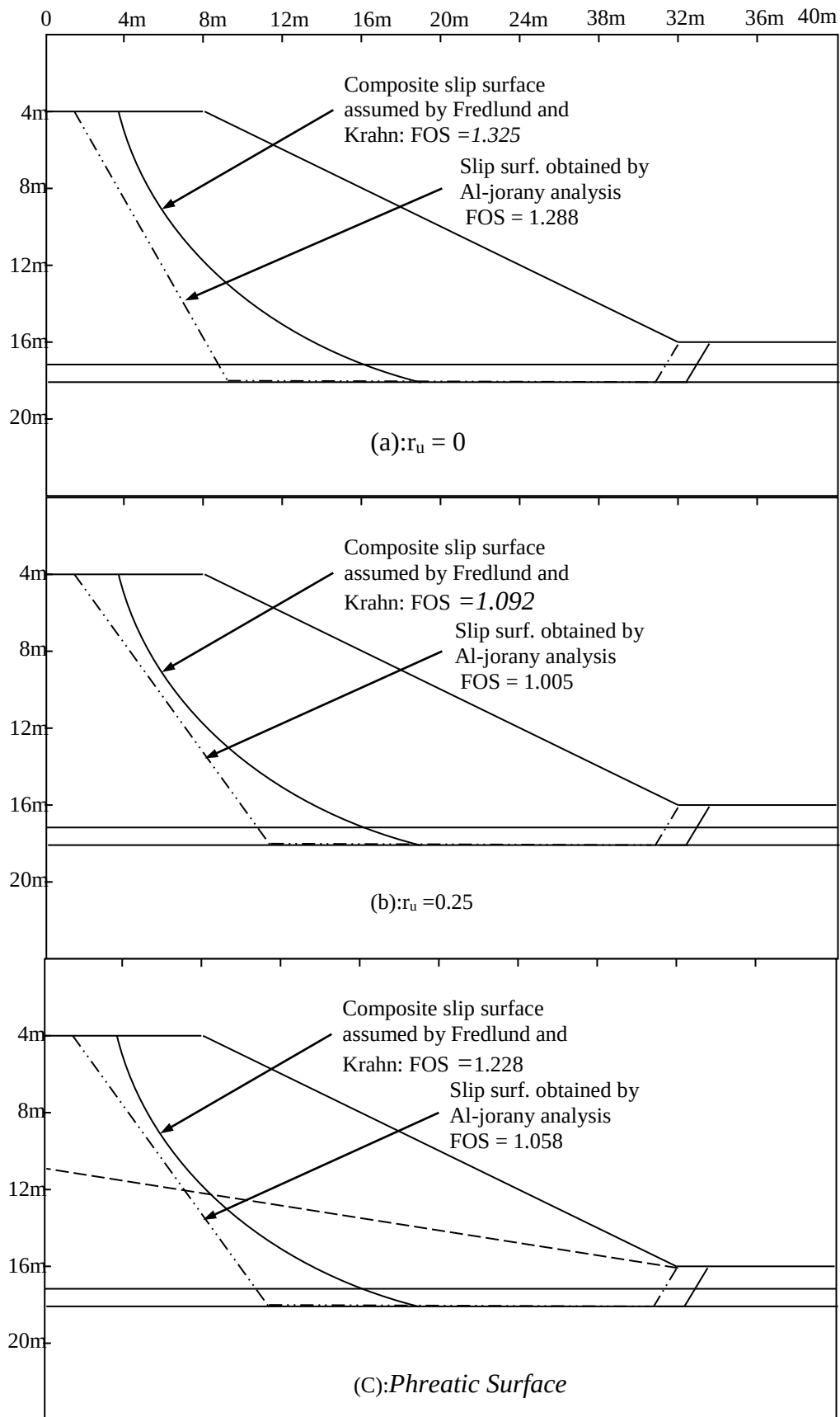


Figure (3.10) Slope stability analysis performed by **Al-Jorany** (1996) with **DPSSA** program, example No. 2

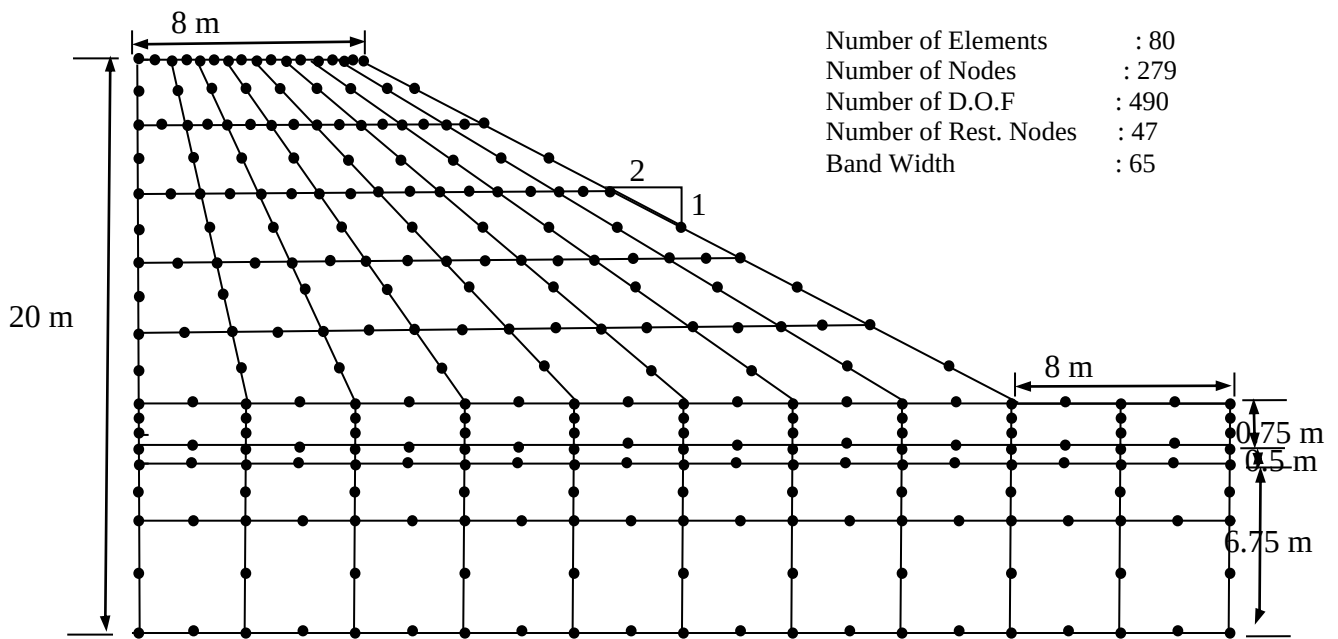


Figure (3.11): The finite element mesh of example No. 2, [Al-Jorany (1996)].

3.3 Rapid drawdown condition analysis

3.3.1 Rapid drawdown example (U.S. Army Corps of Engineers (2003b))

This example, adopted to verify the modelled rapid drawdown condition by **SLIDE** program, is the benchmark example used by the **U.S. Army Corps of Engineers (2003b)**. It is a simple homogenous embankment. The material properties and slope profiles are shown in figure (3.12).

The slope is uniform, with the shear strength parameters as indicated in Figure (3.12). The unit weight of soil is 135 pcf. The unit weight is assumed to be the same above and below the water levels and does not change as a result of drawdown. Drawdown is from a maximum pool level of 103 feet to a minimum pool of 24 feet.

Three methods of analyses are presented by **(U.S. Army Corps of Engineers (2003b))** for rapid drawdown condition. The first uses the Corps of Engineers' 1970 procedure (USACE 1970) for rapid drawdown, and the

Modified Swedish Method for the stability calculations. The second uses the improved (and recommended) procedure for rapid drawdown and the Simplified Bishop Method for the stability calculations. The third uses the improved procedure for rapid drawdown, and the Modified Swedish Method for stability calculations, with side force inclinations determined using Spencer's Method.

The results of these methods are presented in table (3.6) according to **(U.S. Army Corps of Engineers (2003b))**, and compared with result of the **SLIDE** program in the present work.

From table (3.7), it can be observed that the factor of safety obtained by **SLIDE** program is slightly differ than that obtained by the other procedures.

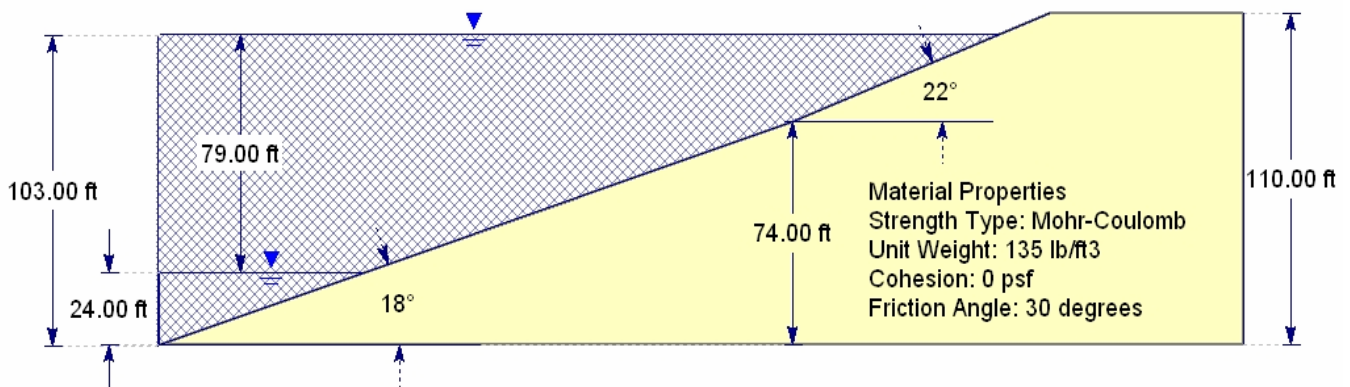


Figure (3.12) Profiles of benchmark example according to **(U.S. Army Corps of Engineers (2003b))**

Table (3.6) Comparison between **SLIDE** program results and other methods of analysis presented by **(U.S. Army Corps of Engineers (2003b))**.

Methods or procedure	Factor of safety	
	FOS steady state	FOS undrained condition
(USACE, 1970)	3.49	1.39
Simplified Bishop Method	2.2	1.44
Modified Swedish/Spencer Procedure	2.23	1.44
SLIDE. program	2.334	1.305

Chapter four

Parametric and sensitivity analysis

This chapter deals with the effect of some design parameters on the stability of side slope of earth dams. These parameters are divided into three groups according to their type and as follows:

- Geometry parameters (height of dam, slope angle of dam).
- Physical parameters (cohesion of dam material, angle of internal friction of dam material, total unit weight of dam material, phreatic surface location).
- Dynamic load parameters (seismic load coefficient, rapid drawdown in water level).

The sensitivity effect of the considered design parameters on the factor of safety shall be investigated by using two analysis approaches, deterministic and probabilistic analyses. The general example as the basic problem has been adopted from [Usama, 2007] is shown in the figure (4.1a). The results are obtained by the **SLIDE V.5.0** computer program.

4.1 Geometry Parameters

4.1.1 Effect of the Height of Dam

To study the effect of height of embankment, different analyses have been made for different heights ($H = 8, 10$ and 12 m) where other parameters remained constant. Figure (4.1 a) shows the case of a slope angle (α) = $\tan^{-1}(1/2)$, total unit weight (γ) = 19.2 kN/m^3 , angle of internal friction (ϕ) = 20° , cohesion (c) = 29 kPa , height of phreatic surface as shown in the figure, and $H=12$ m. When using the **SLIDE** program the factor of safety was 1.858 .

Table (4.1) shows the values of factor of safety that were found by using **SLIDE** program when the height of embankment is set to be (12, 10 and 8m).

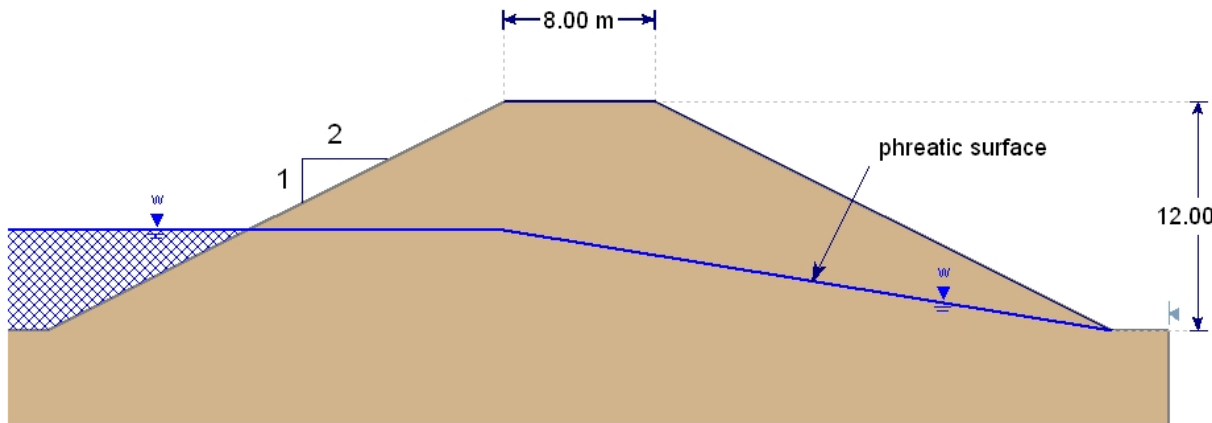


Figure (4.1a) General example as a basic problem used in the parametric analysis.

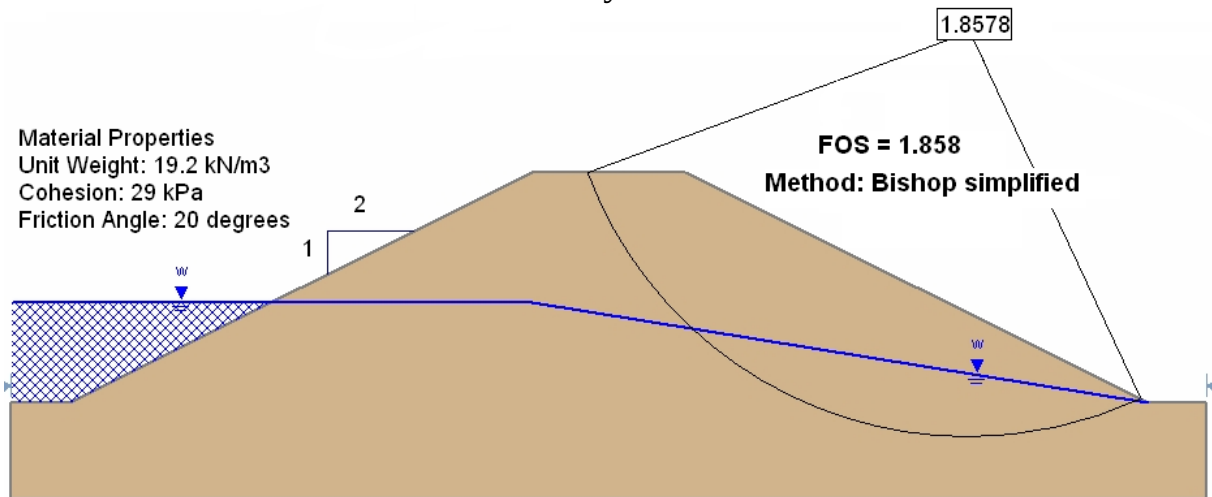


Figure (4.1b) Most critical slip surface in downstream side for general example.

Table (4.1): The input and output parameters for the general example

Input Parameters						Output Parameters
H m	$\alpha = \tan^{-1}(a/b)$ Degree	c kPa	ϕ°	γ kN/m ³	Phreatic surface	FOS
12	$\tan^{-1}(1/2)$	29	20	19.2	$\tan^{-1}(5.23/32)$	1.858
10					$\tan^{-1}(5.23/28)$	1.982
8					$\tan^{-1}(5.23/24)$	2.214

Curves in Figures (4.2) and (4.3) represent the general example with different values of slope for different values of heights of embankment ($H=8, 10$ and 12m). All of these results were obtained using **SLIDE** program. It can be observed that increasing the value of slope angle (α) from $= \tan^{-1}(1/2)$ to $\tan^{-1}(3/4)$ for different heights of embankment decreased the factor of safety between (11.83% - 15.23%). However, increasing the value of slope angle (α) from $= \tan^{-1}(1/2)$ to $\tan^{-1}(1/1)$ for different heights of embankment decreased the factor of safety between (19.55% - 23.84%). It is clear that the factor of safety increases with the decrease in height of the embankment.

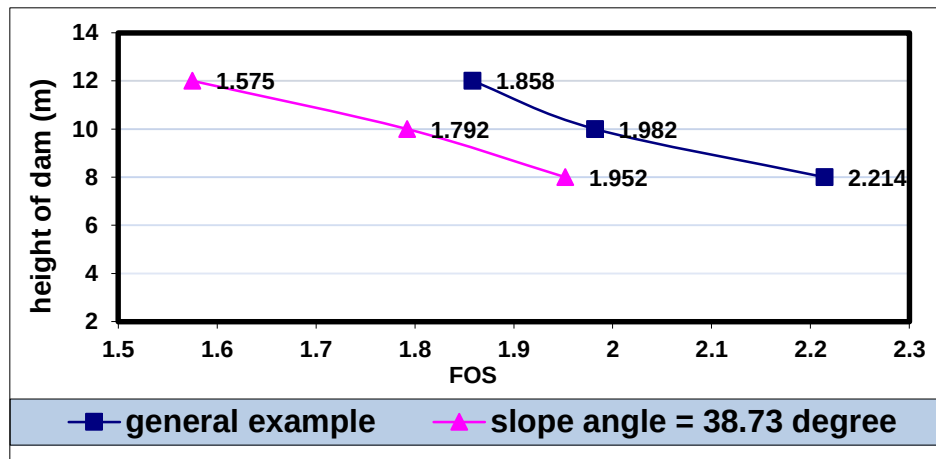


Figure (4.2) Effect of height of dam on FOS for slope angle = 38.73° .

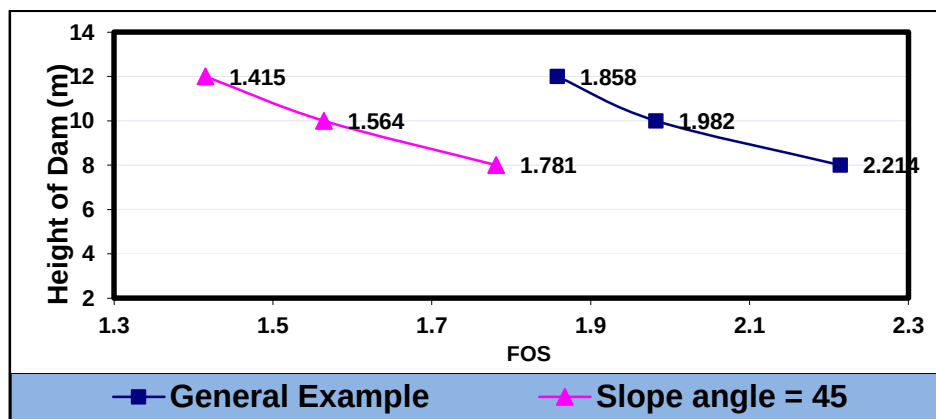


Figure (4.3) Effect of height of dam on FOS for slope angle = 45° .

Table (4.2): Input and output parameters when effect of height of dam is studied.

Ex. No.	Input						Output
General Ex.	H m	$\alpha^{\circ}=\tan^{-1}(a/b)$ Degree	c kPa	ϕ°	γ kN/m ³	Phreatic surface	FOS by SLIDE Prog.
	<u>12</u>	$\tan^{-1}(1/2)$	29	20	19.2	$\tan^{-1}(5.23/32)$	1.858
	<u>10</u>					$\tan^{-1}(5.23/28)$	1.982
	<u>8</u>					$\tan^{-1}(5.23/24)$	2.214
1	12	<u>$\tan^{-1}(3/4)$</u>				$\tan^{-1}(5.23/32)$	1.575
	10					$\tan^{-1}(5.23/28)$	1.729
	8					$\tan^{-1}(5.23/24)$	1.952
2	12	<u>$\tan^{-1}(1/1)$</u>				$\tan^{-1}(5.23/20)$	1.415
	10					$\tan^{-1}(5.23/18)$	1.564
	8					$\tan^{-1}(5.23/16)$	1.781

*Underlined values represent the parameters studied

4.2 Physical parameters

4.2.1 Effect of Cohesion

This effect has been studied by considering different values of cohesion of soil (c) (20, 29, 40 kPa), where other parameters are remained constant (height of embankment = 12 m, slope angle (α) = $\tan^{-1}(1/2)$, total unit weight (γ) = 19.2 kN/m³, angle of internal friction (ϕ) = 20° and phreatic surface).

The input and output data are as shown in Table (4.3) which contains the changed parameters in addition to the input and output parameters of the general example.

It can be observed that the factor of safety increases with the increase in cohesion of soil, and it is fact. The increase in cohesion of soil will increases in the value of shear resisting force on the slip surface.

Curves in Figure (4.4) represent the general example for different angle of internal friction for different values of cohesion (c) (20, 29, 40 kPa). It is observed that increasing the value of angle of internal friction (ϕ) from = 20° to

25° for different values of cohesion, the factor of safety is increased between (12.24% - 16.21%).

Curves in Figure (4.5) represent the general example for different location of phreatic surface for different values of cohesion (c) (20, 29, 40 kPa). It can be observed that dropping the position of Phreatic Surface from the position in the general example to $h_w = -4$ m from the toe with different values of cohesion of soil, the factor of safety is increased between (8.01% - 12.46%).

Curves in Figure (4.6) represent the general example for different unit weight (γ) for different values of cohesion (c) (20, 29, 40 kPa). It can be observed that decreasing the value of unit weight (γ) from $= 19.2 \text{ kN/m}^3$ to 15 kN/m^3 for different values of cohesion of soil, the factor of safety is increased between (6.89% - 7.755%).

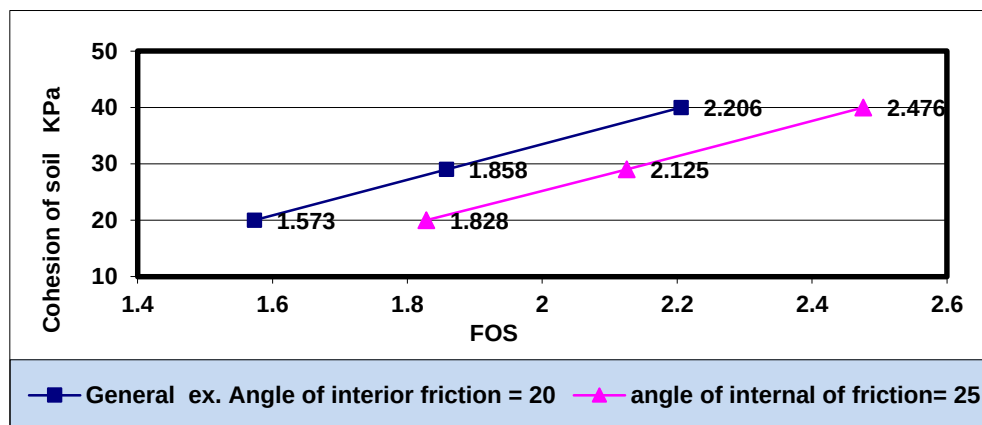


Figure (4.4) Effect of cohesion on FOS for different angles of Internal friction.

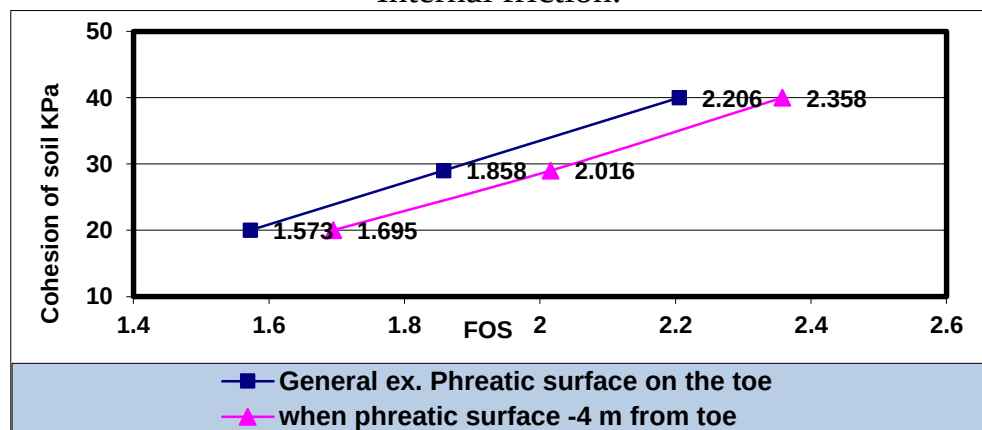


Figure (4.5) Effect of cohesion on FOS for different locations of phreatic surface.

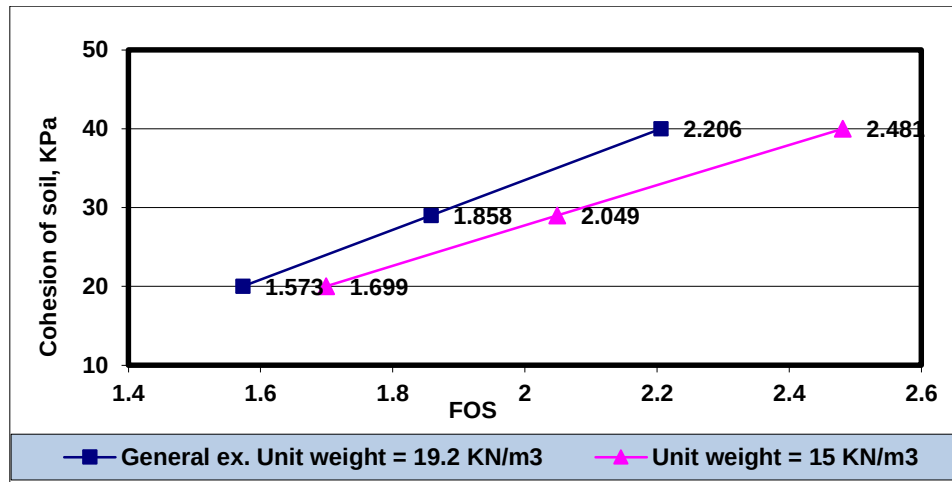


Figure (4.6) Effect of cohesion on FOS for different unit weights.

Table (4.3): The input and output parameters when effect of the cohesion is studied using **SLIDE** program deterministic approach

Ex. No.	Input				Output
General Ex.	c kPa	ϕ^o	γ kN/m ³	Phreatic surface	FOS BY SLIDE Prog.
	<u>20</u>	20	19.2	$\tan^{-1}(5.23/32)$	1.573
	<u>29</u>				1.858
	<u>40</u>				2.206
1	20	<u>25</u>	19.2	$\tan^{-1}(5.23/32)$	1.828
	29				2.125
	40				2.476
2	20	20	<u>15</u>	$\tan^{-1}(5.23/32)$	1.699
	29				2.049
	40				2.481
3	20	20	19.2	<i>hw= -4m under toe</i>	1.695
	29				2.016
	40				2.358

*Underlined values represent the parameters studied.

4.2.2 Effect of the angle of internal friction

Effect of angle of internal friction (ϕ) has been assessed by considering different values of (ϕ) while other parameters are kept constants. The considered values are ($\phi = 15^\circ, 20^\circ$, and 25°).

The input and output data are as shown in table (4.4) which contains the input and output parameters of general example. It can be observed that the factor of safety increases with the increase of the angle of internal friction of soil.

Figure (4.7) represent the general example for different cohesion of soil (c) for different values of angle of internal friction ($\phi = 15^\circ, 20^\circ, 25^\circ$). It can be observed that decreasing the value of cohesion from = 29 kPa to 20 kPa for different angle of internal friction, the factors of safety are decreased between (14.09% - 17.76%).

Figure (4.8) represent the general example for different unit weight (γ) for different values of angle of internal friction ($\phi = 15^\circ, 20^\circ, 25^\circ$). It can be observed that decreasing the value of unit weight (γ) from = 19.2 kN/m³ to 15 kN/m³ for different values of angle of internal friction, the factors of safety are increased between (8.53% - 11.98%).

Figure (4.9) represent the general example for different location of phreatic surface for different values of angle of interior friction ($\phi = 15^\circ, 20^\circ, 25^\circ$). It can be observed that decreasing the value of phreatic surface from the values shown in Table (4.4) to $h_w = -4$ m from the toe for different values of angle of internal friction, the factors of safety are increased between (6.9% - 7.96%).

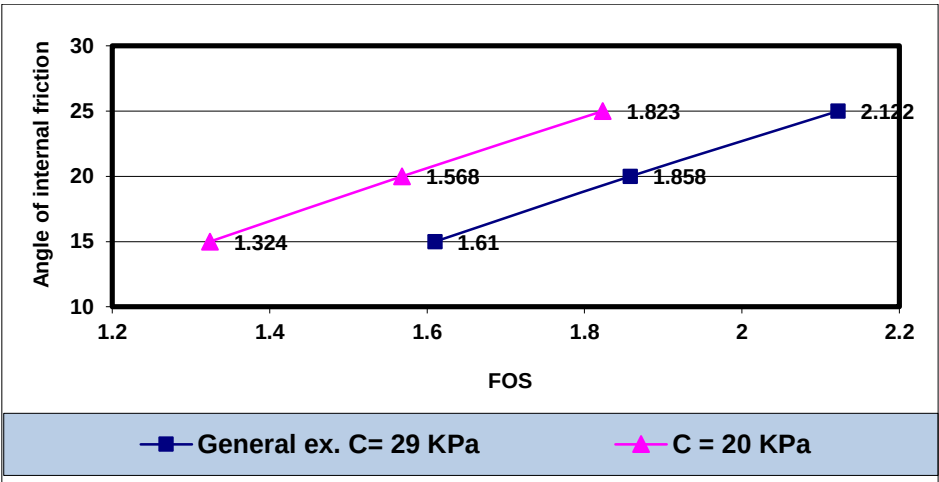


Figure (4.7) Effect of angle of internal friction on FOS for different values of cohesion.

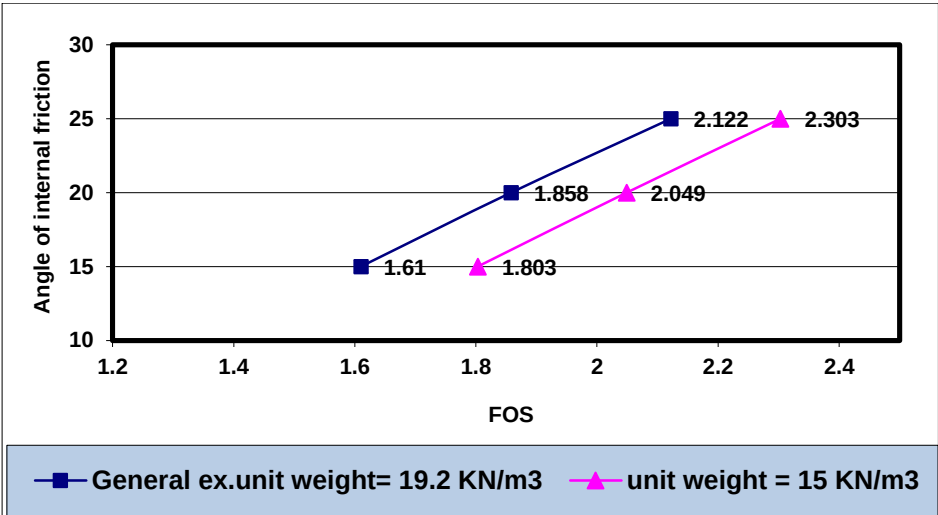


Figure (4.8) Effect of angle of internal friction on FOS for different values of unit weight of soil.

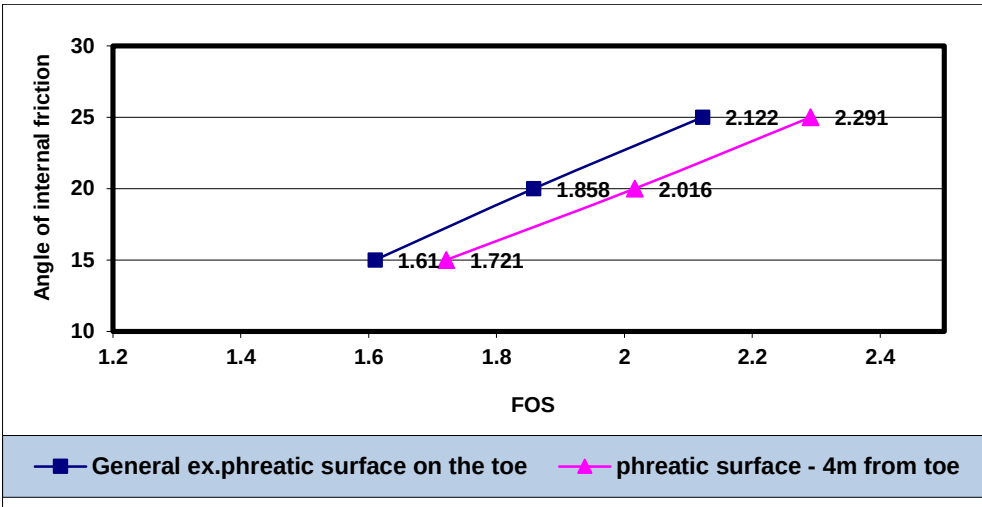


Figure (4.9) Effect of angle of internal friction on FOS for different locations of phreatic surface.

Table (4.4): The input and output parameters when effect of angle of internal friction is studied, deterministic approach.

Ex. No.	Input				Output
General Ex.	ϕ°	c kPa	γ kN/m ³	Phreatic surface	FOS by SLIDE Prog.
	<u>15</u>	29	19.2	$\tan^{-1}(5.23/32)$	1.610
	<u>20</u>				1.858
	<u>25</u>				2.122
1	15	<u>20</u>	19.2	$\tan^{-1}(5.23/32)$	1.324
	20				1.568
	25				1.823
2	15	29	<u>15</u>	$\tan^{-1}(5.23/32)$	1.803
	20				2.049
	25				2.303
3	15	29	19.2	<u>hw = -4m under toe</u>	1.721
	20				2.016
	25				2.291

*Underlined values represent the parameters studied.

4.2.3 Effect of unit weight of soil

To study the effect of unit weight of soil (γ), several analyses have been made and the results are illustrated in the respective figures

Figure (4.10) shows the general example for different cohesion of soil (c) for different values of unit weight of soil ($\gamma = 15, 19.2$, and 24 kN/m^3). It can be observed that decreasing the value of cohesion from 29 kPa to 20 kPa for different unit weight, the factor of safety is decreased between (15.68% - 17.82%).

Figure (4.11) shows the general example for different angle of internal friction (ϕ) for different values of soil unit weight ($\gamma = 15, 19.2, \text{ and } 24 \text{ kN/m}^3$). It can be observed that increasing the value of internal of friction (ϕ) from $= 20^\circ$ to 25° for different total unit weight, the factor of safety is increased between (11.85% - 14.81%).

Figure (4.12) shows the general example for different location of phreatic surface for different values of soil unit weight ($\gamma = 15, 19.2, \text{ and } 24 \text{ kN/m}^3$). It can be observed that decreasing the value of phreatic surface as shown in Table (4.5) to $h_w = -4\text{m}$ from the toe for different values of soil unit weight, the factor of safety is increased between (10.19% - 18.13%). It can be observed that the factor of safety decreases with the increase of unit weight of soil. The increases in unit weight lead to increase on the driving force that lead to reduce FOS.

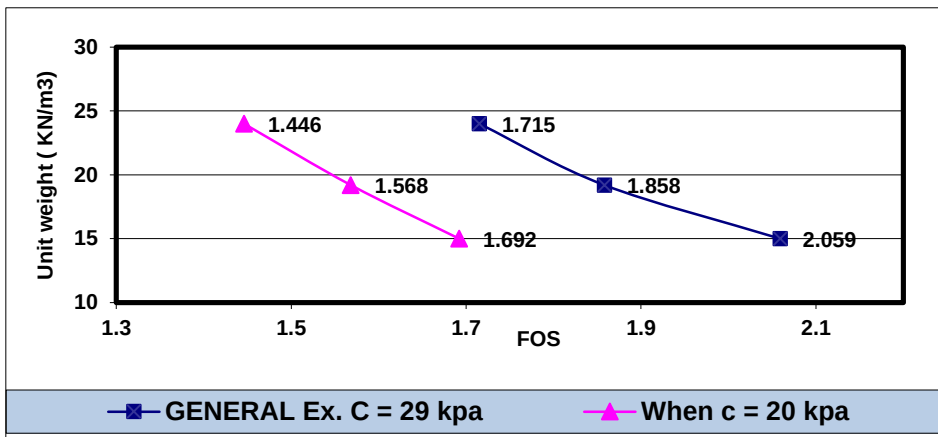


Table (4.10) Effect of soil unit weight on FOS for different values of cohesion.

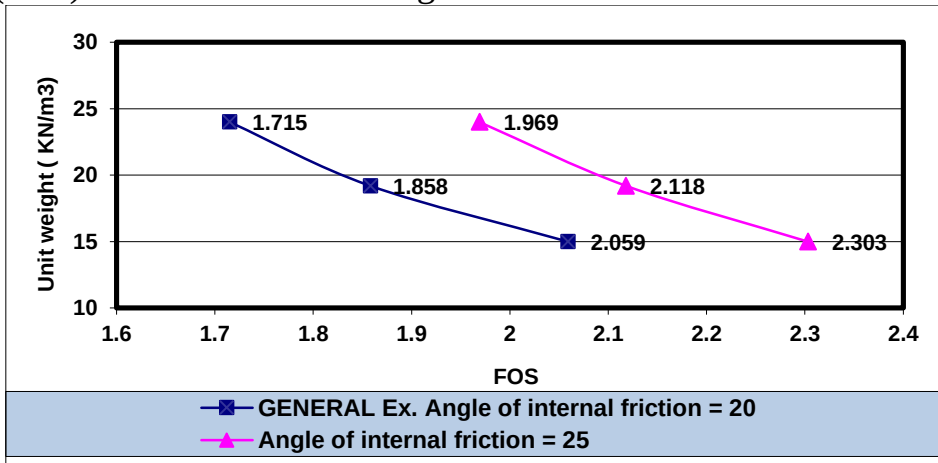


Figure (4.11) Effect of soil unit weight on FOS for different angles of internal friction.

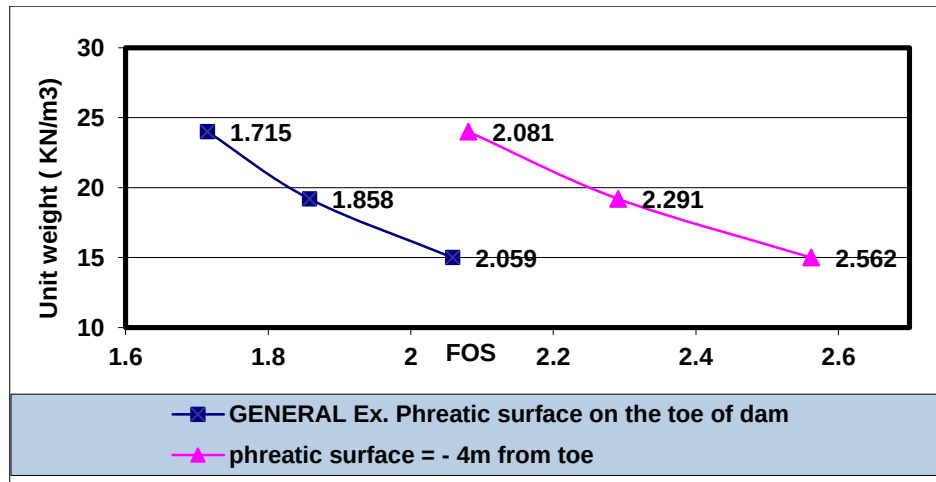


Figure (4.12) Effect of soil unit weight on FOS for different position of phreatic surfaces.

Table (4.5) The input and output parameters when effect of soil unit weight is studied, deterministic approach

Ex. No.	Input				Output
General Ex.	γ kN/m ³	c kPa	ϕ°	Phreatic surface	FOS by SLIDE Prog.
	<u>15</u>	29	20	$\tan^{-1}(5.23/32)$	2.059
	<u>19.2</u>				1.858
	<u>24</u>				1.715
1	15	<u>20</u>	20	$\tan^{-1}(5.23/32)$	1.692
	19.2				1.568
	24				1.446
2	15	29	<u>25</u>	$\tan^{-1}(5.23/32)$	2.303
	19.2				2.118
	24				1.969
3	15	29	20	<i>hw= -4m under toe</i>	2.562
	19.2				2.291
	24				2.081

*underlined values represent the parameters studied.

4.2.4 Effect of phreatic surface

To study the effect of phreatic surface location on the dam safety, three different locations (phreatic surface on the toe, + 4 m from toe, - 4 m from toe) are considered and shown in the figure (4.13).

Table (4.6) contains the input and output parameters of the general example to assess effect of location of phreatic surface on FOS.

Figure (4.14) shows the general example for different cohesion of soil (c) for different location of phreatic surface ($h_w = +4$, zero and -4 from toe). It can be observed that decreasing the value of cohesion from $= 29$ kPa to 20 kPa for different location of phreatic surface, the factor of safety is decreased between (15.92% - 17.48%).

Figure (4.15) shows the general example for different values of angle of internal friction of soil (ϕ) for different location of phreatic surface ($h_w = +4$, zero and -4 from toe). It can be observed that increasing the value of interior of friction (ϕ) from $= 20^\circ$ to 25° for different of location of phreatic surface, the factor of safety is increased between (12.36% - 13.64%).

Figure (4.16) shows the general example for different values of soil unit weight (γ) for different location of phreatic surface ($h_w = +4$, $+2$ and -4 from toe). It can be observed that decreasing the value of soil unit weight (γ) from $= 19.2$ kN/m³ to 15 kN/m³ for different values of location of phreatic surface, the factor of safety is increased between (9.55% - 11.50%).

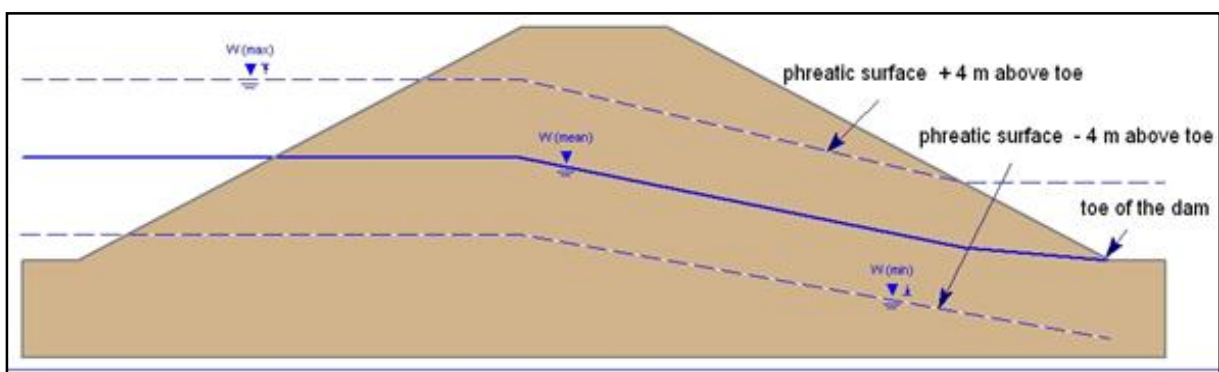


Figure (4.13) The upper and lower positions of phreatic surface.

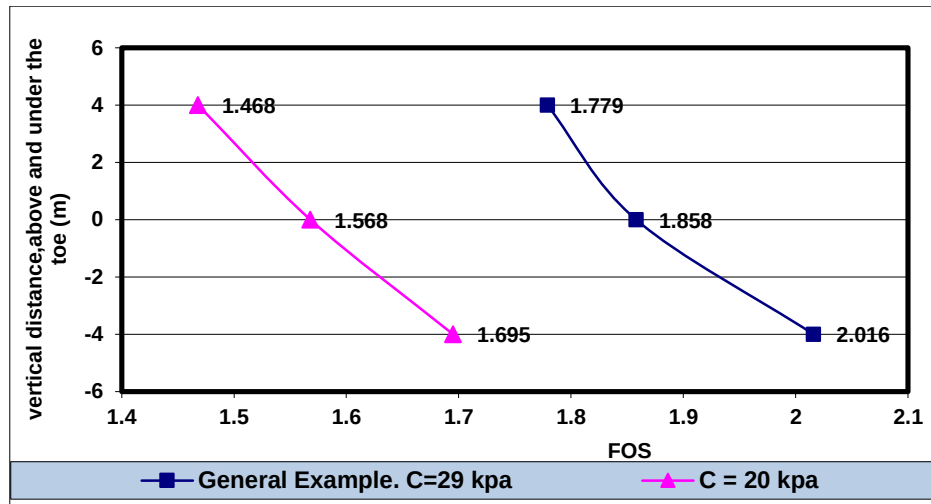


Figure (4.14) The effect of location of phreatic surface on FOS for different values of cohesion.

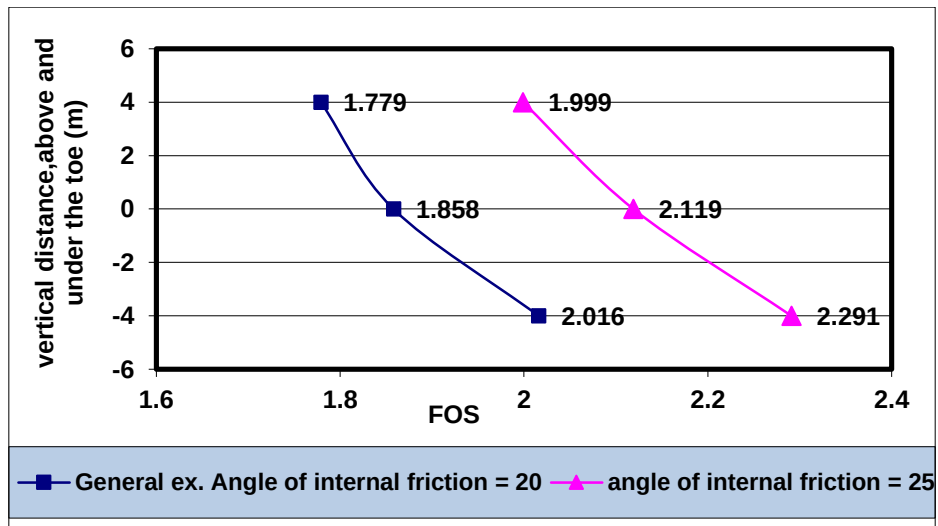


Figure (4.15) The effect of location of phreatic surface versus FOS for different angles of internal friction.

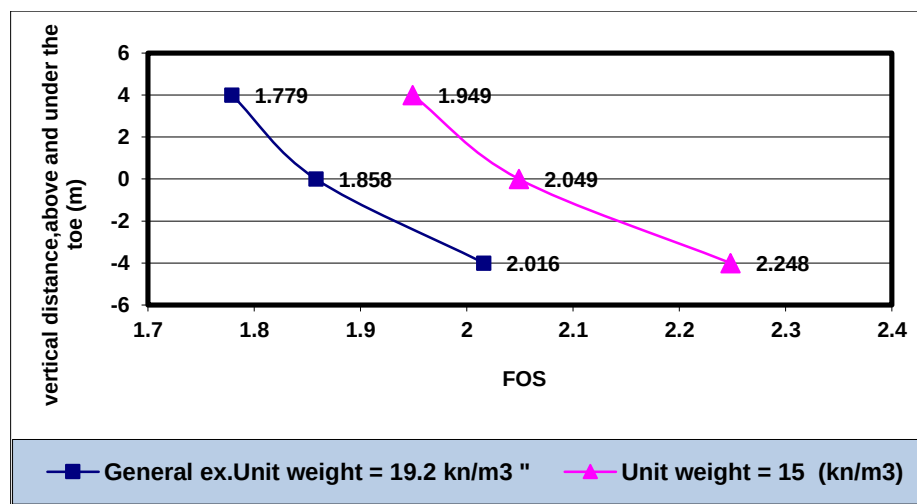


Figure (4.16) The effect of location of phreatic surface on FOS for different unit weights of soil.

Table (4.6) The input and output parameters for different locations of phreatic surface, deterministic approach.

Ex. No.	Input				Output
General Ex.	Location of phreatic surface	c kPa	ϕ°	γ kN/m ³	FOS by SLIDE Prog.
	<u>+ 4 m above toe</u>	29	20	19.2	1.779
	<u>On the toe</u>				1.858
	<u>- 4m below toe</u>				2.016
1	+ 4 m above toe	<u>20</u>	20	19.2	1.468
	On the toe				1.568
	- 4m below toe				1.695
2	+ 4 m above toe	29	<u>25</u>	19.2	1.999
	On the toe				2.119
	- 4m below toe				2.291
3	+ 4 m above toe	29	20	<u>15</u>	1.949
	On the toe				2.049
	- 4m below toe				2.248

*Underlined values represent the parameters studied.

4.3 Dynamic load parameters

4.3.1 Seismic load parameters

The seismic load condition allows including the effect of pseudo-static earthquake, and will be defined as a seismic coefficient effect in the study area. The seismic coefficients are dimensionless coefficients which represent the (maximum) earthquake acceleration as a fraction of the acceleration due to gravity.

To study the effect of seismic load coefficients, three different load coefficients (0.05, 0.1, and 0.15) are considered. If seismic coefficients are defined, a seismic force will be applied to each slice as follows:

$$\text{Seismic Force} = \text{Seismic Coefficient} * \text{Slice Weight} \quad \dots\dots\dots (4.1)$$

From this definition it can be observed that the seismic force increases when slice weight increases.

Table (4.7) presents a summary of results of studying the effect of the seismic force coefficients.

Figure (4.17) shows different values of soil unit weight for different values of seismic load coefficients (0.05, 0.2, and 0.15). It can be observed that decreasing the value of unit weight from 19.2 (kn/m³) to 15 (kn/m³) for different coefficients of seismic force, the factor of safety is increased between (8.9% - 9.78%).

Figure (4.18) shows different values of cohesion of soil(c) for different values of seismic load coefficients (0.05, 0.1, and 0.15). It can be observed that decreasing the value of cohesion from 29 kPa to 20 kPa for different coefficient of seismic force, the factor of safety is decreased between (13.47% - 15.47%).

Figure (4.19) shows different angles of internal friction of soil (ϕ) for different values of seismic force coefficients (0.05, 0.1, and 0.15). It can be observed that increasing the value of internal friction (ϕ) from = 20° to 25° for different of seismic load coefficients, the factor of safety is increased between (14.19% - 16.74%).

Figure (4.20) shows different locations of phreatic surface for different values of seismic load coefficients (0.05, 0.1, and 0.15). It can be observed that decreasing the value of Phreatic Surface as shown in Table (5.7) to hw= -4m from the toe for different values of seismic load coefficients, the factor of safety is increased between (8.99% - 11.79%).

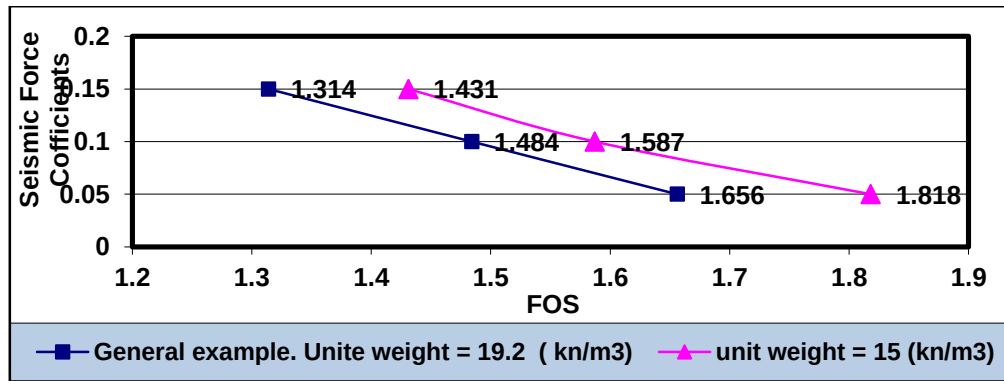


Figure (4.17) Effect of different values of soil unit weight on FOS for different values of seismic force coefficient.

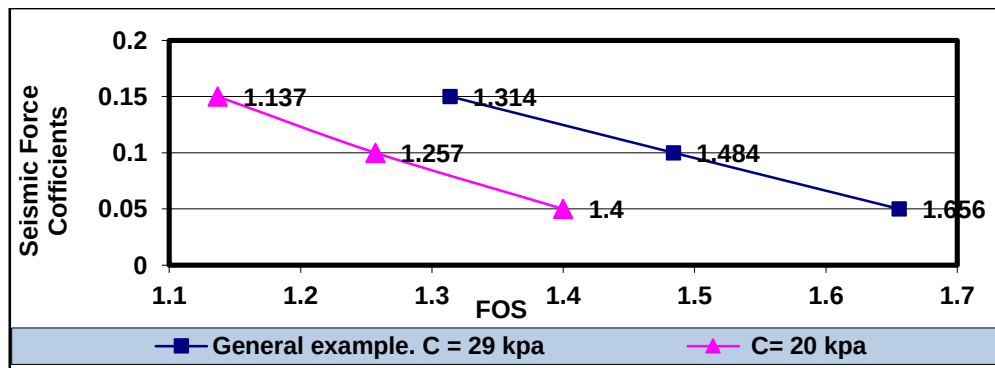


Figure (4.18) Effect of different values of cohesion on FOS for different values of seismic force coefficient.

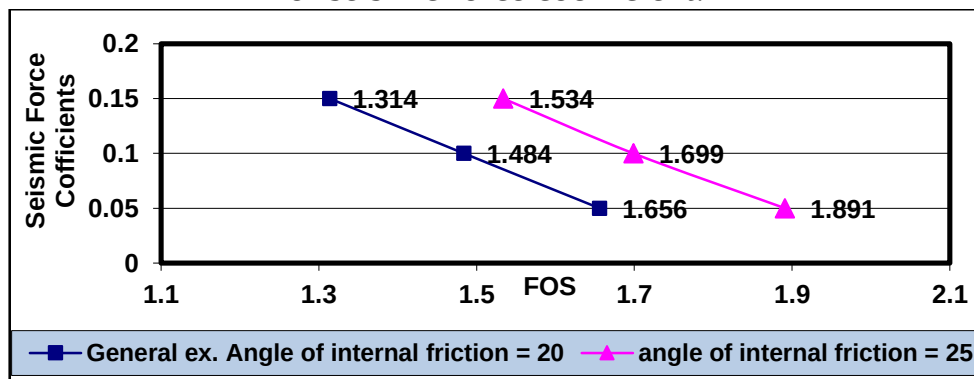


Figure (4.19) Effect of different values of angle of internal friction on FOS for different values of seismic coefficient.

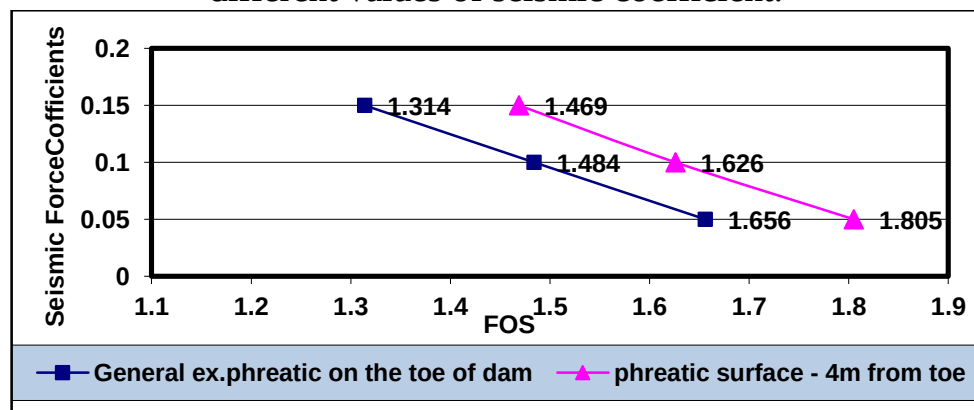


Figure (4.20) Effect of different values of phreatic surface location on FOS for different values of seismic force coefficient.

Table (4.7): The input and output parameters when effect of seismic force deterministic approach.

Ex. No.	Input					Output
General Ex.	Seismic force coefficient	γ kN/m ³	c kPa	ϕ°	Phreatic surface	FOS by SLIDE Prog.
	<u>0.05</u>	19.2	29	20	$\tan^{-1}(5.23/32)$	1.656
	<u>0.1</u>					1.484
	<u>0.15</u>					1.314
1	0.05	<u>15</u>	29	20	$\tan^{-1}(5.23/32)$	1.818
	0.1					1.587
	0.15					1.431
2	0.05	19.2	<u>20</u>	20	$\tan^{-1}(5.23/32)$	1.400
	0.1					1.257
	0.15					1.137
3	0.05	19.2	29	<u>25</u>	$\tan^{-1}(5.23/32)$	1.891
	0.1					1.699
	0.15					1.534
4	0.05	19.2	29	20	<i>hw= -4m under toe</i>	1.805
	0.1					1.626
	0.15					1.469

*underlined values represent the parameters studied.

4.3.2 Rapid drawdown condition

The preceding items focussed on delineating the effect of geometry parameters, shear strength parameters, and seismic load coefficient on FOS of the downstream side slope of the dam.

The drawdown is known as one of the most dangerous conditions for the upstream side slope. When the countervailing upstream water pressure has disappeared, it causes a danger to the upstream slope. The upstream shell cannot stay stable under the hydrodynamic pressure due to rapid drawdown. Soils inside the dam body remain saturated and seepage commences from it towards the upstream slope. Seepage and hydrodynamic pressures create downward forces acting on the upstream slope. Those are adverse to the stability and create a critical condition to the upstream slope.

When the phreatic line (free surface) falls slowly or remains almost at the same position, it is considered as ‘rapid’ drawdown. The lag of the phreatic line or the rate of drawdown depends on four factors: permeability coefficient of the dam fills, drawdown rate, active pore volume, and upstream slope gradient. The case in which the water has to be quickly lowered for the special purpose of rehabilitation, or material of the dam slope is impermeable so the phreatic line does not fall so much even if this drawdown has lasted for several months, is a common problem in reality. The need to study the slope stability of earthfill dams during drawdown is necessary and imperative not only for the existing dams, but also for the design and construction of other earthfill dam projects in the region [Tran, 2008].

Events following a rapid drawdown may usefully, but approximately be divided into four stages as shown in figure (4.21).

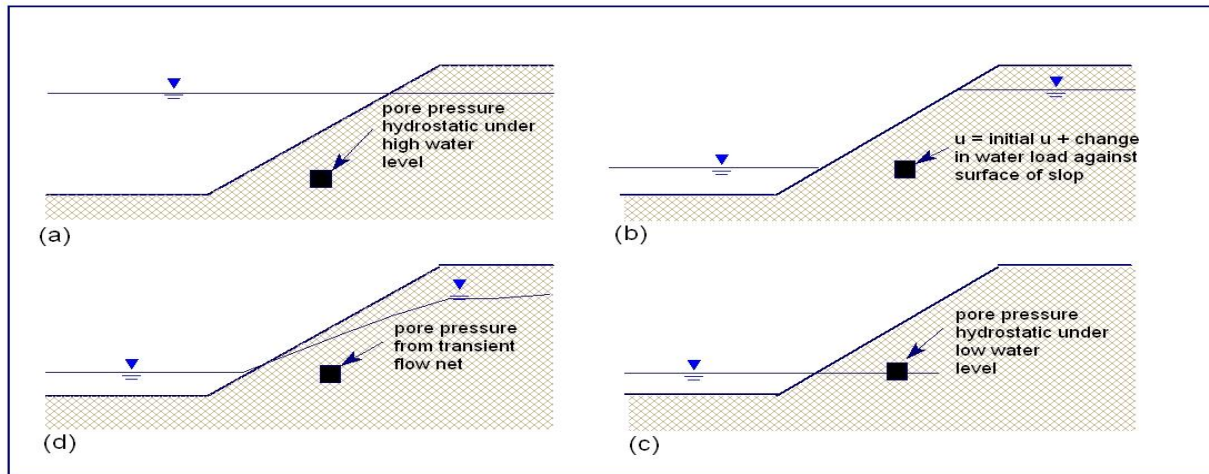


Figure (4.21) Response of slope to rapid drawdown. (a) Initial equilibrium condition. (b) After drawdown but before consolidation adjustment. (c) After consolidation adjustment. (d) Final equilibrium condition. [Lambe and Whitman , 1969].

If the drawdown time is much less than the time in which consolidation adjustment can occur within the slope, the pore pressures immediately following the drawdown will equal the pore pressures before drawdown plus the change in pore pressure due to the change in water load against the slope. In time, consolidation adjustments will occur, but pore pressures will still remain high until the excess water drains from the slope and a new equilibrium is reached corresponding to the low level of water against the slope. With free draining soils, such as coarse sands and gravels, the consolidation time will generally be less than any actual drawdown time so that the stage depicted in figure (4.21b) never occurs and stability of slopes in such soils can be analyzed using a transient flow net as shown in figure (4.21c). With slowly draining soils, the situation depicted in figure (4.21b) is critical with regard to stability of slopes [Lambe and Whitman, 1969].

The general example is considered to study the effect of some design parameters on the values of the drawdown rate and FOS for upstream side slope. Figure (4.22) shows the general example with same materials properties of the dam (which are the same as in previous analysis in this chapter).

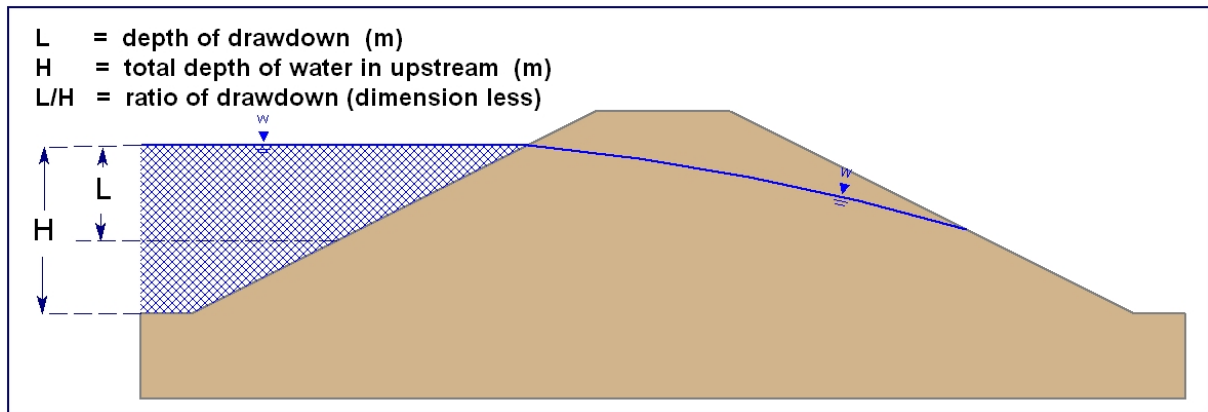


Figure (4.22) General example with same material properties used in previous analysis.

4.3.3 Parameters affecting drawdown condition.

4.3.3.1 Drawdown ratio (L/H)

In this part of analysis the stability of upstream side slope under rapid drawdown condition are studied for different ratio of drawdown, FOS for different ratio of (L/H), ratio of decreases FOS from steady state condition, shown in the table (4.8) below. The factor of safety for steady state condition before drawdown is 2.612.

Table (4.8) Factor of safety for upstream side slope under rapid drawdown condition for different depth of drawdown.

Depth of drawdown (m)	L/H Dimension less	FOS	$Rd = \frac{2.612 - FOS}{2.612} * 100$
1	0.1	2.122	18.76 %
2	0.2	1.990	23.81 %
3	0.3	1.872	28.33 %
4	0.4	1.768	32.31 %
5	0.5	1.679	35.72 %
6	0.6	1.604	38.60 %
7	0.7	1.545	40.85 %
8	0.8	1.500	42.57 %
9	0.9	1.468	43.80 %
10 (full depth)	1	1.457	44.22 %

Rd = ratio of decreases FOS from steady state (%)

The interesting results shown in Figure (4.23) indicate that the FOS decreases noticeably from the beginning of drawdown to ratio of drawdown equal to $2/3$. When $L/H = 2/3$ the value of $FOS = 1.545$ can be considered nearly minimum. The differences of FOS are not so big in the range of drawdown ratio lower than $2/3$, which may be considered almost constant. The value of FOS decreases approximately 40.85% at the drawdown ratio of $2/3$ and nearly 44.22% at the emptying.

It can be concluded that the critical degree of FOS during the rapid drawdown can be considered at the drawdown ratio of $2/3$, not until the emptying. An explanation of the critical FOS is due to the cohesive strength of the slope and trade-off between soil weight and soil shear strength as the drawdown ratio is varied. The fully submerged slope is more stable than the dry slope, as indicated by a higher FOS.

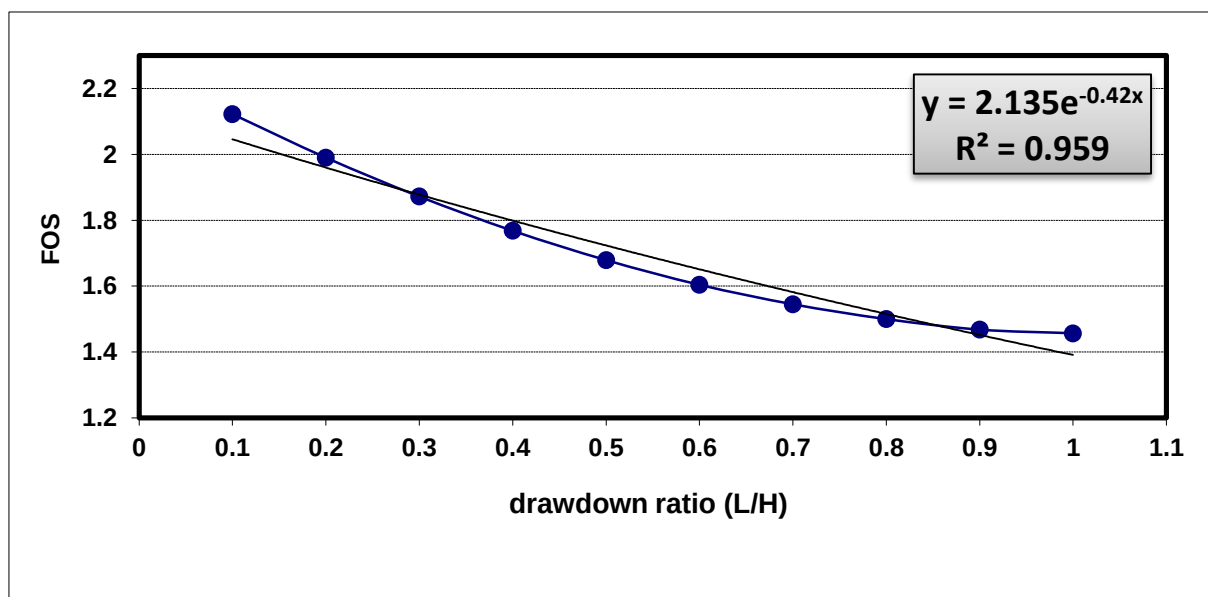


Figure (4.23) FOS for different drawdown ratio.

*Representative equation is given in item (4.4.3)

4.3.3.2 Pore water pressure along slip surface

Pore water pressure in the dam's body seems not to change very much at all drawdown ratios. Even when the reservoir is almost empty, pore water pressure in the dam's body is still high because the phreatic line does not drop. However, pore water pressure in the upstream toe decreases due to a disappearance of the overburden water load. The permeability and consolidation process of the filling materials also have effect on the change of pore water pressure in the upstream shell during drawdown.

In order to lower the phreatic line and pore water pressure as well as the hydrodynamic pressure in the upstream shell, drains in the upstream slope's base and at the intermediate level should be installed, if appropriate. These upstream drains are capable of draining the upstream slope and making the equipotential lines tend to become horizontal. They have a very significant effect on the stability of the upstream slope during drawdown [Tran, 2008].

The study of the changes in pore water pressure along slip surface for different drawdown ratios are illustrated in figures (4.24) through (4.28). The figures show the changes in pore water pressure along slip surface. The initial pore water pressure represents the steady state condition before drawdown and pore water pressure represents final pore water pressure after drawdown. These are defined as:

$$u = u^- + \Delta u \quad \dots (4.1)$$

where:

u = final pore water pressure.

u^- = initial pore water pressure before drawdown.

Δu = change in pore water pressure due to rapid drawdown.

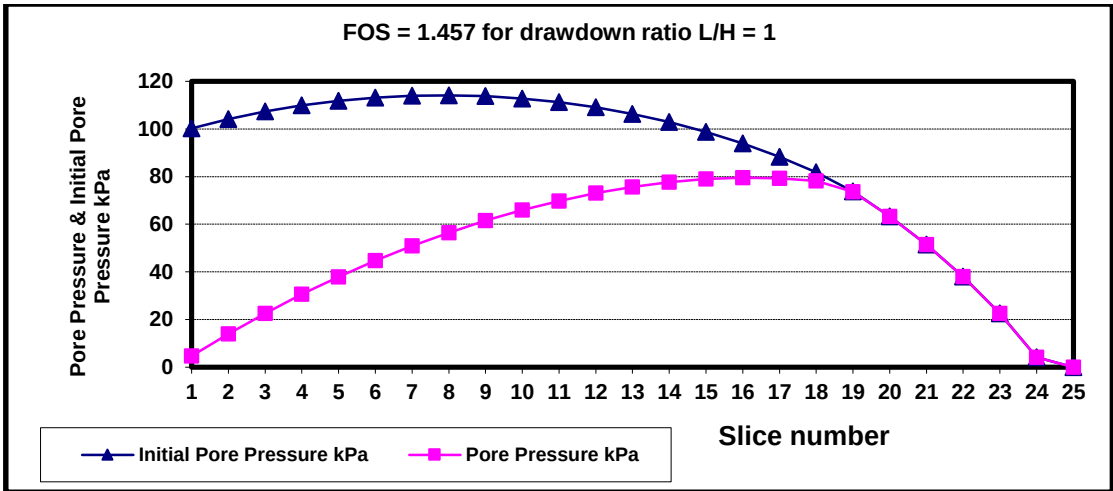


Figure (4.24) Pore water pressure and initial pore water pressure along slip surface for drawdown ratio = 1, FOS = 1.457.

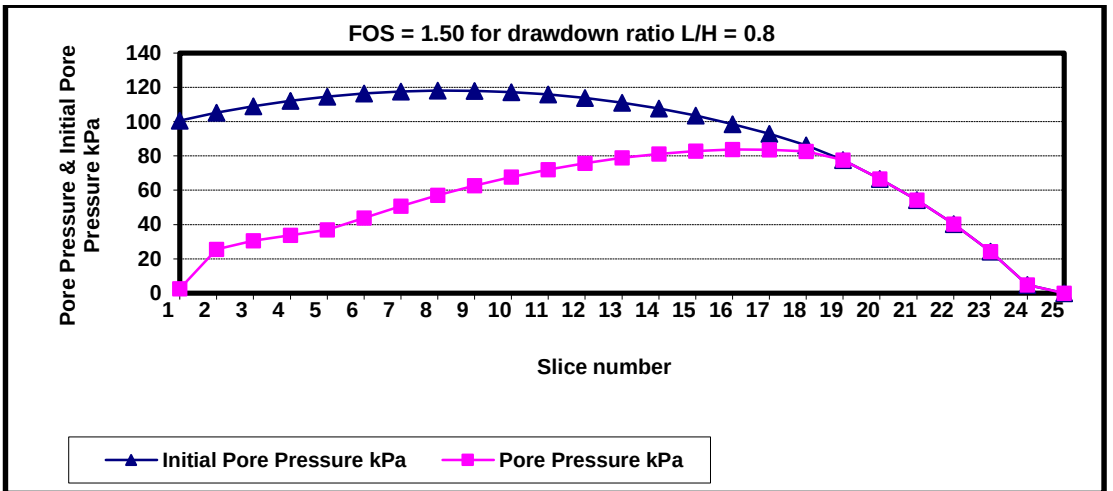


Figure (4.25) Pore water pressure and initial pore water pressure along slip surface for drawdown ratio = 0.8, FOS = 1.50.

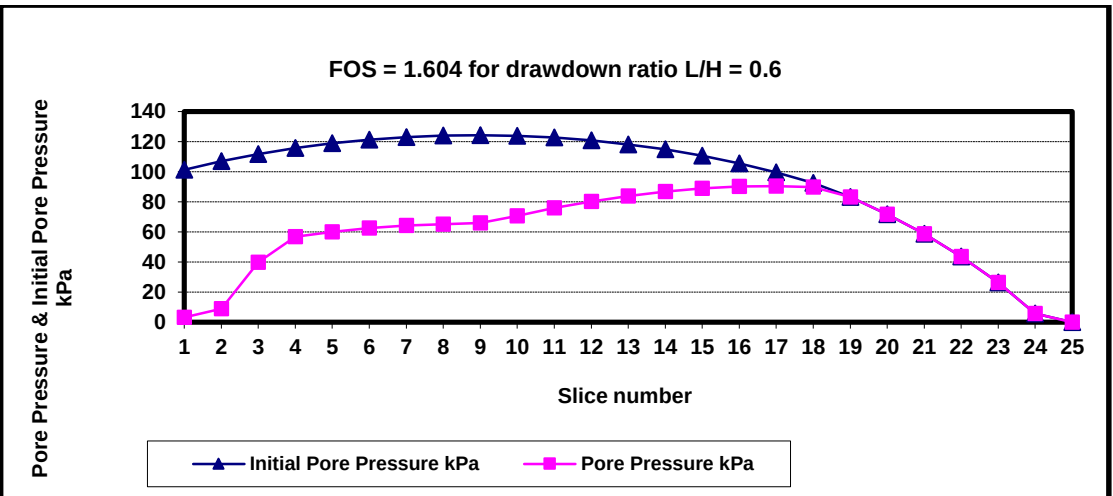


Figure (4.26) Pore water pressure and initial pore water pressure along slip surface for drawdown ratio = 0.6, FOS = 1.604.

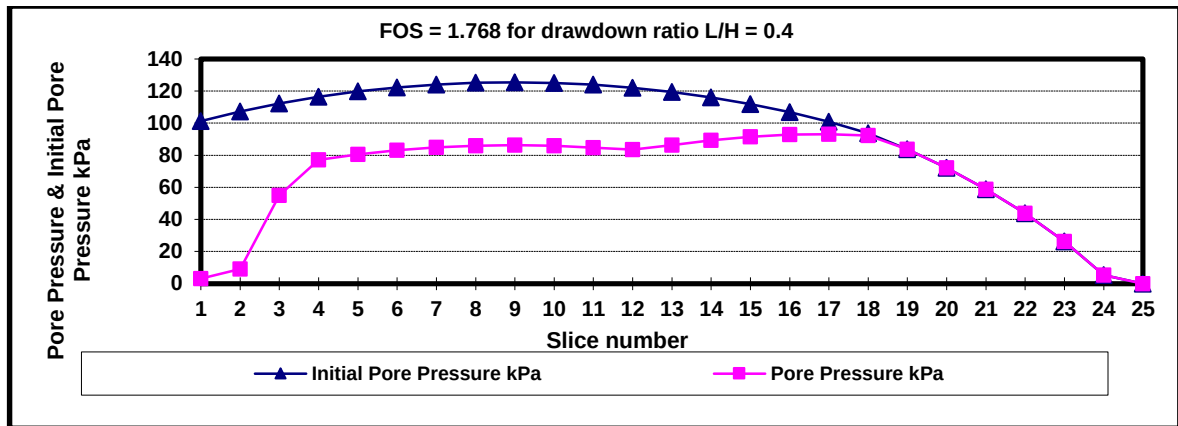


Figure (4.27) Pore water pressure and initial pore water pressure along slip surface for drawdown ratio = 0.4, FOS = 1.768.

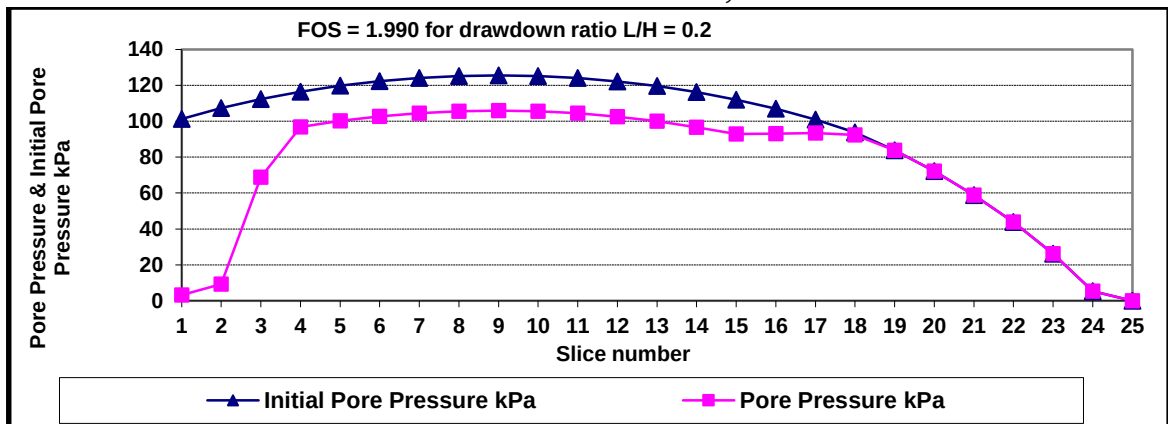


Figure (4.28) Pore water pressure and initial pore water pressure along slip surface for drawdown ratio = 0.2, FOS = 1.990.

From these figures, it can be seen that the pore water pressure decreases with increases of drawdown ratio (L/H). This increase in (L/H) will increase the negative pore water pressure in upstream side slope. In general, the pore water pressure has large effect on the stability of upstream side slope. It can be understood that the decrease in the effective vertical stresses due to (negative pore water pressure) is greater than the increase in the effective horizontal stresses in the dam's body, which means that the changes in effective vertical stresses are greater than the changes in effective horizontal stresses. The decrease in the major effective stresses is greater than the increase in the minor effective stresses. The decrease of the major effective stresses results in a decrease in the effective shear strength of soils in the upstream slope, leading to a decline of the upstream slope stability.

4.3.3.3 Effect of material properties on the drawdown condition

To study the effect of material properties on the stability of upstream side slope under rapid drawdown condition, three different ratios of drawdown L/H , (1, 0.6, and 0.2) are considered. This effect has been studied by considering the aforementioned (L/H) values whereas other parameters are kept constant (height of embankment = 12 m, slope angle (α) = $\tan^{-1}(1/2)$, soil unit weight (γ) = 19.2 kN/m³, angle of internal friction (ϕ) = 20° and total height of water in upstream (H) = 10 m. The general example is shown in figure (4.29).

To study the effect of unit weight of soil on the FOS for different values of drawdown ratio, several analyses have been made and presents in table (4.9).

Figure (4.30) shows the different values of soil unit weight for different values of drawdown ratio (0.2, 0.6, and 1). It can be observed that decreasing the value of unit weight of soil from (γ) = 19.2 KN/m³ to (γ) = 15 KN/m³ for different values of drawdown ratio L/H (0.2, 0.6, and 1), the factor of safety is increased between (15% - 7.66% - 5.83%) respectively for drawdown ratio, and can be observed that increasing the value of soil unit weight from (γ) = 19.2 KN/m³ to (γ) = 24 KN/m³ for different values of drawdown ratio L/H (0.2, 0.6, and 1), the factor of safety is decrease between (10.15% - 6.05% - 5.08%) respectively for drawdown ratio.

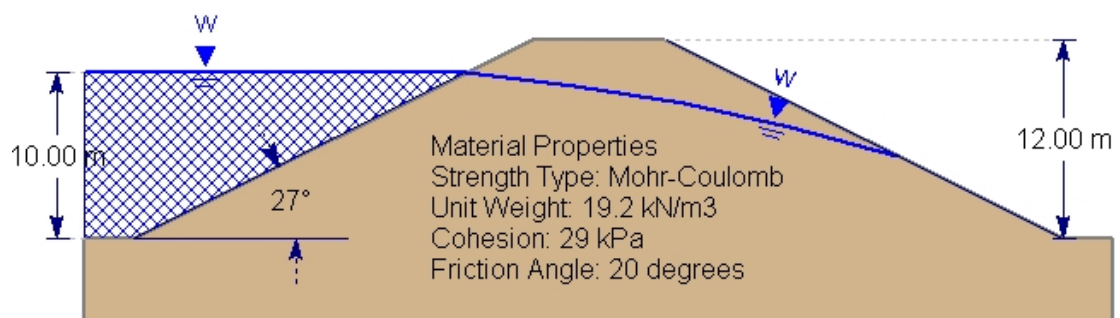


Figure (4.29) General example with material properties and total depth of water (H) = 10 m in upstream side slope.

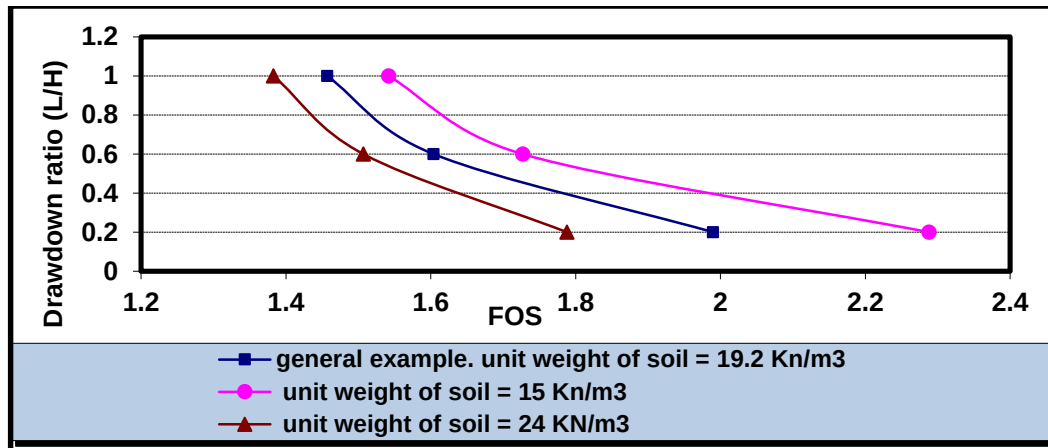


Figure (4.30) The effect of different values of unit weight of soil for different values of drawdown ratio on FOS.

Table (4.9) The input and output parameters for different values of soil unit weight, $C = 0$, $\phi^\circ = 20$, and seismic force coefficient = 0

Ex. No.	Input		Output
General Ex.	L/H Dimension less	γ kN/m ³	FOS by SLIDE Prog.
	0.2	19.2	1.990
	0.6		1.604
	1		1.457
1	0.2	15	2.288
	0.6		1.727
	1		1.542
2	0.2	24	1.788
	0.6		1.507
	1		1.383

To study the effect of cohesion of soil on the FOS for different values of drawdown ratio, several analyses have been made; table (4.10) shows input and output parameters.

Figure (4.31) shows the different values of cohesion for different values of drawdown ratio (0.2, 0.6, and 1). It can be observed that decreasing the value of cohesion of soil from $(c) = 29 \text{ kN/m}^2$ to $(c) = 20 \text{ kN/m}^3$ for different values of drawdown ratio L/H (0.2, 0.6, and 1), the factor of safety is decreased between

(18.3% - 19.07% - 20.04%) respectively for drawdown ratio, and can be observed that increasing the value of cohesion from $(c) = 29 \text{ KN/m}^2$ to $(c) = 40 \text{ KN/m}^2$ for different values of drawdown ratio L/H (0.2, 0.6, and 1), the factor of safety is increased between (21.65% - 22.44% - 23.47%) respectively for drawdown ratio.

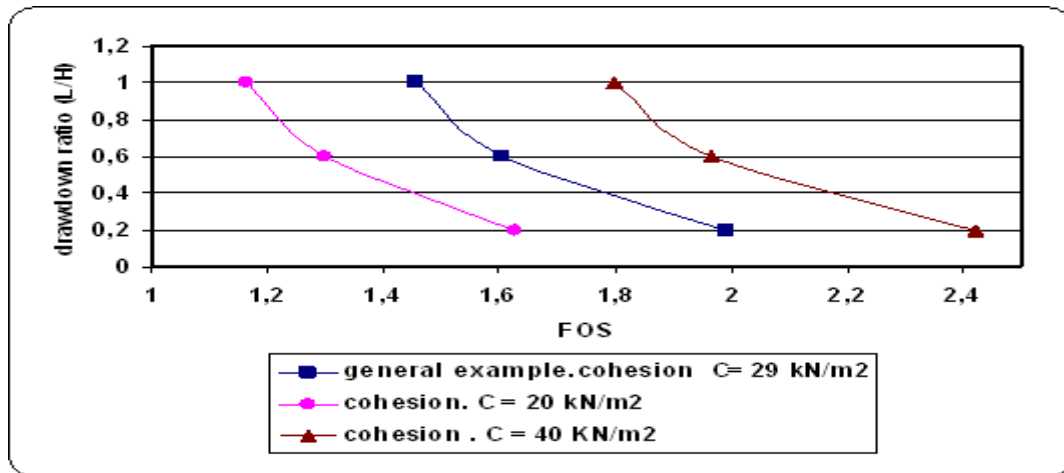


Figure (4.31) Effect of different values of cohesion of soil on FOS for different values of drawdown ratio.

Table (4.10) The input and output parameters for different values of cohesion of soil, $\gamma = 19.2$, $\phi^\circ = 20$, and seismic force coefficient = 0.

Ex. No.	Input		Output
General Ex.	L/H Dimension less	c kPa	FOS by SLIDE Prog.
	0.2	29	1.990
	0.6		1.604
	1		1.457
1	0.2	20	1.626
	0.6		1.298
	1		1.165
2	0.2	40	2.421
	0.6		1.964
	1		1.799

To study the effect of angle of internal friction of soil on FOS for different values of drawdown ratio, several analyses have been made; table (4.11) shows input and output parameters.

Figure (4.32) shows the different values of angle of internal friction for different values of drawdown ratio (0.2, 0.6, and 1). It can be observed that decreasing the value of cohesion of soil from $(\phi) = 20$ degree to $(\phi) = 15$ degree for different values of drawdown ratio L/H (0.2, 0.6, and 1), the factor of safety is decreased between (11.3% - 10.78% - 10.02%) respectively for drawdown ratio, and can be observed that increasing the value of internal friction from $(\phi) = 20$ degree to $(\phi) = 25$ degree for different values of drawdown ratio L/H (0.2, 0.6, and 1), the factor of safety is increase between (11.60% - 10.97% - 10.09%) respectively for drawdown ratio.

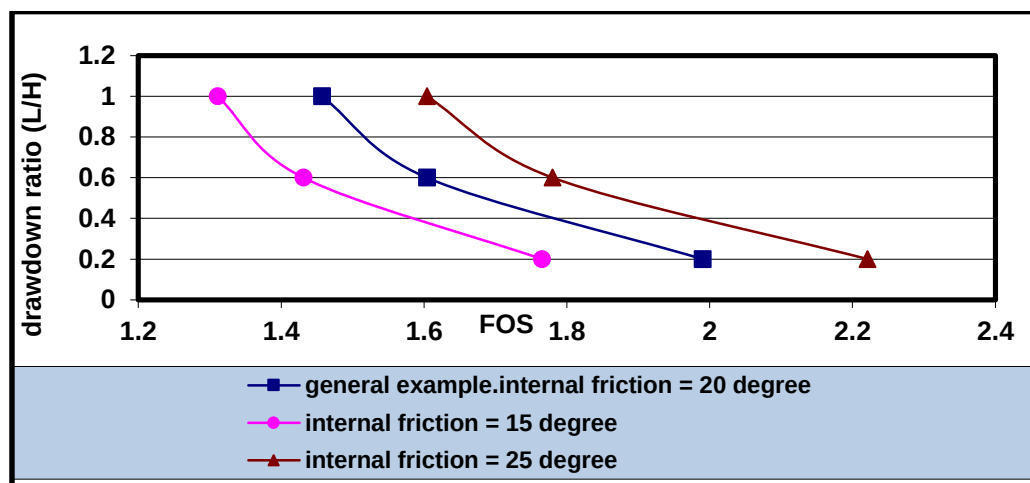


Figure (4.32) Effect of different values of internal friction of soil on FOS for different values of drawdown ratio.

Table (4.11) The input and output parameters for different values of internal friction of soil, $C = 29$, $\gamma = 19.2$, Seismic force coefficient = 0.

Ex. No.	Input	Output	
General Ex.	L/H Dimension less	ϕ°	FOS by SLIDE Prog.
	0.2	20	1.990
	0.6		1.604
	1		1.457
1	0.2	15	1.765
	0.6		1.431
	1		1.311
2	0.2	25	2.221
	0.6		1.780
	1		1.604

To study the effect the value of seismic load coefficients on FOS for different values of drawdown ratio, several analyses have been made; table (4.12) shows input and output parameters.

Figure (4.33) shows the different values of seismic load coefficients for different values of drawdown ratio (0.2, 0.6, and 1). It can be observed that increasing the value of seismic load coefficient from zero to 0.1 for different values of drawdown ratio L/H (0.2, 0.6, and 1), the factor of safety is decreased between (25.27% - 19.88% - 19.76%) respectively for drawdown ratio, and can be observed that increasing the value of seismic load coefficients from zero to 0.15 for different values of drawdown ratio L/H (0.2, 0.6, and 1), the factor of safety is decreased between (34% - 29.42% - 27.45%) respectively for drawdown ratio.

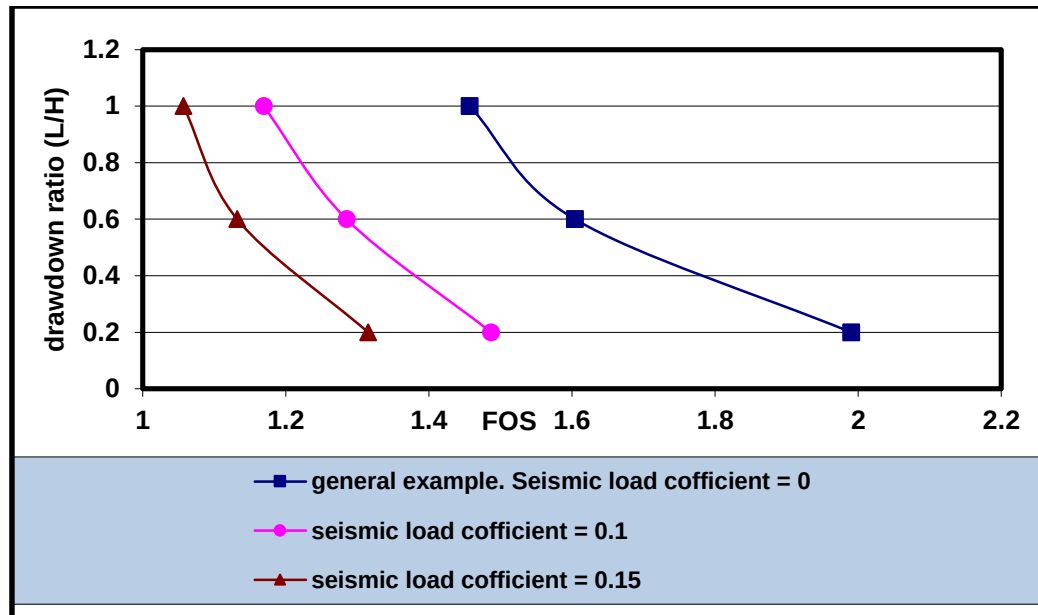


Figure (4.33) Effect of different values of seismic load coefficient on FOS for different values of drawdown ratio.

Table (4.12) The input and output parameters and FOS for different values of seismic load coefficients, $c = 29$, $\phi^0 = 20$, and $\gamma = 19.2$.

Ex. No.	Input		Output
General Ex.	L/H Dimension less	Seismic force coefficient	FOS by SLIDE Prog.
	0.2	0	1.990
	0.6		1.604
	1		1.457
1	0.2	0.1	1.487
	0.6		1.285
	1		1.169
2	0.2	0.15	1.315
	0.6		1.132
	1		1.057

4.4 Probabilistic and sensitivity analyses of the stability of earthen dams

In a traditional slope stability analysis, it is assumed that the values of all model input parameters are exactly known. For a given slip surface, a single value of safety factor is calculated. This type of analysis can be referred to as a **deterministic** analysis.

For most real world slope stability problems, the values of many input parameters are not very well known; therefore a **probabilistic** approach to the analysis of slope stability can be useful. In a probabilistic slope stability analysis, certain statistical distributions to model input parameters, such as material properties (cohesion, unit weight, angle of internal friction), support properties (reinforcement of side slope), loads (seismic load), and water table location, must be assigned.

By assigning a statistical distribution to one or more model input parameters, this allows to account for the degree of uncertainty in the value of the parameters. Input data samples are randomly generated based on the defined statistical distributions. A given slip surface may then have many different values of safety factor calculated. This results in a distribution of safety factors, from which a probability of failure for the slope can be calculated.

In a **sensitivity** analysis, individual input parameters are varied between minimum and maximum values. This result in a plot of safety factor versus the parameter value, which allows determining which input parameters have the greatest effect on safety factor, and which parameters do not.

4.4.1 Probabilistic analysis

In order to carry out a probabilistic analysis by using **SLIDE** computer program, at least one (or more) of model input parameters should be defined as random variables. Almost all model input parameters in **SLIDE** can be defined as random variables, for example, material properties, support properties, load magnitudes, and water table location.

In this analysis, the material properties (cohesion of soil, unit weight of soil, angle of internal friction of soil), loads (seismic force), and phreatic surface location, will be defined as a random variables. This is done by selecting a statistical distribution for each selected parameter (random variable), and entering the appropriate parameters for the distribution (standard deviation, minimum and maximum values, correlation coefficient).

There are several different statistical distributions available for defining random variables. In most cases, a normal distribution will be used. The normal (or Gaussian) distribution is the most common probability density function (PDF), and is generally used for probabilistic studies in geotechnical engineering [**Duncan and Stephen, 2005**].

In the present work the normal distribution will be used as a statistical distribution for defining random variables. For the normal distribution, 99.73 % of all samples will fall within 3 standard deviations of the mean value. This fact leads to Eq. (4.2), (the "Three Sigma Rule"), which allows to estimate the standard deviation for a normally distributed random variable.

$$\sigma = \frac{HCV - LCV}{6} \quad \dots \dots \dots (4.2)$$

where:

σ = estimated standard deviation

HCV = highest conceivable value of the random variable

LCV = lowest conceivable value of the random variable

Equation (4.2) is useful if actual data of the random variables is not available [**Duncan and Stephen, 2005**].

In this part of analysis, the general example mentioned in the beginning of this chapter and shown in figure (4.1a) will be used. All parameters defined as random variables are shown in table (4.13). Standard deviations of all random variables in table (4.13) are calculated by using equation (4.2).

Table (4.13) Input data for probabilistic analysis, $n = 1000$

Random variables	Distribution	Mean value	Minimum value	Maximum value	Standard deviation
Cohesion (KN/m ²)	Normal	29	0	58	9.66
Angle of internal friction (degree)	Normal	20	0	40	6.66
Unit weight(KN/m ³)	Normal	19.2	15	23.4	1.4
Seismic load coefficient	Normal	0.2	0	0.4	0.066
Phreatic surface	Normal	Represented graphically in figure (4.34) by drawing boundaries of minimum, maximum, and mean position of phreatic surface			

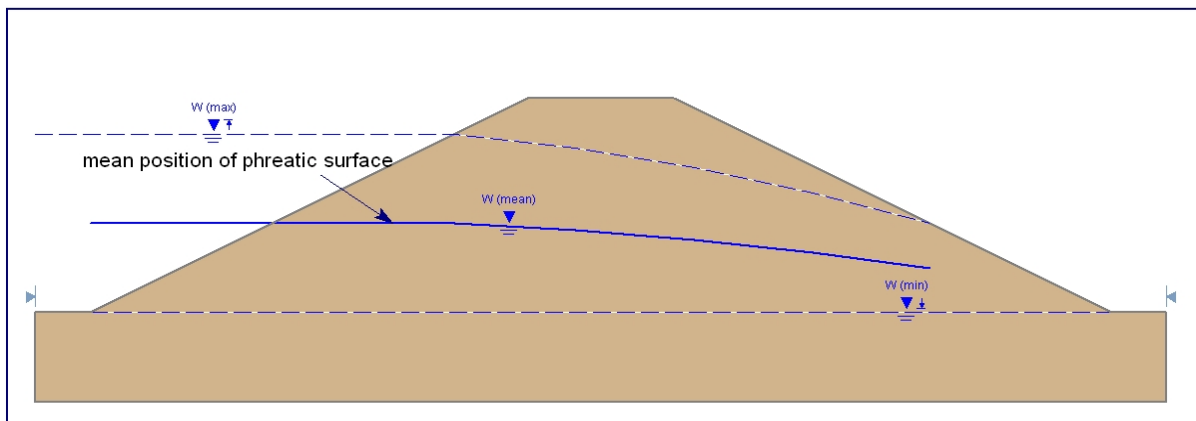


Figure (4.34) Boundaries of minimum, maximum, and mean positions of phreatic surface.

In addition to the parameters for the normal distribution (standard deviation, minimum and maximum values) mentioned in table (4.13), the correlation coefficient between the involved parameters is very important and has to be considered in the analysis.

It is known that cohesion and friction angle are related in a general way, such that materials with low friction angles tend to have high cohesion, and materials with low cohesion tend to have high friction angles. If a correlation coefficient is defined, this means that when the statistical samples are generated for cohesion and friction angle, the two variables will be correlated. The value of cohesion will be related to the value of friction angle, and vice versa. This

simulates the relationship between cohesion and friction angle which exists in real materials.

When no correlation has been defined, cohesion and friction angle are treated as completely independent variables.

In the present work, no correlation coefficient will be defined for seismic load coefficient and phreatic surface location; the two variables are treated as completely independent variables.

4.4.2 Results of probabilistic analysis

The primary results of the probabilistic analysis can be defined or displayed by mean FOS, probability of failure, reliability index (normal) and reliability index (lognormal), and correlation coefficient between random variables; these relations are defined hereinafter.

- **Mean FOS:** The Mean Safety Factor is the mean (average) safety factor, obtained from the Probabilistic Analysis. It is simply the average of the safety factors calculated for the Global Minimum slip surface.
- **Probability of Failure:** The probability of failure is defined as the number of analyses with safety factor less than 1, divided by the total number of samples, and shown in equation (4.3).

$$PF = \frac{NUM.FAILED}{NUM.TOTAL} * 100\% \quad \dots\dots\dots (4.3)$$

where:

PF = probability of failure.

$NUM.FAILED$ = Number of Analyses with Safety Factor < 1.

$NUM.TOTAL$ = Total number of analyses (samples).

- **Reliability Index:** The Reliability Index is an indication of the number of standard deviations which separate the Mean Safety Factor from the critical safety factor (= 1). The Reliability Index can be calculated assuming either a

Normal or Lognormal distribution of the safety factor results. If it is assumed that the safety factors are normally distributed, then Equation (4.4) is used to calculate the Reliability Index.

$$\beta = \frac{\mu_{FOS-1}}{\sigma_{FOS}} \quad \dots\dots\dots (4.4)$$

where:

β = reliability index.

μ_{FOS} = mean safety factor.

σ_{FOS} = standard deviation of safety factor.

If it is assumed that the safety factors are best fitted by a lognormal distribution, then Equation (4.5) is used to calculate the Reliability Index.

$$\beta_{LN} = \frac{\ln \left[\frac{\mu}{\sqrt{1 + Cv^2}} \right]}{\sqrt{\ln(1 + Cv^2)}} \quad \dots\dots\dots (4.5)$$

where:

β_{LN} = lognormal Reliability Index.

μ = the mean safety factor.

Cv = coefficient of variation of the FOS ($= \sigma / \mu$), [Duncan and Stephen, 2005].

- Correlation coefficient between random variables: The correlation coefficient indicates the degree of correlation between the two variables. A correlation coefficient close to 1 (or -1) indicates a high degree of correlation. A correlation coefficient close to zero indicates little or no correlation.

Results of probabilistic analysis are given in the table (4.14). Correlation coefficient between shear strength parameters estimated close to (-0.5) according to natural of relationship between cohesion of soil and angle of internal friction.

Table (4.14) Results of the probabilistic analysis, correlation coefficient between shear strength parameters estimated close to (-0.5).

Probabilistic analysis results	
Parameters	value
Factor of Safety, mean	1.74
Factor of Safety, standard deviation	0.39
Factor of Safety, minimum	0.64
Factor of Safety, maximum	3.16
Probability of Failure	2.500 % (25failed surfaces / 1000 valid surfaces)
Reliability index (assuming normal distribution)	1.86
Reliability index (assuming lognormal distribution)	2.35

Beside the results of probabilistic analysis in table (4.14), other results are presented graphically as follows. The figures display also the critical data points (with FOS < 1).

Figure (4.35) shows the results and location of the most critical slip surface for deterministic and probabilistic analyses.

Figure (4.36) shows the relationship between the cohesion of soil and Safety Factor when cohesion of soil is defined as a random variable.

Figure (4.37) shows the relationship between the friction angle of soil and Safety Factor when friction angle of soil is defined as a random variable.

Figure (4.38) shows the relationship between the location of phreatic surface and Safety Factor when the former is defined as a random variable.

Figure (4.39) shows the relationship between the value of seismic load coefficient and Safety Factor when the former is defined as a random variables.

Figure (4.40) shows the relationship between the unit weight of soil and Safety Factor when unit weight is defined as random variables.

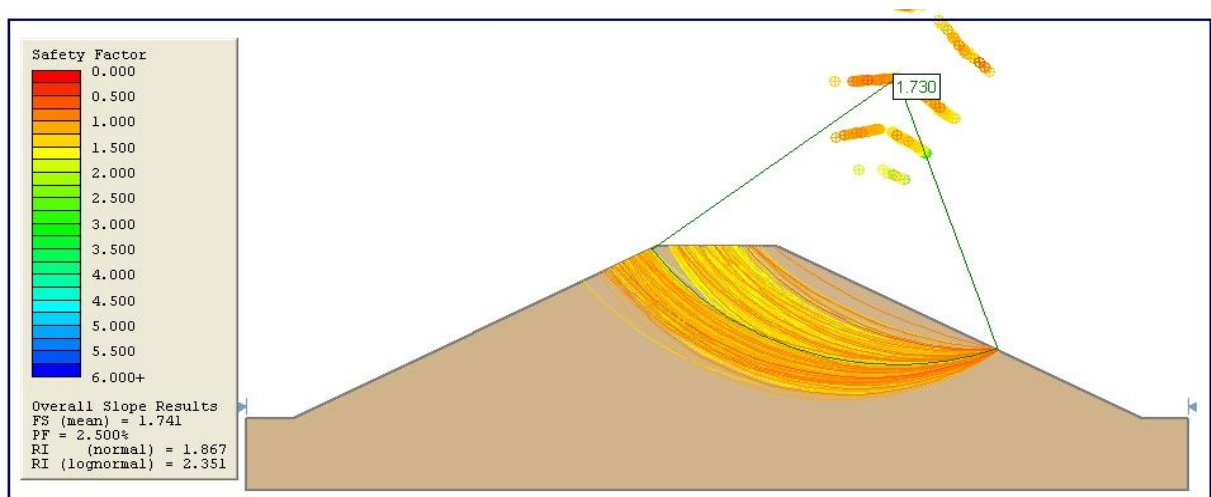


Figure (4.35) Results and location of the most critical slip surface for deterministic and probabilistic analyses.

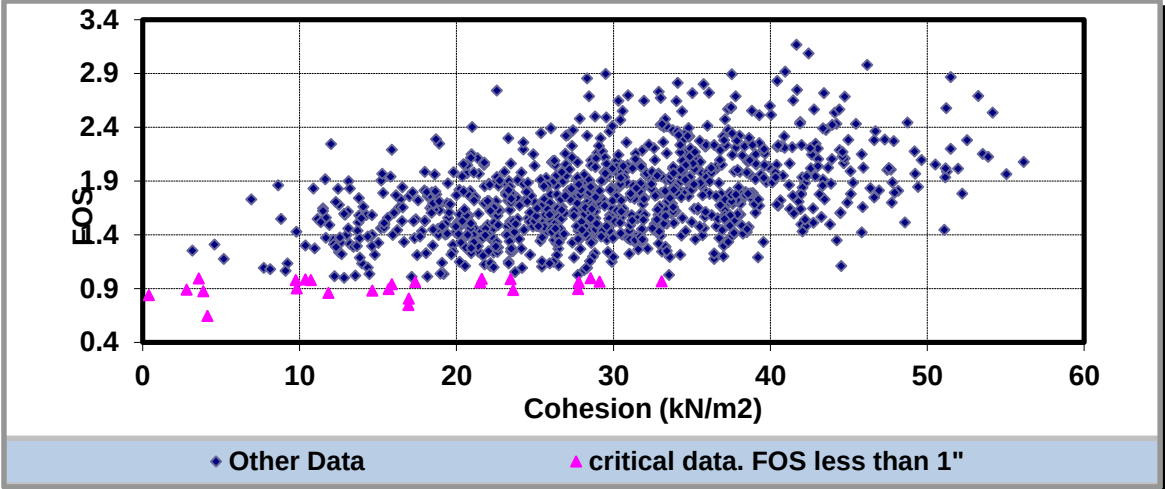


Figure (4.36) Relationship between cohesion of soil and Safety Factor for probabilistic analysis approach.

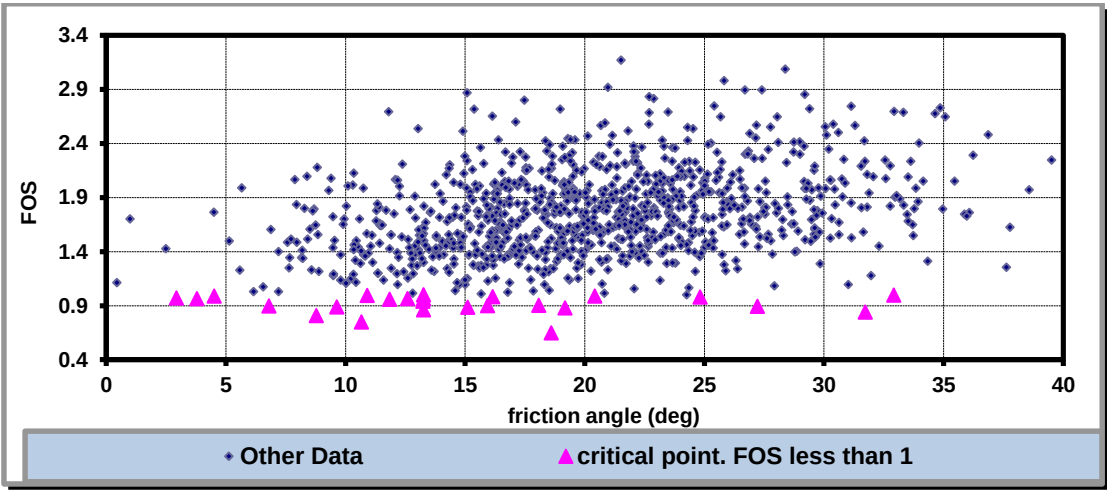


Figure (4.37) Relationship between friction angle of soil and Safety Factor for probabilistic analysis approach.

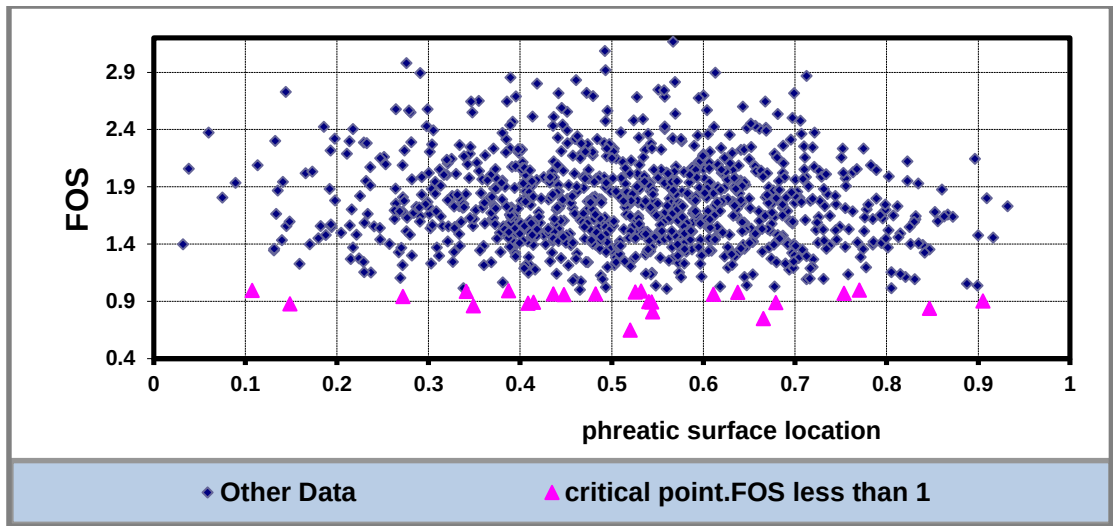


Figure (4.38) Relationship between phreatic surface location and Safety Factor for probabilistic analysis approach

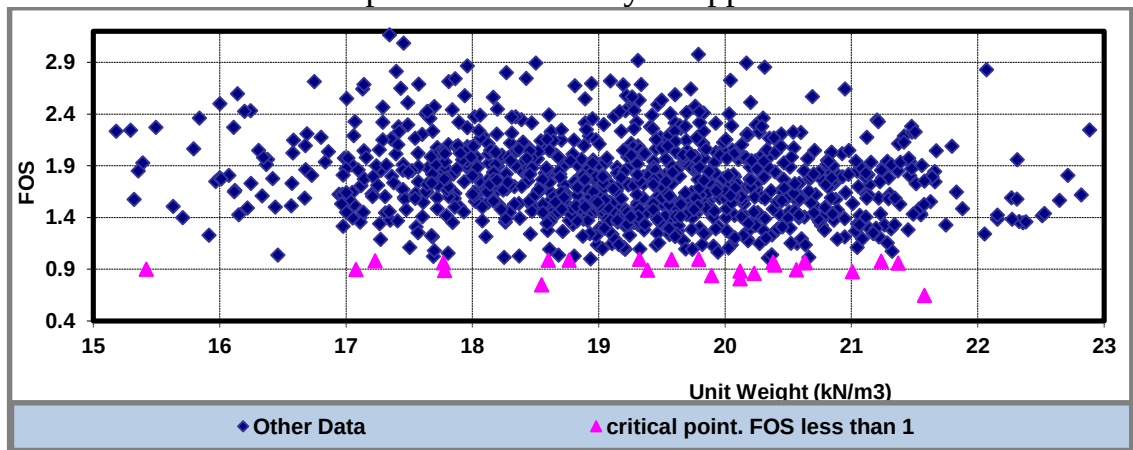


Figure (4.39) Relationship between unit weight of soil and Safety Factor for probabilistic analysis approach.

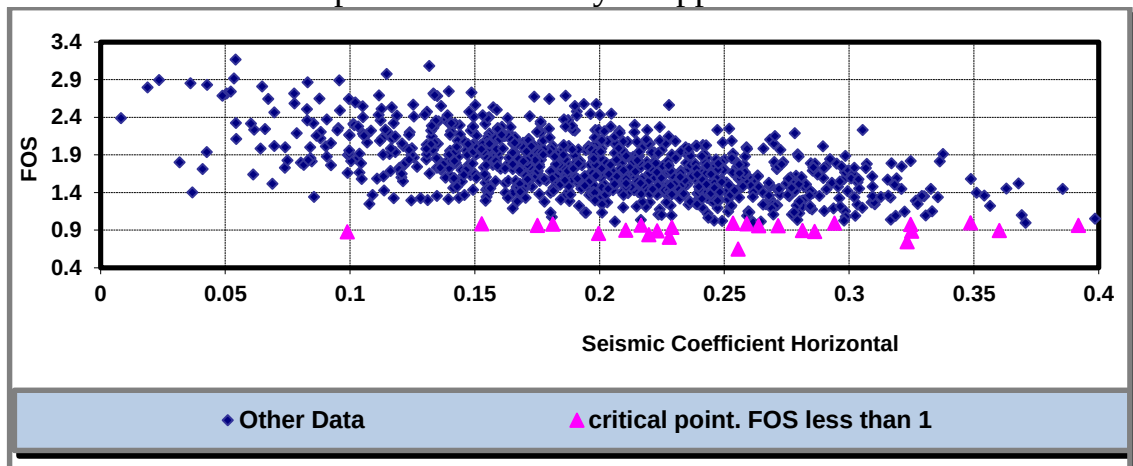


Figure (4.40) Relationship between seismic coefficient and Safety Factor for probabilistic analysis approach

The best-fit curves of the relationships of FOS with the five aforementioned parameters, when considering them as random variables, are shown in figure (4.41).

From this figure it can be observed that the factor of safety is more sensitive to the friction angle, cohesion, and seismic load coefficient (steep curves), and least sensitive to the unit weight and phreatic surface location (curves are almost flat). Percent of range = 0 represents the minimum value of each variable, and percent of range = 100 represents the maximum value of each variable.

It is clear that all curves intersect at percent of range = 50%. Percent of range = 50% always represents the mean value of each variable.

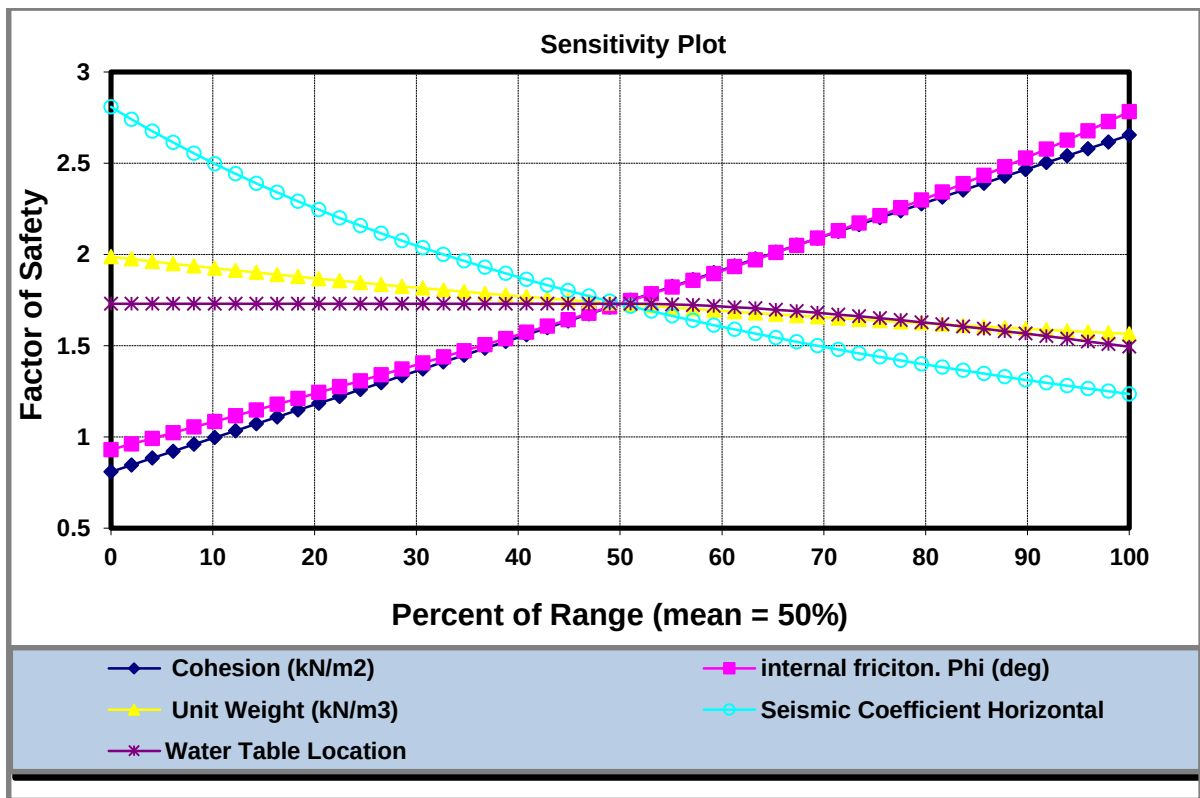


Figure (4.41) Sensitivity plots of parameters that affect FOS.

Finally, cohesion of soil and angle of internal friction are very important parameters to determine factor of safety of earthfill dams and any geotechnical engineering problem; these parameters in most times gave a guide-line to make a good design criteria. So, correlation coefficient must be investigated between cohesion and internal friction of soil. In reality, the cohesion and friction angle of Mohr-Coulomb materials are generally correlated such that materials with low cohesion often have high friction angles, and vice versa. In general, it is recommended that a correlation coefficient is defined between cohesion and friction angle, for a Mohr-Coulomb material. This will generate values of cohesion and friction angle which are more likely to occur in the field. In the current work, a negative correlation coefficient between the two variables is defined ; a negative correlation coefficient simply means that when one variable increases, the other is likely to decrease, and vice versa.

Based on the generated data points, the results are plotted in figure (4.42). The probability of failure of an existing earth dam depends on many factors; one of these factors is the correlation coefficient between cohesion of soil and angle of internal friction. It is interesting to note that the probability of failure decreases significantly, as the correlation between cohesion and friction angle increases (i.e., closer to -1). This implies that the use of a correlation coefficient, and the generation of more realistic combinations of cohesion and internal friction, tends to decrease the calculated probability of failure.

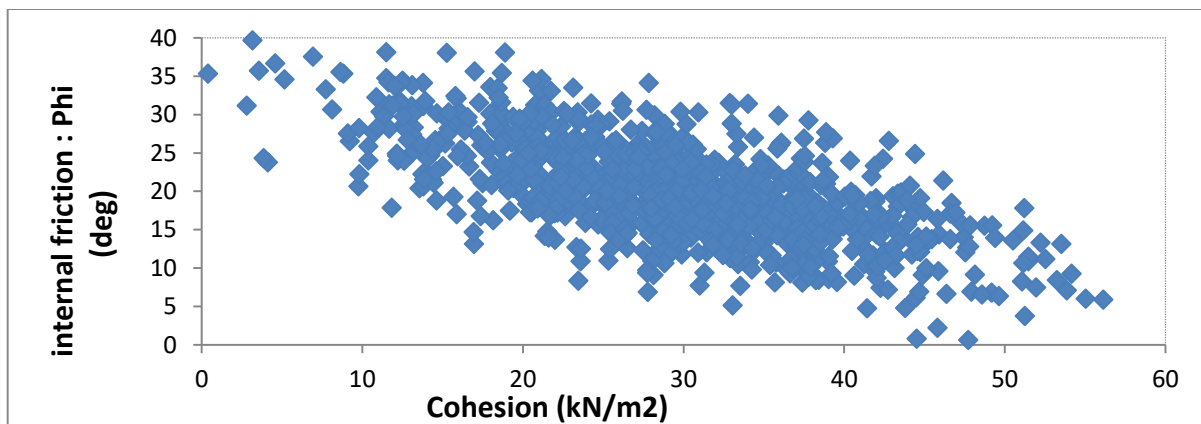


Figure (4.42) Correlation coefficient between cohesion and internal friction.

4.4.3 Proposed Equations for a Minimum Factor of Safety:

After performing the probabilistic and sensitivity analyses of parameters that affect FOS, Figure (4.41) contains the variables that have been taken to suggest proposed equations to estimate the minimum factor of safety.

From variations of the values of design parameters of soil and values of factor of safety, **SLIDE** V5.0 is used to estimate or propose more practical equations for the relationships between these parameters.

Equation (4.6) represents the relationship between cohesion of soil and minimum factor of safety obtained in present work.

$$\text{FOS}_{\min} = 0.8069 + 0.0319 C \quad [0 \leq C \leq 58] \quad \text{---} \quad (4.6)$$

$$R^2 = 1$$

where:

C = cohesion of soil, kpa.

$\text{FOS}_{\min}$ = minimum factor of safety.

R = regression

Equation (4.7) represents the relationship between angle of internal friction of soil and minimum factor of safety.

$$\text{FOS}_{\min} = 0.8646 + 0.0454 \phi \quad [0 \leq \phi \leq 40] \quad \text{---} \quad (4.7)$$

$$R^2 = 0.99$$

where:

ϕ = angle of internal friction of soil, degree

$\text{FOS}_{\min}$ = minimum factor of safety.

Equation (4.8) represents the relationship between unit weight of soil and minimum factor of safety.

$$\text{FOS min} = 2.9983 e^{-0.0283\gamma} \quad [15 \leq \gamma \leq 23.4] \dots\dots\dots (4.8)$$

$$R^2 = 0.99$$

where:

γ = unit weight of soil, KN/m³

FOS min = minimum factor of safety.

Equation (4.9) represents the relationship between seismic load coefficient and minimum factor of safety.

$$\text{FOS min} = 2.6563 e^{-2.0189a} \quad [0 \leq a \leq 0.4] \dots\dots\dots (4.9)$$

$$R^2 = 0.99$$

where:

a = seismic load coefficient.

FOS min = minimum factor of safety.

Equation (4.9) represents the relationship between drawdown ratio (L/H) and minimum factor of safety, see figure (4.29)

$$\text{FOS min} = 2.135 e^{-0.42(L/H)} \quad [0 \leq (L/H) \leq 1] \dots\dots\dots (4.10)$$

$$R = 0.96$$

where:

(L/H) = drawdown ratio, dimensionless

FOS min = minimum factor of safety.

4.4.4 Applicability of the Proposed Equations

The following example describes the applications of the aforementioned proposed equations.

Given A side slope with ($H=12\text{m}$; $\alpha=26.565^\circ$) and a cohesive soil having ($\phi = 20^\circ$; $c = 29 \text{ kPa}$). The phreatic surface position is as shown in figure (4.34), and ($\gamma = 19.2 \text{ kN/m}^3$). Find the minimum factor of safety.

In case of solved this example by using **SLIDE** program, the minimum factor of safety for this example is about 1.730.

However, the minimum safety factor for the given example it can be calculated by the aforementioned proposed equations and as follows:

- By equation (4.6): $\text{FOS}_{\min} = 1.732$.
- Compared with the value obtained by using the program: the relative difference is (+ 0.11 %).
- By equation (4.7) : $\text{FOS}_{\min} = 1.772$.
- Compared with that obtained by using the program: the relative difference is (+ 2.46)
- By equation (4.8) : $\text{FOS}_{\min} = 1.742$.
- The relative difference between this value and that obtained by using the program is (+ 0.7%).
- By equation (4.9): $\text{FOS}_{\min} = 1.774$.
- Compared with the minimum factor of safety obtained by using the program: the relative difference is (+ 2.6%).

The comparisons stated hereinbefore justify the use of the proposed equations in calculating minimum factor of safety for any values of the involved parameters (cohesion, unit weight, friction angle, seismic load coefficient, and drawdown ratio).

Chapter Five

Stability Analysis of Zoned Earth dams

5.1 Stability Analysis

After achieving the verification of the computer programming (**SLIDE V.5.0**) that has been presented in this study, the program is approved to be suitable for use to analyze seepage through earthen dam and stability analysis of side slope.

Slope stability analysis of high earth and rockfill dams is very important to ascertain the stability of the structure. As the geotechnical and hydraulic properties of the material in different zones of the earth and rockfill dam differ to a great extent, the stability of both upstream slope and downstream slope under static and pseudo static conditions under different conditions have to be analyzed. The developments in geotechnical engineering and the advancement in the earth moving machineries enabled the construction of high earth and rockfill dams to store huge quantities of water for irrigation, power generation and other public uses. The earth and rockfill dams are preferred mostly for many construction sites in comparison with the concrete or masonry dams because of their economic considerations as they are constructed using the cheap locally available materials. However, the lack of appropriate construction methodology such as compaction equipment, maintenance of appropriate density, moisture content, etc. may lead to the construction of an unstable structure. Therefore, the construction with the natural materials needs good quality control. Zoned dams, specifically earth and rockfill dams, have become more popular as they allow the use of available soils of different gradations in appropriate zones of the dam cross section. Moreover, the flexibility of the materials provides excellent seismic stability to the structure. An earth and rockfill dam should be stable under various conditions. The stability of an earth slope subjected to changing water levels depends on many factors. The dams located in active seismic prone

areas need to be analyzed extensively for the safety of the structure, [**Rajesh and Gupta, 2008**].

It is always required to answer the critical question to what extent the side slopes of an existing earth dam are safe. In this respect, the stability of side slopes of one important Iraqi dam will be re-analyzed in this chapter.

The Haditha Dam is one of the important dams in Iraq, which had been designed by international consulting corporation. The designer analyzed the stability of the side slope of Haditha Dam under different loading conditions. The values of factor of safety obtained by the designer are moot. The main reason behind that is that very conventional methods were utilized in that analysis.

The aim of this chapter is to analyze seepage through a zoned earth dam aiming at investigating the effects of different loading conditions and different water condition in the reservoir on the overall stability of the dam (deterministic and probabilistic approaches). The same loading conditions that had been considered by the designer will be studied in this analysis and the results of the computer programs (**SLIDE V.5.0**) will be compared with those obtained by the designer.

5.2 Zoned Earth Dams (Haditha Dam as a case study)

5.2.1 General Description

The Haditha Dam is a multi-purpose structure designed to control the Euphrates River flow in the interests of irrigation, electric power generation and for partial accumulation of extreme Euphrates River inflows into Haditha reservoir.

It is an earthen dam with a central core located on the Euphrates River about 7 km upstream the town of Haditha in the middle west of Iraq. The total length of the dam is about (9km) and its maximum height is about (55m). The shell, which is composed mainly of sand-gravel and / or rock muck, and the central

dolomite core, are constructed by hydraulic filling. Beside the dolomite as an impervious core, an asphaltic concrete diaphragm was constructed inside the central core as an anti-seepage element. This vertical diaphragm is located (10 m) upstream the dam axis, and has a thickness of (1m) up to elevation (125.00). Above that level, the diaphragm thickness is reduced to only (0.5m). Figure (5.1) shows a typical cross section of the central part of the dam at the channel portion where its constructed height is maximum [Al-Jorany, 1996].

Table (5.1) displays the most important properties of the different materials composing of Haditha Dam.

Table (5.1) Material properties for Haditha Dam [Al-Jorany, 1996]

Material type	Parameter	Value
Mealy dolomite (Foundation)	Horizontal permeability, k_x cm/sec	1.15×10^{-5}
	Vertical permeability, k_y cm/sec	1.15×10^{-5}
	Total unit weight, γ_t kN/m ³	22
	Saturated unit weight, γ_s kN/m ²	22
	Cohesion, C kN/m ²	20-40
	Angle of internal friction ϕ	24-28
Clay (Foundation)	Horizontal permeability, k_x cm/sec	2.31×10^{-6}
	Vertical permeability, k_y cm/sec	2.31×10^{-6}
	Total unit weight, γ_t kN/m ³	18.7
	Saturated unit weight, γ_s kN/m ²	19.0
	Cohesion, C kN/m ²	30-50
	Angle of internal friction ϕ	14-16
Limestone (Foundation)	Horizontal permeability, k_x cm/sec	1.15×10^{-6}
	Vertical permeability, k_y cm/sec	1.15×10^{-6}
	Total unit weight, γ_t kN/m ³	22
	Saturated unit weight, γ_s kN/m ²	22
	Cohesion, C kN/m ²	100-200
	Angle of internal friction ϕ	30-35
Rock muck sandy gravel (shell)	Horizontal permeability, k_x cm/sec	2.31×10^{-4}
	Vertical permeability, k_y cm/sec	2.31×10^{-4}

Material type	Parameter	Value
Rock muck sandy gravel (shell)	Total unit weight, γ_t kN/m ³	17
	Saturated unit weight, γ_s kN/m ²	20.3
	Cohesion, C kN/m ²	0
	Angle of internal friction ϕ	27-30
Dolomite (Core)	Horizontal permeability, k_x cm/sec	1.15×10^{-6}
	Vertical permeability, k_y cm/sec	1.15×10^{-6}
	Total unit weight, γ_t kN/m ³	20.3
	Saturated unit weight, γ_s kN/m ²	21.4
	Cohesion, C kN/m ²	0
	Angle of internal friction ϕ	30-33

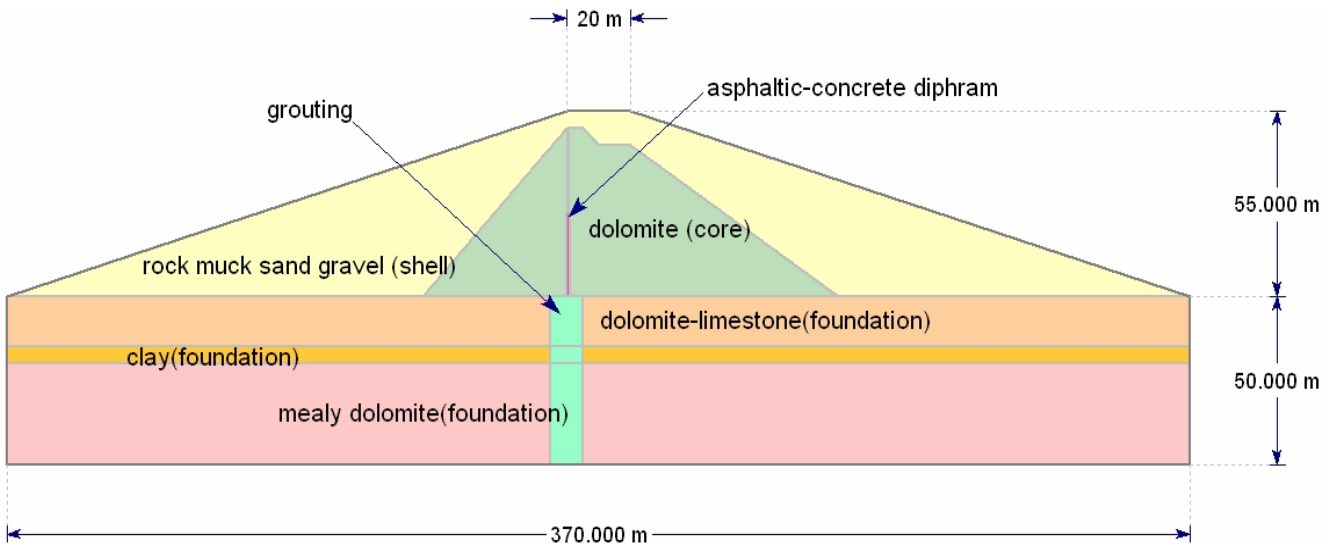
5.3 Analysis of the Haditha Dam under different load conditions

Stability analysis of zoned earthen dams is more sensitive analysis than that of another big building structure because the great mass of the dam has a complex design structure, sensitive to water condition, and load condition.

In the present work the stability analysis of zoned earth dam (Haditha Dam) is divided into two parts and as follows:

- Seepage analysis (pore water pressure distributions and position of phreatic surface) under different water conditions.
- Overall stability (stability of upstream and downstream side slopes) under different loads conditions.

Seepage analysis results are taken to be boundary conditions in overall stability analysis. The finite element mesh used in this analysis is as shown in figure (5.2); quadratics four node elements are used to describe the domains. The mesh contains 2082 element and 2166 node; the number of elements is selected from mesh generation as shown in figure (5.3).



NUMBER OF ELEMENT = 2082
NUMBER OF NODES = 2166

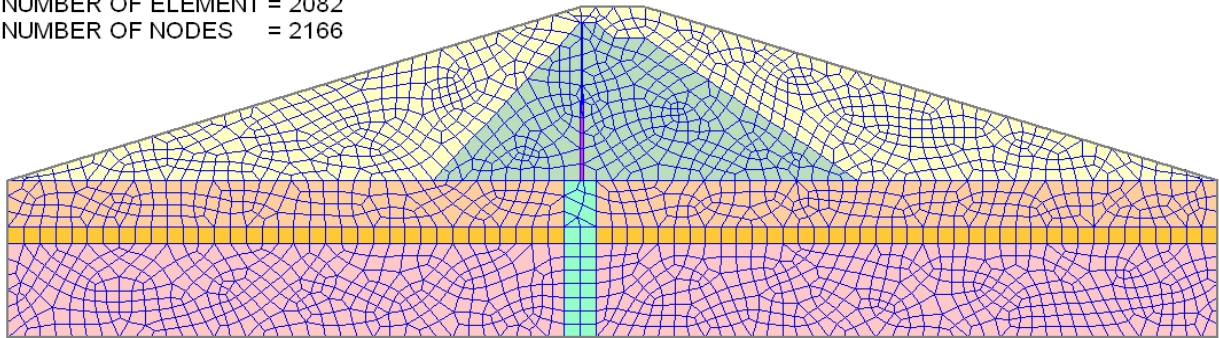


Figure (5.2) Finite Element Mesh for the Haditha Dam

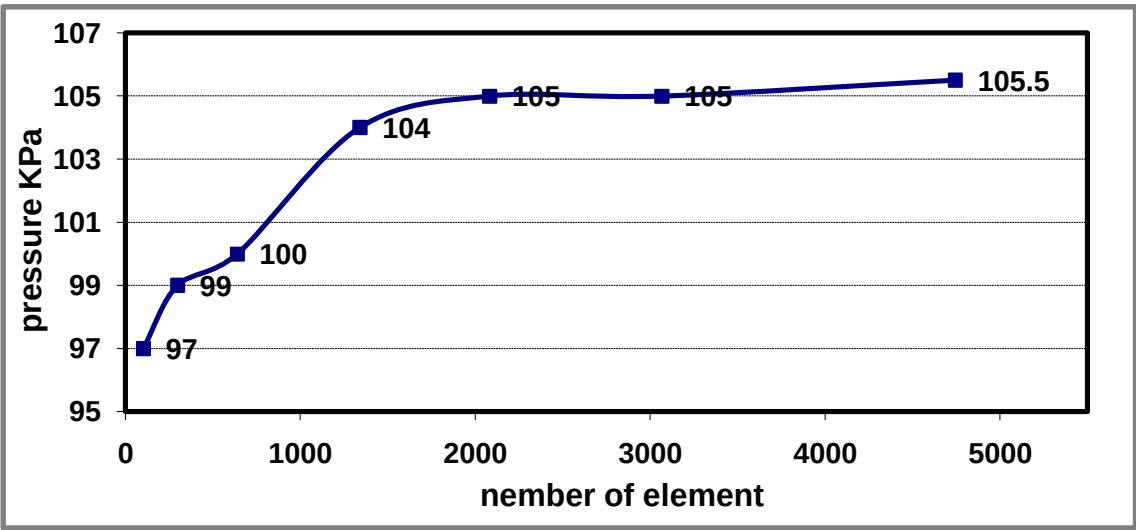


Figure (5.3) Mesh Generations

5.4 Results and Discussion

5.4.1 Seepage Analysis Results

Seepage through Haditha Dam under different water conditions is presented as follows:

- Normal reservoir level (143.00 m.a.s.l.)
- Maximum reservoir level (152.20 m.a.s.l.)
- Minimum reservoir level (112.00 m.a.s.l.)

5.4.1.1 Normal Reservoir Level:

The water level in the upstream for the case of normal operation is considered as 143.00 m.a.s.l. Figure (5.4) shows computed location of free surface. The figure indicates that free surface is constructed from several discontinuous surfaces because the rate of water movement is different in each material within the dam body. Furthermore, the free surface elevation drops from (141.8m) to (129.8m) at the boundary of diaphragm, which reflects the efficiency of the diaphragm as anti seepage device. Figures (5.5) and (5.6) show contour maps for total head and pressure head distributions throughout the dam body at normal operation conditions.

5.4.1.2 Maximum reservoir level

In this case, the water level in the upstream is considered maximum (i.e. 152.20m.a.s.l). Figure (5.7) shows computed locations of the free surface in the shell and core for this water condition. The head and pressure distributions are demonstrated in Figures (5.8) and (5.9), respectively. The free surface elevation drops from (151.2m) to (138.2m) at the boundary of diaphragm, which reflects the efficiency of the diaphragm as anti seepage device.

5.4.1.3 Minimum reservoir level

The water level is considered as 112.00 m.a.s.l. in the upstream side. The computed location of free surface corresponding to such a water condition is shown in Figure (5.10). It can be noticed that a free surface is relatively horizontal in the shell and extension of the free surface in core with a small drop near the junction of the core and diaphragm. Figures (5.11) and (5.12) display the total head and pressure head distributions due to the minimum water level in reservoir.

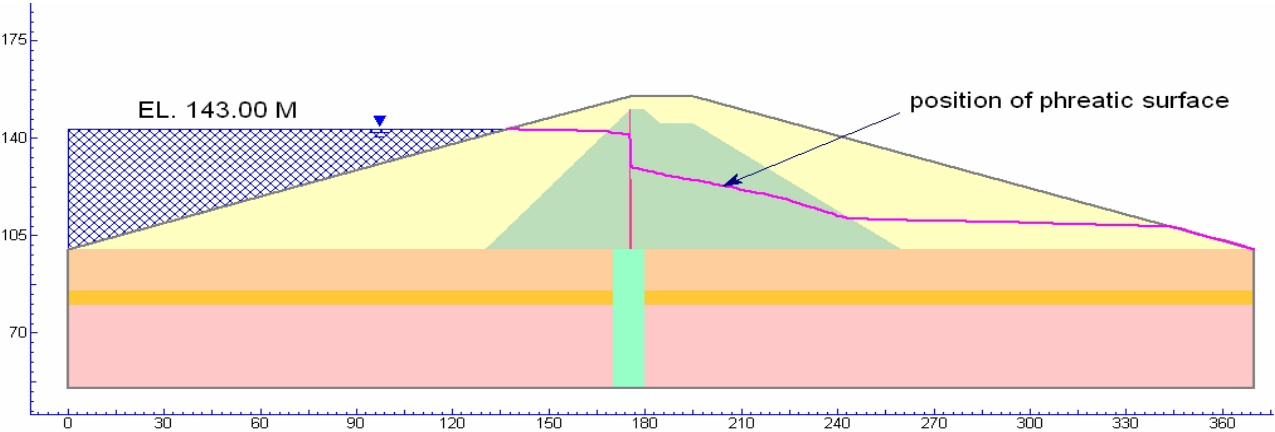


Figure (5.4) Computed Location of Phreatic Surface for Normal Water Level (143.00 m)

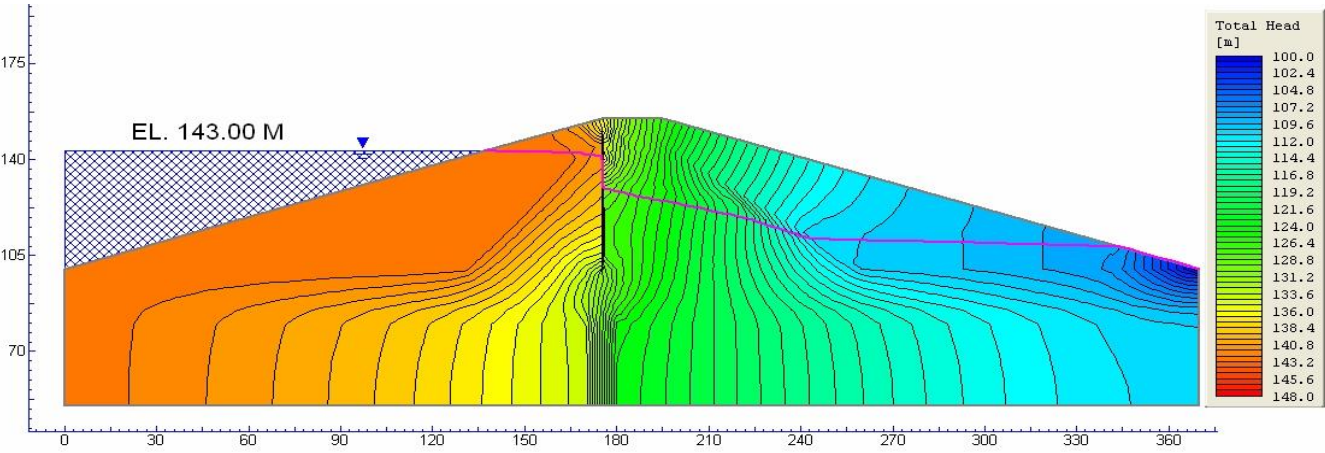


Figure (5.5) Computed Contour Map of Total Head Distribution for Normal Water Level (143.00 m)

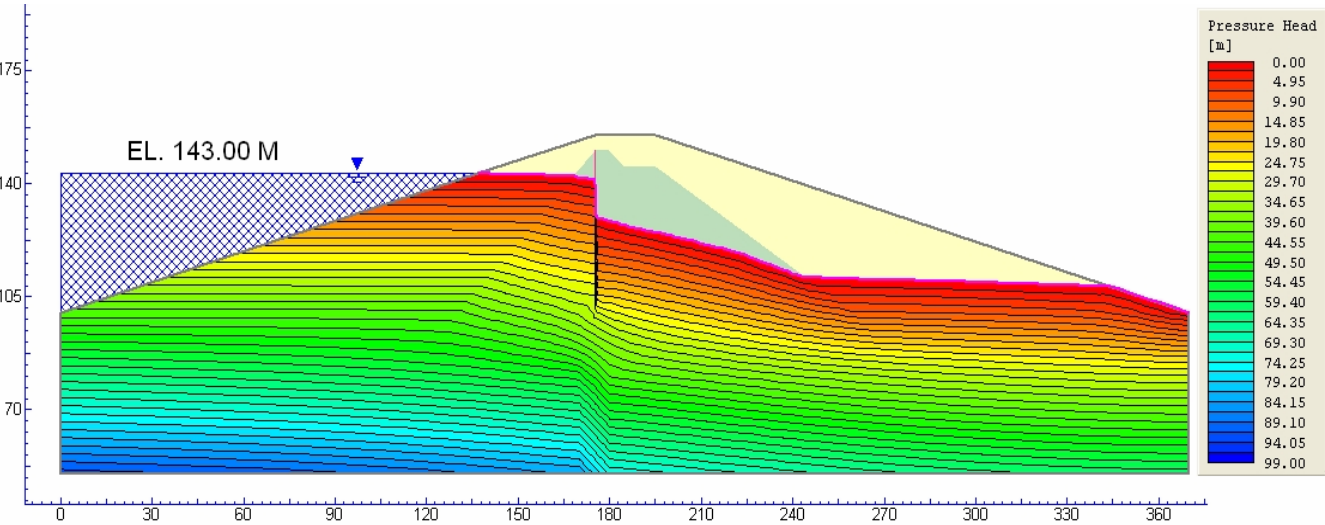


Figure (5.6) Computed Contour Map of Pressure Head Distribution for Normal Water Level (143.00 m)

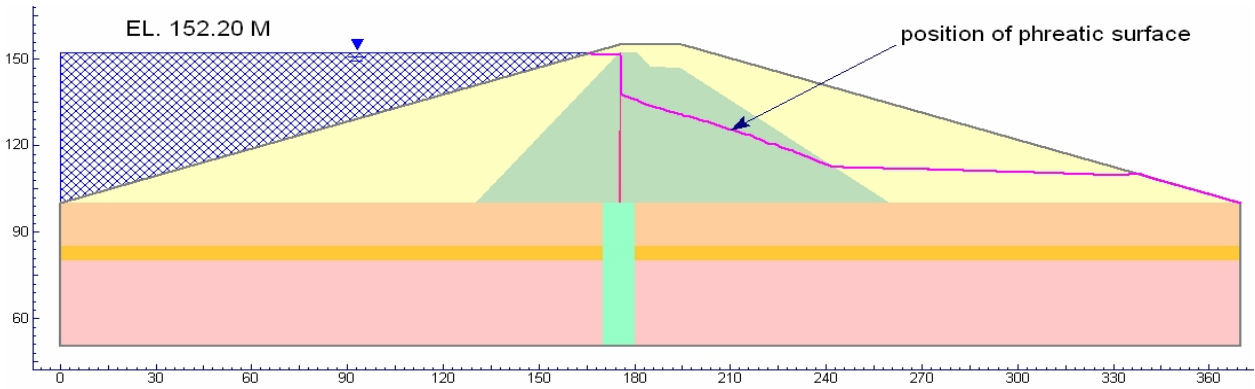


Figure (5.7) Computed Location of Phreatic Surface for Maximum Water Level (152.20 m)

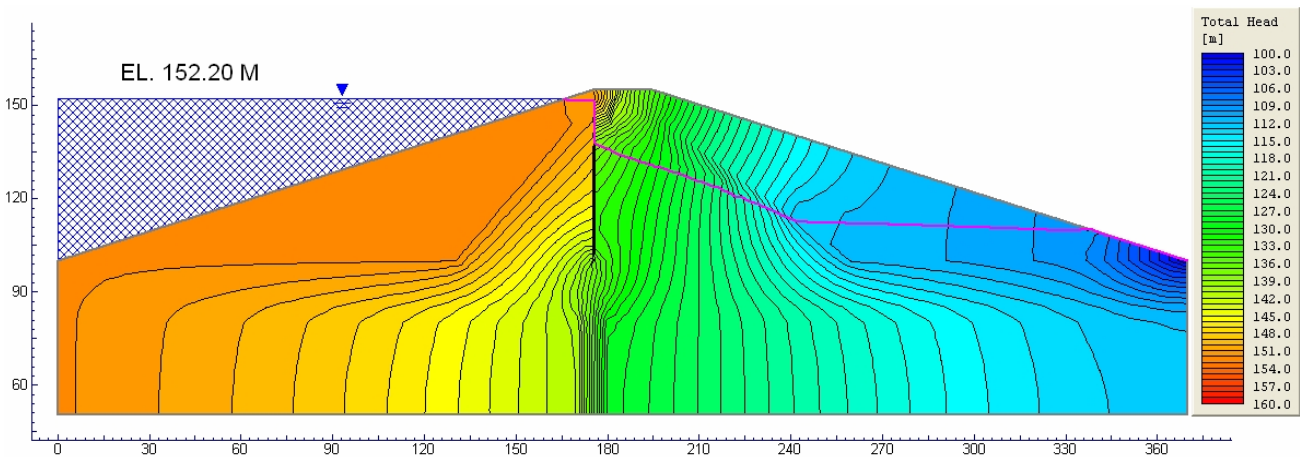


Figure (5.8) Computed Contour Map of Total Head Distribution for Maximum Water Level (152.20 m)

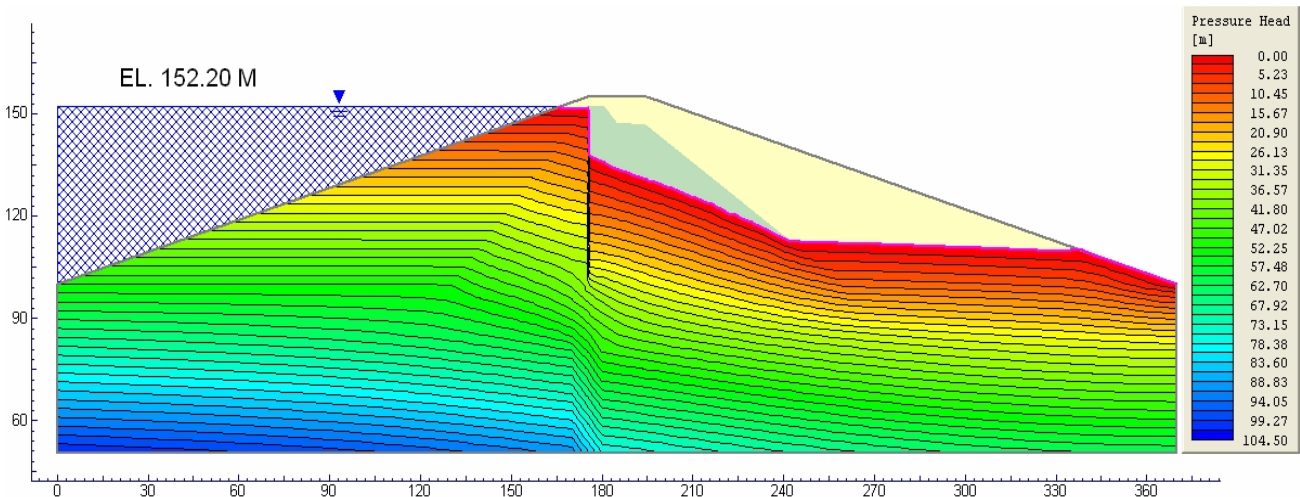


Figure (5.9) Computed Contour Map of Pressure Head Distribution for Maximum Water Level (152.20 m)

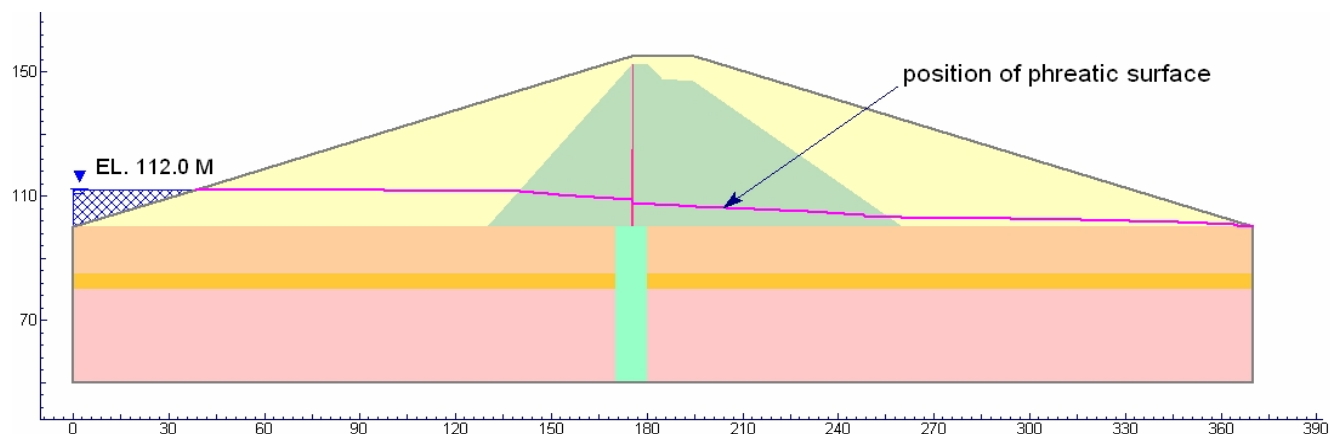


Figure (5.10) Computed Location of Phreatic Surface for Minimum Water Level (112.00 m)

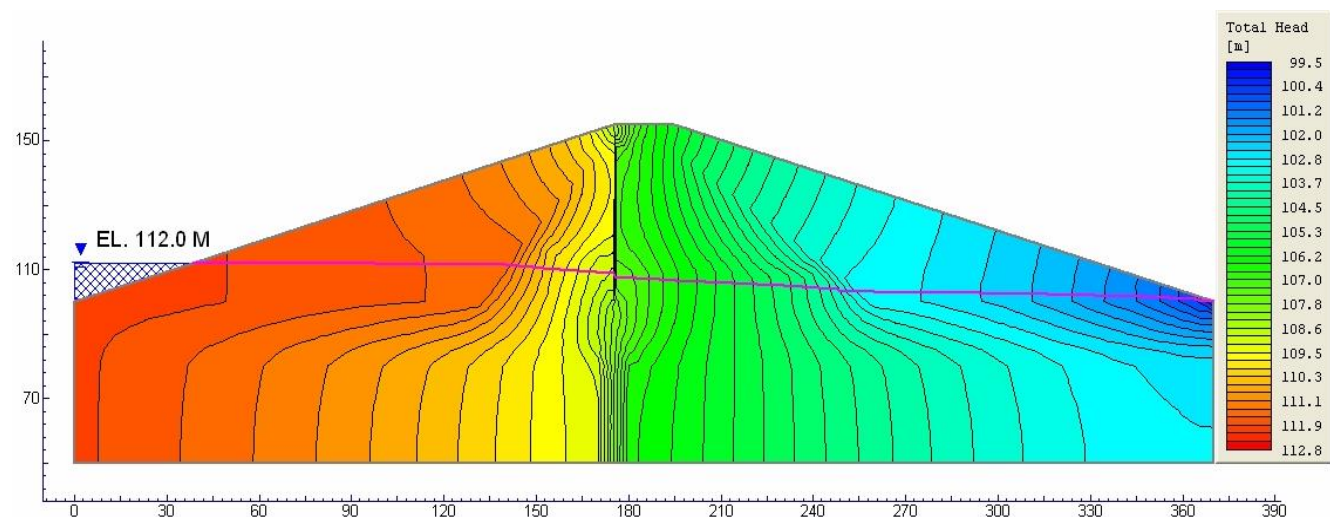


Figure (5.11) Computed Contour Map of Total Head Distribution for Minimum Water Level (112.00 m)

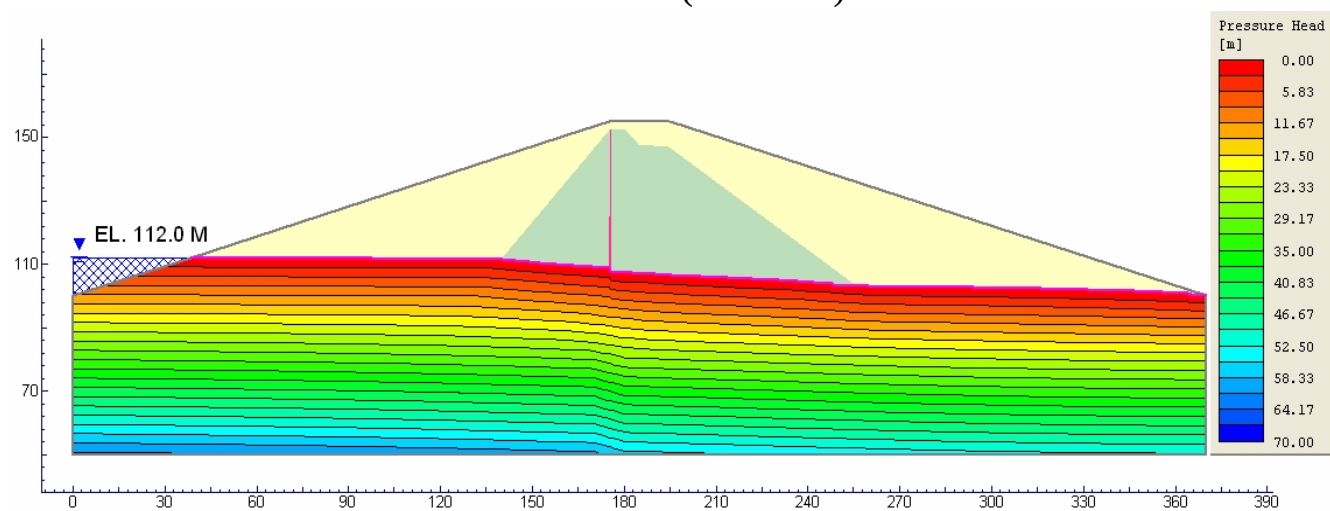


Figure (5.12) Computed Contour Map of Pressure Head Distribution for Minimum Water Level (112.00 m)

5.4.2 Importance of Using the Grouting and Asphaltic Concrete Diaphragm (Cut-off wall)

In order to show the effect of existing (cut-off wall) grouting and the asphaltic concrete diaphragm, figures (5.13a) and (5.13b) show zoned earth dam with cut-off wall, and without cutoff wall, respectively. The quantity of seepage through discharge section in both cases is clearly different. The quantity of seepage is equal to 2.388 (m^3/month) for case of existing cut-off wall, and equal to 2.825 (m^3/month) for the other case. From these values, the existing cutoff wall reduced the seepage quantity downstream of dam with percent more than 18 %. Therefore, the cutoff wall have a large effect on the seepage quantity and position of phreatic surface as illustrated in figures (5.13a, 5.13b).

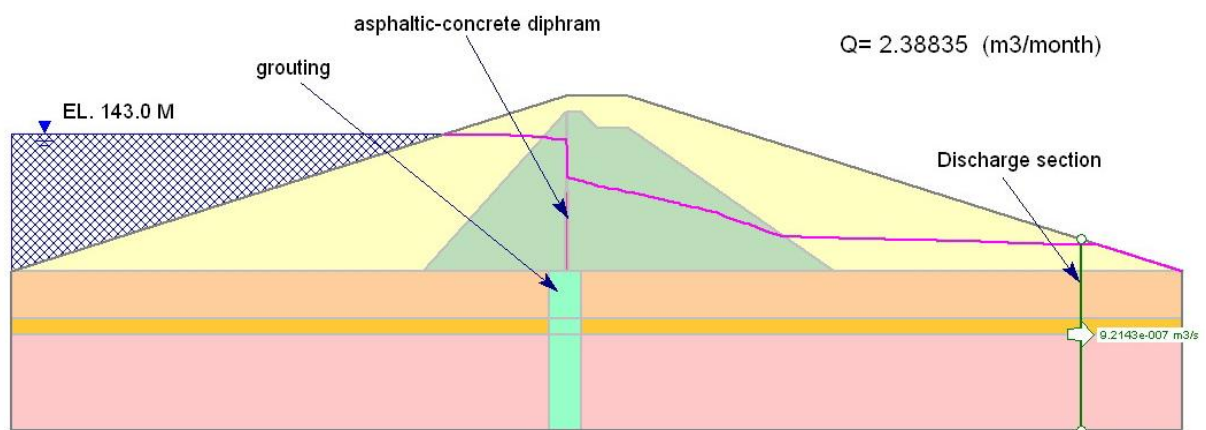


Figure (5.13a) Haditha Dam for Normal Water Level with Existing Asphaltic Concrete Diaphragm, grouting and discharge section.

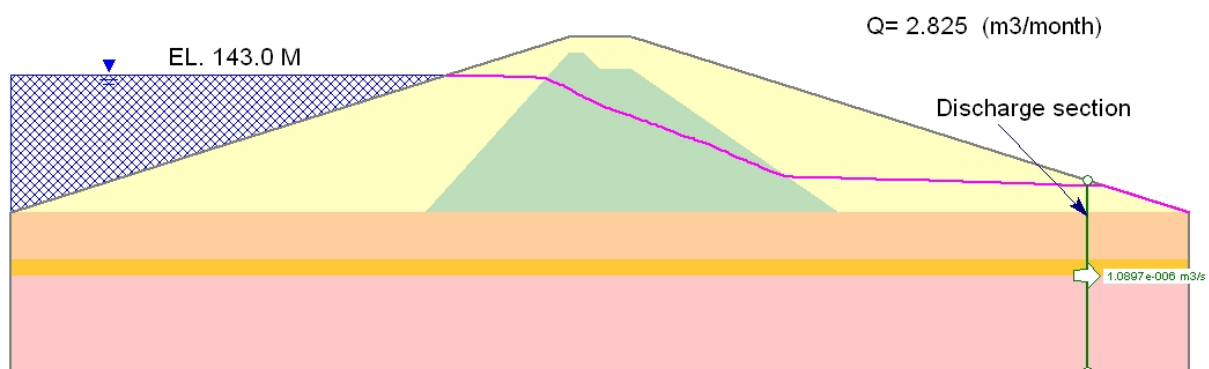


Figure (5.13b) Haditha Dam for Normal Water Level without Asphaltic Concrete Diaphragm and grouting.

Figures (5.14a) and (5.14b) are presented to study the effect of cutoff wall on the pressure head distribution in the core of the zoned earth dam. A straight line with ten points has been taken to show this effect. Figure (5.15) shows this effect.

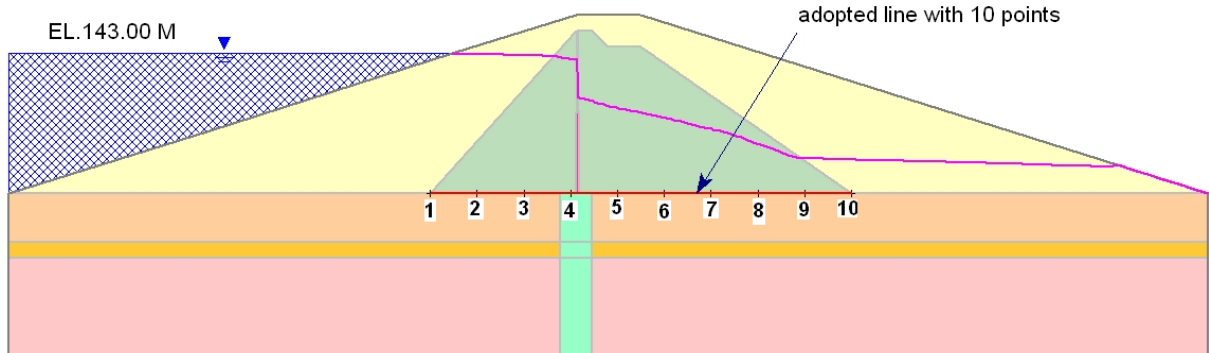


Figure (5.14a) Body of Dam with asphaltic concrete diaphragm and grouting.

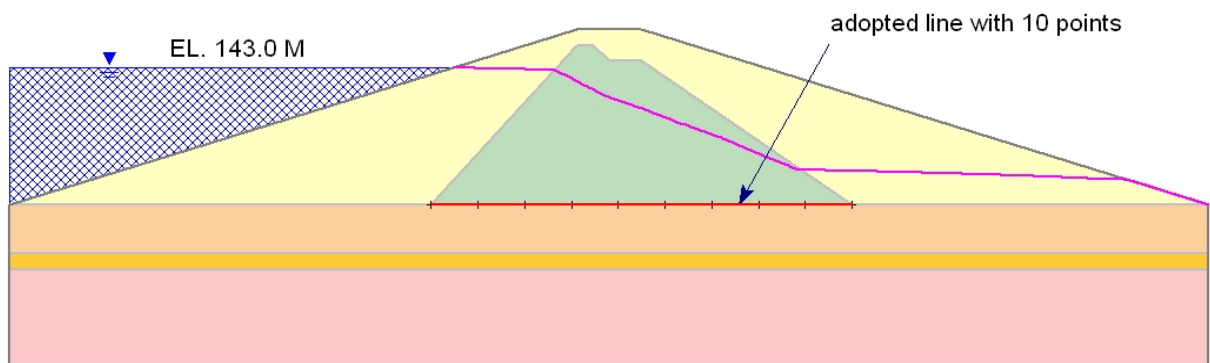


Figure (5.14b) Body of dam without asphaltic concrete diaphragm and grouting.

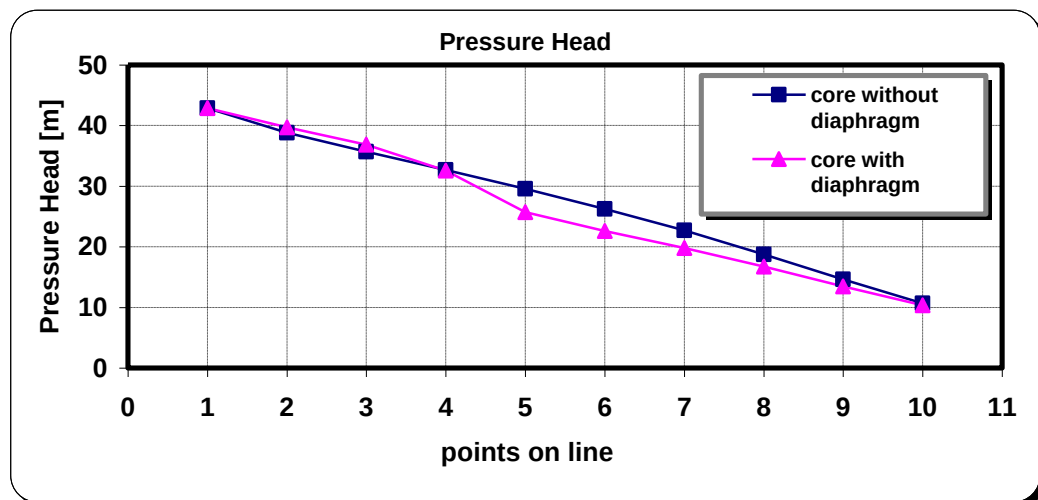


Figure (5.15) Pressure head distribution along the line in the base of core.

5.4.3 Stability analysis results and discussion

5.4.3.1 The designer stability analysis of Haditha Dam

The designer (USSR Hydropromexport) had analyzed the stability of the dam in the channel portion where the height of the dam is about (55.00 m) (the maximum height). The analysis was accomplished according to the following methods [Al-Jorany, 1996]:

1. The ordinary or Fellenius method.
2. Predetermined polygonal slip surface method.
3. Slip along a plan method.

According to the above-mentioned methods of analysis, slip surface of a certain shape and location is to be assumed and the value for the factor of safety is then to be obtained for that surface. In addition to this presumption, which is in most cases not in the safe side, the ordinary method neglects completely the effects of the interslice forces. This assumption gives an under estimated value for the factor of safety and violates Newton's second law of action and reaction at the slice interfaces.

The results of the designer analysis (values of factor of safety to each shape of slip surface) for different water conditions are shown in figure (5.16) and they will be discussed and compared with the results of the present analysis in the following paragraphs.

5.4.4 Methods of solution and scenario used in the present analysis

The computer program (**SLIDE V.5.0**) has been utilized to analyze the stability of the Haditha Dam. The loading conditions considered in the present analysis are to an extent similar to those studied by the designer. The limit equilibrium method (LEM) according to Morgenstern-price presented by computer program (**SLIDE V.5.0**) is applied to define the potential slip surface and calculate the factor of safety of the dam slopes. The failure area is assumed

and divided into a number of sections. The equilibrium of each section is considered and finally a factor of safety for the assumed slip surface is determined, considering the equilibrium of the whole mass. The potential slip surface and factor of safety are iteratively determined until a critical slip surface and minimum factor of safety have been found.

It is very important to define the techniques that are used to select the shape of the slip surface and location of the critical slip surface. The **[U.S. Army Corps of Engineers, 2003a]** recommended that the shape and location of the critical slip surface are subjected to the limitations bellow:

- **Shape of the slip surface.** All of the limit equilibrium methods require a potential slip surface to be assumed in order to calculate the factor of safety. Calculations are repeated for a sufficient number of trial slip surfaces to ensure that the minimum factor of safety has been calculated. For computational simplicity the candidate slip surface is often assumed to be circular or composed of a few straight lines. However, the slip surface will need to have a more complicated shape in complex strait graph. The assumed shape is dependent on the problem geometry and strait graph, material characteristics (especially anisotropy), and the capabilities of the analysis procedure used.
- **Location of the critical slip surface:** The critical slip surface is defined as the surface with the lowest factor of safety. Because different analysis procedures employ different assumptions, the location of the critical slip surface can vary somewhat among different methods of analysis. The critical slip surface for a given problem analyzed by a given method is found by a systematic procedure of generating trial slip surfaces until the one with the minimum factor of safety is found. Searching schemes vary with the assumed shape of the slip surface and the computer program used.

From these assumptions and limitation of **(U.S. Army Corps of Engineers, 2003a)** the slip surface types used in the present analysis are non-

circular slip surfaces (polygonal, arc or composite, and block). The surface definition and search method used are encountered in the block search. The analysis method used is the limit equilibrium method (LEM) according to Morgenstern-price. The strength model used in the present analysis, is Mohr-Coulomb.

After reviewing the methods and scenario, which are used in the present analysis, the stability of the dam side slope is investigated under for different conditions and as follows:

5.4.4.1 Minimum Reservoir Level:

In this case, the water level in the upstream is considered as (112.00 m). The values of factor of safety obtained by the designer for this extreme case are shown in table (5.2), according to the polygonal slip surface method and cylindrical slip surface method. The polygonal slip surfaces considered by the designer are shown in figure (5.16); meanwhile he did not give any information about the location of the cylindrical slip surface he presumed.

Table (5.2) Factor of safety for stability of side slope presented by the designer of Haditha Dam for minimum reservoir level (112.00 m).

Type of slip surface	upstream side slope	downstream side slope
Polygonal slip surface	1.63	1.54
Cylindrical slip surface	1.86	1.71

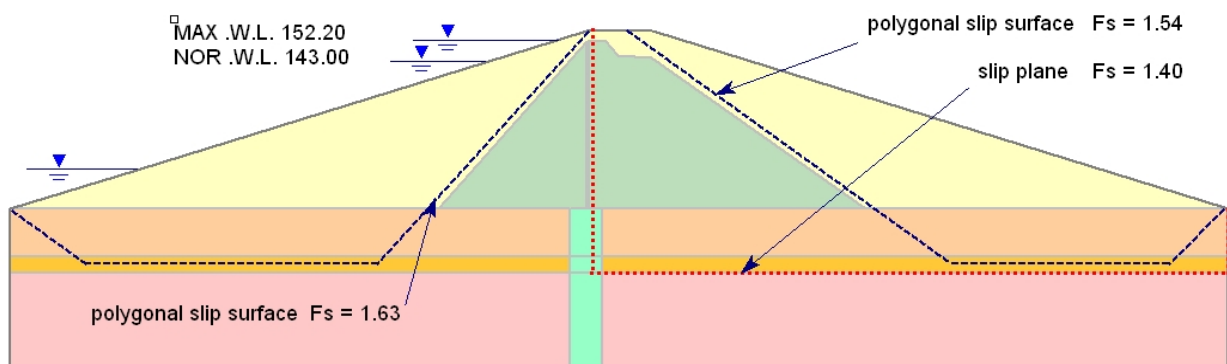


Figure (5.16) The shape and location of slip surface proposed by designer.

According to the present analysis of slope stability, using program **(SLIDE V.5.0)**, the most critical slip surfaces in the upstream and downstream for these loading conditions for each possible slip surfaces are shown in figures (5.17),(5.18),and(5.19). The values of factor of safety obtained in the present work for upstream and downstream are shown in table (5.3).

According to the results in table (5.3) and figures (5.17),(5.18),and (5.19), the shape and location of the most critical slip surface in the upstream side slope for these possible slip surfaces is completely different from that presumed by the designer. The designer presumed two possible slip surfaces, cylindrical and polygonal slip surface; the polygonal slip surface is shown in figure (5.16), whereas he did not give any information about location of the cylindrical slip surface.

The value of factor of safety for each slip surface depends on the location and shape of slip surface and other parameters, so in the present work three types (according to shape of surface) of slip surface are used to calculate the minimum factor of safety. Table (5.2) shows the values of factor of safety obtained by designer and table (5.3) shows the values of factor of safety obtained in the present work. From these tables the values of factor of safety for polygonal slip surface related to the designer slip surface as found by the present analysis are 2.416 for upstream slip surface and 2.352 for the downstream. These values are considerably different from those obtained by the designer. This may be related to the conventional methods of stability analysis the designer had utilized.

In many problems, the potential or actual failure surface can be approximated closely by two or three straight lines. This situation arises when there are weak strata within or beneath the slope and also when the slope rests upon a very strong stratum [**Lambe and whitman, 1969**]. From the above definition, the block slip surface is more reasonable to describe the most critical

slip surface in present work according to the hydraulic conditions and material properties considered in the case study.

Table (5.3) Values of the factor of safety in the present work for upstream and downstream side of the Haditha Dam for normal reservoir level

Method used	Condition	Type of slip surface		
		Block	Composite	polygonal
Morgenstern-price	upstream	1.561	1.949	2.416
	downstream	1.932	2.125	2.352

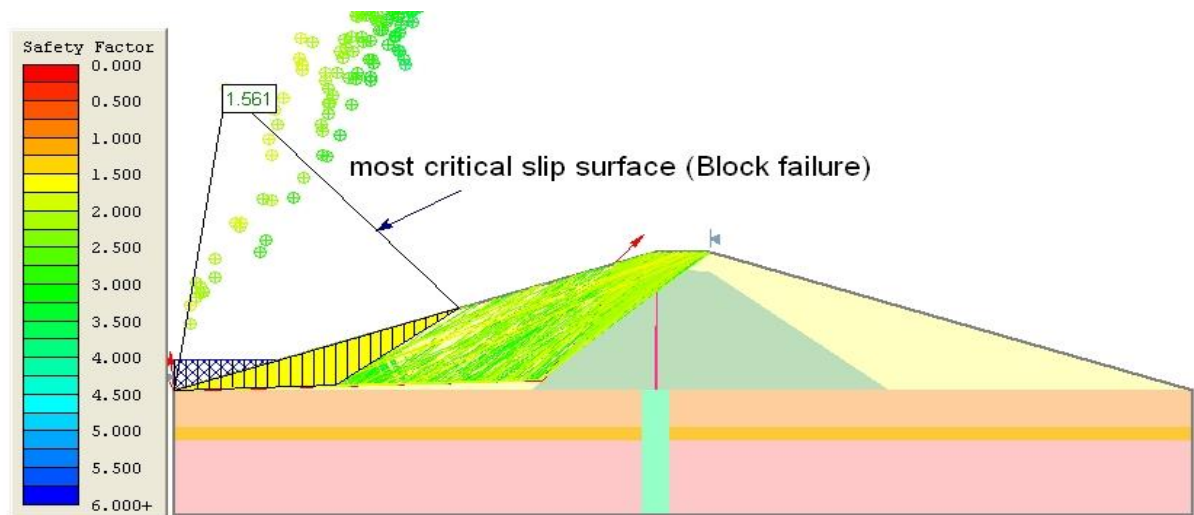


Figure (5.17) Location and value of the most critical slip surface (block slip surface) obtained in the present work, FOS = 1.561

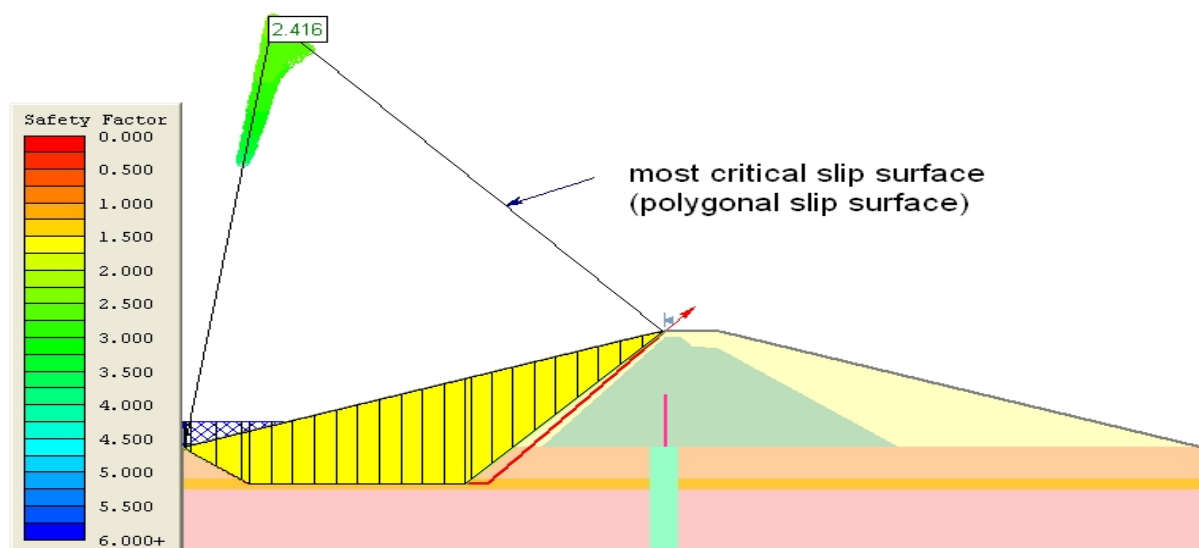


Figure (5.18) Location and value of the most critical slip surface (polygonal slip surface) obtained in the present work, FOS = 2.416

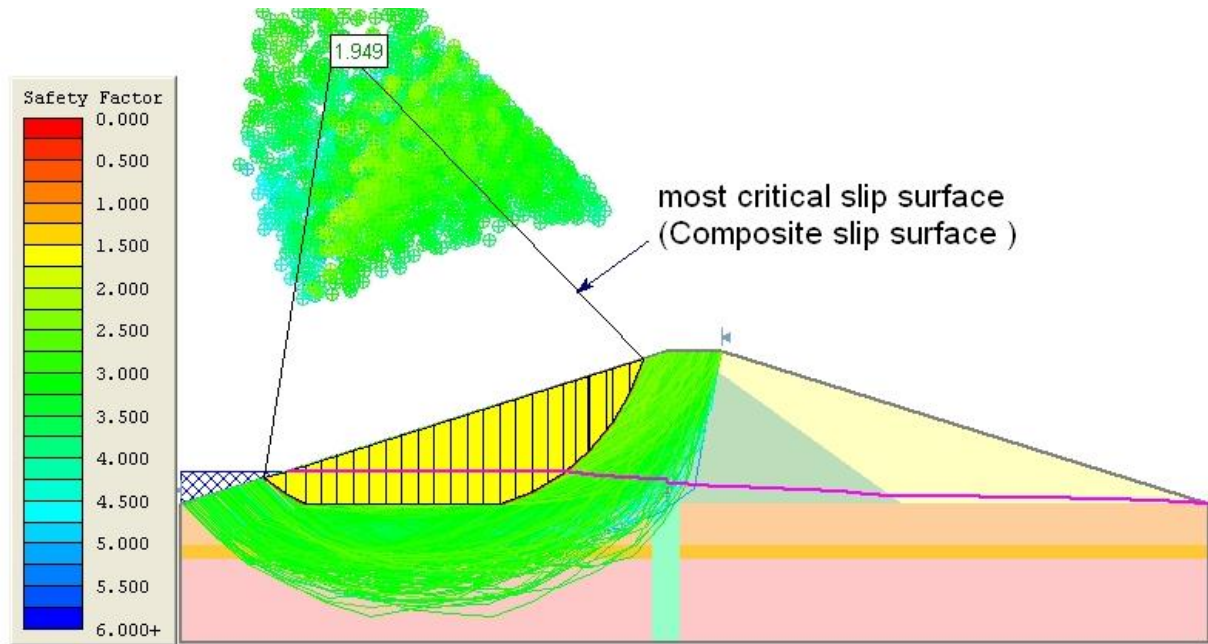


Figure (5.18) Location and value of the most critical slip surface (composite slip surface) obtained in the present work, FOS = 1.949

5.4.4.2 Minimum reservoir level with seismic effects:

The seismic load condition has been studied to investigate the stability of the Haditha Dam under this condition. In order to study the effect of seismic load coefficient on the stability of the dam, and recalling what is mentioned in item (4.3.1), a seismic load coefficient of 0.1 have been considered for the analysis.

The designer investigated the seismic effects on the dam due to an earthquake of about (0.1g) lateral acceleration. The inertia forces due to this proposed acceleration was assumed to act at each slice cantroids. The dam was considered to be at the end of the construction stage and before filling the reservoir. The values of factor of safety obtained by the designer based on the ordinary or Fellenius method and according to the polygonal slip surface method are show in table (5.4).

The results of the present work for these load conditions are shown in table (5.5), the position and shape of most critical slip surface for minimum water level with seismic effects for block , composite, and polygonal slip surfaces are shown in figures (5.19a),(5.19b),and(5.19c), respectively. The seismic load coefficient used in this analysis is the same as that used by designer.

From figures (5.19a), (5.19b), and (5.19c), the shape and location of the most critical slip surface are different from those presumed by designer. It can be noticed that the values of factor of safety are slightly less than those found by the designer.

Table (5.4) Factor of safety obtained by designer for minimum reservoir level with seismic effect (coefficient 0.1)

Method used	Upstream side slope	Downstream side slope
Ordinary or Fellenius	1.29	1.23
Polygonal slip surface	1.20	1.19

Table (5.5) The values of factor of safety obtained in the present work for minimum water level with seismic effect (coefficient 0.1).

Method used	Condition	Type of slip surface		
		Block	Composite	polygonal
Morgenstern-price	upstream	1.09	1.542	1.726
	downstream	1.423	1.679	1.761

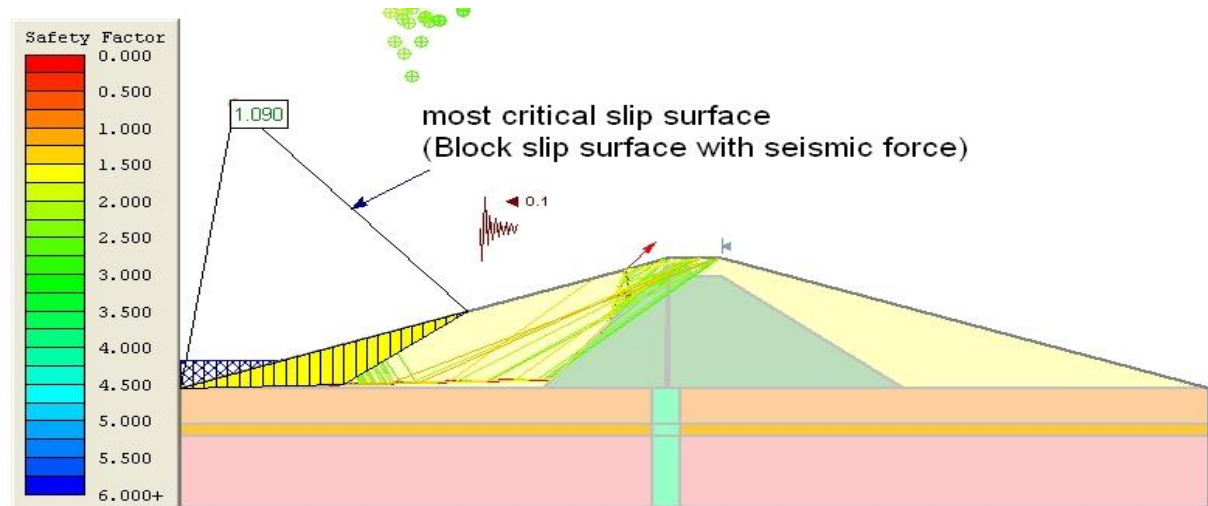


Figure (5.19a) Location and value of the most critical slip surface (block slip surface) for minimum water level with seismic effect (0.1), FOS = 1.09.

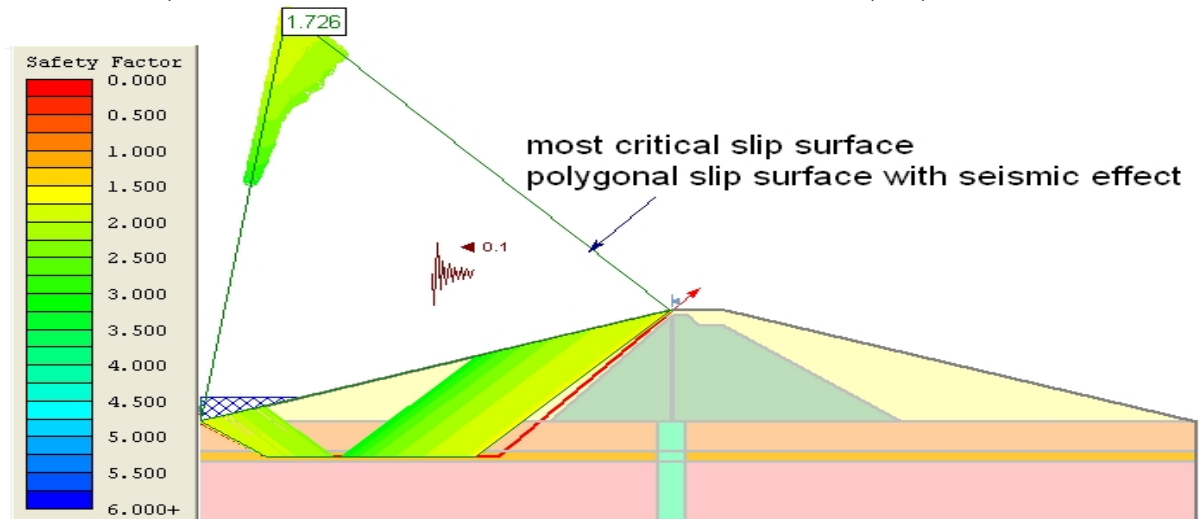


Figure (5.19b) Location and value of the most critical slip surface (polygonal slip surface) for minimum water level with seismic effect (0.1), FOS = 1.726.

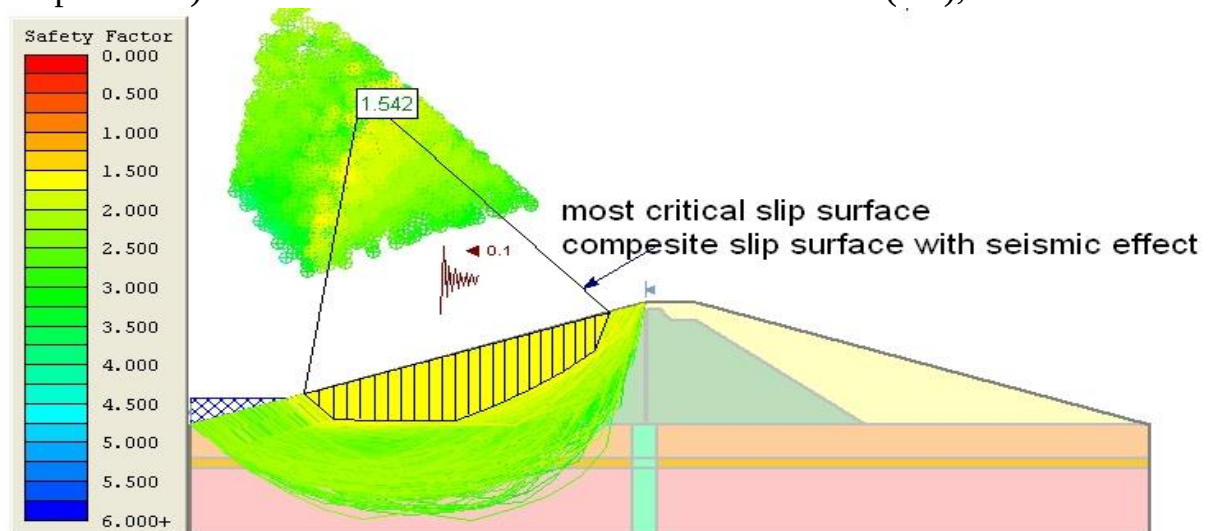


Figure (5.19c) Location and value of the most critical slip surface (composite slip surface) for minimum water level with seismic effect (0.1), FOS = 1.542.

5.4.4.3 Maximum Reservoir Level

In this loading condition, the water level in the upstream is assumed to be maximum (152.20 m). The values of factor of safety obtained by the designer for this case are, 2.40 (according to the ordinary method) and 1.40 (based on the designer slip plane method). In fact, the second value of factor of safety is very crude and moot since the designer assumes that upstream side of the dam will try to move as a rigid body in the upstream direction slipping in the clay layer. The unreliability of this method of analysis is demonstrated by the very under estimated value of factor of safety obtained by the designer himself.

The most critical slip surfaces as determined by the present analysis for this water condition are presented by two critical shapes of slip surfaces, composite slip surface that passes close to the upper surface of upstream slope shown in figure (5.20 a), and block slip surface that passes close to toe of the dam in upstream. This slip surface, which is shown in figure (5.21a), indicates that the mode of critical failure for this loading condition is almost a local failure in the upper face of the upstream (shell).

The minimum or most critical slip surfaces values of FOS against this type of failure are 2.140 for block failure and 2.148 for composite failure in upstream side.

Figures (5.20b) and (5.21b) show the minimum values of FOS against failure in downstream side of dam for block failure and composite failure, respectively.

The polygonal slip surface proposed by the designer is shown in figure (5.22); the value of FOS corresponding to this surface is relatively high (about 3.045) indicating that this surface could never be a critical one according to the other slip surfaces in the present work for block failure and composite failure.

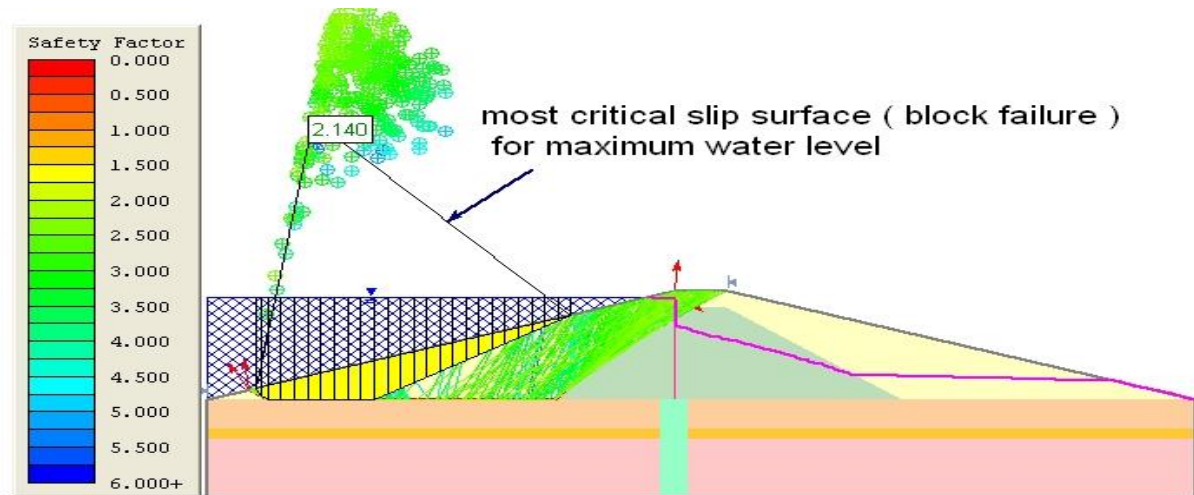


Figure (5.20a) The most critical slip surface (Block slip surface) in upstream side for maximum water level in reservoir, FOS = 2.140

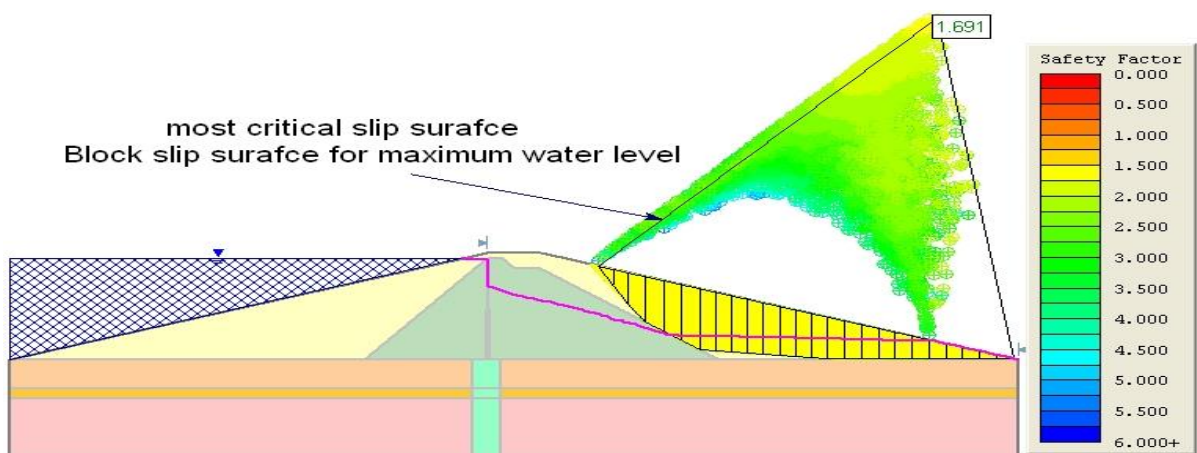


Figure (5.20b) The most critical slip surface (Block slip surface) in downstream side for maximum water level in reservoir, FOS = 1.691.

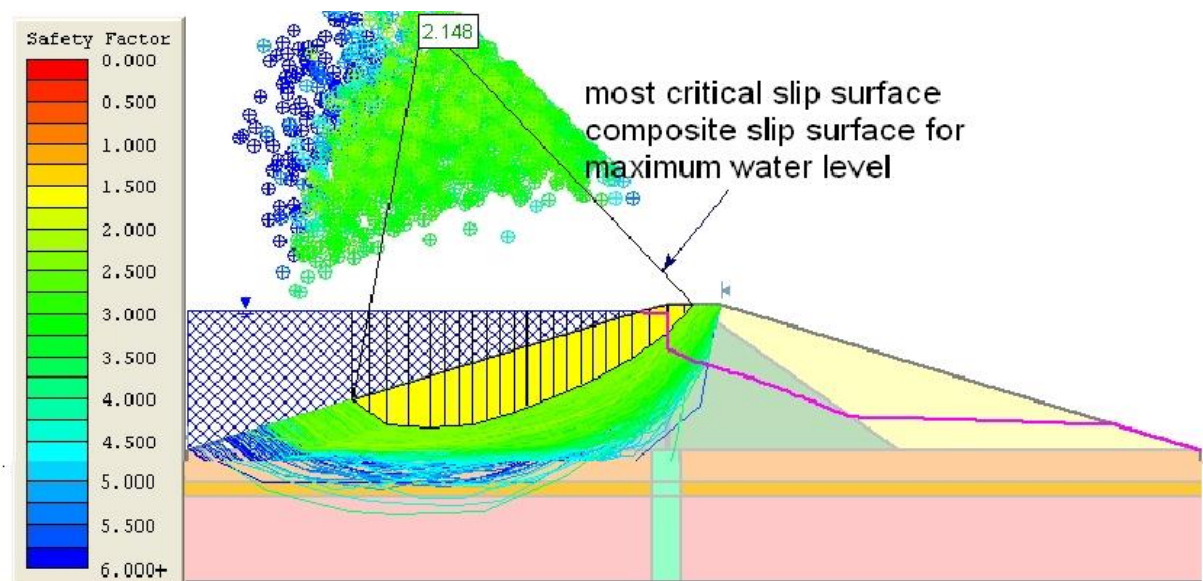


Figure (5.21a) The most critical slip surface (composite slip surface) in upstream side for maximum water level in reservoir, FOS = 2.148

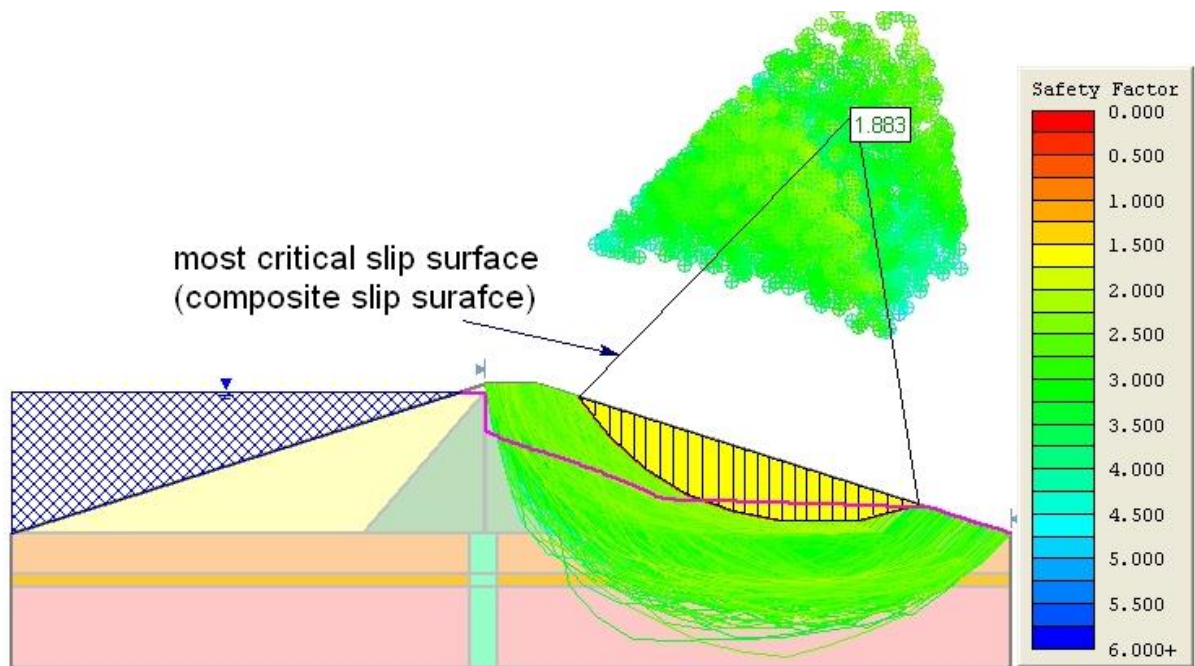


Figure (5.21b) The most critical slip surface (composite slip surface) in downstream side for maximum water level in reservoir, FOS = 1.883.

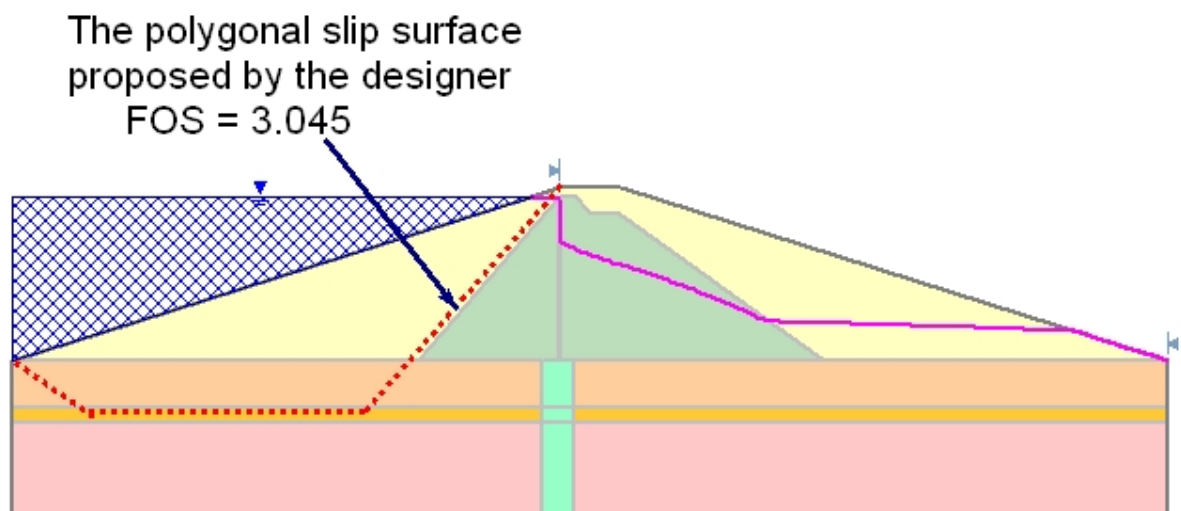


Figure (5.22) The polygonal slip surface proposed by the designer of the dam for maximum water level in upstream side, FOS = 3.045.

5.4.4.4 Rapid Drawdown Condition

The designer had analyzed the case of rapid drawdown of the reservoir water level from elevation (143.00 m to 129.50 m). The factor of safety obtained by the designer is 1.89 according to the ordinary method. This value is greater than that obtained by the designer himself for the case of normal operation (143.0m), which is about 1.60. These results are not fair since the designer implies that the case of rapid drawdown is safer than normal operation condition.

The drawdown is known as one of the most dangerous conditions for the upstream slope. When the countervailing upstream water pressure has disappeared, it causes a danger to the upstream slope. The upstream shell cannot stay stable under the hydrodynamic pressure due to rapid drawdown. Soils inside the dam body remain saturated and seepage commences from it towards the upstream slope. Seepage and hydrodynamic pressures create downward forces acting on the upstream slope.

In the present analysis a greater range of rapid drawdown is investigated from normal elevation (143.00 m to elevation 112.00 m).

The computer program (**SLIDE** V.5.0) is used to calculate the FOS of upstream side under rapid drawdown condition.

Generally, sudden drawdown stability computations are performed for conditions occurring when the water level adjacent to the slope is lowered rapidly. For the analysis purposes, it is assumed that drawdown is very fast, and no drainage occurs in materials with low permeability; thus the term “Sudden” drawdown. Materials with values of permeability greater than 10^{-4} cm/sec can be assumed to drain during drawdown, and drained strengths are used for these materials [**U.S Army Corps of Engineers, (2003a)**].

Excess pore pressure refers to short term (transient) changes in pore pressure within a soil due to rapidly drawdown of ponded water in upstream side conditions (undrained loading). Materials with low permeability such as

clays, may exhibit this behavior. With the so-called "B-bar" method, the change in pore pressure is assumed to be directly proportional to the change in vertical stress. The excess pore pressure is given by :

$$\Delta u = \bar{B} \Delta \sigma_v \quad \dots\dots\dots (5.1)$$

where:

Δu = excess pore water pressure caused by drawdown condition.

$\bar{B}$ = (B-bar) overall pore pressure coefficient for earth fill material.

$\Delta \sigma_v$ = change in vertical effective stress.

From equation (5.1), it can be observed that the value of the excess pore pressure is dependent on the value of (B-bar) coefficient. The value of (B-bar) coefficient is dependent on the type of soil and properties of soil. If (B-bar) coefficient is defined about 0, the soil is free to drain and no excess pore water pressure is developed in upstream side. If (B-bar) coefficient is defined about 1, the undrained condition is applied and excess pore pressure is developed in upstream side. The value of (B-bar) coefficient close to 1 represents critical condition of rapid drawdown in upstream side and should be selected to any soils which have low permeability.

U.S Army Corps of Engineers (2003a) recommended that the materials with values of permeability greater than 10^{-4} cm/sec can be assumed to drain during drawdown and drained strengths are used for these materials.

Figure (5.23) shows the value and location of most critical slip surface for steady state condition (before drawdown). Figure (5.24) shows the value and location of most critical slip surface for rapid drawdown condition from EL. 143.0 m to EL. 112.0 m. Figures (5.25),(5.26), and (5.27) show the value and location of most critical slip surface for rapid drawdown condition with seismic load effect for three different values of seismic coefficient, namely, (0.05, 0.15, and 0.2).

From the values recorded for factor of safety in this part of analysis, it can be observed that the values of safety factor decreases dramatically with the increase of the value of seismic coefficient under rapid drawdown condition, and minimum value of safety factor recorded is about 1.006.

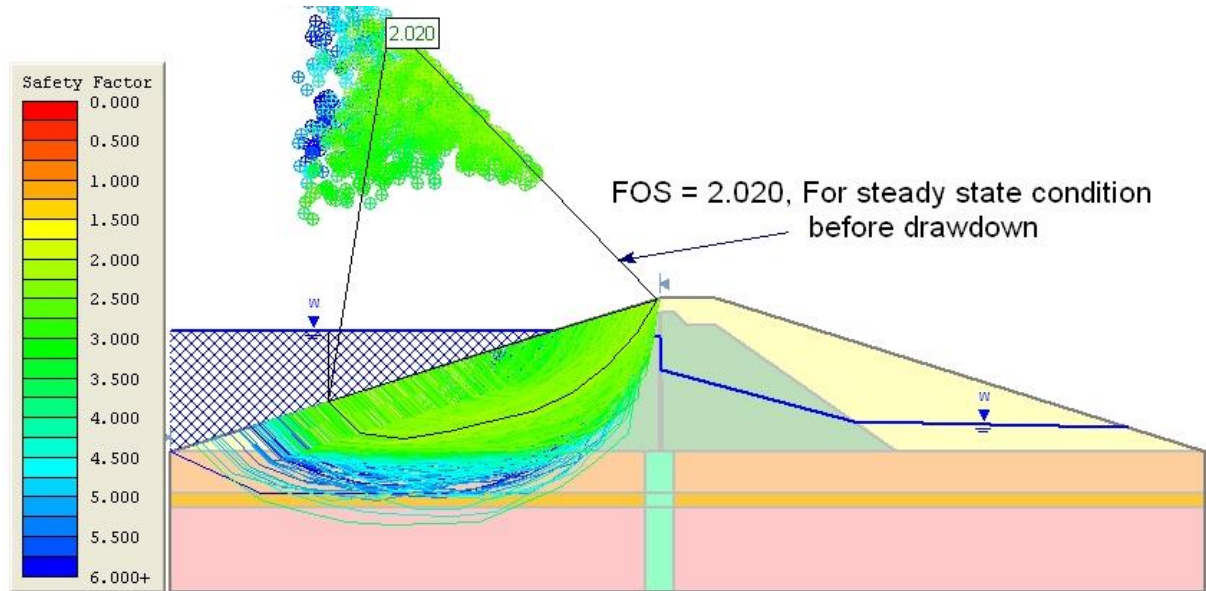


Figure (5.23) The most critical slip surface for steady state condition, before drawdown, FOS = 2.020

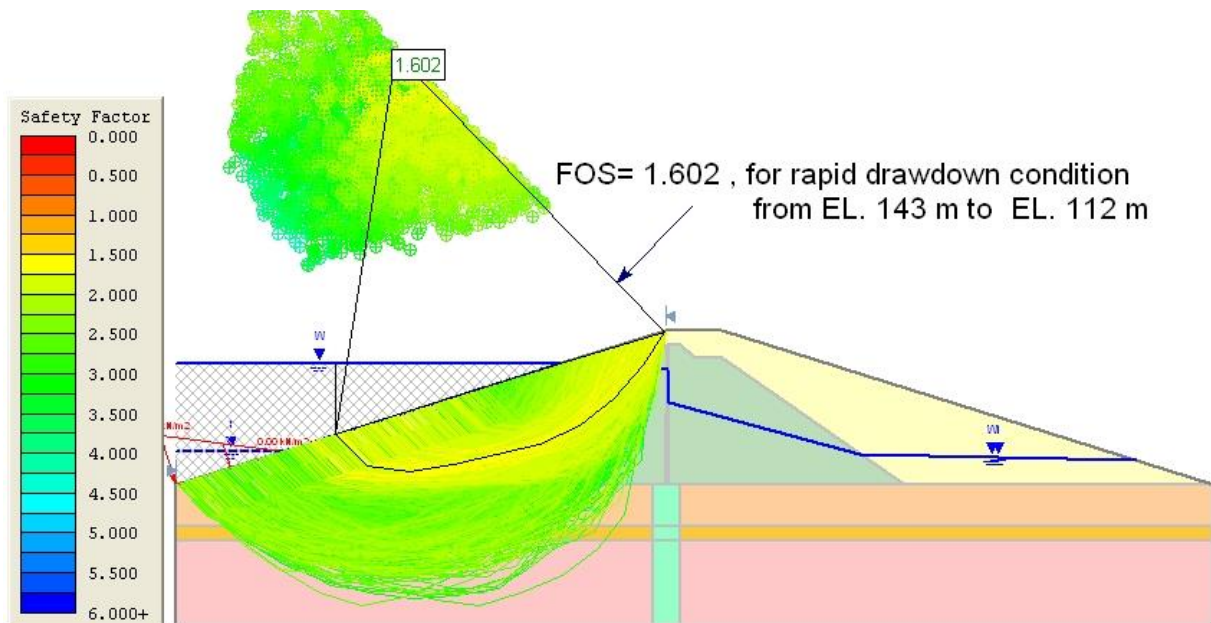


Figure (5.24) The most critical slip surface for rapid drawdown condition from EL. 143.0 m to EL. 112.0 m, FOS = 1.602

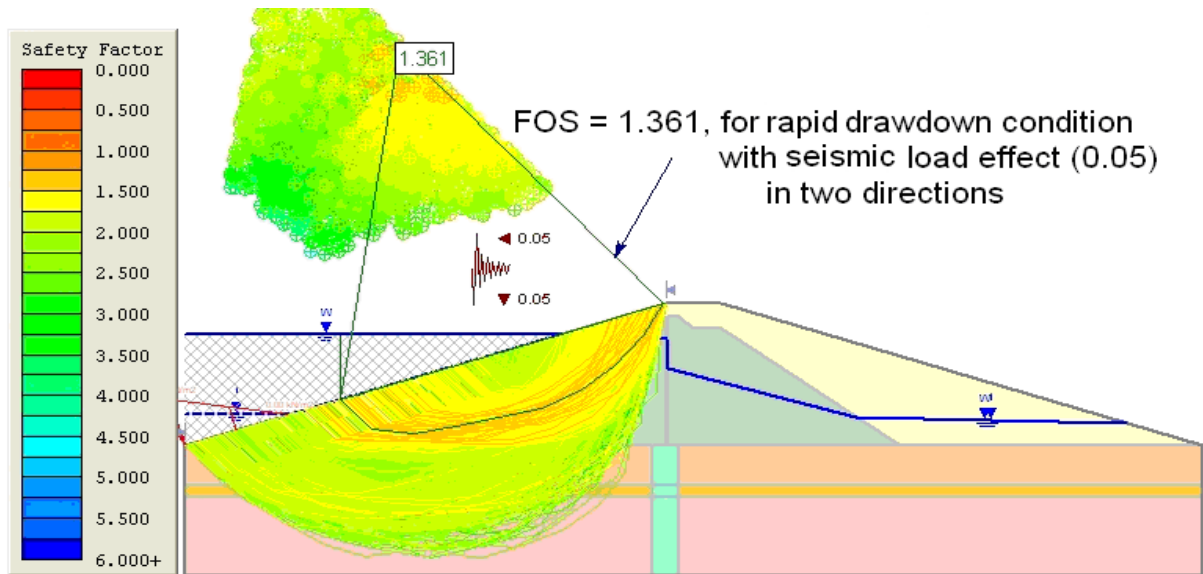


Figure (5.25) The most critical slip surface for rapid drawdown condition with seismic load effect (0.05) in two directions, FOS = 1.361.

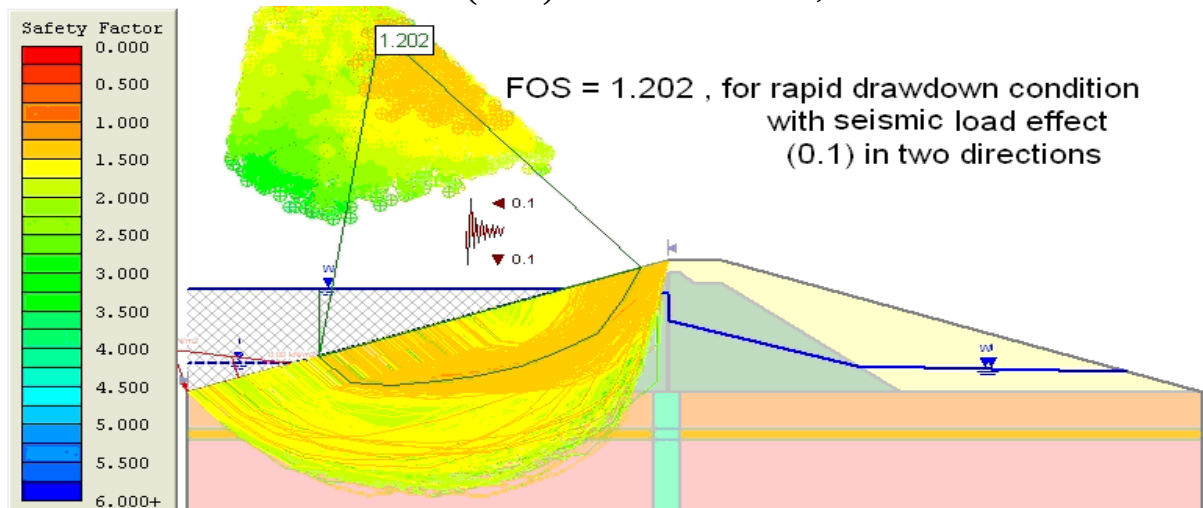


Figure (5.26) The most critical slip surface for rapid drawdown condition with seismic load effect (0.15) in two directions, FOS = 1.202.

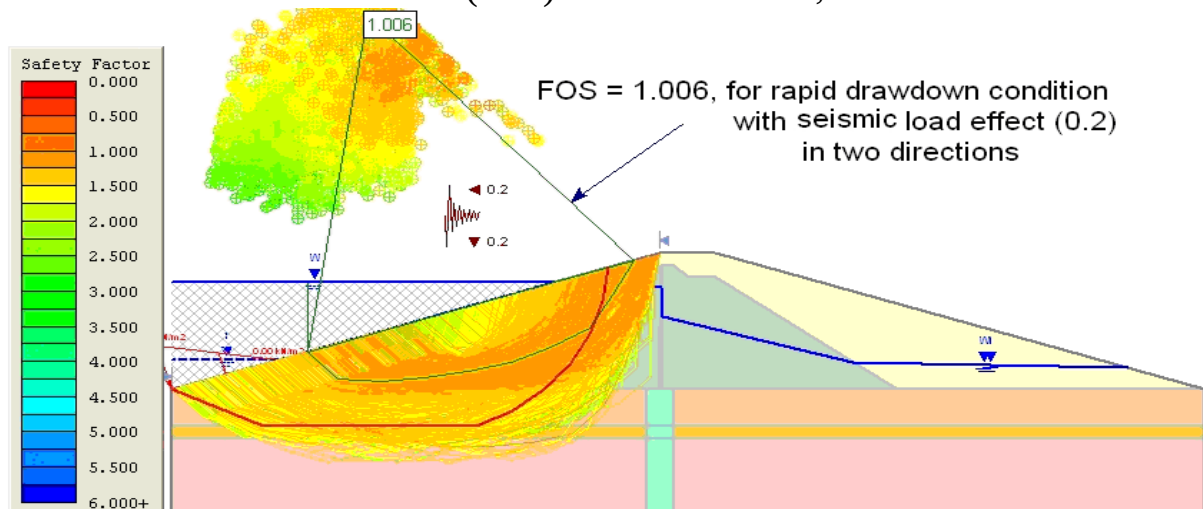


Figure (5.27) The most critical slip surface for rapid drawdown condition with seismic load effect (0.2) in two directions, FOS = 1.006.

5.5 Probabilistic analysis of zoned earth dams

Recalling what is mentioned in item (4.4.1), for most real world slope stability problems, the values of many input parameters are not very well known; therefore, a **probabilistic** approach to the analysis of slope stability can be useful.

From the results of most critical slip surfaces for upstream and downstream side slopes for the Haditha Dam, it can be observed that the most critical slip surfaces in most cases are located at the shell of the dam in upstream and downstream. For these reasons, material properties for the shell of the dam will be defined as random variables (cohesion, unit weight, angle of internal friction), and load (seismic load).

Table (5.6) contains variables that were defined as random variables in the shell of the dam in this part of analysis.

Results of probabilistic analysis of stability of the upstream and downstream sides are shown in table (5.7). Figures (5.28) and (5.29), present the sensitivity analysis of parameters that affect the factor of safety for upstream and downstream sides, respectively. From these figures; it can be observed that the values of FOS are very sensitive to the value of angle of internal friction of soil more than the other variables. Figures (5.30) and (5.31) show the correlation coefficient between FOS and angle of internal friction of the shell of dam in upstream side and downstream side.

From the high correlation between the FOS and angle of internal friction for stability of upstream and downstream sides, proposed equations have been made to represent real correlation between FOS and angle of internal friction of soil.

Table (5.6) Input data for probabilistic analysis and variables that are defined as random variables in the shell of dam, $n = 3000$.

Random variables	Distribution	Mean value	Minimum value	Maximum value	Standard deviation
Cohesion (KN/m ²)	Normal	0	0	20	3.333
Angle of internal friction(degree)	Normal	28	10	46	6
Unit weight(KN/m ³)	Normal	17	15	19	0.667
Seismic load coefficient	Normal	0	0	0.2	0.033

*Standard deviations for variables are calculated by equation (4.2)

Table (5.7) Results of probabilistic analysis for upstream and downstream side.

Results of probabilistic analysis		
Side failure	downstream	upstream
Factor of Safety, deterministic	1.89	1.76
Factor of Safety, mean	1.71	1.55
Factor of Safety, standard deviation	0.40	0.36
Factor of Safety, minimum	0.69	0.61
Factor of Safety, maximum	3.25	2.96
Probability of Failure	2.56 % (77 failed surfaces / 3000 valid surfaces)	4.66 % (140 failed surfaces / 3000 valid surfaces)
Reliability index (assuming normal distribution)	1.78	1.52
Reliability index (assuming lognormal distribution)	2.21	1.79

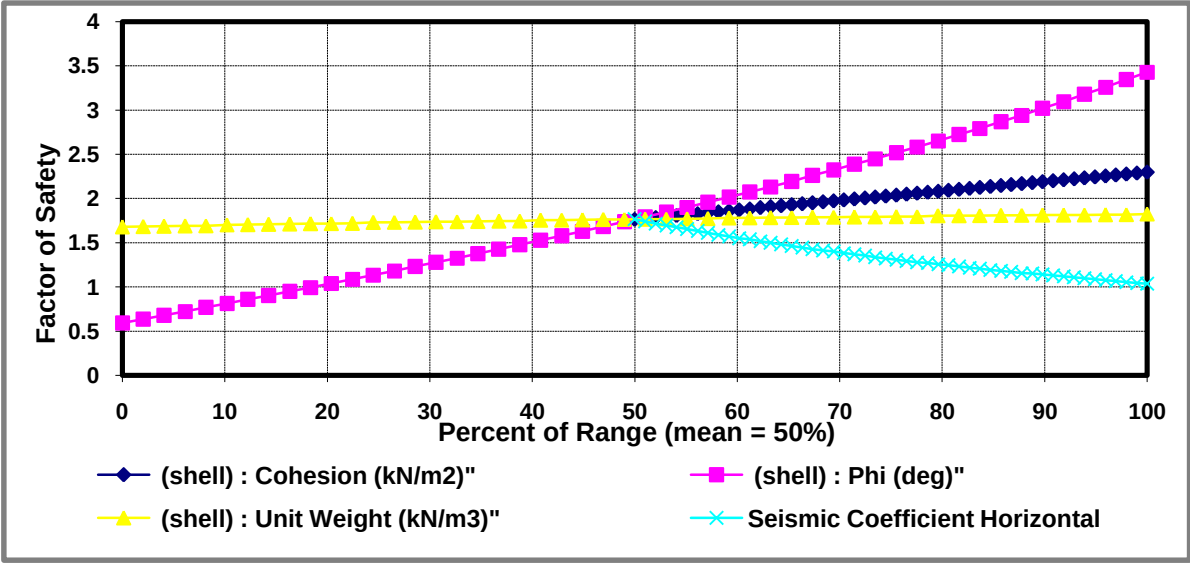


Figure (5.28) Sensitivity analysis plot of parameters that affect FOS for upstream side of dam (shell).

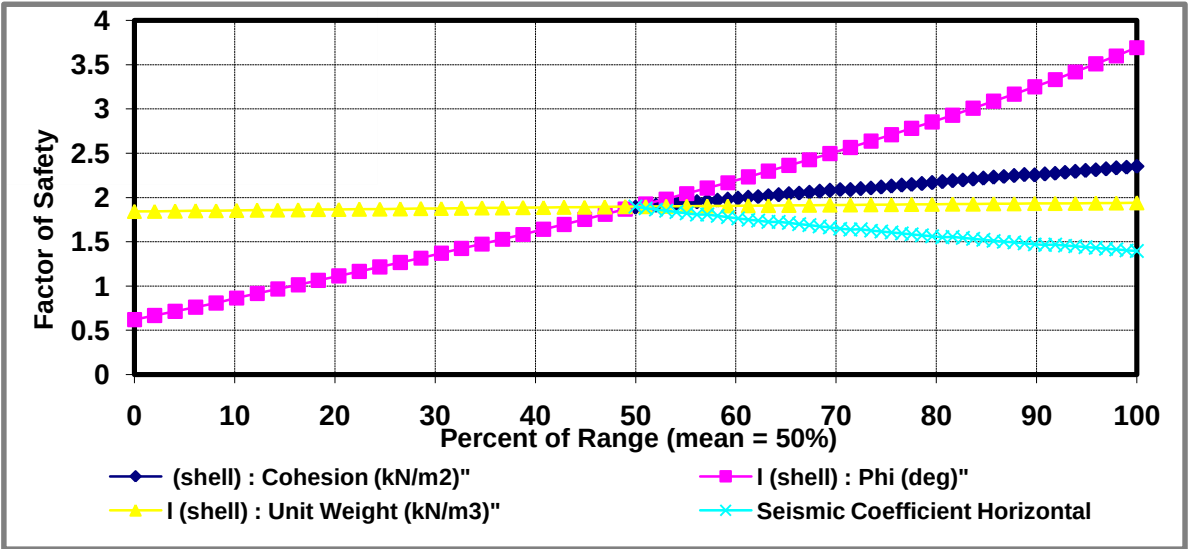


Figure (5.29) Sensitivity analyses plot of parameters that affect FOS for downstream side of dam (shell).

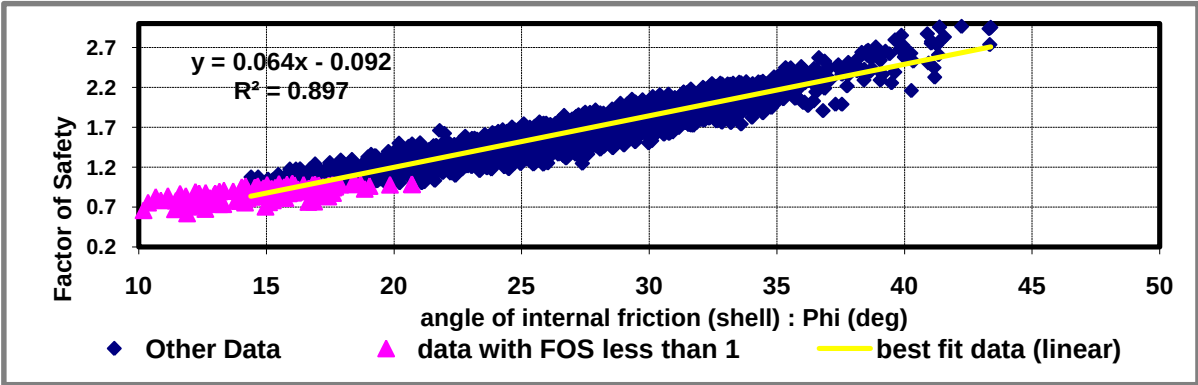


Figure (5.30) The correlation coefficient between FOS and angle of internal friction of the shell of dam in upstream side.

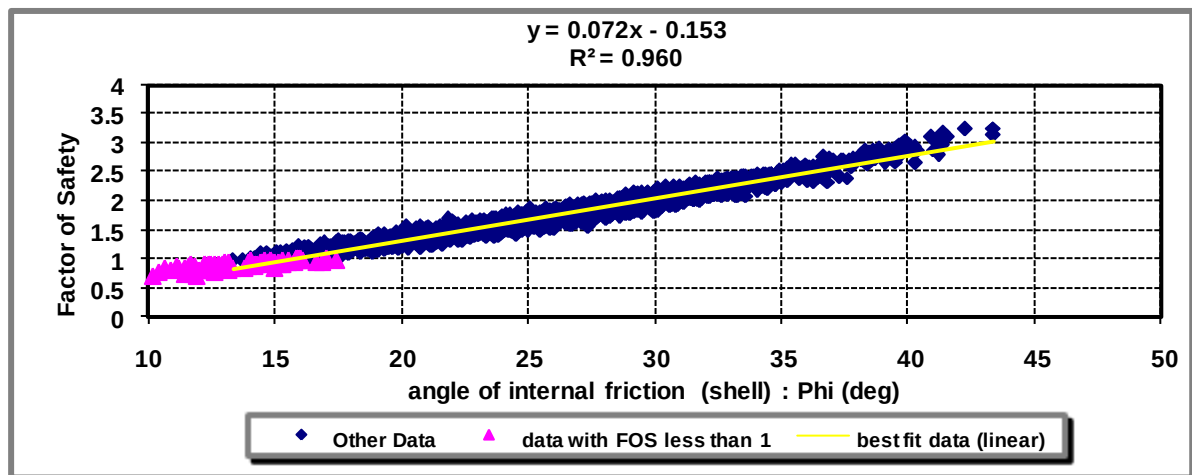


Figure (5.31) The correlation coefficient between FOS and angle of internal friction of the shell of dam in downstream side.

Equations (5.2) and (5.3) are proposed for use to calculate the minimum factor of safety of upstream and downstream sides of Haditha Dam.

$$\text{FOS min} = 0.0645 \varnothing - 0.0928 \quad [\varnothing > 0] \quad \text{----- (5.2)}$$

$$R^2 = 0.90$$

where:

FOS min = minimum factor of safety for upstream side.

$\varnothing$ = angle of internal friction of soil (shell), degree.

$$\text{FOS min} = 0.0792 \varnothing - 0.1531 \quad [\varnothing > 0] \quad \text{----- (5.3)}$$

$$R^2 = 0.96$$

where:

FOS min = minimum factor of safety for downstream side.

$\varnothing$ = angle of internal friction of soil (shell), degree.

Note: These proposed equations are obtained by using the facilities which provided with **SLIDE** program by using 3000 input data

5.6 Methods of Increasing the Factor of Safety:

It is a very well known fact that earthen dam slopes fail. They fail for a variety of reasons. One reason may be excavation of the toe of an embankment, resulting in removing the balancing moment of the slope and causing failure. Another could be an increase in pore pressure along the failure plane during rapid drawdown condition and a corresponding decrease in vertical effective stress and a loss of strength.

There have been methods developed to help in stabilizing failing slopes and reinforce slopes that could fail. These methods are listed below [Yaeger, 2002]:

- End anchored support, (e.g., mechanically end anchored rock bolts).
- Geotextiles (Strip reinforcement).
- Grouted tiebacks / Ground anchors.
- Micro piles.
- Soil nails.
- Stone columns.

This section will discuss the technique that is used to reinforce existing failures, installation of tiebacks or reinforcing the slopes by tiebacks. It will be seen that this method can be very effective, if utilized correctly, stabilizing existing failures and increasing the strength of slopes on the slip surface of failure [Yaeger, 2002].

5.5.1 Grouted Tiebacks / Ground Anchors.

The concept of tiebacks is basically that one carries the lateral earth pressure with a “tie”. The tie transfers the lateral load of the soil to a zone of soil or rock located beyond the failure plane as can be seen in Figure (5.32).

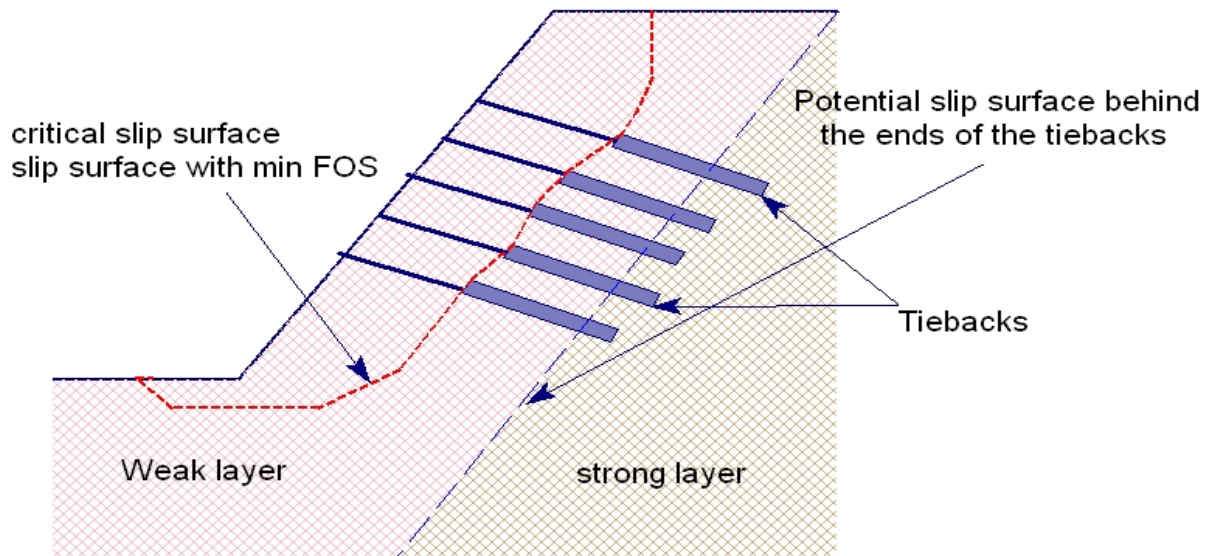


Figure (5.32) Reinforcing the side slope of earth fill by Tiebacks mode of stabilization [Yaeger, 2002].

The following design guidelines should be used when working with tiebacks [Yaeger, 2002].

- Tiebacks are typically installed in cohesion-less soils.
- Typically have a design load of 50-130 tons (for ease of installation and size of equipment required).
- The length of the tieback should be selected such that the anchorage zone is beyond the original failure plane.
- Shallow failure surfaces typically only require one row of tiebacks.
- The inclination of the tiebacks should be between 10 and 30 degrees from the horizontal.
- There should be a minimum of 15 feet of overburden above the zone of embedment.
- All permanent tiebacks should be corrosion resistant.

It is very important to select the zones which need support by using (tiebacks mode). The back analysis method will be used to determine the critical slip surface which requires the maximum amount of support force in order to achieve a specified factor of safety. It is very important to select the minimum required length of support. The minimum required length of support and magnitude of force which it needs to achieve the factor of safety are discussed in the procedure below.

This part of analysis will be adapted to increase the value of factor of safety for some cases which had been analyzed in this chapter previously. (1.09) is the value of factor of safety for upstream side of the Haditha dam under seismic load coefficient about (0.1). In order to increase this value from 1.09 to 1.5 a support pattern equivalent to this increasing value (support force to achieve the new factor of safety) must be added.

Figure (5.33) shows the details of the critical slip surface before adding the support. Figure (5.34) shows the zoned of the most critical slip surface requiring (5076) KN reinforcing force to attain factor of safety = 1.5. This value of requiring force is calculated by using back analysis method in **SLIDE** program. The support force had been calculated can be used to estimate the required capacity and spacing of supports. The slip surface which is determined can be used to estimate the required length of support.

Figure (5.35) shows the details of the tiebacks used in two positions in upstream side to achieve the specified factor of safety. The tiebacks supports are divided into two groups according to the distribution of most critical slip surface of upstream side. Group one contains 5 tiebacks selected near the crest of the dam in upstream side with length equal to 56m, tensile capacity 600 KN, plate capacity 600 KN, and pullout strength 400 KN/m. Group two contains 10 tiebacks selected near the toe of the dam with same properties for group one.

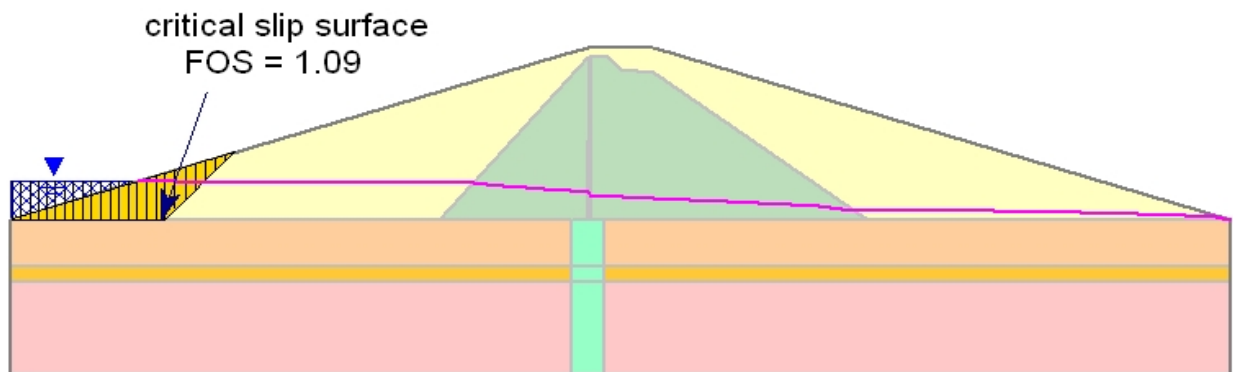


Figure (5.33) Details of the most critical slip surface before adding the support, FOS = 1.09

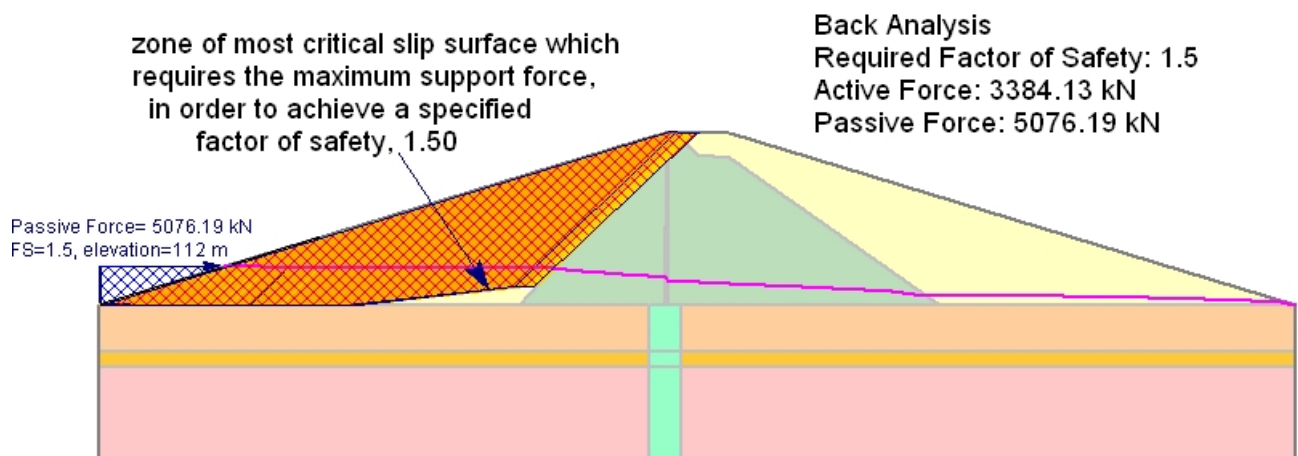


Figure (5.34) Back analysis results, zone of the most critical slip surface which requires the maximum support force.

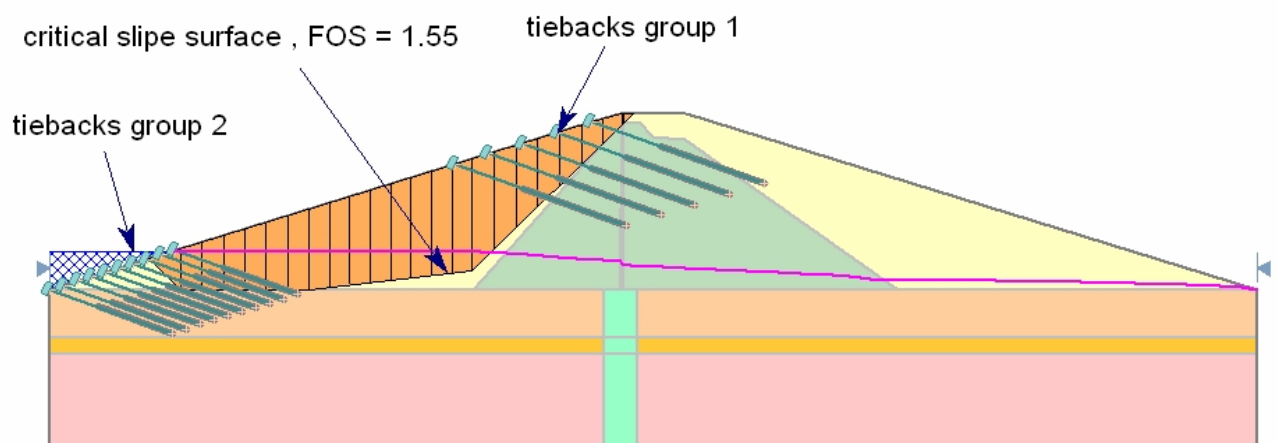


Figure (5.35) Details of the tiebacks support in upstream side, position of most critical slip surface, FOS = 1.55.

Chapter Six

Conclusions and Recommendations

6.1 Conclusions

The following main conclusions can be drawn from the results which are based on seepage and stability analyses:

1. The **SLIDE** computer program is suitable to modelling a complex geometry of earth dams than other applicable methods; the modelling is very close to the realities of zoned earth dams.
2. The factor of safety decreases for the following reasons, listed according to their importance:
 - (a) Increase of angle of slope.
 - (b) Increase of height of embankment.
 - (c) Decrease in cohesion of soil.
 - (d) Decrease in angle of internal friction.
 - (e) Increase of unit weight of soil.
 - (f) Increase of the seismic load coefficients.
 - (g) Increase of the drawdown ratio for rapid drawdown condition.
3. The value of factor of safety is more sensitive to the values of angle of slope, angle of internal friction, and cohesion of soil, than those of other parameters.
4. Probabilistic analysis results make a guide to any designer of earthen dams to check reliability index and degree of confidence of design.
5. The stability of the upstream side slope dramatically decreases during rapid drawdown. The most dangerous state may occur at the drawdown ratio ($L/H = 2/3$), and not always at the complete emptying.

6. In the case of zoned earth dams, particularly in the case of the Haditha Dam, the shape and location of the failure surface as determined by the present work are different from those proposed by the designer.
7. In the case of zoned earth dams, the upstream slope is still stable during rapid drawdown, and the minimum value obtained for FOS is about 1.09 during rapid drawdown and seismic load coefficient 0.2.
8. The cut-off wall (asphaltic concrete diaphragm) seems not to resist any failures in the upstream shell for Haditha Dam.
9. Back analysis approach, it is suitable to obtain the critical zone which it need to support in case of increasing the values of FOS from the critical value to values would to be more stable.

6.2 Recommendations for future work:

Based on the results obtained during assessing this work, the following suggestions can be put forward:

1. The analysis can be modified further if three dimensional analysis of the seepage problem under transient conditions is added, and three dimensional analysis for slope stability.
2. Studying the effect of external loading on slope stability, such as surcharge loads.
3. Studying the effect of infiltration through dam body on slope stability of earth dams.
4. Studying the effect of tension crack on slope stability.