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FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

CHAIR OF HYDROLOGY AND HYDRAULIC ENGINEERING

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EVALUATION OF SLOPE STABILITY AND SEEPAGE ANALYSIS USING PORE WATER PRESSURE, A CASE STUDY OF MALKA WAKANA DAM

A Thesis Submitted to the School of Graduate Studies of Jimma University in Partial fulfillment of the requirements for the Degree of Masters of Science in Hydraulic Engineering

By: Hassen Hussien

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As a member of the board of examiners of the MSc. Thesis open defense examination, we certify that we have read, evaluated the thesis prepared by: Mr. Hassen Hussien entitled: *Evaluation of Slope Stability and Seepage Analysis Using Pore Water Pressure, A Case Study of Malka Wakana Dam*. We recommended that the thesis be accepted as fulfilling the thesis requirement for the degree of Master of Science in Hydraulic Engineering.

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DECLARATION

I declare that the thesis entitled "*Evaluation of Slope Stability and Seepage Analysis Using Pore Water Pressure, A Case Study of Malka Wakana Dam*" is my own work under close direction and instruction of my advisor and all sources of materials used for this thesis have been duly acknowledged. This thesis submitted in partial fulfillment of the requirements for Master of Science degree in Hydraulic Engineering at Jimma University.

I solemnly declare that this thesis is not submitted to any other institution for the award of any academic degree. Any copying or publication of this thesis for commercial purposes or for financial gain is not allowed without my permission.

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December, 2017

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DEDICATION

I dedicate this thesis to my beloved father Hussien Kedu and my mother Amane Alo, who poured me the spirit of hard working, love and for their dedicated partnership in the success of my life. Dad may Allah rest your soul in peace!

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ABSTRACT

When a dam fails, large quantities of water are suddenly released, creating major flood waves capable of causing disastrous damage to down-stream. It may cause loss of life, erosional damage, spoiling of agricultural land, and adverse ecological and environmental impact. This thesis deals with slope stability evaluations and seepage analysis for Malka Wakana embankment dam carried out by commonly used limit equilibrium and finite element methods.

The study utilize a limit equilibrium software SLOPE/W due to its simplicity and better prediction of factor of safety and finite element based software SEEP/W was used to analyze the saturated or unsaturated soil region. A limit equilibrium analysis was carried out using the SLOPE/W software for the stability of the dam slope. The geometry was created in the SEEP/W program and transferred to SLOPE/W by setting the model for steady state condition stability analysis. The dam is analyzed under normal water level taking the water elevation at 2516 m. This is used as upstream boundary condition and zero pressure boundary condition at the downstream horizontal toe drain. The soil properties parameters such as cohesion value, internal friction angle, unit weight, hydraulic conductivity and water content, assigned to prepared regions and for seepage analysis the saturated or unsaturated model was used while for slope stability analysis the Mohr-coulomb model was used. The steady state seepage analysis used as parent analysis for slope stability analysis that means the software use the pore water pressure comes from the initial steady state analysis as piezo-metric line in the body of the dam. Also for Transient seepage analysis the steady state analysis used as initial pore water pressure condition. For sudden drawdown slope stability analysis the transient seepage analysis used as initial pore water pressure condition that means the software reads the pore water pressure condition from previous analysis. Then the analysis type selected and slip surface drawn for slope stability analysis by using entry and exit and it follow a right to left path for upstream slope and for downstream slope it follow a left to right path. In SLOP/W different methods were available for slope stability analysis, among these methods Morgenstern-price, Bishop, Spencer and Janbu method are some of them. Among these the Morgenstern-Price analysis and half-sine function for interslice forces was selected because this method is satisfy both the force and moment equilibrium condition and also gives the better results for factor of safety as compare to Spencer, Bishop and Janbu analysis methods.

The overall minimum stability factor of safety for the steady state condition was equal to 1.985, which means the slope is stable under this condition. Also for transient slope stability analysis the minimum factor of safety obtained were 1.613 for rapid draw down and 1.958 for slow drawdown. These means the slopes were stable for rapped and slow draw down condition. In this study, the seepage through the dam as per the SEEP/W software model that includes foundation seepage is 0.022m^3/day per1m of length and the exit gradient at the downstream toe is less than 1.0. The slope is potentially stable throughout the steady state and transient analysis and the exit gradient is always less than one which means the dam is overall stable under all conditions.

Keywords: *Embankment Dam, GeoStudio, Seepage, Slope Stability.*

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ACRONYMS

BSM	Bishop Simplified Method
CD	Consolidated Drained
CSS	Critical Shear Surface
CU	Consolidated Undrained
EEG	Ethiopian Electric Generation
EEPHO	Ethiopian Electric Power Head Office
FE	Finite Element
FS	Factor of safety
HEP	Hydroelectric Power
ICOLD	International Commission on Large Dams
JSM	Janbu simplified Method
LE	Limit Equilibrium
M-PM	Morgenstern-Price method
MWHPP	Malka Wakana Hydroelectric Power Plant
RDD	Rapid drawdown
SDD	Slow Drawdown
SEEP/W	Software application used for the seepage analysis
SLOPE/W	Software application used for the slope stability analysis
SPM	Spencer Method
USACE	United State Army Corps of Engineer
USBR	United State Bureau of Reclamation
USSD	United State Society on Dams
UU	Unconsolidated Undrained

CHAPTER -1

INTRODUCTION

1.1 Background

In ancient times, dams were built for the single purpose of water supply or irrigation. As civilizations developed, there was a greater need for water supply, irrigation, flood control, navigation, water quality, sediment control and energy. Therefore, dams are constructed for different purpose such as water supply, flood control, irrigation, navigation, sedimentation control, and hydropower. A dam is the cornerstone in the development and management of water resources development of a river basin. The multipurpose dam is a very important project for developing countries, because the population receives domestic and economic benefits from a single investment.*(Tainji, 2015).*

Water storage in dams started early in history and dams are one of the oldest man-made constructions. Dams are vital elements in modern society and represent large economic values. They also represent a potential risk, something that was recognized early on. ICOLD divide dams into embankment dams about 70% of the total number, concrete dams about 28% and masonry dams about 2% *(Johansson, 1997).*

Dams clearly make a significant contribution to the efficient management of finite water resources that are unevenly distributed and subject to large seasonal fluctuations. Most of the dams are single-purpose dams, but there is now a growing number of multipurpose dams. Using the most recent publication of the World Register of Dams, irrigation is by far the most common purpose of dams. Among the single purpose of dams, 48% are for irrigation, 17% for hydropower (production of electricity), 13% for water supply, 10% for flood control, 5% for recreation and less than 1% for navigation and fish farming *(Tainji, 2015*).

The construction of dam ranks with the earliest and most fundamental of civil engineering activities. All great civilizations have been identified with the construction of storage reservoirs appropriate to their needs, in the earliest instances to satisfy irrigation demands arising through the development and expansion of organized agriculture. Operating within constraints imposed by local circumstance, notably climate and terrain, the economic power of successive civilizations was related to proficiency in water engineering. Prosperity, health and material progress became increasingly linked to the ability to store and direct water. Dams are individually unique structures. Irrespective of size and type they demonstrate great complexity in their load response and in their interactive relationship with site hydrology and geology. In recognition of this, and reflecting the relatively indeterminate nature of many major design inputs, dam engineering is not a stylized and formal science. As practiced, it is a highly specialist activity which draws upon many scientific disciplines and balances them with a large element of engineering judgment; dam engineering is thus a uniquely challenging and stimulating field of endeavor (*Novak, 2004*).

Instability related issues in engineered as well as natural slopes are common challenges to both researchers and professionals. In construction areas, instability may result due to rainfall, increase in groundwater table and change in stress conditions. Similarly, natural slopes that have been stable for many years may suddenly fail due to changes in geometry, external forces and loss of shear strength *(Abramson et al., 2002; Aryal, 2006).*

The slope stability analyses are performed to assess the safe and economic design of human-made or natural slopes (e.g. embankments, road cuts, open-pit mining, excavations, and landfills). In the assessment of slopes, engineers primarily use factor of safety values to determine how close or far slopes are from failure. When this ratio is greater than 1, resistive shear strength is greater than driving shear stress and the slope is considered stable. When this ratio is close to 1, shear strength is nearly equal to shear stress and the slope is close to failure, if FS is less than 1 the slope should have already failed. Limit equilibrium types of analysis for assessing the stability of earth slopes have been in use in geotechnical engineering for many decades. The software code SLOPE/W allows geotechnical engineers to carry out limit equilibrium slope stability analysis of existing natural slopes, unreinforced man-made slopes, or slopes with soil reinforcement. The program uses many methods such as: Bishop's modified method, Janbu's simplified method, Spencer method, Morgenstern-Price method and others. SLOPE/W allows these methods to be applied to circular, composite, and non-circular surfaces (*Krahn, 2004*).

The Malka wakana earth and rock fill dam, is constructed by local materials on Wabe Shebelle River, for hydroelectric power production. The total length of the dam along the 10 m crests of which 7 m road way is 1800 m and 38 m high. The dam creates a reservoir with the surface area of 816 ha of the 763 Mm^3 storage capacity, the reservoir provides water withdrawal up to 60 m^3/sec. to the open headrace canal for hydroelectric power production (*Malka Wakana Detailed Project report, 1985*). Details of the case study slopes and a corresponding site description are given in chapter 3.

1.2 Problem statement

In recent years, dam safety draws increasing attention from the public. This is because floods resulting from dam failures can lead to devastating disasters with tremendous loss of life and property, especially in densely populated areas. For instance, the breaching of the levees in New Orleans in August 2005 during Hurricane Katrina caused damage of US$100-200 billion and a regional death count of about 1600 (*Zhang and Jia, 2007*).

When a dam fails or is deliberately demolished, large quantities of water are suddenly released, creating major flood waves capable of causing disastrous damage to down-stream. Major flood waves may seriously damage or destroy power plants, industrial plants, dwellings, and bridges; may disrupt irrigation, navigation, transportation, and socio-economic activities; and may cause loss of life, erosional damage, spoiling of agricultural land, and adverse ecological and environmental impact. These damages and losses could constitute a national disaster and adversely affect a nation's economic, social effort (*Singh, 1996*).

As per (*Jansen, 1980; Singh, 1996*) there have been approximately 2,000 dam failures around the world since the 12th century AD. About 10 percent of those failures have occurred so far in the 20th century causing damage worth millions of dollars and loss of more than 8,000 lives. Three types of earth embankment problems commonly found are seepage, slope stability and vegetation outgrowth. Data from 111 failures show three main reasons for embankment dam failure (*ICOLD, 1995*). Overtopping at high flood discharge about 30% of the total failures, internal erosion and seepage problems in the embankment about 20% and internal erosion and seepage problems in the foundation about 15% (*Johansson, 1997*).

Many embankment dams are constructed in Ethiopia most of which are used for irrigation purpose. However, their capacity reduces frequently before their design life time due to a number of reasons. The main causes of capacity reduction are Hydrological, Structural and Hydraulic failure of which hydraulic failures contributes 58% in Amhara region (*Tefra, 2006*). The Malka Wakana earth and rock fill dam also has the seepage problems that observed at the downstream toe of the dam. Specially, at the right side downstream toe of the dam the excess seepage were observed and the downstream face of the dam also eroded. Therefore, the evaluation of slope stability and seepage analysis in Malka Wakana dam is vital to know and study characteristics of other dams to be built in the future and to assess existing conditions of previously constructed dams.

1.3 Objective of the study

1.3.1 The general objective:

The general objectives of this research is to evaluate Malka Wakana Embankment Dam stability.

1.3.2 The specific objectives:

1. To evaluate the seepage for steady state condition and Transient condition
2. To assess the slope stability for steady state condition
3. To assess the slope stability for drawdown conditions

1.4 Research questions

The research questions that this study will go to explain; are as follows:

1. Does the dam stable during steady state conditions?
2. Does the dam slope is stable with steady state condition?
3. Does the dam slope is stable with the drawdown of the reservoir?

1.5 Significance of the study

This study is to evaluate the stability of Malka Wakana embankment dam using pore water pressure will provide helpful information to various stake holders as follows;

1. Owners, contractors and consultants will benefit from the study as a source of information for earthen dam construction projects, in case of Ethiopia.

2. The study will provide lessons that will help the concerned body can come up with appropriate measures to address problems resulting from instability of slope and seepage through the dam on the stability of embankment dam.

3. Other researchers will use the findings as a reference for further research on stability of the earthen Dam.

CHAPTER -2

REVIEW OF LITERATURE

2.1 Embankment dams

Dams, which are constructed from earth and rock materials, are generally referred as embankment dams. The history of construction of embankment dams is much older than that of concrete dams. It is evident that some earth dams were constructed about 3,000 years ago in the cradles of ancient cultures such as east countries (*Narita, 2000*). According to (*Asawa, 2005*) Embankment dams are water impounding structures composed of natural fragmental materials such as soil and rock and consist of discrete particles which maintain their individual identities and have spaces between them. These materials derive strength from their position, internal friction, and mutual attraction of their particles.

According to the standard manual provided by the International Commission on Large Dams (ICOLD), in which about 63 member countries are now associated, dams with the height of more than 15m are referred to as "high dams". About 14,000 high dams have been registered up to the present, and more than 70 percent of them are embankment dams. A recent report on the construction of high dams has also noted that among about 1,000 of high dams constructed in recent two decades, just about 20 percent are concrete dams and remaining 80 percent are embankment dams *(Narita, 2000).*

Embankment dams can be of many types, depending upon how they utilize the available materials. The initial classification into earthfill or rockfill embankments provides a convenient basis for considering the principal variants employed. According to (*Narita, 2000*) Embankment dams are classified into two main categories by types of soil mainly used as construction materials, such as earthfill dams and rockfill dams. The latter ones further can be classified into a few groups by configurations of dam sections, as one with a centrally located core, one with an inclined core and one with a facing, The main body of rockfill dams, which should have a structural resistance against failure, consists of rockfill shell and transition zones, and core and facing zones have a role to minimize leakage through embankment. Filter zone should be provided in any type of rockfill dams to prevent loss of soil particles by erosion due to seepage flow through embankment. In earthfill dams, on the other hand, the dam body is the only one which should have both structural and seepage resistance against failure with a provided drainage facilities.

Also earthfill embankments may be categorized as an earthfill dam if compacted soils account for over 50% of the placed volume of material. An earthfill dam is constructed primarily of selected engineering soils compacted uniformly and intensively in relatively thin layers and at a controlled moisture content. An earth dam is composed of suitable soils obtained from borrow areas or required excavation and compacted in layers by mechanical means *(USACE, 1994)*.

Rockfill embankments, in the rockfill embankment the section includes a discrete impervious element of compacted earthfill or a slender concrete or bituminous membrane. The designation ‘rockfill embankment’ is appropriate where over 50% of the fill material may be classified as rockfill, i.e. Coarse grained frictional material. A rock-fill dam is one composed largely of fragmented rock with an impervious core. The core is separated from the rock shells by a series of transition zones built of properly graded material. A membrane of concrete, asphalt, or steel plate on the upstream face should be considered instead of an impervious earth core only when sufficient impervious material is not available. Modern practice is to specify a graded rockfill, heavily compacted in relatively thin layers by heavy plant. The construction method is therefore essentially similar to that for the earthfill embankment *(USACE, 1994)*.

2.2 Basic requirements

2.2.1 Design considerations

The following criteria must be met to ensure satisfactory earth and rock-fill structures *(USACE, 1994)*:

a) The embankment, foundation, and abutments must be stable under all conditions of construction and reservoir operation including seismic.
b) Seepage through the embankment, foundation, and abutments must be collected and controlled to prevent excessive uplift pressures, piping, sloughing, and removal of material by solution, or erosion of material by loss into cracks, joints, and cavities. In addition, the purpose of the project may impose a limitation on the allowable quantity of seepage. The design should consider seepage control measures such as foundation cutoffs, adequate and non-brittle impervious zones, transition zones, drainage blankets, upstream impervious blankets, and relief wells.
c) Freeboard must be sufficient to prevent over-topping by waves and include an allowance for the normal settlement of the foundation and embankment as well as for seismic effects where applicable.

d) Spillway and outlet capacity must be sufficient to prevent overtopping of the embankment.

A homogeneous earthfill dam should be designed with relatively flat slopes to reduce the possibility of failure; Generally 1:3 in upstream side and 1:2 in downstream side. Unlike other dams, the dam body is the only structure which provides structural and seepage resistance against failure and required drainage facilities for a homogeneous earthfill dam *(Narita, 2000).*

The design is unique for each earthfill dams because of the location of the dam and the variety of materials to be used for construction. Purpose of the dam also plays an important role on design criteria. The factors mentioned under section 2.2.1 of this study bring hard to define a general design criteria (*Kutzner, 1997).* However, every design criteria must be included the following fundamental design aspects (*Jansen, 1988):*

1. Stability of embankment and foundation in critical conditions such as Earthquake and flood.
2. Control of seepage and pressure in both embankment and foundation
3. Safety measures to control overtopping situation
4. Erosion control methods

Dam may lose its performance by time because of the long term changes in the properties of constructed materials. A typical example is the material may become more anisotropic than when it was at the stage of construction. Also, deposition, displacement and biological growth are some other considerable process which may impact on the performance of a dam *(Jansen, 1988*). To maintain the performance of a dam, critical conditions such as earthquake, overtopping and un-expectable increase of seepage quantity are should be overcome with controlling structures such as filter protected chimney drains, horizontal drain blankets, foundation cut-offs, relief wells and abutment drainage curtains (*Jansen, 1988).*

Evaluation of slope stability requires (*USACE, 2003*):

a. Establishing the conditions, called “design conditions ” or “loading conditions,” to which the slope may be subjected during its life, and
b. Performing analyses of stability for each of these conditions. There are four design conditions that must be considered for dams: (1) during and at the end of construction, (2) steady state seepage, (3) sudden drawdown, and (4) earthquake loading. The first

three conditions are static; the fourth involves dynamic loading. In this case study the steady state seepage and sudden drawdown load conditions are considered.

2.2.2 Materials

A good embankment soil material has to be water insoluble and should contain inorganic substances as long as possible. Hence, the clay with higher water content more than 80% and crushed rock powders are strongly avoidable materials. Generally fine grained soils are very suitable for embankment construction but, those should be within a particular range of moisture content and fulfill the requirements for compaction *(USACE, 2004).* Because in the case of fine grained soils with higher water content, the self-weight of the embankment may develop the higher pore-water pressure within the embankment dam.

A well graded wide range of particle size soil is always preferable than a uniform soil when the other properties of the both soils are equal. Because the well graded soils are less susceptible to piping and liquefaction and soil erosion *(USACE, 2004).* However, for any soils, the large boulders which have the particle size greater than the required thickness of compacted layer must be removed before compaction. This operation will raise the performance of compaction.

2.2.3 Geotechnical parameters

Before a geotechnical analysis can be performed, the parameters values needed in the analysis must be determined.

2.2.3.1 Unit weight

Unit weight of a soil mass is the ratio of the total weight of the soil to the total volume of the soil. Unit weight (γ), is usually determined in the laboratory by measuring the weight and volume of a relatively undisturbed soil sample obtained from the field. Measuring unit weight of soil directly in the field might be done by sand cone test, rubber balloon or nuclear densiometer (*Das, 2008*). In this study the unit weights presented in a report by "Hydro-project" scientific and research center is used.

2.2.3.2 Cohesion

Cohesion (c), is usually determined in the laboratory from the Direct Shear Test. Unconfined Compressive Strength (S_{uc}) can be determined in the laboratory using the Triaxial Test or the Unconfined Compressive Strength Test (Das, 2008). "Hydro-project" scientific and research center has already determined the cohesions for this project.

2.2.3.3 Friction angle

The angle of internal friction, ϕ, can be determined in the laboratory by the Direct Shear Test or by Triaxial test *(Das, 2008).* The values determined by “Hydro-project” scientific and research center are used for this analysis.

2.3 Determination of shear strengths

Before discuss the determination of shear strengths for fill materials, it is necessary to provide background information on drained and undrained soil conditions, and total and effective stresses.

As discussed in *(Duncan, 2005):-* “Drained is the condition under which water is able to flow into or out of a mass of soil in the length of time that the soil is subjected to some change in load. Under drained conditions, changes in the loads on the soil do not cause changes in the water pressure in the voids in the soil, because the water can move in or out of the soil freely when the volume of voids increases or decreases in response to the changing loads.

Undrained is the condition under which there is no flow of water into or out of a mass of soil in the length of time that the soil is subjected to some change in load. Changes in the loads on the soil cause changes in the water pressure in the voids, because the water cannot move in or out in response to the tendency for the volume of voids to change.”

Depending on the loading conditions and the permeability of the fill material within the embankment, an engineer could be considering drained or undrained conditions, or both (in the case of a free-draining shell material and impervious core material), in the analysis of the stability of an embankment dam.

Total and effective stresses are defined in *(Duncan, 2005)* as follows:

“Total stress (σ) is the sum of all forces, including those transmitted through inter particle contacts and those transmitted through water pressures, divided by the total area. Total area includes both the area of voids and the area of solid.”

“Effective stress (σ') includes only the forces that are transmitted through particle contacts. It is equal to the total stress minus the water pressure (u).” The equation for effective stress is given as: $\sigma' = \sigma - u$

Table 2.1: Shear strength and pore pressure for static design conditions

Design Condition	Shear Strength	Pore Water Pressure
During Construction and End-of-Construction	Free draining soils – use drained shear strengths related to effective stresses	Free draining soils - Pore water pressures can be estimated using analytical techniques such as hydrostatic pressure computations if there is no flow, or using steady seepage analysis techniques (flow nets or finite element analyses).
	Low-permeability soils – use undrained strengths related to total stresses	Low-permeability soils -Total stresses are used; pore water pressures are set to zero in the slope stability computations.
Steady-State Seepage Conditions	Use drained shear strengths related to effective stresses.	Pore water pressures from field measurements, hydrostatic pressure computations for no-flow conditions, or steady seepage analysis techniques (flow nets, finite element analyses, or finite difference analyses).
Sudden Drawdown Conditions	Free draining soils – use drained shear strengths related to effective stresses.	Free draining soils- First-stage computations (before drawdown)-steady seepage pore pressures as for steady seepage condition. Second- and third-stage computations (after drawdown) – pore water pressures estimated using same techniques as for steady seepage, except with lowered water level.
	Low-permeability soils -Three-stage computations: First stage-use drained shear strength related to effective stresses; second stage-use undrained shear strengths related to consolidation Pressures from the first stage; third stage-use drained strengths related to effective stresses, or undrained strengths related to consolidation pressures from the first stage, depending on which strength is lower-this will vary along the assumed shear surface.	Low-permeability soils-First-stage computations--steady-state seepage pore pressures as described for steady seepage condition. Second-stage computations – total stresses are used; pore water pressures are set to zero. Third-stage computations -- same pore pressures as free draining soils if drained strengths are used; pore water pressures are set to zero where undrained strengths are used.

Source: (*USACE, 2003)*

Table 2.2: Slope stability criteria

Design Condition	Primary Assumption	Remarks	Shear strength to be used
End of construction (upstream or downstream slop)	Zones of the embankment or layers of the foundation are expected to develop significant pore pressures during construction	Embankment soils that are slowly permeable Saturated slowly permeable foundation soils that are not predicted to fully consolidate during Construction Permeable embankment zones and/or foundation strata	Unconsolidated undrained(UU)–includes Triaxial UU tests, unconfined compression (qu) tests, and field vane shear tests Consolidated undrained(CU') or consolidated drained(CD)
Rapid drawdown (upstream slope)	Drawdown from the highest normal pool to the lowest ungated outlet	Consider failure surfaces both within the embankment and extending into the foundation	Lowest shear strength from a composite envelope of CU and CD
Steady seepage (downstream slope)	Phreatic line developed from pool at the principal spillway crest Uplift pressure simulated by phreatic line developed from auxiliary spillway crest applied to saturated embankment and foundation soils	Consider failure surfaces both within the embankment and extending into the foundation	Lowest shear strength from a composite envelope of CU and (CU+CD)/2 envelopes

Source: (*USDA, 2005*)

2.3.1 Drained and undrained shear strengths

Shear strength is defined as the maximum value of shear stress that the soil can withstand. The shear stress on the horizontal plane in the direct shear test specimen is equal to the shear force divided by the area: (*Duncan, et.al. 2014)* discussed in the book titled as Soil strength and slope stability, "The shear strength of soils is controlled by effective stress, no matter whether failure occurs under drained or undrained conditions The relationship between shear strength and effective stress can be represented by a Mohr-Coulomb strength envelope, as shown in Figure 2.1, the relation-ship between τ *and* σ shown in Figure 2.1

can be expressed as. The shear strength of a soil is a function of the cohesion of the soil (c), the internal angle of friction of the soil (ϕ), and the normal stress (σ). The shear stress at failure (τ) is expressed by the Mohr-Coulomb failure law as *(Duncan, et.al. 2014).*

$$\tau = c + \sigma \tan\phi \qquad 2.1$$

$$\tau' = c' + \sigma' \tan\phi' \qquad 2.2$$

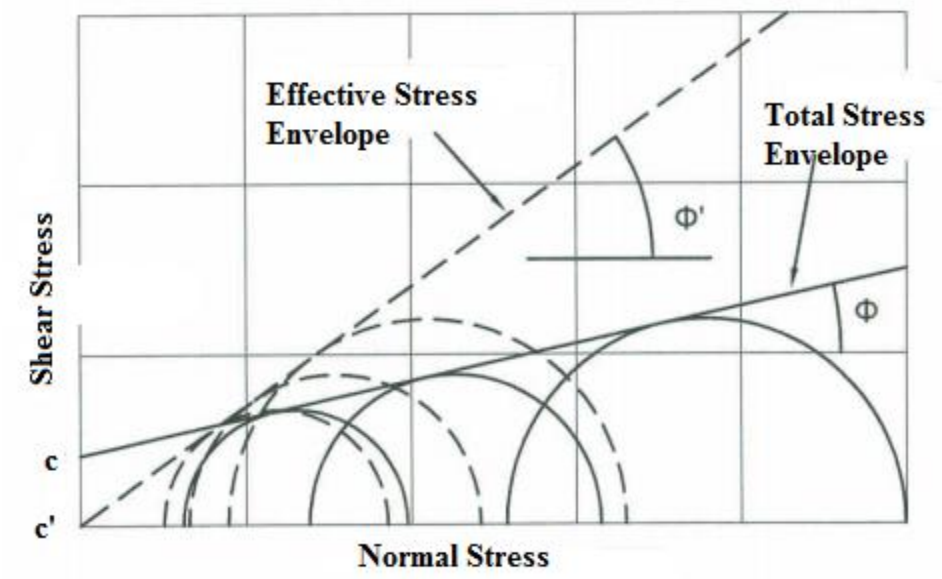


Figure 2.1: Shear strength envelopes for total and effective stresses *(Duncan, et.al. 2014).*

Where c and c' are the cohesion intercepts and ϕ *and* ϕ' are the friction angles for the total and effective stress shear strength envelopes, respectively. Figure 2.1 shows the shear strength envelopes that are developed from Mohr circles for total and effective stresses

2.4 Factor of safety

Factors of safety provide a quantitative indication of slope stability. As (*Duncan, et.al. 2014*) discussed a value of FS = 1.0 indicates that a slope is on the boundary between stability and instability; the factors tending to make the slope stable are in balance with those tending to make the slope unstable. A calculated value of FS less than 1.0 indicates that a slope would be unstable under the conditions contemplated, and a value of FS greater than 1.0 indicates that a slope would be stable.

The most widely used and most generally useful definition of factor of safety for slope stability is (*Duncan, et.al. 2014*).

$$FS = \frac{Shear\ Strength\ of\ the\ soil}{Shear\ Stress\ required\ for\ equilibrium} \qquad 2.3$$

Uncertainty about shear strength is often the largest uncertainty involved in slope stability analyses, and for this reason it is logical that the factor of safety should be related directly to shear strength. One way of judging whether a value of FS provides a sufficient margin of safety is by considering the question: What is the lowest conceivable value of shear strength? A value of FS = 1.5 for a slope indicates that the slope should be stable. When shear strength is represented in terms of c and ϕ, or c' and ϕ', the same value of FS is applied to both of these components of shear strength. It can be said that this definition of factor of safety computed using limit equilibrium procedures is based on the assumption that FS is the same for every point along the slip surface (*USSD, 2007*).

For stability analyses of embankment dams, the recommended factors of safety will vary with loading conditions. Long-term loading conditions (i.e., steady seepage) require higher factors of safety while short-term loading conditions (i.e., rapid drawdown) will require lower factors of safety. Presented in Table 2.3 is a list of different design standard and their recommended criteria for factors of safety for the different loading conditions.

Table 2.3: Factors of safety for embankment dams

Design Standard	Loading Condition	Stress Parameter	FS
USACE	During Construction and End of Construction	Total and Effective	1.3
	Steady State seepage	Effective	1.5
	Sudden Drawdown	Effective	1.3
USBR	During Construction and End of Construction	Effective	1.3
	Steady State seepage	Effective	1.5
	Sudden Drawdown	Effective	1.3

Source*: (USSD, 2007)*

2.5 Fundamentals of groundwater flow

It is well known in geotechnical engineering that groundwater seepage often plays a significant role in slope stability and deformation of geotechnical structures. In order to grasp how groundwater seepage behaves in a particular soil mass, geotechnical engineers conduct various types of seepage analyses. To conduct a seepage analysis, it often requires a fundamental understanding of seepage theory, engineering principals/concepts, soil mass properties, soil geometry, and subsurface soil conditions *(Raymond, 1988).*

This subtitle provides a review of groundwater basics, groundwater theory and the equations that are fundamental to groundwater seepage. It is important to note that the term "permeability" in the bulletin and geotechnical applications is synonymous with "hydraulic conductivity." However, in other industries (such as the oil and gas industry), permeability is taken to mean the "intrinsic permeability" which is a soil property and independent of the permeating fluid *(Raymond, 1988).* To show the relation between intrinsic permeability and hydraulic conductivity it need to review Darcy's Law.

Darcy's law

Darcy's Law is an equation that relates flow velocity to hydraulic gradient under laminar flow conditions *(Darcy, 1856; Das, 2008).*

$$Q = KiA \qquad 2.4$$

$$Hydraulic\,Conductivity = K = \overline{K}\frac{\gamma_w}{\mu} \qquad 2.5$$

$$Hydraulic\,Gradient = i = \frac{\Delta h}{\Delta L} \qquad 2.6$$

Where, Q is the flow rate (flow volume over time), K is the hydraulic conductivity, $\overline{K}$ is the intrinsic permeability, γ_w is the unit weight of water, μ is the viscosity of water, i is the hydraulic gradient, Δh is the head loss, ΔL is the change in length, and A is the cross-sectional area.

Based on the equation above it is demonstrated that hydraulic conductivity is in fact a property of both the soil and the permeating fluid. In most geotechnical applications, water is the permeating fluid. Although the viscosity of water varies with temperature, in geotechnical engineering, the variations are often small enough that changes in hydraulic conductivity can be neglected *(Das, 2008).*

Flow nets

According to (*Krahn, 2004*), "A flow net is a graphical solution to the equation of steady groundwater flow. A flow net consists of two sets of lines which must always be orthogonal (perpendicular to each other): flow lines, which show the direction of groundwater flow, and equipotential lines (lines of constant head), which show the distribution of potential energy." Flow nets can be used to determine the quantity of seepage and upward lift pressure below hydraulic structures. The Figure 2.2 is taken from software just to show the flow net principles.

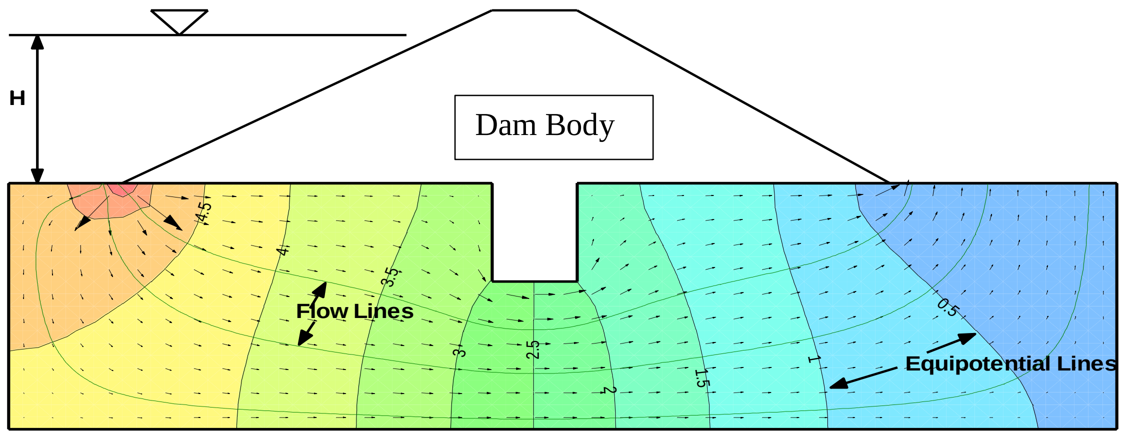


Figure 2.2: SEEP/W result of flow nets

Flow quantities can be estimated from a flow net as the total head drop times the conductivity times a ratio of the number of flow channels to the number of equipotential drops *(Krahn, 2004).*

$$Q = \Delta h * K * \frac{n_f}{n_d} \tag{2.7}$$

Where Q is the flow rate, Δh is the change in head, K is the hydraulic conductivity, n_f is the number of flow lines and n_d is the number of drops.

2.5.1 Determining hydraulic conductivity

Hydraulic conductivity is a quantitative measure of a soil's ability to transmit water when subjected to a hydraulic gradient. The potential for piping through dam is directly related to hydraulic conductivity. If foundation soils underneath a dam have high hydraulic conductivity and fluid velocity is uncontrolled, internal erosion can develop and transport fines within the embankment. According to (*USBR, 2014*) the permeability of soil and rock materials in or beneath an embankment is the most obvious factor that plays a key role in seepage behavior. That being said, it can also be a very difficult parameter to measure, which implies that several methods to estimate permeability may be useful to get an understanding of the potential range in permeability values at a given dam and foundation. Permeability, or more precisely "coefficient of permeability," is at times used interchangeably with the term "hydraulic conductivity." Throughout this study, the two terms will be used interchangeably to refer to the flow rate through a saturated porous medium under a unit (1.0) hydraulic gradient.

Therefore, hydraulic conductivity plays an important role in a variety of applications and scenarios. This section provides information on how to approximate or determine the hydraulic conductivity of various soil types using field methods, empirical methods, and laboratory testing.

Field method

One reliable and easy way to determine the hydraulic conductivity in the field is to use the Auger-hole Method. The Auger-hole Method obtains the average hydraulic conductivity of soil layers extending from the water table *(Beers, 1983)*. This is done by boring a hole into the soil to a finite depth below the water table; groundwater seeps into the hole and reaches equilibrium. The water in the hole is then removed and water begins to seep back into the hole. "The rate at which the water rises in the hole is measured and then converted by a suitable formula to the hydraulic conductivity for the soil" *(Beers, 1983).* Since the auger-hole method is rarely used for determining hydraulic conductivity compared to empirical and laboratory methods, the methodology is excluded in the scope of this thesis.

Empirical method

Hazen's Approximation is an empirical relation between hydraulic conductivity with grain size and is shown below in the equation 2.8 (*Hazen, 1930*; *West, 1995*).

$$k \approx C(D_{10})^2 \qquad 2.8$$

Where C is a constant (for simplicity purposes use C=1) and D_{10} is the diameter (effective size), in mm, of the 10^{th} percentile grain size of the sample. Table 2.4 provides the hydraulic conductivity for various soil types using Hazen's Approximation.

Table 2.4: Hazen's approximation

Materials	k (cm/sec)	Effective Size, D_{10} (mm)
Uniform Coarse sand	0.4	0.6
Uniform medium sand	0.1	0.3
Clean, well-graded sand and gravel	0.01	0.1
Uniform, fine sand	$4*10^{-3}$	0.06
Well-graded silty sand and gravel	$4*10^{-4}$	0.02
Silty sand	10^{-4}	0.01
Uniform silt	$5*10^{-5}$	0.006
Sandy clay	$5*10^{-6}$	0.002
Silty clay	10^{-6}	0.0015
Clay (30% to 50% clay size)	10^{-7}	0.0008
Colloidal clay (minus 2 $\mu m \geq 50\%$)	10^{-9}	$4*10^{-6}$

Source: (*West, 1995*)

Laboratory methods

When conducting a seepage analysis, measurement of hydraulic conductivity is often performed on soil samples collected from the field. There are two common tests for measuring hydraulic conductivity in a laboratory setting: The Constant Head Permeability Test and the Falling Head Permeability Test. The Constant Head Permeability Test is preferred for soils with k > 10^{-3} cm/sec (granular soils), and the Falling Head Test is preferred for soils with k <10^{-5} cm/sec (fine grained soils) *(Das, 2008).*

Constant head permeability test

The constant head permeability test is based on the equation 2.9 *(Das, 2008)*

$$\frac{Q}{t} = Av \qquad 2.9$$

Where Q is the flow rate, t is time, A is the cross-sectional area and v is the flow velocity. Flow velocity is measured using Darcy's Law.

Constant Head Permeability Equation for Flow Velocity *(Das, 2008).*

$$v = ki = k\frac{h}{L} \qquad 2.10$$

Solving for k yields the following.

$$k = \frac{QL}{Aht} \tag{2.11}$$

Where k is the hydraulic conductivity, Q is the flow rate, L is the length of the specimen, A is the cross-sectional area, h is the difference in head, and t is time.

Falling head permeability test

Falling-head Method: The falling head permeability test is based on the equation: 2.12 (*Das, 2008*).

$$k = \frac{2.3aL}{At}\log\left(\frac{h_1}{h_2}\right) \tag{2.12}$$

Where k is the hydraulic conductivity, a is the cross-sectional area of the supply reservoir, L is the length of the soil specimen, A is the cross-sectional area of the soil specimen, t is time, and h_1 is the hydraulic head at time zero, and h_2 is the hydraulic head at time, t.

Typical values for hydraulic conductivity

To provide a better understanding of how hydraulic conductivity relates to soil type, typical values for hydraulic conductivity are provided in the following tables.

Table 2.5: Hydraulic conductivity categorization by degree of permeability

Degree of permeability	Hydraulic conductivity (cm/sec)
High	$>10^{-1}$
Medium	10^{-1} to 10^{-5}
Low	10^{-3} to 10^{-5}
Very Low	10^{-5} to 10^{-7}
Practically Impermeable	Less than 10^{-7}

Source: (*Terzaghi and Peck, 1996*)

Table 2.6: USBR hydraulic conductivity categorization by natural soil type

Permeability k_H of Unconsolidated Natural Soils (k_H inversely related to % finer grains)	
Soils	K_H Range ($*10^{-6}$ cm/ses)
Gravel, open work	>2,000,000
Gravel (Poorly graded)	200,000 to 2,000,000
Gravel (Well graded)	10,000 to 1,000,000
Sand, coarse (poorly graded)	10,000 to 500,000
Sand, medium (Poorly graded)	1,000 to 100,000
Sand, fine (poorly graded)	500 to 50,000
Sand (Well graded)	100 to 50,000
Sand, silt	100 to 10,000
Sand Clayey (SC)	1 to 1000
Silt (ML)	1 to 1000
Clay (CL)	0 to 3

Source: (*USBR, 2014*)

Table 2.7: USBR hydraulic conductivity categorization by natural rock type

Permeability k_H of Unfractured Rock (k_H increases with pore size)	
Rock	K_H Range ($*10^{-6}$ cm/ses)
Sandstone, medium	100 to 200,000
Sandstone, silty	0 to 5000
Limestone	0 to 15,000
Granite, weathered	200 to 10,000
Schist	0 to 2000
Tuff	0 to 1000
Gabbro, weathered	50 to 500
Basalt	0 to 50
Dolomite	0 to 5

Source: (*USBR, 2014*)

Table 2.8: USBR hydraulic conductivity categorization by embankment soil type

Permeability (k_v) of Embankment core materials (k_v inversely related to % fines)		Permeability (k_v) of Embankment shell materials (k_v inversely related to % fines)	
Unified soil classification	K_v Range ($*10^{-6}$ cm/sec)	Unified soil classification	Kv Range ($*10^{-6}$ cm/sec)
Silty gravel(GM-SM)	0.0 to 10.0	Poorly Graded gravel	2,000- 1,000,000
Clay gravel (GC)	0.0 to 10.0	Well graded gravel	1,000 to 100,000
SP-SM	0.0 to 10.0	Poorly graded sand-gravel	1000 to 50,000
Silty-sand(SM)	0.0 to 10.0	Well graded sand-gravel	500 to 5,000
SM-SC	0.0 to 3.0	Silty gravel	10 to 500
SM-ML	0.0 to 10.0	SP(medium-coarse)	10,000 to 20,000
SC	0.0 to 3.0	SP (fine to medium)	5,000 to 10,000
ML	0.0 to 10.0	SP (very fine to fine)	500 to 5,000
ML-CL	0.0 to 1.0	Well graded sand (SW)	300 to 5,000
CL	0.0 to 1.0	SP with silt (SP-SM)	10 to 1,000
MH	0.0 to 0.1	Silty sand	10 to 500

Source: (*USBR, 2014)*

Table 2.9: USBR hydraulic conductivity categorization by embankment soil type

Permeability (k_v) of washed Embankment Drain materials (k_v increases with grain size)		Anisotropy(k_h/k_v) of embankment materials (k_h/k_v increases with placement water content)	
Material	K_v Range ($*10^{-6}$ cm/s)	Material	K_h/k_v Range
Coarse sand and gravel	150,000- 500,000	$\frac{\text{Embankment core}}{\text{Reclamation standard placement}}$	4 to 9
Medium to coarse sand	50,000 - 150,000	Nonstandard placement	9 to 36
Fine to medium sand	10,000 - 50,000	Hydraulic fill	64 to 225
		$\frac{\textit{Embankment shell}}{\textit{Reclamation standard}}$	4 to 9
		$\frac{\textit{Embankment Drain}}{\textit{Reclamation standard}}$	1 to 4

Source: (*USBR, 2014)*

2.6 Causes of embankment dam failure

The failure mode of an embankment dam is directly connected with the type of cause of failure and the type of the dam. Singh, 1996 documented that Biswas and Chatterjee (1971) examined the case of 300 dam failure and they have concluded that the 35% of the world's dam failure is caused by the direct overflow. Other 25% of failure is caused because of foundation problems such as excessive seepage, abnormal increases of pore-pressure and internal erosion. Improper design and construction caused the remaining 40% of the failure.

Table 2.10: Causes of failure of earth dams

Cause of partial or complete failure	Percentages of total
Overtopping	30
Seepage	25
Slides	15
Conduit leakage	13
Slope paving	5
Miscellaneous	7
Unknown	5

Source: (*Singh, 1996).*

2.6.1 External erosion

External erosion is caused by flow over embankment (overtopping). The overtopping situation is occurred when *(Costa and Schuster, 1988);*

1. Insufficient capacity of spillway design
2. Partly or fully blocked spillway
3. Losses of storage capacity of the dam
4. Huge water displacement due to earthquake

In case of excess rainfall, the upstream water level increases instantly. When this level exceeds the maximum drainage capacity of the dam, water stared to flow over embankment. This over flowing water causes the breaching followed by slide at downstream slope of the embankment as a consequence of external erosion.

2.6.2 Internal erosion

Internal erosion causes relatively higher number of the embankment dam failure. When compared with the external erosion, it is a long term process and several factors involved. Abnormal increases of seepage quantity and leakage of turbid water are the visual indication of ongoing erosion. In some cases, internal erosion and piping may appear similar because, the induced force is common for both that obtained from the water flow with higher hydraulic gradient *(Fell, 2003).* But both have completely different mechanisms. Piping effect is a result from the intergranular flow of water. Internal erosion is a very common cause of embankment failure in hydraulically fractured structures such as cracks and joints (*Singh, 1996).*

2.6.3 Piping

Piping is a result of soil erosion which takes place through the embankment because of the seepage water flow. The water flow exerts force on particles and washes out them through an unexpected seepage discharge point. This discharge point undergoes further erosion towards upstream side and form an open like "pipe" through the embankment (*Fell, 2003*).

2.7 Methods for seepage control.

Seepage control measures aim to collect or direct seepage into engineered features, where it can be controlled to minimize the development of adverse behavior such as high gradients, excessive pore pressures, large seepage flows. In general, these methods focus on proper filtering and drainage of seepage flows (*USBR, 2014*).

i. Embankment internal filter or drain

As discussed in (*USBR, 2014*) Internal filter and drainage features for an embankment dam typically include a chimney filter or drain located immediately downstream of the core of the dam, connected to a horizontal filter or drainage blanket that extends to the downstream toe of the dam. Quite often, this filter or drain system is comprised of two separate zones to ensure both filter compatibility and adequate drainage capacity. Natural, processed sands and gravels serve as the best internal filter and drain components. Both the chimney and blanket portions of the filter are designed to ensure that finer materials in the core or foundation cannot erode into downstream zones. Filters and drains should extend deep enough in the foundation and high enough in the embankment to ensure that all potential pathways for internal erosion are properly protected.

ii. Toe drains

Toe drains typically serve as the collection system for the internal drainage system in the embankment, as well as a drainage source for foundation seepage. As such, toe drains need to be carefully designed to fully satisfy filter criteria for both embankment and foundation soils. Toe drains typically consist of perforated or slotted pipe surrounded by a gravel or small rock envelope which, in turn, is surrounded by filter sand or gravel *(USACE, 2004).*

iii. Drainage trenches

Downstream drainage trenches running parallel to the toe of the dam can be used when downstream drainage of the foundation is needed beyond what is normally provided by a toe drain. In essence, the deeper trenches provide relief of pressures and a filtered outlet for seepage layers that are located at a greater depth than would be encountered with a typical toe drain. Trenches are excavated and filled with filter/drainage materials of specified gradation to prevent piping of adjacent foundation soils into the trench. As with a toe drain, a perforated or slotted collector pipe is typically included and set at the lowest possible elevation that will still allow downstream outfall *(USACE, 2004).*

iv. Relief wells

Relief wells are used to reduce excessive pore pressures in pervious foundations to a tolerable level. Relief wells provide safety against high exit gradients or uplift pressures. Frequently, relief wells are used to reduce artesian pressures in confined aquifers. Carefully designed "filter packs" are placed around the well screen to ensure that foundation materials are not piped into the wells *(USACE, 2004).*

v. Horizontal drains

Horizontal or semi-horizontal drains can be bored into foundations (frequently in abutment areas) to relieve excessive pore pressures or intercept seepage. Horizontal drains have been constructed in both rock and soil materials. Careful attention to screening and filtering is essential to prevent the potential for internal erosion into the drains *(USACE, 2004).*

2.8 Seepage reduction measures

There are a number of different seepage reduction measures, with almost all of them essentially reducing seepage by means of extending the seepage path through the use of vertical or horizontal barriers. This lengthening of the seepage path results in a lowering of the hydraulic gradient and, thus, a reduction in seepage flows (*USBR, 2014*). These methods are discussed in (*USBR, 2014*) in detailed and in this study some of them are discussed below that taken form this reference.

i) Embankment core

The effectiveness of a wide embankment core acting as a seepage barrier should not be underestimated. Due to low gradients through wide cores, seepage is minimized. The location of the core varies in reclamation embankments. Most commonly, the core is located in the center of the embankment, which has the advantage of providing the highest contact pressure at the base of the core and typically leads to a cutoff trench located in the center of the dam. The placement tends to enhance slope stability for dams that have a weak foundation layer left in place (by limiting the extent of both upstream and downstream failure surfaces passing through the foundation) (*USBR, 2014*).

ii) Cutoff trenches

A well-constructed cutoff trench located beneath the core of a dam and backfilled with impermeable soils is a very reliable means of minimizing seepage through pervious foundation soils. In addition, since the excavation of this feature enables a complete view of foundation conditions, it enables a designer to gain firsthand knowledge of the foundation materials, provides the ability to adjust the design if needed, and permits foundation treatment at the bottom of the excavation and filter protection along the downstream face of the excavation (*USBR, 2014*).

iii) Slurry trench cutoff wall

Cutoff walls constructed by slurry trench methods can effectively cut off seepage in the embankment or foundation of dams. For new dams, slurry trench cutoff walls have been used as the impermeable water barrier for an embankment (instead of an impervious earth core) or as a foundation cutoff when the bedrock (or other suitable impermeable layer) is relatively deep, making a traditional cutoff trench excavation very costly. On existing dams, slurry trench cutoff walls have been used to reduce seepage through embankments, soil foundations, and rock foundations (*USBR, 2014*).

iv) Grout curtains

Grout curtains have often been used to reduce seepage through foundation and abutment rock, but as a seepage cutoff feature, their effectiveness varies greatly depending on geologic conditions. Although grouting can be dependable for reducing total seepage flow through the foundation, a single “window” in the curtain can allow a shorter flow path with concentrated seepage. The effectiveness may be increased by use of multiple grout lines. Neat cement grout is most commonly used in Reclamation applications and is generally reserved for grouting in rock foundations containing joints and fractures (*USBR, 2014*).

v) Upstream blankets

Upstream blankets are a horizontal extension of the embankment water barrier usually an earthfill core, typically used at a site underlain by high permeability foundation materials that are too deep to allow economical construction of a fully penetrating cutoff. Relatively impermeable soil materials are frequently used in an upstream blanket, although geomembranes can be an economical alternative. Because a high gradient will typically occur across an upstream blanket, it is important to ensure that blanket materials cannot pipe into the underlying foundation. This can be accomplished by designing a transition or filter material beneath the impermeable soil that meets filter criteria for the blanket and the foundation (*USBR, 2014*).

2.9 Embankment slope stability

Variations of the loads acting on slopes, and variations of shear strengths with time, result in changes in the factors of safety of slopes. As a consequence, it is often necessary to perform stability analyses corresponding to several different conditions reflecting different stages in the life of a slope. As conditions change, the factor of safety against slope instability may increase or decrease (*Duncan, et.al. 2014*).

2.9.1 Loading conditions for embankment dams

The stability of the upstream and downstream slopes of the dam embankment is analyzed for the most critical or severe loading conditions that may occur during the life of the dam. These loading conditions typically include (*USSD, 2007*):

i. End of Construction: when significant pore pressure development is expected either in the embankment or foundation during construction of the embankment.
ii. Steady-State Seepage: when the long-term phreatic surface within the embankment has been established.
iii. Rapid (or sudden) Drawdown: when the reservoir is drawn down faster than the pore pressures can dissipate within the embankment after the establishment of steady-state seepage conditions.
iv. Earthquake: when the embankment is subjected to seismic loading. This is not concerned in this thesis.

For the evaluation of embankment dam stability, the applicable loading conditions need to be determined. These loadings conditions are discussed in the following subsections.

a) End of construction

The end-of-construction loading condition is usually analyzed for new embankments that 1) include fine-grained soils, and 2) are constructed on fine-grained saturated foundations that may develop excess pore pressures from the loading of the embankment. The embankment is constructed in layers with soils at or above their optimum moisture content that undergo internal consolidation because of the weight of the overlying layers.

Embankment layers may become saturated during construction as a result of consolidation of the layers or by rainfall. Because of the low permeability of fine-grained soils and the relatively short time for construction of the embankment, there is little drainage of the water from the soil during construction: resulting in the development of significant pore pressures. Soils with above optimum moisture content will develop pore pressures more readily when compacted than soils with moisture contents below optimum. Both the upstream and downstream slopes of the embankment are analyzed for this condition (*USSD, 2007*).

b) Steady-state seepage

After a prolonged storage of reservoir, water percolating through an embankment dam will establish a steady-state condition of seepage. The upper surface of seepage is called the phreatic line.

It is general practice to analyze the stability of the downstream slope of the dam embankment for steady-state seepage(or steady seepage) conditions with the reservoir at its normal operating pool elevation (usually the spillway crest elevation) since this is the loading condition the embankment will experience most (*USSD, 2007*).

c) Rapid (or Sudden) drawdown

This loading condition assumes that steady-state seepage conditions have been established within the embankment as a result of maintaining a reservoir at the normal pool elevation and that the embankment materials beneath the phreatic surface are saturated. The reservoir is then drawn down faster than the pore pressures within the embankment materials can dissipate, resulting in a reduced factor of safety. This loading condition is analyzed for the upstream slope of the dam *(USSD, 2007).* In case of this study the steady state, rapid drawdown and slow drawdown slope stability are analysed

Generally the stability of an embankment slope depend on the height of the slope H, slope angle β and the shear strength parameters such as cohesion C and the friction angle ϕ. Among these three parameters, the height and the slope angle reduces the stability with respect to increased amount but, increasing shear strength parameters giving a more stable slope *(Das, 2008).*

2.10 Pore water pressure

Pore water pressure can be defined as pressure experienced by water contained in the pores of earth materials, concrete structures or rock. Via instrumentation associated with large civil engineering structures such as dams, underground tunnels, tall buildings and other mega structures measurement of pore water pressure enables to study detail geotechnical aspects of the structures (*Krahn, 2004)*.

The study of pore pressure has following main purposes:

a) Effect of water in pores of soil or rock is to reduce load bearing capacity of soil or rock. Effect is more pronounced with higher pore water pressure leading eventually in some cases to total failure of load bearing capacity of the soil.

b) Ground water level and flow pattern determination

c) Determine flow pattern of water in embankment & concrete dams and their foundations and to delineate the phreatic line.

The most common way of defining pore-water pressure conditions is with a piezo-metric line. With this option, SLOPE/W simply computes the vertical distance from the slice base mid-point up to the piezo-metric line, and multiplies this distance times the unit weight of water to get the pore-water pressure at the slice base (*Krahn, 2004)*.

Also, SLOPE/W is fully integrated with the finite element products available in GeoStudio. This makes it possible to use finite element computed pore-water pressures in a stability analysis. For example, the pore-water pressures can come from a:

a. SEEP/W steady-state seepage analysis

b. SEEP/W transient analysis at any particular time step

In general, the pore-water pressures can come from any finite element analysis that creates a head or pore-water pressure file. For all the nodes on the ground surface line, when the pore water pressure is positive (i.e., surface ponding condition), SLOPE/W automatically computes the equivalent weight of the water above the ground surface.

When the finite element pore-water pressure analysis has multiple time steps, the pore-water pressure of a certain time step to be used in the analysis can be selected. Alternatively, SLOPE/W allows to select all the time steps to be included automatically in the stability analysis. For example, in the case of transient SEEP/W analysis of a drawdown, this feature will be very useful in assessing the factor of safety versus time (*Krahn, 2004)*.

CHAPTER -3

MATERIAL AND METHOD

3.1 Study area description

Malka wakana hydroelectric power plant is located at 300 km SE of Addis Ababa, between Arsi and Bale Zonal boundary, Oromia, Ethiopia. The dam is constructed on Wabe Shebelle River, one of the largest water courses flowing along the south-east cost of the country and falling in to the Indian Ocean on the territory of Somali. Its geographical coordinates are between 7°5’ - 7°10’40’’N latitude and 39°14’30’’- 39°27’E longitude. The 2300-2400 m elevation of altitude has highly influenced the climatic situation of the vicinity. The mean annual temperature is not greater than 13-14℃ (max. 28℃). The Dam is in the upper course of the river where the mean annual flow of the river is 825 Mm^3 and the maximum flood discharge is 530 m^3/sec. The dam which is an earth and rock fill dam, is constructed by local materials. The total length of the dam along the 10 m crests of which 7 m road way is 1800 m and 38 m high. The dam creates a reservoir with the surface area of 816 ha of the 763 Mm^3 storage capacity, the reservoir provides water withdrawal up to 60 m^3/sec. to the open headrace canal (*Malka Wakana Detailed Project report, 1985*).

The topography of the Malka Wakana hydroelectric project is favourable for the construction of a diversion-type power station: the natural bed drop of the water falls and Considerable River gradients at the downstream stretch make it possible to create a high head at the station. The engineering geological conditions are rather complicated here: together with strong basalts the soft volcanogenic rocks are developed here. The natural water falls 80m and water conveying lines was create a head equal to 300 m at the power station. The installed capacity of the hydroelectric plant (HEP) is $153*10^6$ kW (4 generating units). The average annual power output at the HEP is $543*10^6$ kWh.

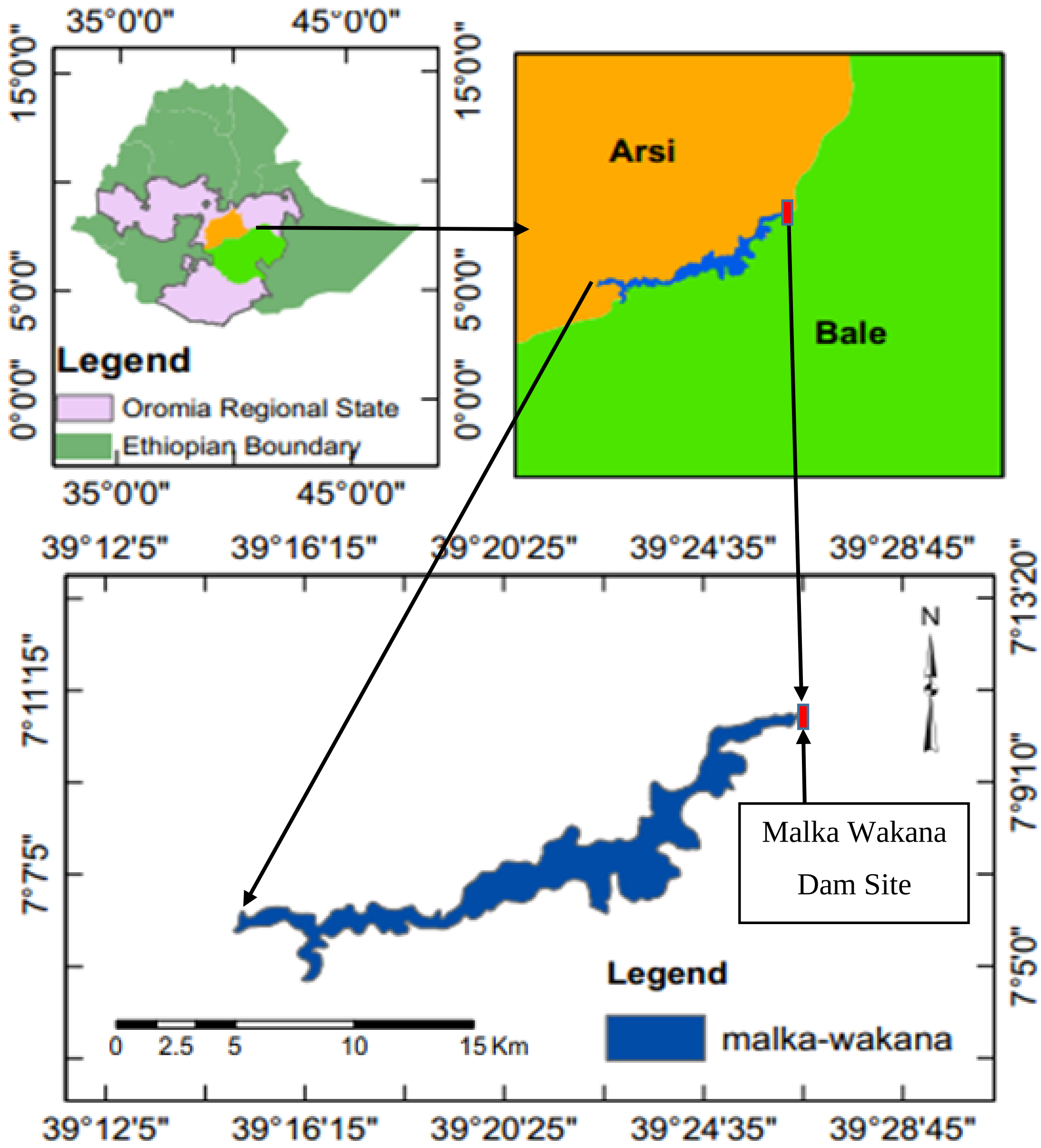


Figure 3.1: Location of the study area

Considering the topography and the foundation rock formation, an earth and rockfill dam section with an impervious clay core has been provided. A detailed resistivity and seismic refraction surveys have been carried out along the proposed dam axis (*Malka Wakana Detailed Project report, 1985*) as a result, unconsolidated sediments with a thickness of 4-5m and three types of basalt have been revealed. Along the dam axis the presence of fault zones have been indicated by seismic refraction, as well as resistivity profiling surveys. It has been found out that the basalts in this area have a high modulus of deformation. The more massive and hard basalts are over lain by semi-consolidated sediments varying in thickness from 0.5 to 25 m. These basalts considered as suitable for foundation purposes, with adequate grouting and filling up of cracks and weaker zones along the dam axis. The dam is proposed with a clay core and shells of sand-gravel materials and the rock. The head upon the structure is 31 m at normal head water level (NHWL) at EL. 2516 m and tail water level (TWL) at EL. 2485 m. The upstream face of the dam is of a 1:2.5 slope, while that of the downstream face is 1:2.2. The clay core with slopes of 10:1 serves as a cut-off structure of the dam. In the core bottom provision is made for concrete slab, 16m wide and 1m thick. Horizontal drainage at the downstream shell foot filled of the slag taken from the effective excavation (*Malka Wakana Detailed Project report, 1985*).

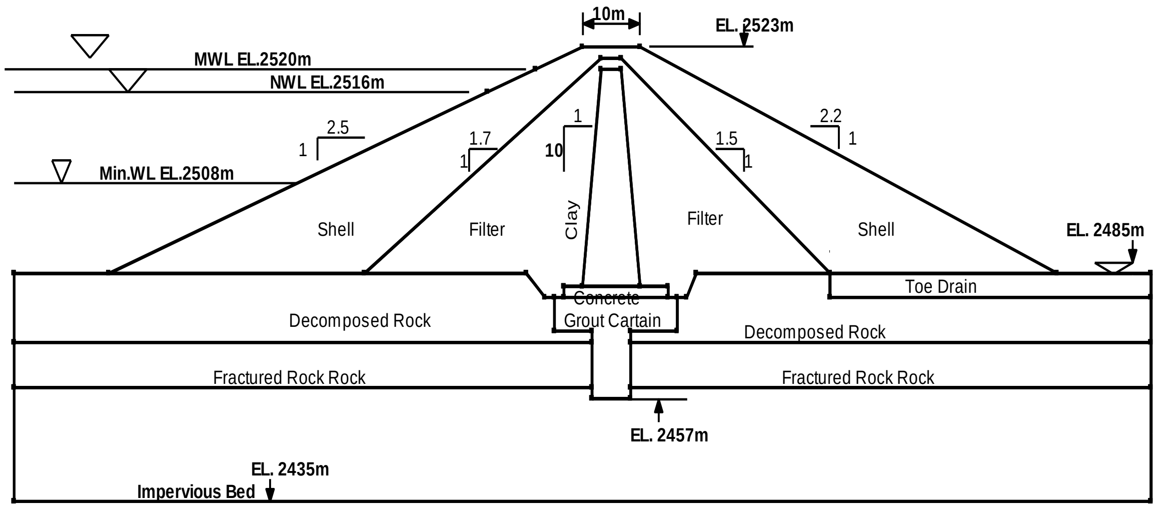


Figure 3.2: Malka wakana dam cross section

Source: (*Malka Wakana Detailed Project report, 1985*)

3.2 Method

Many different solution techniques for slope stability analyses have been developed over the years. Analyze of slope stability is one of the oldest type of numerical analysis in geotechnical engineering. In this study limit equilibrium method for stability analysis and finite element based method used for seepage analysis. Two modern geotechnical software programs are utilized, that is SLOPE/W and SEEP/W.

3.2.1 Slope stability evaluations

The case study slopes were evaluated by LE method, the Computer software SLOPE/W. the basic principles and review of the LE methods are described in section 3.2.1.1.

The aim of the study was not only to evaluate the stability conditions, but also to evaluate the seepage through the cross section of the dam. Moreover, the study aims to compare the selected LE methods that are commonly used in practice. The study focuses on the effect of reservoir water drawdown variations in rapid and slow draw down. First, the steady state analysis based on full of reservoir water level and second, the rapid and slow drawdown of the reservoir water level are carried out respectively.

3.2.1.1 Limit Equilibrium Method

The limit equilibrium method of analysis for static slopes is still the most widely used tool to analyze the stability of a given soil slope. It considers a soil continuum of different strata, and given a particular failure surface in the form of lines or arcs, a "Factor of Safety" is found through the application of force or moment equilibrium. The factor of safety is defined as the ratio of the resisting force to the driving force or resisting moment to the driving moment. So if a particular failure surface has a Factor of Safety (FS) of 1, then it is at the "limit" of equilibrium assumptions. A Factor of Safety less than 1 means that the driving forces are greater than the resisting forces and the slope will fail either in rotation, translation, or a combination thereof *(Krahn, 2004)*.

Analysis of slopes has traditionally been carried out by limit equilibrium methods, which are based on the principles of static equilibrium of forces and moments. According to *(Fredlund and Rahadjo, 1993)*, LE methods are important mainly because of two reasons. First, the methods have proved to be reasonably reliable in assessing the stability of slopes. Second, the methods require a limited amount of input, but can quickly perform an extensive trial and error search for the critical shear surface (CSS). However, *(Krahn, 2003)* says, "LE methods are missing the fundamental physics of stress strain relationship, and thus they are unable to compute a realistic stress distribution". In spite of these limitations, the LE methods are still common in practice because of their simplicity and the reasonably accurate FS obtained.

There are several methods for computation of FS from a particular sliding mass. The methods most commonly used in practice and gives an overview of their use cases and assumptions as follows.

a) Ordinary or Fellenius method

This method is also sometimes referred to as the Swedish method of slices. In this method, all inter-slice forces are ignored. The slice weight is resolved into forces parallel and perpendicular to the slice base. The force perpendicular to the slice base is the base normal force, which is used to compute the available shear strength. The weight component parallel to the slice base is the gravitational driving force. Summation of moments about a point used to describe the trial slip surface is also used to compute the factor of safety. The factor of safety is the total available shear strength along the slip surface divided by the summation of the gravitational driving forces or mobilized shear *(Krahn, 2004).*

The simplest form of the Ordinary factor of safety equation in the absence of any pore-water pressures for a circular slip surface is *(Janbu 1954, Nash, 1987; Aryal, 2006):*

$$FS = \frac{\sum\left(c\ l + N \tan\phi\right)}{\sum W \sin\alpha} \qquad 3.1$$

Where: c = cohesion, l = slice base length, N = base normal (W cos α), ϕ = friction angle, W = slice weight, and α = slice base inclination.

b) Bishop's simplified method

This method is suggested by Professor Bishop in 1950's. The method is a modified version of the ordinary method of slice and the normal forces between inter slices are included. But Bishop did not include the shear forces between the slices and developed a new equation for the factor of safety. The new equation was a non-linear equation because, the normal force between two slices has obtained using the factor of safety hence, the equation contains the variable factor of safety (FS) in both side. Therefore an iterative method is compulsory to solve the equation. The final equation which Bishop derived is *(Bishop, 1950; Krahn, 2004).*

$$FS = \frac{1}{\sum_{i=1}^{n} W_i \sin\alpha} \sum_{i=1}^{n} \left(\frac{c_i l_i + (W_i \cos\alpha - \frac{c_i l_i \sin\alpha}{FS}) \tan\phi_i}{m_a} \right) \qquad 3.2$$

In the equation Bishop has included a new term m_α and defined as;

$$m_\alpha = \cos\alpha + \frac{\sin\alpha . \tan\phi_i}{FS} \qquad 3.3$$

c) Janbu's simplified method

The Janbu's Simplified method is similar to the Bishop's Simplified method except that the Janbu's Simplified method satisfies only overall horizontal force equilibrium, but not overall moment equilibrium *(Krahn, 2004).*

d) Spencer method

Spencer (1967) developed two factor of safety equations; one with respect to moment equilibrium and another with respect to horizontal force equilibrium. He adopted a constant relationship between the interslice shear and normal forces, and through an iterative procedure altered the interslice shear to normal ratio until the two factors of safety were the same. Finding the shear-normal ratio that makes the two factors of safety equal, means that both moment and force equilibrium are satisfied *(Krahn, 2004).*

e) Morgenstern-price method

Morgenstern and Price (1965) developed a method similar to the Spencer method, that it satisfies both force and moment equilibriums and assumes the interslice force function but they allowed for various user specified inter-slice force functions. The inter-slice functions available in SLOPE/W for use with the Morgenstern-Price (M-P) method are: Constant, Half-sine, Clipped-sine, Trapezoidal and Data-point specified *(Krahn, 2004).*

According to M-PM (1965), the interslice force inclination can vary with an arbitrary function (f(x)) *as (Nash, 1987; Aryal, 2006)*:

$$S = f(x) * \lambda * H \tag{3.4}$$

Where, f(x) = interslice force function that varies continuously along the slip surface, λ = scale factor of the assumed function, S = interslice shear force and H = interslice normal forces and u = pore pressure. For a given force function, the interslice forces are computed by iteration procedure until, force equilibrium FS (F_f) is equals to moment equilibrium FS (F_m) in Equations 3.5 and 3.6 *(Nash, 1987; Aryal, 2006).*

$$F_f = \frac{\sum [\{c'l + (N - ul)\tan\phi'\}\sec\alpha]}{\sum \{W - (S - S')\}\tan\alpha + \sum (H - H')} \tag{3.5}$$

$$F_m = \frac{\sum (c'l + (N - ul)\tan\phi')}{\sum W \sin\alpha} \tag{3.6}$$

The forces considered are shown in Figure 3.3.

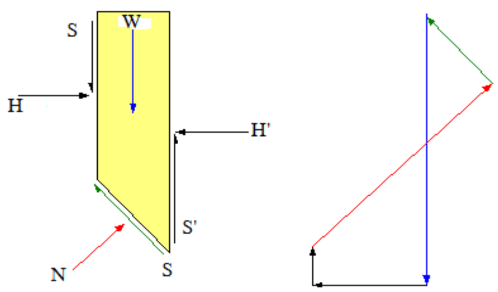


Figure 3.3: Free body and force polygon for morgenstern-price method

3.2.1.2 Selected method for analysis

The most vigorous LE methods, Morgenstern-Price method was selected for analysis due to satisfying the force and moment equilibrium factor of safety equations and considering inter-slices normal and shear forces. In addition, Spencer, Bishop's simplified (BS) and Janbu's Simplified (JS) methods were chosen due to their common use in practice to compare the FS obtained from Morgenstern-Price method.

Further details about all the methods are presented in Table 3.1.

Table 3.1: Equations of statics satisfied

Method	Moment Equilibrium	Force Equilibrium
Ordinary or Fellenius	Yes	No
Bishop's Simplified	Yes	No
Janbu's Simplified	No	Yes
Spencer	Yes	Yes
Morgenstern-Price	Yes	Yes
Corps of Engineers-1	No	Yes
Corps of Engineers-2	No	Yes
Lowe-Karafiath	No	Yes
Janbu Generalized	Yes	Yes
Sarma-vertical slices	Yes	Yes

Source: *(krahn, 2004).*

Table 3.2: Equations of statics satisfied

Methods	Inter-slice normal (H)	Inter-slice shear (S)	Inclination of S/H resultant and S-H relationship
Ordinary or Fellenius	No	No	No inter-slice forces
Bishop's Simplified	Yes	No	Horizontal
Janbu's Simplified	Yes	No	Horizontal
Spencer	Yes	Yes	Constant
Morgenstern-Price	Yes	Yes	Variable; user function
Corps of Engineers-1	Yes	Yes	Inclination of a line from crest to
Corps of Engineers-2	Yes	Yes	Inclination of ground surface at top of slice
Lowe-Karafiath	Yes	Yes	Average of ground surface and slice base inclination
Janbu Generalized	Yes	Yes	Applied line of thrust and moment equilibrium of slice
Sarma-vertical slices	Yes	Yes	$S = C + H \tan\phi$

Source: (*Krahn, 2004).*

3.2.1.3 Selected input parameters

The shear strength parameters obtained from malka wakana detailed project report (the tests for material done by "Hydro-project" scientific and research center) were selected as in put parameters for the stability analysis. The selected input parameters used in the stability evaluations are summarized in Table 3.3. The dam cross sections, upstream and downstream slope of the dam, the upstream water level in the reservoir and downstream water level, hydraulic conductivity and water content of the soil are used as input parameters.

3.2.2 Modeling

The strength parameters c and ϕ can be total strength parameters or effective strength parameters. SLOPE/W makes no distinction between these two sets of parameters. Which set is appropriate for a particular analysis is project- specific, from a slope stability analysis point of view, effective strength parameters give the most realistic solution, particularly with respect to the position of the critical slip surface (*Duncan, et.al. 2014)* . For this study the strength parameters discussed in Table 3.3 are used.

Table 3.3: Input parameters for slope stability

Soil Description	γ (kN/m^3)	γ_S (kN/m^3)	c' (kPa)	ϕ' (°)	Hydraulic Conductivity m/day	Vol. water content
Sand – Gravel	20	21	5	35	10	0.4
Transition(sand-gravel)	19	20	10	31	5	0.45
Clay	15	18	30	11	0.001	0.5
Decomposed rock	23	24	20	24	0.2	0.6
Fractured rock	27	27	10	35	0.2	0.8
Bed rock	28	28	0	40	0.02	0.9

Source: (*Malka Wakana Detailed Project report, 1985)*

3.2.2.1 Simplified slope models

The dam cross section were modelled with nine regions including the foundation as shown in figure 3.2 the outer part of the dam both upstream and downstream shell, transition zone, central core and foundation with three layer of rocks and below the core of the dam there is region of concrete for one meter depth below the core and grout curtain below the central core up to the bed rock depth and a stiffer layer at the base.

3.2.3 Software used

3.2.3.1 SLOPE/W

SLOPE/W is the most common and popular software application which used for the stability analysis of slope. This application is created based on limit equilibrium method and included several types of methods like Bishop, Janbu, Spencer and Morgenstern-Price methods. The stability analysis using SLOPE/W is included following components *(Krahn, 2004).*

1. Drawing geometry
2. Defining soil properties and assigning for the corresponding soil layer
3. Defining the water table
4. Selection of analysis method
5. Problem solving and display the results

The results of stability analysis from the SLOPE/W can be obtained as both visuals and numbers. The visually interpreted results make it possible to easily understand of the results in numbers. The very important advantage of the SLOPE/W analysis is it allows handling all possible slides in a same model with the corresponding factor of safety. In SLOPE/W it is possible to extract individual slip surfaces and their properties. When a particular slip surface selected, the corresponding factor of safety will be displayed. (*Krahn, 2004)*.

The input parameters, were used to search and refine the circular CSS in SLOE/W. the entry and exit search option was used to identify the CSS, and this was verified by the auto-locate option. The Mohr-coulomb soil model, together with a half sine function for interslice forces were selected. Moreover, the minimum FS was computed based on assumption of 30 numbers of slices, no tension cracks and no optimization of the circular CSS.

3.2.3.2 SEEP/W

SEEP/W is a numerical modeling software which used to solve the practical seepage problems. This is a part of the most popular geotechnical software called GeoStudio. The SEEP/W program is created with the combination of seepage theory and finite element method and working on saturated/unsaturated soil region (*Krahn, 2004*).

SEEP/W is a finite element CAD software product for analyzing groundwater seepage and excess pore-water pressure dissipation problems within porous materials such as soil and rock. Its comprehensive formulation enables to consider analyses ranging from simple, saturated steady-state problems to sophisticated, saturated/unsaturated timedependent problems. SEEP/W can be applied to the analysis and design of geotechnical, civil, hydrogeological, and mining engineering projects *(Krahn, 2004)*

The practical seepage problems are never easy to convert into a numerical modeling because of the heterogeneity of the natural soils and the varying boundary condition. Generally the boundary conditions for a seepage problem never being as same as found in the initial stage. Therefore the seepage analysis in SEEP/W program is divided into two categories.

3.2.4 Analysis type

There are two fundamental types of seepage analysis: steady state and Transient seepage analysis.

1.Steady-state analysis

In the steady state the fundamental water flow properties such as water pressure and water flow rates never going to be changed. Since steady state analysis ignore the time domain, it greatly simplifies the equations being solved. Practically achieving steady state is impossible. The purpose of the steady-state analysis is only to know how the initial input parameters respond to a given boundary condition.

This analysis never state that how long it takes to reach a steady state. It returns a set of solved values for water pressures and water flow parameters for particular boundary conditions. A constant pressure H and a constant flux rate Q are the important boundary conditions used for a steady-state analysis.

2.Transient analysis

Transient analysis is used to know how long the embankment takes to responds for a given boundary condition. Therefore the fundamental flow properties, pressures and water flow rate will vary with time. In general, a transient analysis can provide more accurate results when soil conditions are modeled, however, they are significantly more complicated than steady state analysis. The analysis required an initial boundary condition as well as a destination boundary condition. If the initial or future conditions are not accurately represented, the analysis will provide inaccurate results. In general, there are two transient analysis were done in this study, these are rapid and slow drawdown depending on the time of reservoir drawdown.

When developing a numerical steady state and transient condition modeling using SEEP/W, one must determine geometry, assign materials, and assign boundary conditions, and draw flux section across the section to be want to view the seepage then after running the analysis it is possible to draw flux label.

3.2.5 Defining the problem

Seepage is believed to be the most important cause for failure of the embankment dam. Abnormal seepage conditions occurred during the intense rainfall and flooding effected significantly in the stability of the embankment slopes. Therefore, it is important that the stability and seepage analysis for the potential failure slopes with some extreme conditions. First of all, the embankment was analyzed for seepage and stability with full reservoir condition of upstream water level means for steady state condition. SEEP/W and SLOPE/W computer programs were used to analyze the seepage and stability conditions respectively. The maximum water level at the upstream side is at El. 2520 m, the normal water level is at El. 251 6m, the minimum water level is 2508m and the downstream tail water level is at El. 2485 m. this makes a large head difference between the upstream and downstream sides and which causes seepage through the embankment. The upstream boundary conditions are defined by the total head equal to the water level in the reservoir along the upstream slope and zero pressure at the downstream horizontal drain.

At start, a steady state analysis of seepage and corresponding stability analysis were carried out for the normal water level with the total head as a boundary condition. The pressure and water flow conditions obtained from the steady state analysis used as initial pore water pressure conditions for the transient analysis and for slope stability analysis.

The slope stability analysis was carried out using the SLOPE/W and the geometry was created in the SEEP/W program and transferred to SLOPE/W program by setting the model for steady state condition stability analysis. The SEEP/W steady state analysis is used as parent analysis for SLOPE/W. also for sudden drawdown slope analysis the transient seepage analysis is used as parent or initial pore water condition. Then the analysis type selected and slip surface drawn for slope stability analysis by using entry and exit and it follow a right to left path for upstream slope and for downstream slope it follow a left to right path. The Morgenstern-Price analysis and half-sine function for interslice forces were selected but for the comparison the software also gives the result of factor of safety for Spencer, Bishop and Janbu analysis type.

3.2.6 Slip surface for circular failure model

After the material input and pore pressure was assigned, a slip surface was defined. The analysis were performed for circular failure model, there were several methods for defining the slip surface for the circular failure but the entry and exit method was selected. One of the problems with the other methods is how to visualize the extents or the range of the trail slip surface. This difficulty is solved by the entry and exit method because it specifies the location where the trail slip surfaces should enter the ground surface and where should exit.

3.2.7 Verification and computation

In the seepage analysis when the material properties were defined and boundary conditions were specified and flux section drawn across the specified dam section, then SEEP/W runs to verify the input data using the verify data command in the Tools menu. When the verification is completed and there are no errors, then SEEP/W computes the amount of seepage across the dam at the specified section and show the zero pressure line in the dam body.

When the slip surface has been specified, then SLOPE/W runs several checks to verify the input data using the verify data command. When the verification is completed and there are no errors, then SLOPE/W computes the factor of safety using the method of slice selected. The minimum factor of safety is obtained for that particular analysis and its corresponding critical slip surface is displayed.

CHAPTER-4

RESULTS AND DISCUSSION

4.1. Steady state seepage and stability analysis

a. Steady state seepage analysis

The first critical condition to be analysed in this study is when the reservoir is full of water and some steady state seepage into the malka wakana earth and rock fill embankment dam is established. In this case, a Phreatic Surface under steady seepage state is present in the embankment dam body. The water table profile in the reservoir and seepage through the dam cross section were shown in a Figure 4.1.

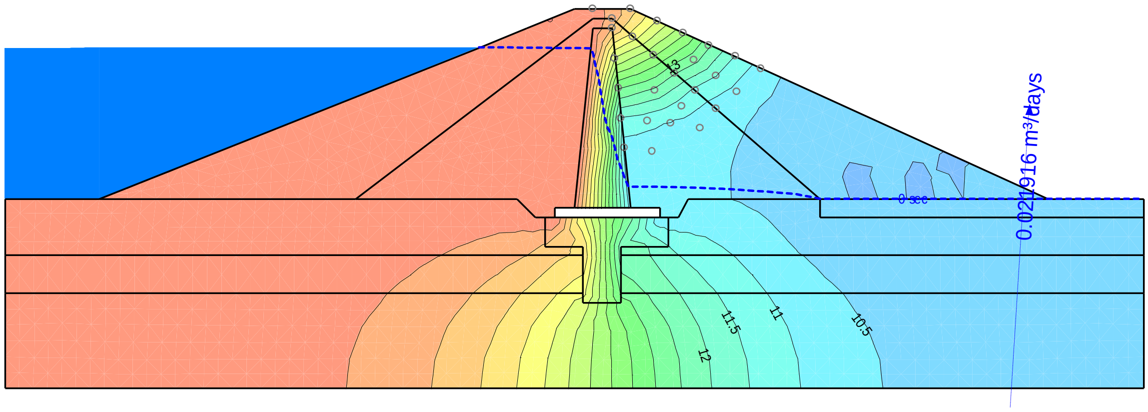


Figure 4.1: Steady state seepage analysis, showing the internal water phreatic surface

In the figure 4.1 the equipotential lines are showed by contours drawn automaticaly by the software, the red contours regions indicate that the pressure head is high at the upstram side and decreasing as it goes from upstream to downstream side of the dam.

Table 4.1: Results of steady state seepage analysis

	elevation of water (m)		
	2508	2516	2520
Seepage (m^3/day/m)	0.011	0.022	0.027
Exit gradient	0.12	0.25	0.35

In steady state analysis, the amount of seepage through the cross section is identified using flux section drawn across the dam and the calculated seepage value is equal to 0.022 m^3/day/m for normal operating water level. This is compared with the quantity seepage estimated in the design document that is 2.25m^3/day/m. Therefore, the design document has no problem of quantifying the expected quantity of seepage.

It can be noticed from Table 4.1 that the exit gradient was always less than 1.0, which means that the dam is safe in these conditions.

b. Slope stability analysis result for steady state condition

The malka wakana earth and rock fill embankment dam and its foundation was analysed against failure by slope instability. Considerations of loading conditions which may result to instability for all likely combinations of reservoir and tail water levels, seepage condition steady state loading conditions was examined in particular, as follows:

The steady state condition have been analysed when some steady state seepage into the malka wakana earth and rock fill embankment dam is established.

The overall minimum stability factor of safety for the steady state condition, i.e. when the reservoir is full of water and some steady state seepage into the malka wakana earth and rock fill embankment dam is established, was calculated equal to 1.985, which means the slope is stable under this condition as per the USACE and USBR the factor of safety for steady state condition is ≥1.5, for downstream slope. The computed analysis results are illustrated in Figure 4.2.

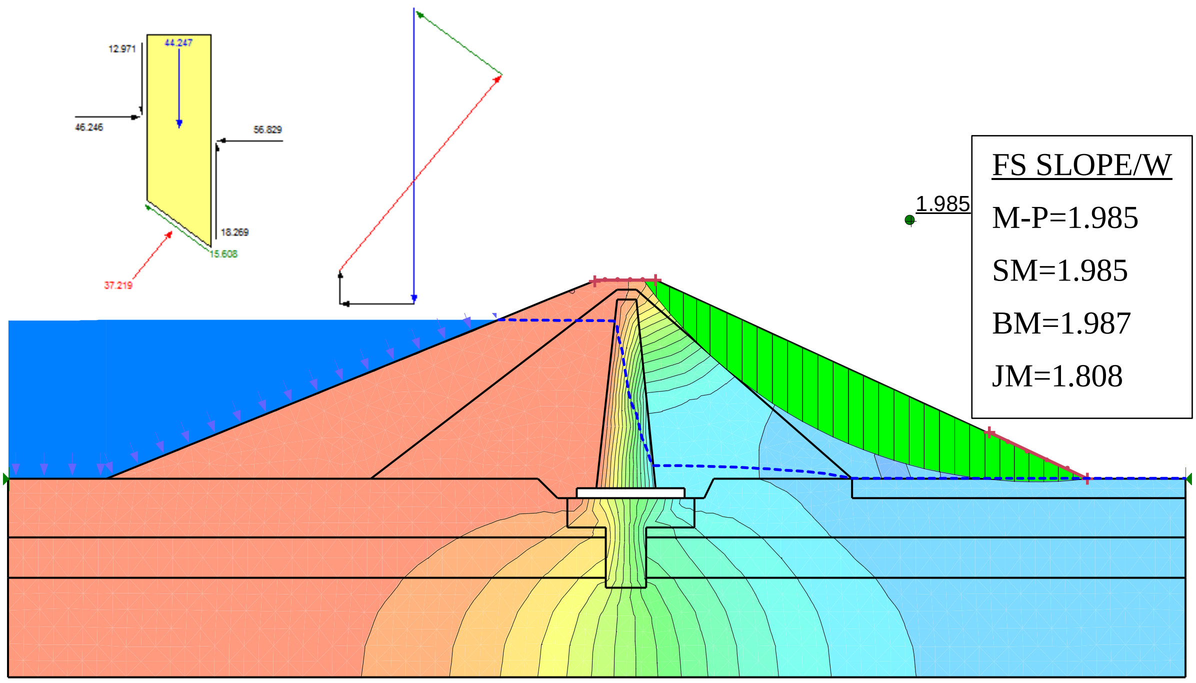


Figure 4.2: Slope stability analysis for steady state condition with free body diagram

The force polygon in Figure 4.2 indicate that the morgenstern-price method considering all forces between adjacent slices and weight of the slice. These forces are normal right and left side forces, right and left side shear forces, base normal force, base shear mobalise force and weight of the slice.

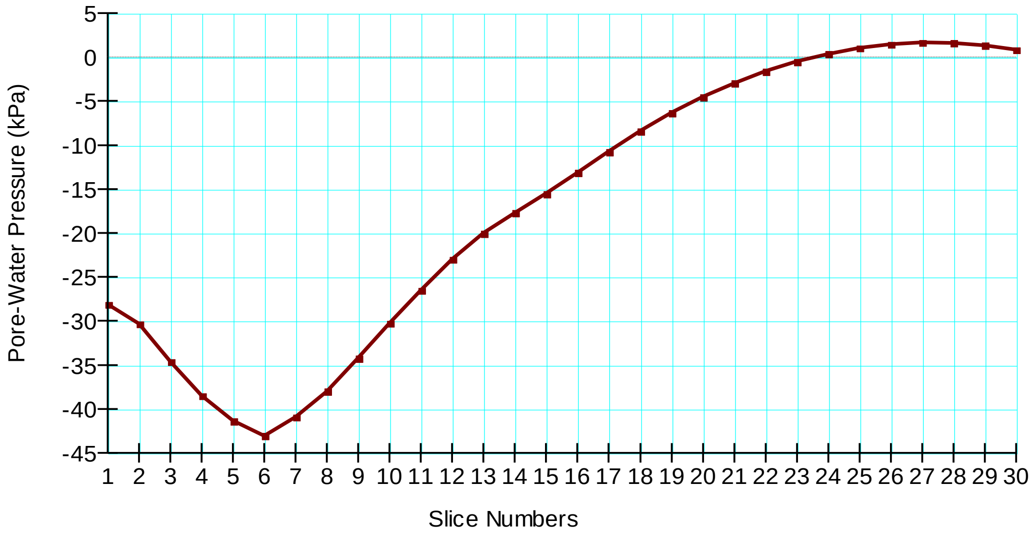


Figure 4.3: Graph of pore water pressure versus slice numbers

Figure 4.3 show that the pore water pressure with slice numbers, this graph indicate that how much the slices were responded to the pore water pressure. The slice with highly negative pore pressure are far from the water table, means from the line with zero pressure. The slices above the phreatic line are in negative pore pressure but the negativity increases from top to downstream face as phreatic line far from the downstream face due to the central core material of less hydraulic conductivity. After the phreatic line crossing the central core of the dam, the pore pressure was increased from negative to posative value. When slices were below the phreatic line, the pore water pressure show the positive value at the downstream toe drain.

Table 4.2: Comparison of factor of safety obtained for steady state condition

<table>
<tr><th>Method of slices</th><th>Factor of safety</th><th>Design document FS by manual calculation</th><th>D/S slope Standard FS</th></tr>
<tr><td>Morgnstern-Price</td><td>1.985</td><td rowspan="4">1.78</td><td rowspan="4">1.5</td></tr>
<tr><td>Spencer</td><td>1.985</td></tr>
<tr><td>Bishop</td><td>1.987</td></tr>
<tr><td>Janbu</td><td>1.839</td></tr>
</table>

In Table 4.2 and in figure 4.2 the results obtained from SLOPE/W for different method of analysis, Morgenstern-Price, Spencer, Bishop and Janbu methods. The values calculated show that even if the results are different between the methods, the slope is stable since the values are greater than FS recommended by the USACE and USBR for steady state load condition for downstream slope stability that is (>1.5). Therefore, the downstream slope of the malka wakana embankment dam is stable under steady state seepage condition.

4.2 Transient condition seepage and stability analysis

c. Rapid drawdown seepage analysis

Rapid Drawdown in the reservoir water level may cause the upstream face instability mainly due to the removal of the supporting water and also due to the development of the adverse seepage forces inside the embankment dam body during pore water pressure dissipation process. In this case, there is no water table present in the reservoir but in the embankment dam body there are still full pore water pressures. Effective or drained shear strength parameters of soils are used in this loading case.

In a particular analysis, the Geo-Slope program allows to import the results which obtained from another analysis result to define the functions as well as the boundary conditions. So, the transient analysis could be done on the steady-state analysis as the parent analysis. Therefore, the pressure head and the pore water pressure at each node which obtained from the steady-state analysis were transferred to the transient analysis as the boundary condition. The properties of the soil such as permeability and the volumetric water content which defined in the steady-state analysis also imported to the transient analysis.

The time duration for the analysis was defined as 90 days with 20 time steps and the time increment was selected as exponential manner. Every time step in the model was saved and the times which corresponding to the significant changes in the flow properties were taken as the results.

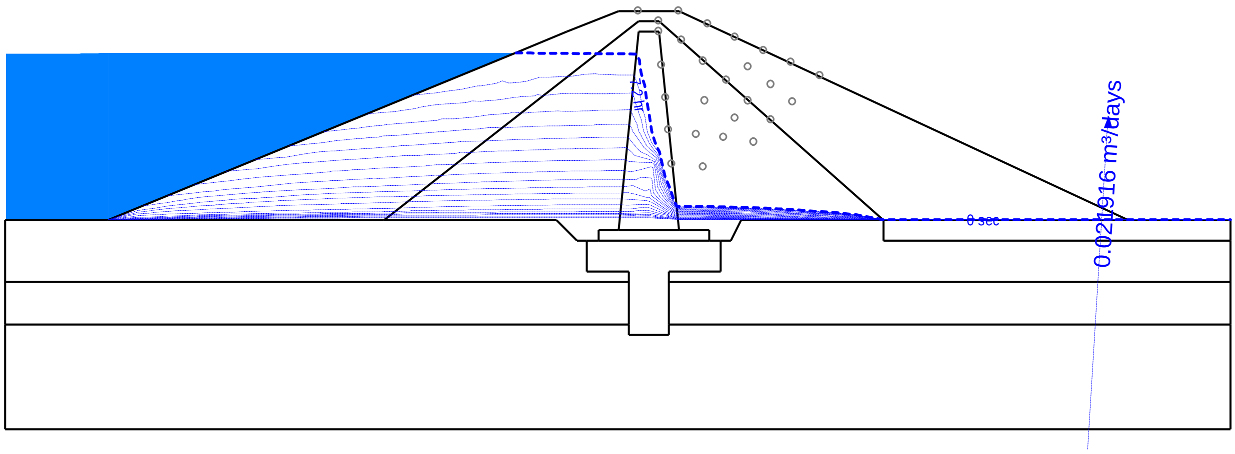


Figure 4.4: Transient analysis showing isolines of drawdown.

Figure 4.4 show the iso-lines for various time periods after the drawdown. The water table is decreasing in to lower position in assumed time period and the seepage through the dam also shows same variation as water table decreased. These changes happened because of the decreasing head of water level at the upstream side. These decrease in seepage with time period is indicated in Figure 4.5.

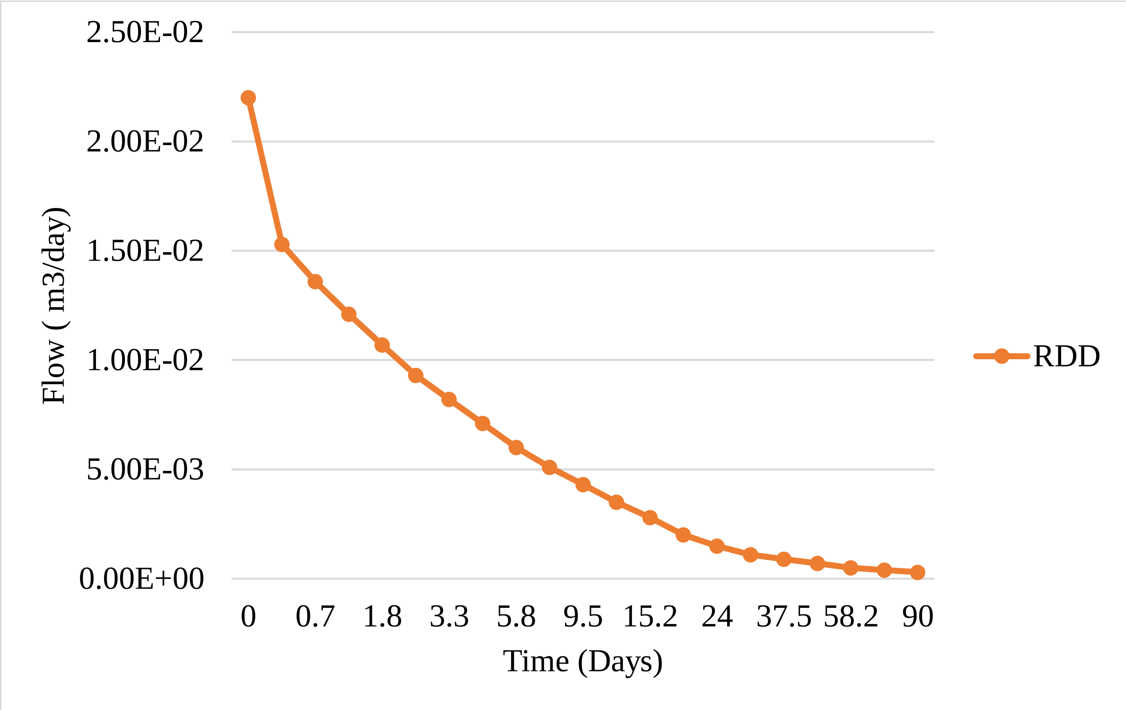


Figure 4.5: Seepage through the dam recorded at the downstream toe of the dam

The Figure 4.5 show the decreasing of the seepage through the dam cross section due to the drawdown of the water in the reservoir. The seepage is high at the initial time but as the time of drawdown was increased the seepage also decreased continuously up to the end of the analysis.

After 7 hours of drawdown, the phreatic line assumed just below the water table which corresponds to the initial condition. After a certain period it reached a maximum drawdown which the flux and water table show a small variation.

The SEEP/W program helps to analyze the various pressure conditions, flow conditions and the change in the material properties at any point or region of the embankment. The pressure condition could be analyzed in different forms such as total pressure, pressure head, pore water pressure and the hydraulic gradient separately. Here, some nodes from the upstream of the dam have been selected for the analysis. When the water is moved from normal water level to the bottom of the reservoir what the pore water pressure, total head and volumetric water content are look likes below in Figure 4.7 - 4.9.

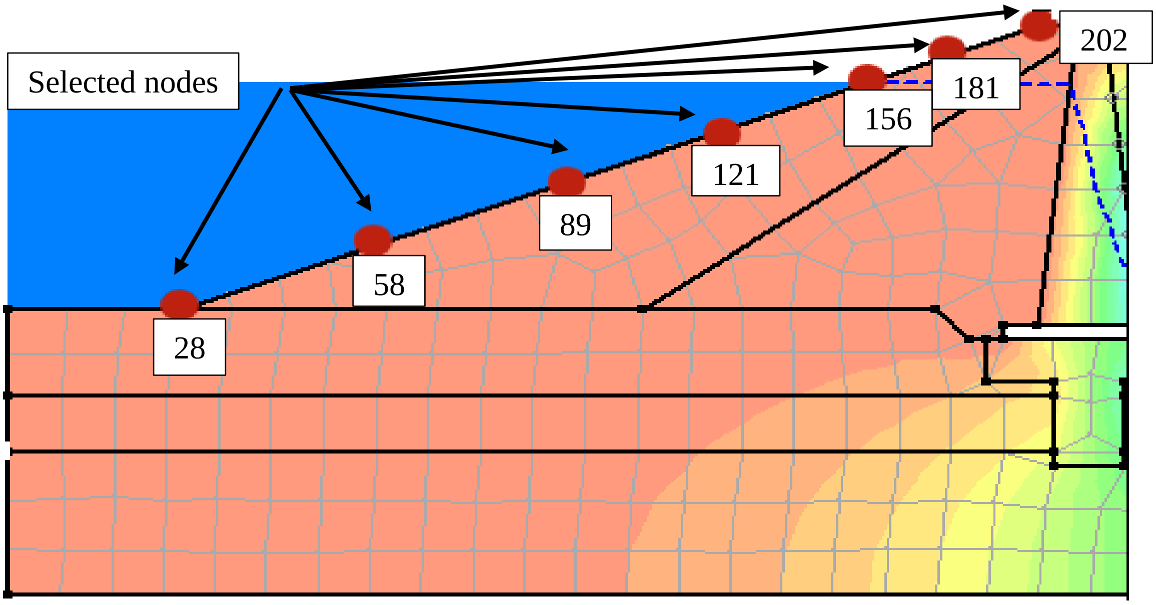


Figure 4.6: Selected nodes on the upstream slope of the dam

Figure 4.6 shows the nodes on the upstream slope of the dam which are selected for the analysis of pressure variation. SEEP/W program allow generating the graphs with distance and time as independent variables.

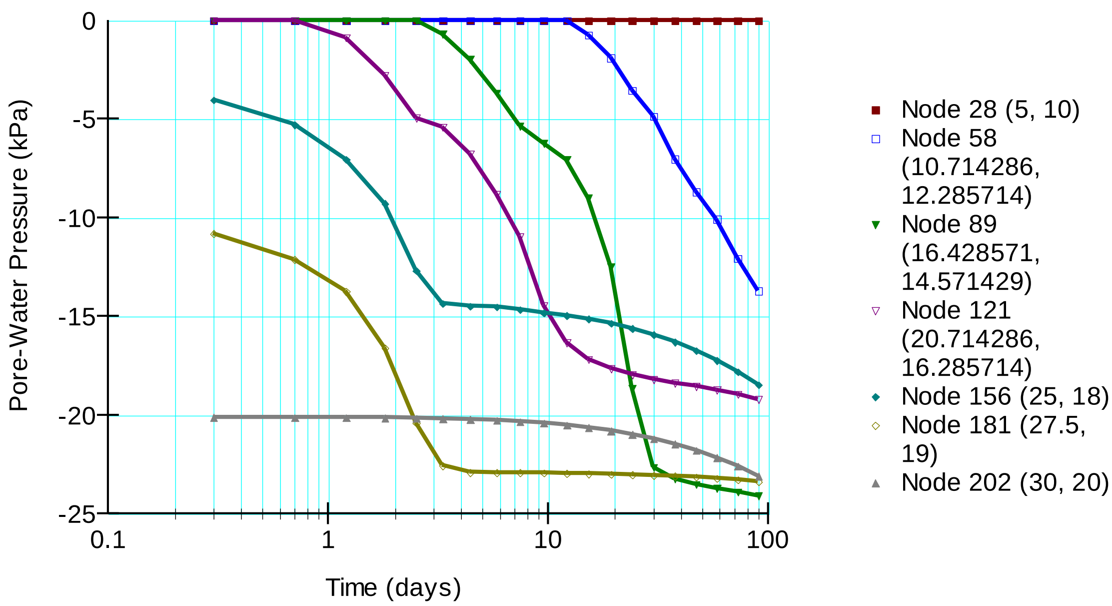


Figure 4.7: Pore water pressure changes versus time of rapid drawdown

Pore water pressure changes with time in selected 7 nodes on the upstream slope of the dam as shown in Figure 4.7. Pore water pressure changes in some nodes in similar way. The nodes at the bottom of the geometry (node 28) has zero pore water pressure, because the node is at the level of reservoir drawdown, so it assumed that the pore pressure is zero at this point. The node at the top of water table or the node above normal water level (node 202) almost straight line this indicate that the pressure head is not change with time, it is highly negative. But the drawdown of the reservoir caused the other nodes pore pressure to be fall from high to low level. The pore water pressure which corresponding to these nodes would be changed with time of the analysis and arrange them according to the degree of saturation at each nodes, these are nodes (58, 81, 110, 156 and 181). Therefore, as indicated in Figure 4.7, the reservoir is empty within a few hours but the pore water pressure take the time to be dissipated from the body of the dam, this reduces the shear strength of the embankment materials.

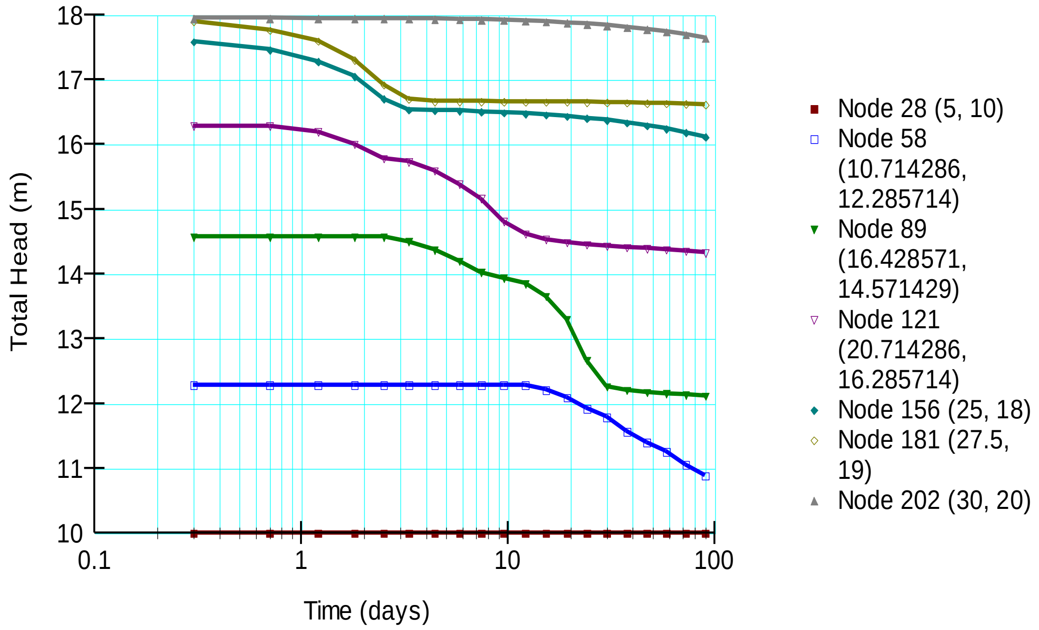


Figure 4.8: Total head changes versus time of rapid drawdown

Figure 4.8 shows the change of total head with time at same nodes on the upstream face of the dam. The node at the top of water table or the node above zero pore water pressure (node 202) is almost straight line this indicate the head is not change with time. Also the node at the bottom node (28) is straight line because the node is saturated at all times. At all other nodes, the total pressure head continuously decreases with time until the end of analysis. Another important thing to be analyzed in the SEEP/W program is material properties.

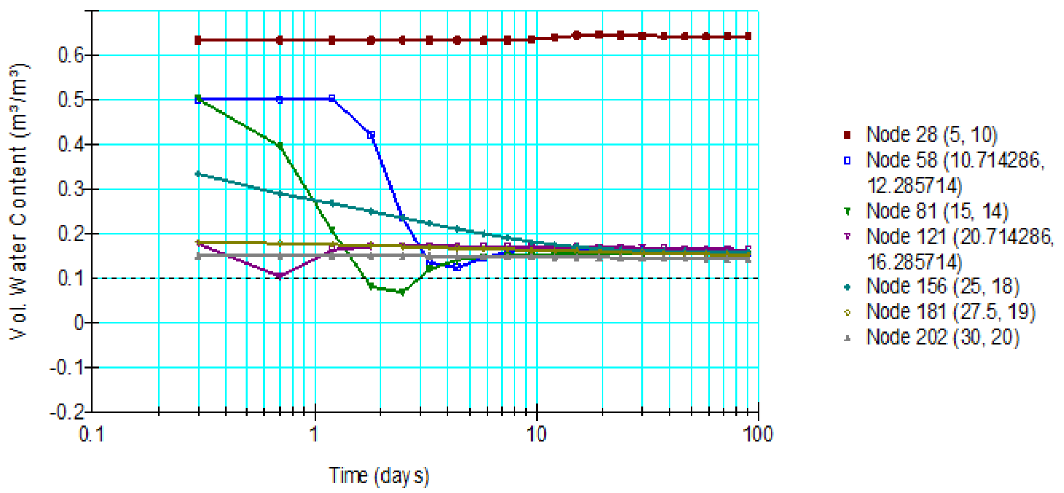
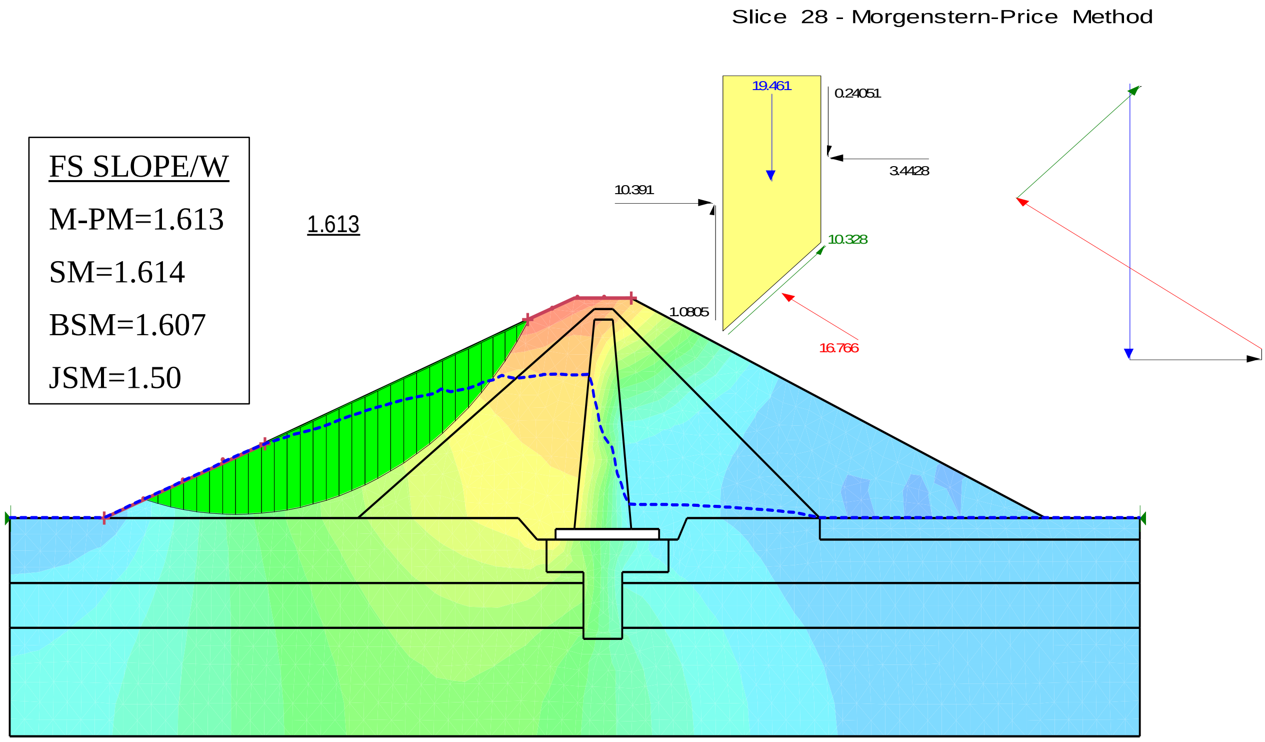


Figure 4.9: Change of volumetric water content with time of rapid drawdown

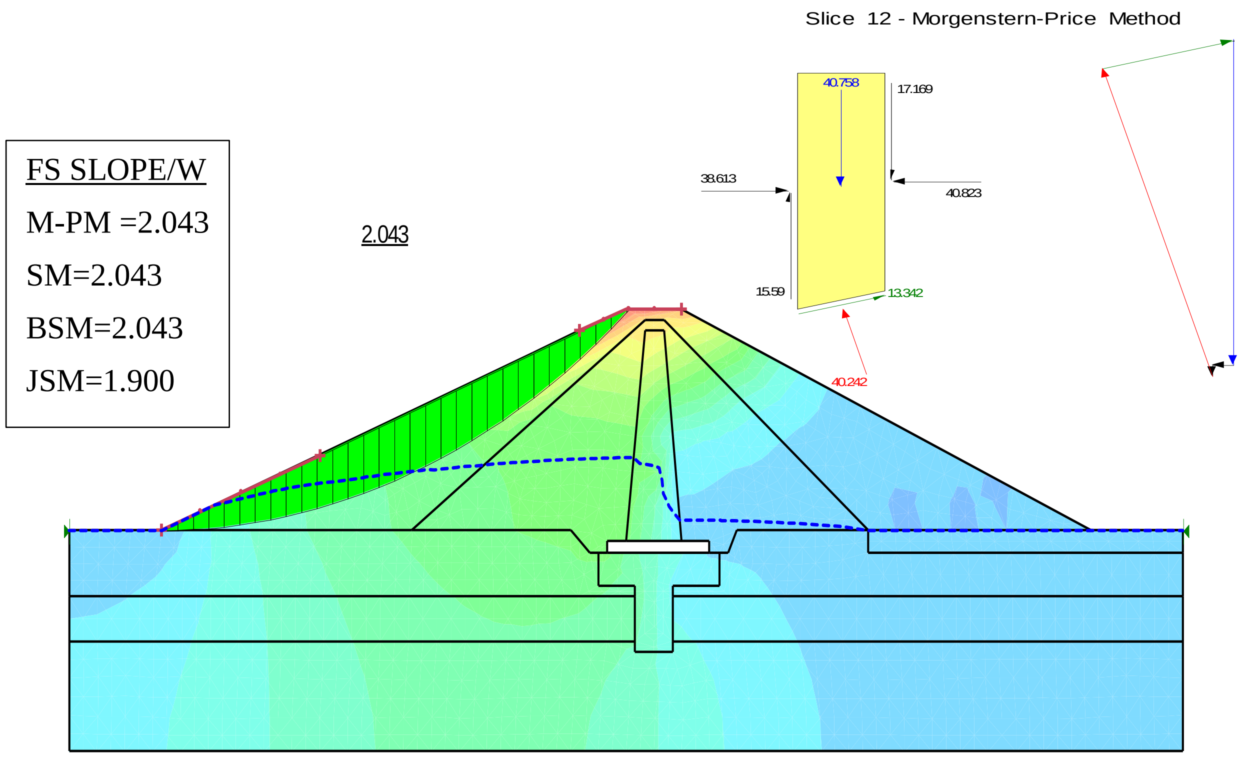
The volumetric water content is one of the most important material properties in a seepage problem. Figure 4.9 shows the change of volumetric water content with time on the selected nodes. Node 28 are representing the soil which have higher volumetric water content. Because the node always lay below the water table, hence it is fully saturated. Similarly, the nodes 202, 181 and 156 are always above the water table and hence, partially saturated. A large changed observed in the nodes 58, 81 and 121; because initially the water table is above the nodes at a time and the soil is fully saturated. But after the drawdown the water table is below the nodes this made the nodes partially saturated. Due to this reason the volumetric water content of these nodes are showing the high variation with time.

d. Stability analysis result for rapid drawdown condition

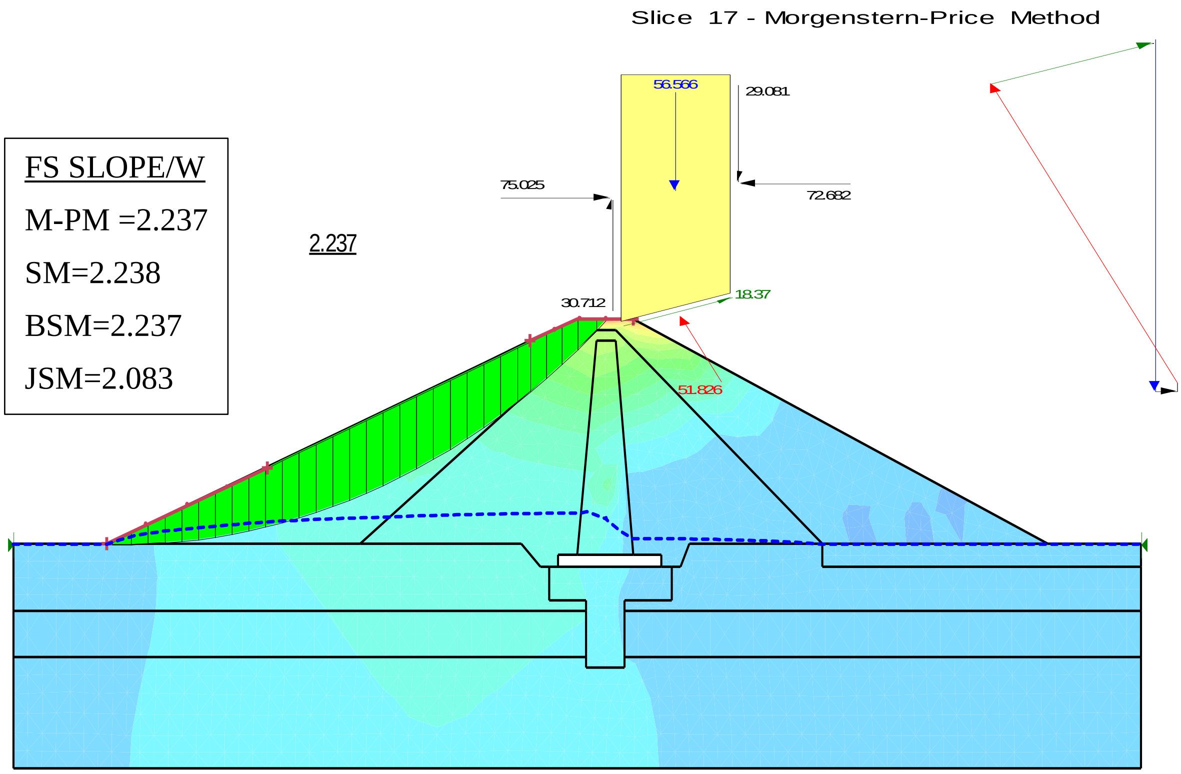
The drawdown of the reservoir is occurred due to different reasons among these, the drawdown due to unusually high water use demands, drawdown for the emergency release of the reservoir and drawdown for construction modifications are some of them. The stability has been analyzed for 90day of drawdown with 20 time steps and the results of five different time periods after the transient (Rapid drawdown) analysis are shown in Figure 4.10. They are 7.2hours, 2 days and 12 hours, 9 days and 12 hours, 30 days and 90 days (last time period). All the analysis carried out based on Morgenstern-Price method and compered with Spencer, Bishop simplified, and Janbu simplified method. The factor of safety was obtained for each analysis with keeping the same slip surface for all analysis. Stability conditions for selected time period are shown in Figures 4.10 (a)-4.10(e).



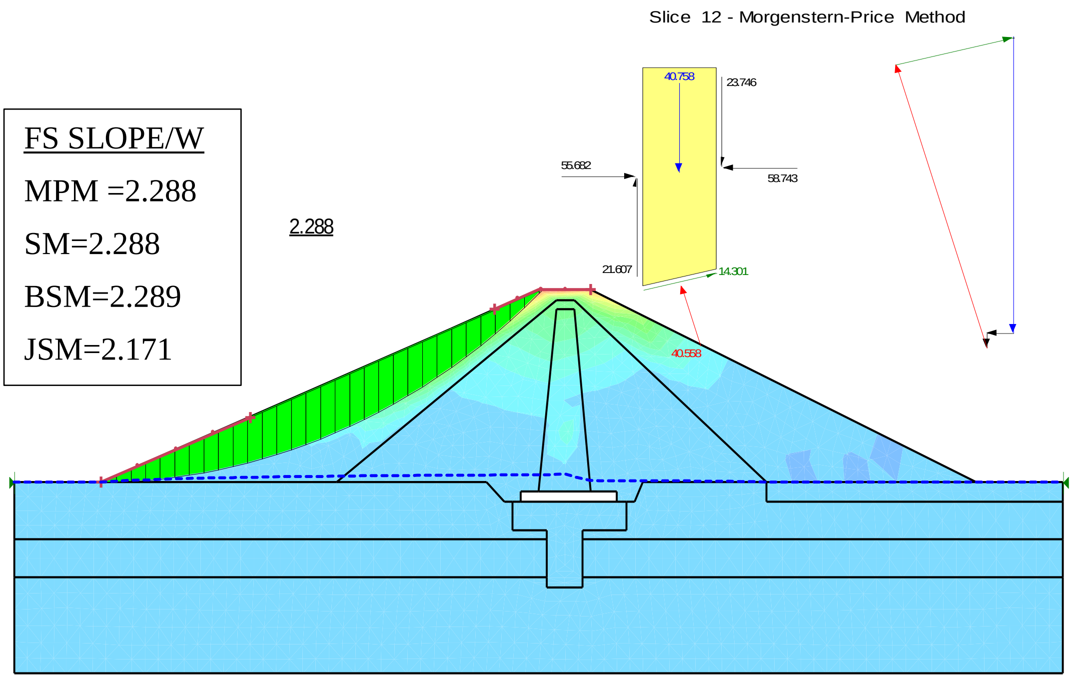
a) Stability analysis after 7.2 hours critical shear surface and slice free body diagram with force polygon



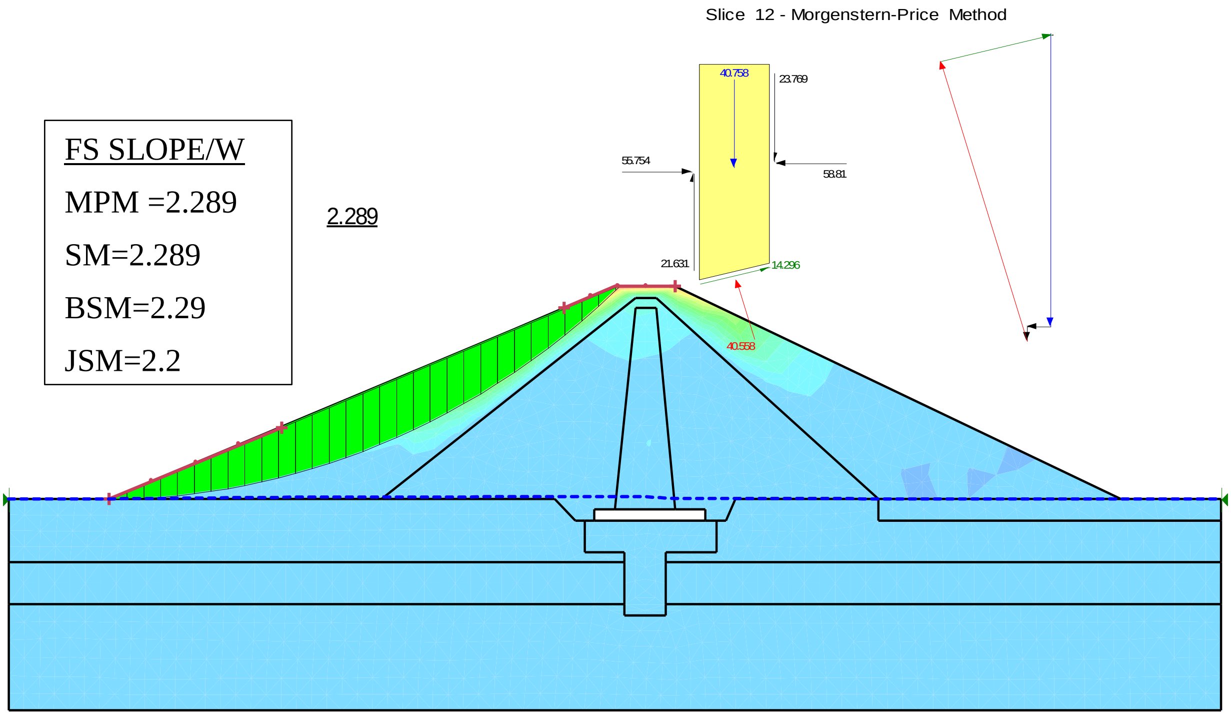
b) Stability analysis after 2 days and 12 hours critical shear surface and slice free body diagram with force polygon



c) Stability analysis after 9day and 12 hours, critical shear surface and slice free body diagram with force polygon



d) Stability analysis after 30 days, critical shear surface and slice free body diagram with force polygon



e) Stability analysis after 90 days critical shear surface and slice free body diagram with force polygon

Figure 4.10: Critical shear surface and slice free body diagram with force polygon

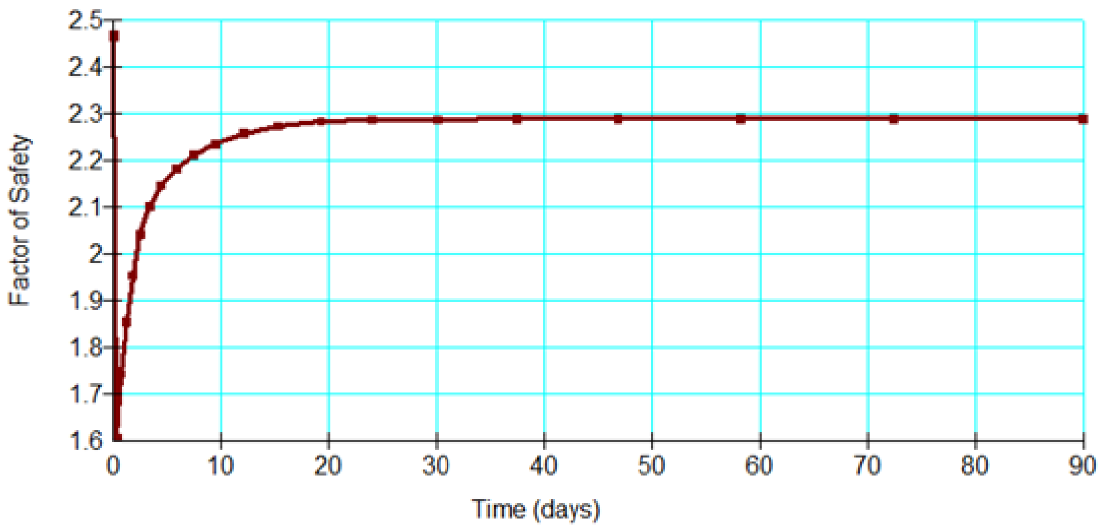


Figure 4.11: Factor of safety versus time of rapid drawdown

The factor of safety is decreasing for initial drawdown time, then it is increased until the end of analysis for the other time steps.

As indicated in the Figure 4.11 the factor of safety is initially high, for the decreasing of the first drawdown time the factor of safety is decreasing dramatically but not less than the factor of safety recommended by USACE for drawdown stability standard which is (>1.3); then after the first drawdown time step the factor of safety is increasing with the time steps until the end of analysis. The results show that the slope is stable throughout the transient analysis or for rapid drawdown of the reservoir. But at the beginning of the drawdown at time 7.2 hours the factor of safety is 1.613, this indicates that initially pore water pressure is high in the body of the dam when the water level is reduced, the pore water pressure takes time to dissipate from the dam. This reduce the shear strength of the soil and the saturation of soil reduces the frictional strength. But when the time elapsed was increased the pore water pressure is dissipated from the soil the shear resistant of the soil is increased and the factor of safety also high. Figure 4.10 (a) – 4.10 (e) also show the rapid drawdown of the reservoir is decreasing the factor of safety and when the time of drawdown is elapsed the factor of safety increases. In Figure 4.10 (a) at 7.2 hours' time of drawdown the FS is 1.613, this small when comparing with other FS in Figure 4.10 (b) – Figure 4.10 (e), it is increasing with increasing time of drawdown up down. But after almost 30 days of drawdown the graph in Figure 4.11 show horizontal line due to the same FS obtained since the reservoir is almost empty.

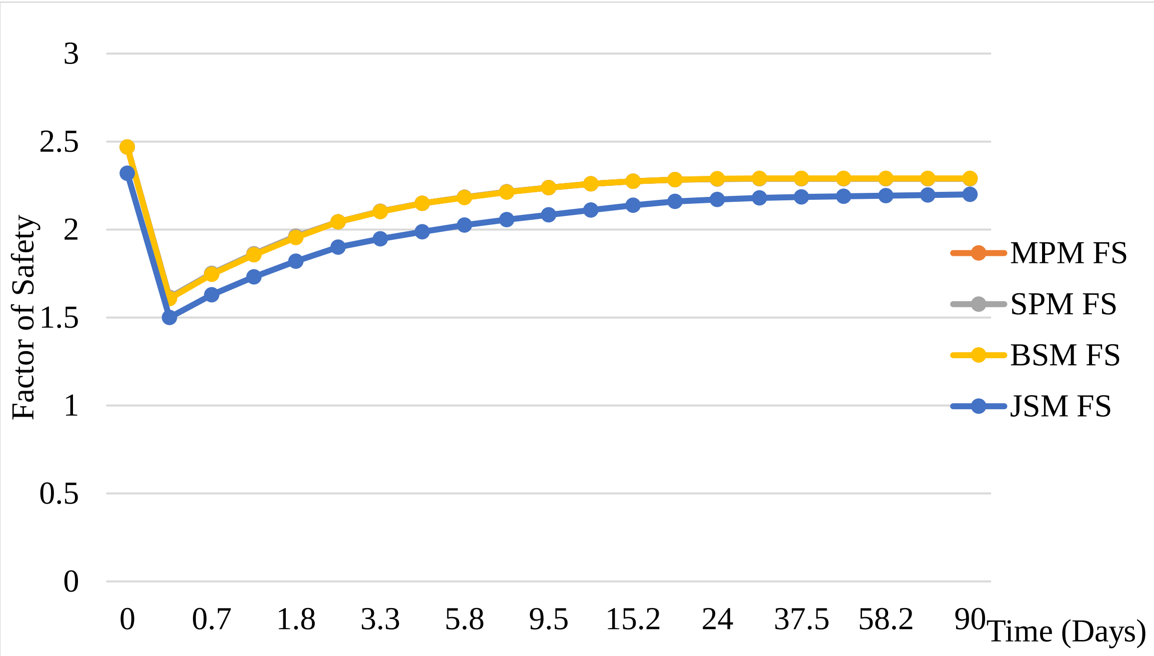


Figure 4.12: Factor of safety versus time of rapid drawdown

As shown in Figure 4.12, the curve of the graph are changed sharply after 7.2 hours. This indicate that the drawdown of the reservoir is reducing the factor of safety since it is rapid drawdown after 7.2 hours the reservoir is assumed empty. After this time means 7.2 hours the factor of safety is increasing since the slope materials gain its strength back and the factor of safety also increased until the end of analysis due to the pore water pressure is dissipated from the dam body as time of drawdown is elapsed. Eventually, the slope is stable since the factor of safety are greater than the minimum required factor of safety stated by USBR and USACE which is (>1.3) for upstream slope stability under drawdown of reservoir.

e. Slow drawdown seepage analysis

The slow drawdown analysis could be done on the steady-state analysis as the parent analysis means the initial water level is transferred from steady state analysis. Therefore the pressure head and the pore water pressure at each node which obtained from the steady-state analysis are transferred to the slow drawdown analysis as the boundary condition. The properties of the soil such as permeability and the volumetric water content which defined in the steady-state analysis also imported to this analysis. The only difference is the variation of FS depend on the time. The material properties are the same for steady state, rapid drawdown and slow drawdown

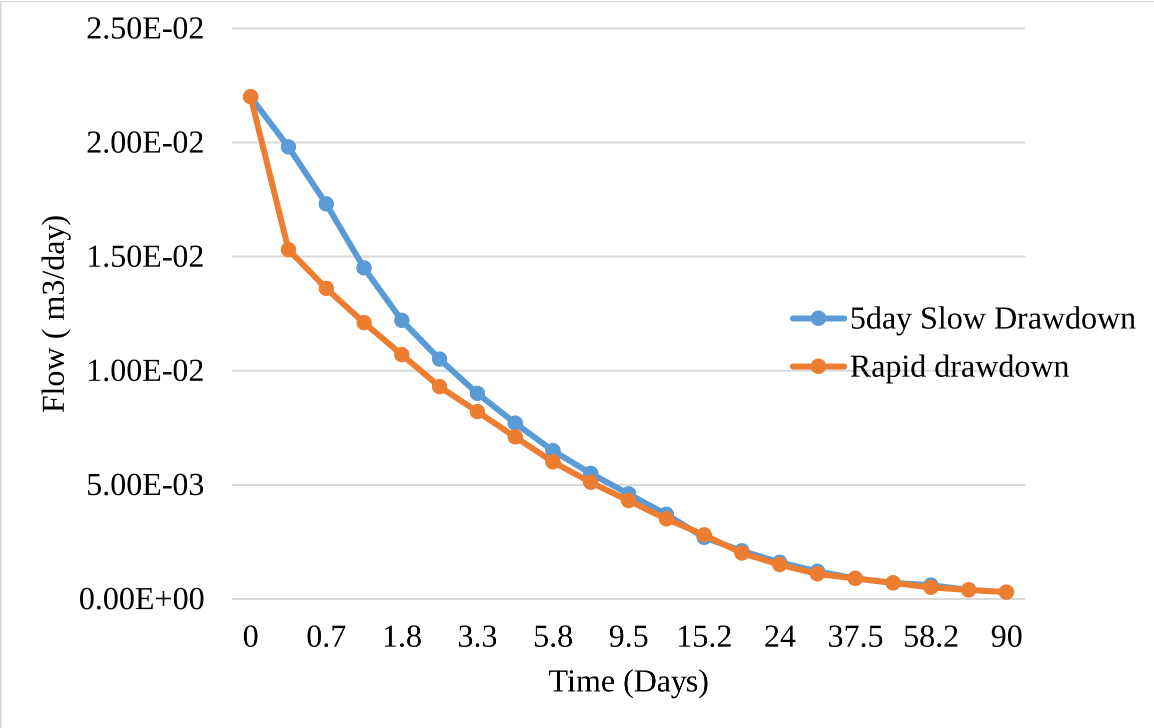


Figure 4.13: Seepage through the dam recorded at the downstream toe of the dam

The seepage through the dam is decreased slower than rapid drawdown with the same time of drawdown as indicated in Figure 4.13 this is due to the lowering of water level in the reservoir slower than rapid drawdown. The water that contribute for seepage is reduced with time, the recorded seepage also decreasing and decreasing until it comes to the end of analysis.

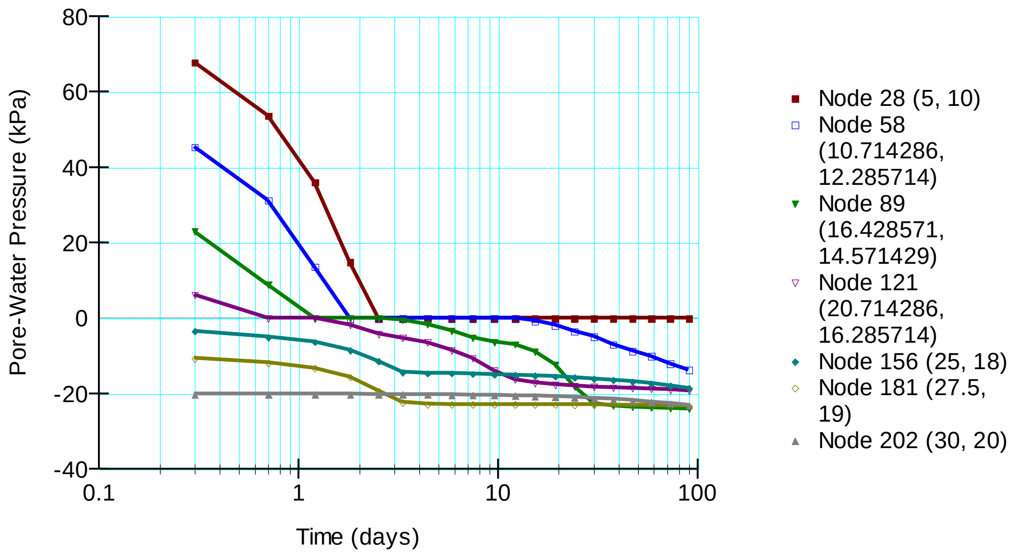


Figure 4.14: Pore water pressure versus time of slow drawdown

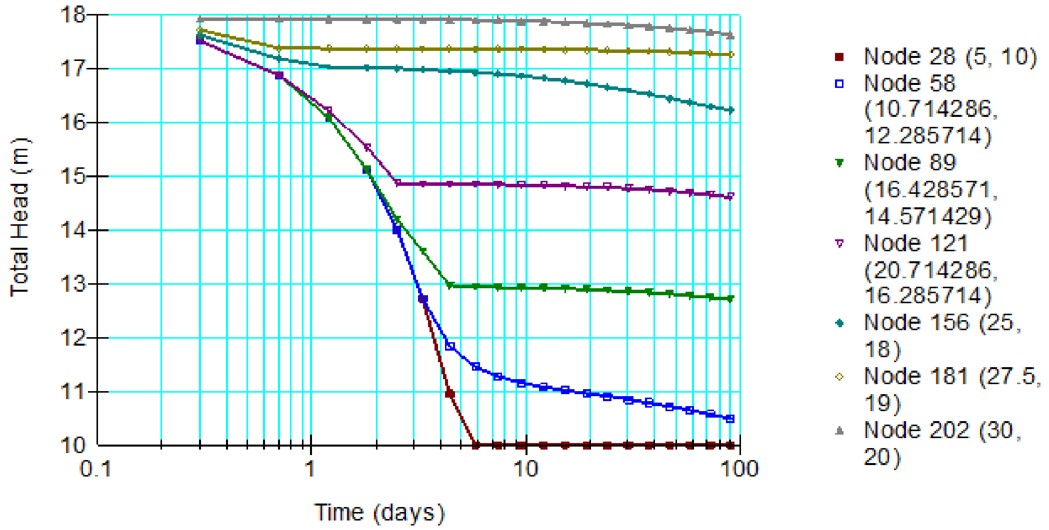


Figure 4.15: Total head versus time of slow drawdown

The pore water pressure changes with time in 7 selected nodes on the upstream slope of the dam as shown in figure 4.14. Pore water pressure changes in all nodes are in similar way. The nodes at the bottom of the geometry (node 28) initially has high pore water pressure, but the pore water pressure is reduced continuously until the reservoir water level comes to zero, but not less than zero. The pore water pressure which corresponding to other nodes (nodes 58, 89 and 121) are changed with reservoir drawdown; initially when the water level is above the nodes, the nodes show positive pore water pressure but when the water level in the reservoir is below these nodes the nodes show negative pore water pressure. Other nodes above the water level in the reservoir are showing negative pore water pressure at any time of the analysis and arrange themselves according to the degree of saturation at each nodes. These are nodes (156, 181 and 202).

In Figure 4.15; the nodes at the top of water table or the nodes above zero pore water pressure (node 202, 181 and 156) are almost straight lines this indicate that the head is not change with time. But in the other nodes (Nodes, 28, 58, 89, and 121) the drawdown of the reservoir caused the graph to be fall from high to low level of pressure head and arrange them according to their saturation degree to each nodes. In general the changes of pore water pressure and heads with time of drawdown in slow drawdown were not steep as much as the rapid drawdown.

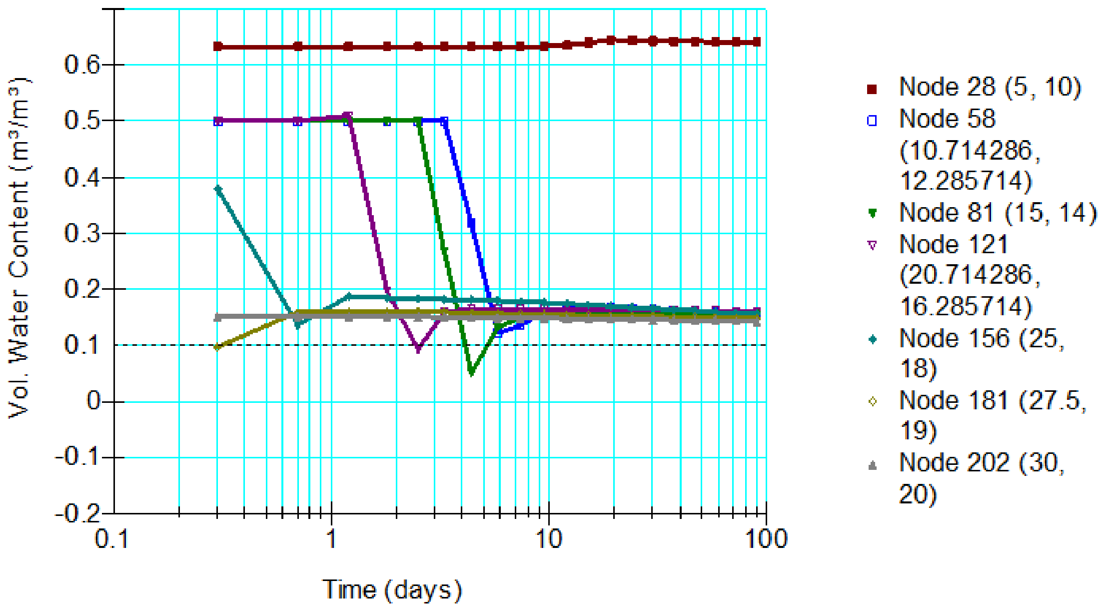


Figure 4.16: Volumetric water content versus time of slow drawdown

The Figure 4.16 and 4.17 shows the changes in the volumetric water content and hydraulic conductivity of the soil at selected nodes. Node 28 are representing the soil which have higher volumetric water content Figure 4.16. Because the node always lay below the water table, hence it is fully saturated. Similarly, the nodes 202, 181 and 156 are always above the water table and hence, partially saturated.

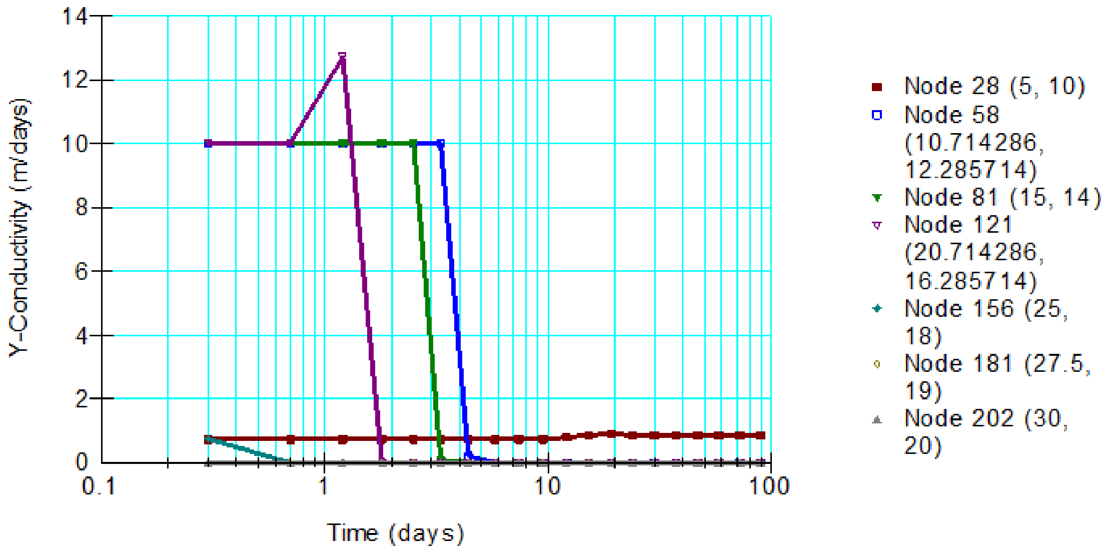
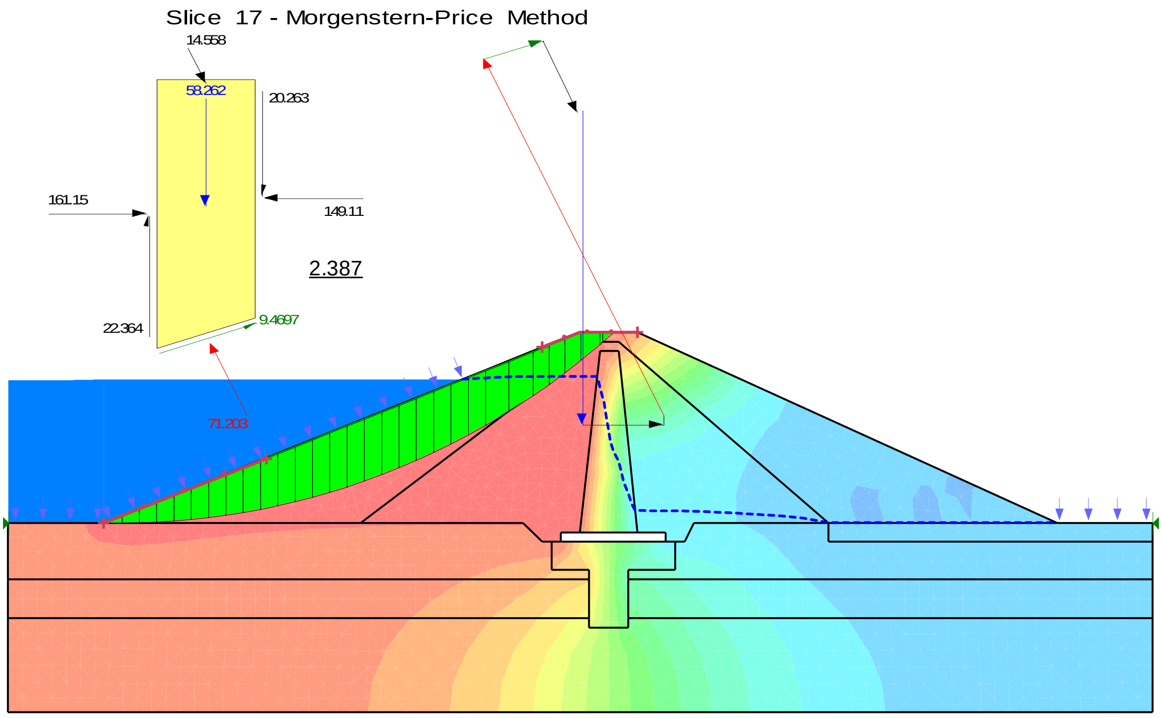


Figure 4.17: Hydraulic conductivity versus time of slow drawdown

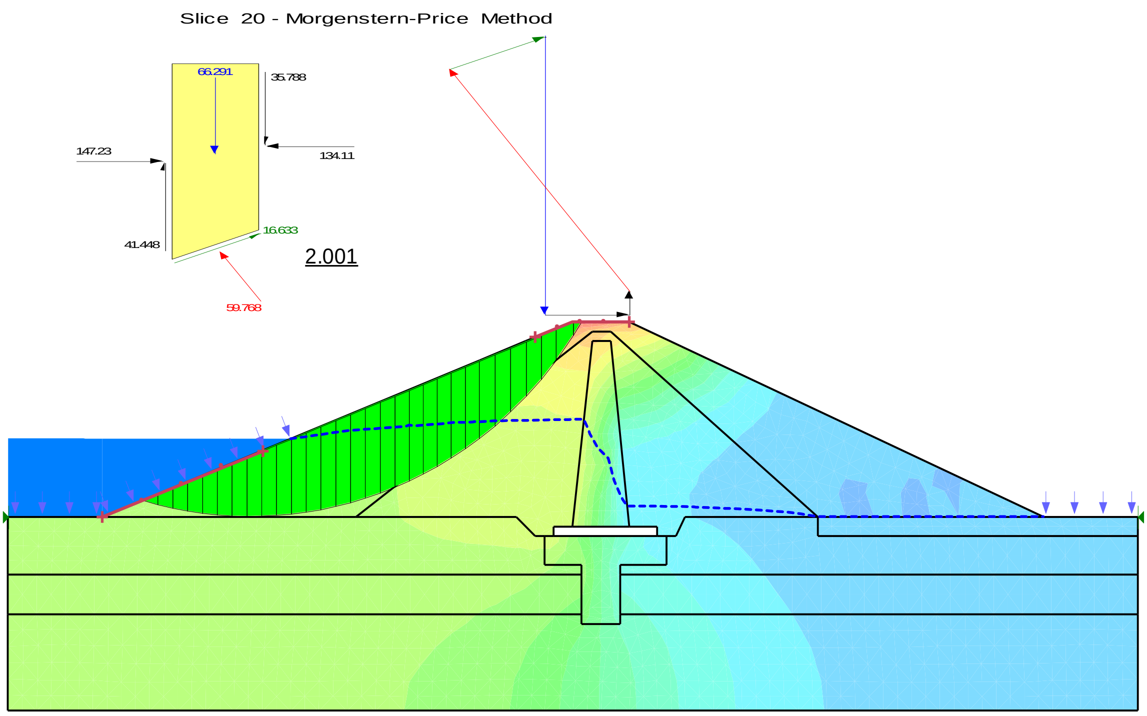
A large changed observed in the nodes 58, 81 and 121; because initially the water table is above the nodes at a time and the soil is fully saturated. But after the drawdown the water table is below the nodes this made the nodes partially saturated. Due to this reason the volumetric water content of these nodes are showing the high variation with time. Also for the figure 4.17 showing the Y-conductivity of the materials versus time. As shown in the figure the nodes at the top and bottom of the geometry (28, 156, 181 and 202) are almost straight lines, this indicate no more conductivity variation in these nodes. But in other nodes (nodes 58, 81 and 121) show the high variation in Y-conductivity, this is because of the change of water level in the reservoir.

f. Stability analysis result for slow drawdown condition

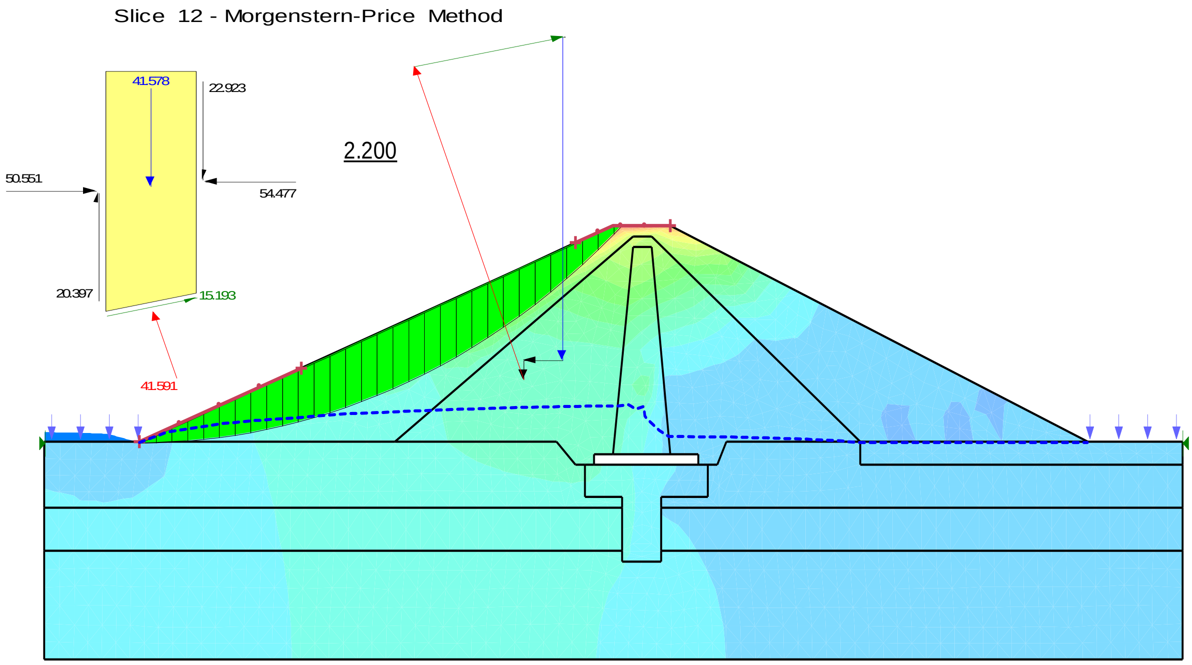
The stability of the upstream slope has been analyzed for the same time as in the rapid drawdown. Also five different time periods after the drawdown are selected. These are 7.2 hours, 2 days and 12 hours, 9 days and 12 hours, 30 days and 90 days (last time period). All the analysis carried out based on Morgenstern-Price method and compered with Spencer, Bishop simplified and Janbu simplified method. The factor of safety was obtained for each analysis with keeping the same slip surface for all analysis. The results indicate Stability conditions for each selected time period are shown in Figures 4.18 (a)-4.18(e).



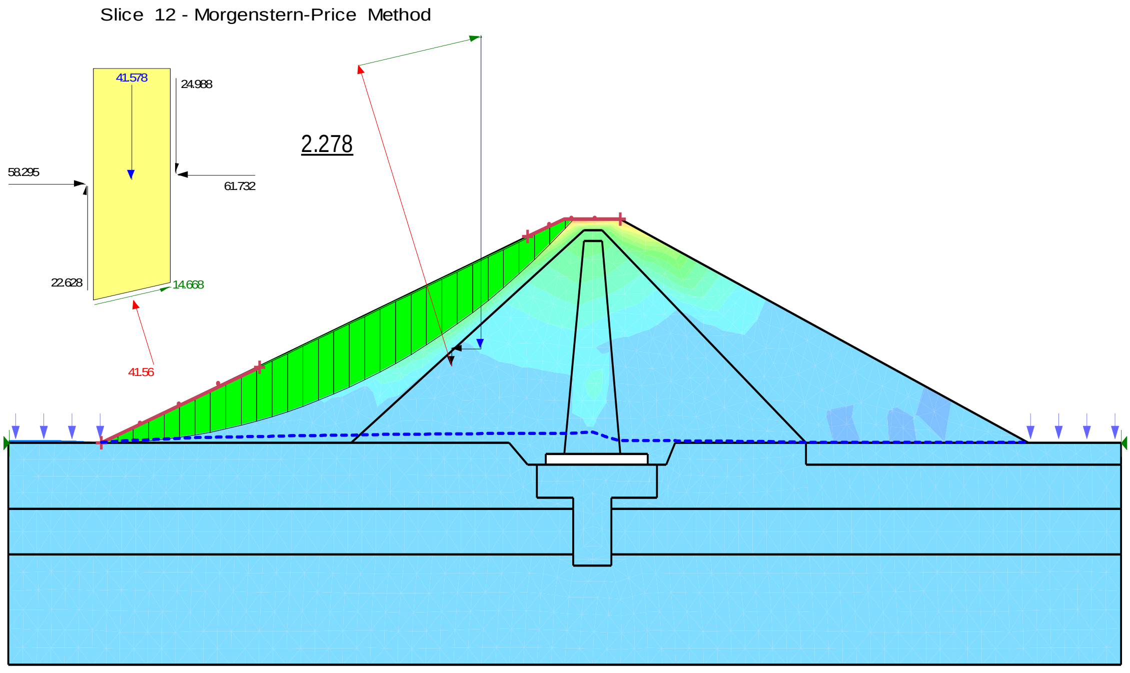
(a) Stability analysis after 7.2 hours critical shear surface and slice free body diagram with force polygon



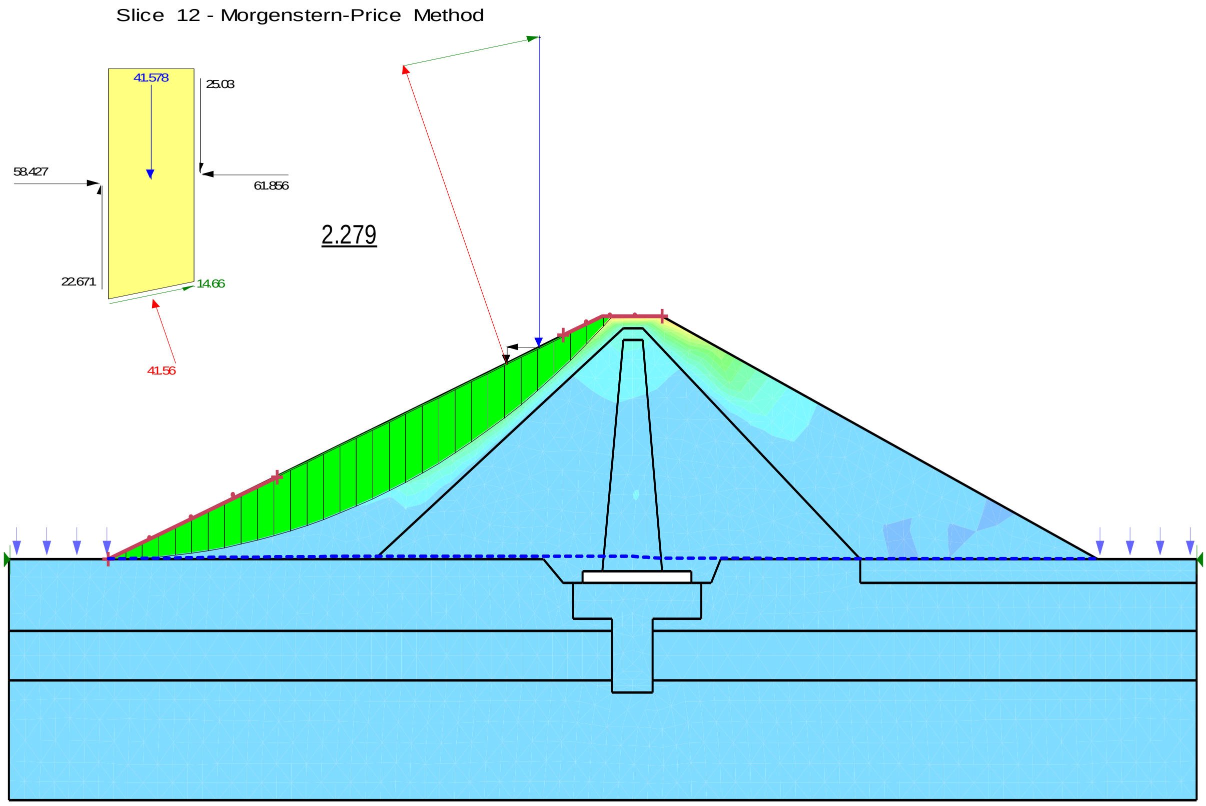
b) Stability analysis after 2 days and 12 hours critical shear surface and slice free body diagram with force polygon



(c) Stability analysis after 9 days and12 hours critical shear surface and slice free body diagram with force polygon.



d) Stability analysis after 30 days critical shear surface and slice free body diagram with force polygon.



(e) Stability analysis after 90 days critical shear surface and slice free body diagram with force polygon for slow drawdown

Figure 4.18: Critical shear surface and slice free body diagram with force polygon

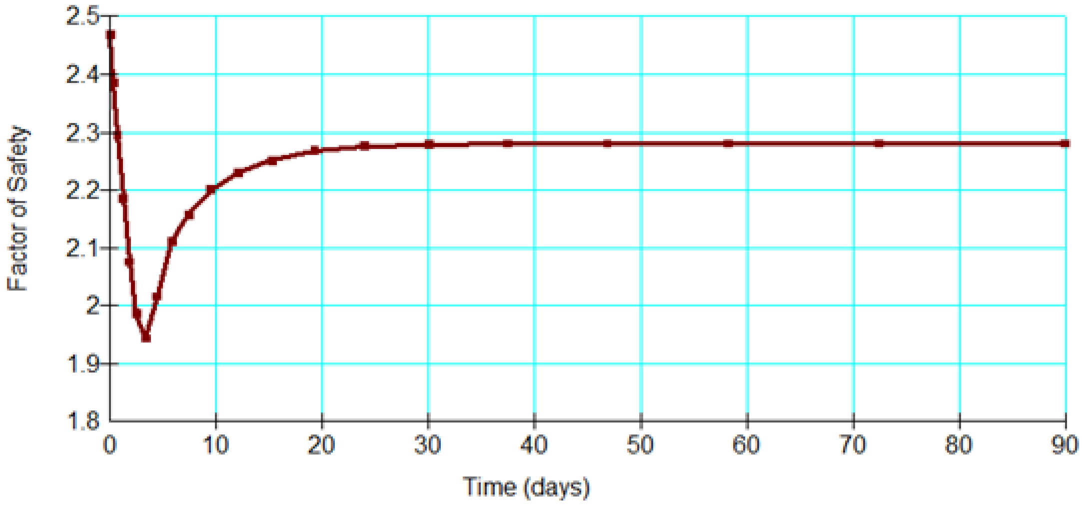


Figure 2.19: Minimum factor of safety versus time for slow drawdown

As indicated in the Figure 4.18 (a) to (e) the factor of safety is initially decreased slowly with the time steps until reach 3 days and 7 hours then it is increasing until the end of analysis. The results show that the slow drawdown of the reservoir is no much effect on the upstream slope of the dam. But as the water level that support the slope is decreased the factor of safety also decrease until the pore water pressure is dissipated from the body of the dam then the dam material gain its strength buck. Overall the slope is stable throughout the slow drawdown of the reservoir analysis more in appreciable way than rapid drawdown of the reservoir. With FS 1.958 >1.613 for RDD and also greater than FS recommended by USACE and USBR which is 1.3.

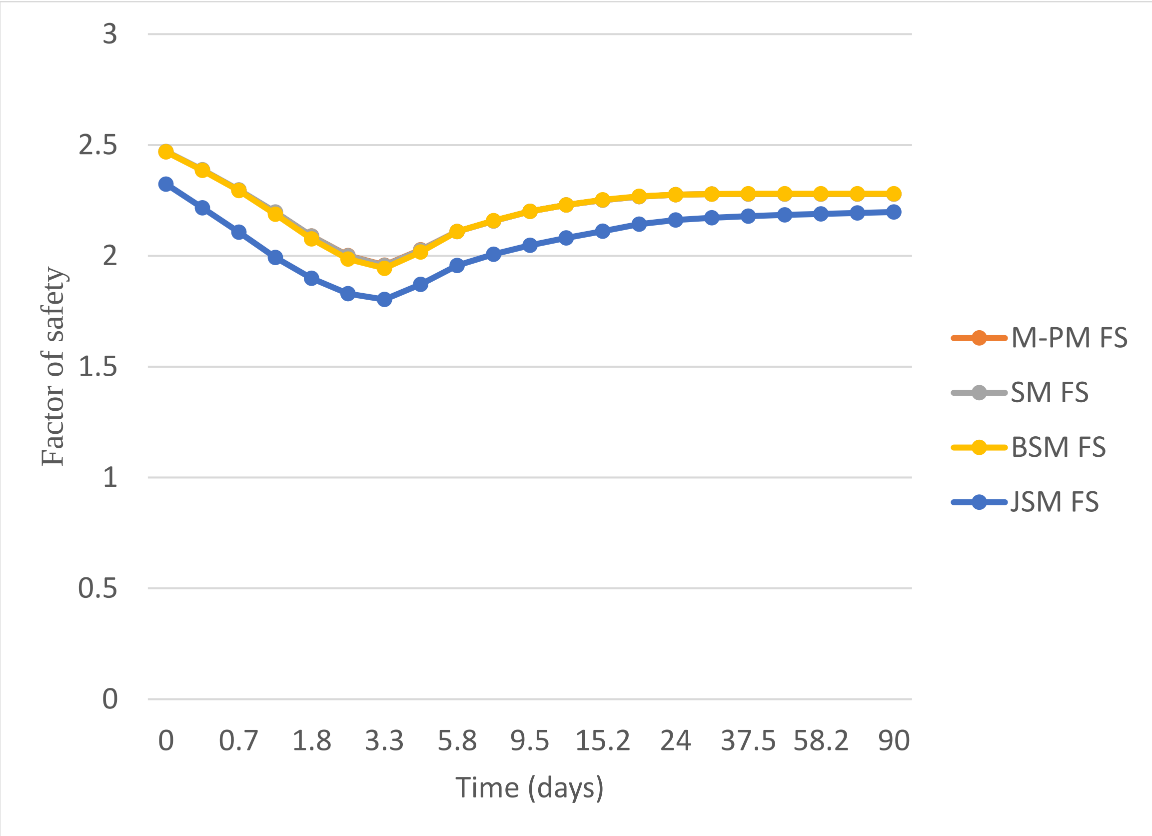


Figure 4.20: Factor of safety versus time of slow drawdown

In the Figure 4.20 indicated that the curve of FS with time of drawdown is changing smoothly from initial FS when the water level is at normal operating level to the critical FS at which the reservoir water assumed to be empty and from minimum FS to the end of the analysis whith the time of drawdown. This smooth curve of FS show that the pore water pressure have enough time to dissipate from the body of the dam than the sharply changed curve of FS in rapid drawdown shown in Figure 4.12. therefore, if the water in the reservoir needed to be reduced for modification of structure or maitenance process the releasing of water from reservoir should be recommended to give the enough time before reservoir is empty. Unless it could make the upstream slope unstable.

g. Comparison of LE methods

The Spencer, BS and JS methods are compared with reference to MP method for the steady state and transient condition (rapid and slow drawdown) analysis for upstream and downstream slope as shown in Figure 4.22, Table B.2 and Table B.4.

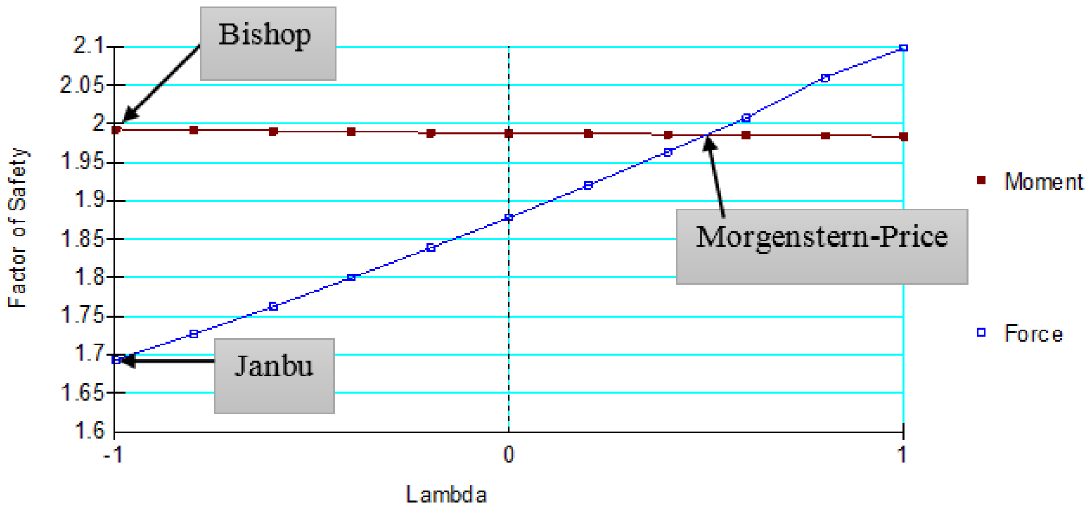


Figure 4.21: Factor of safety versus lambda

As indicated in Figurev 4.21, the FS from BSM is found almost equal compared to MPM. The reason is that the moment equilibrium FS (F_m) curve is mostly unaffected for a circular shear surface. As Krahn (2003) says, " Generally the slope of F_m curve is found nearly horizontal for a circular shear surface, and for such conditions, there is no effect of the interslice force function (f(x))". This is because the whole sliding mass can rotate without any significant movement of slices. However, BSM may overestimate FS if the external loads are applied.

In contrast, JSM has computed 8-20% lower FS compared to the FS from MPM. The larger difference indicates the sensitivity of the force equilibrium FS (F_f) due to the interslice forces. A substantial amount of interslice movement is required in this case before sliding take place.

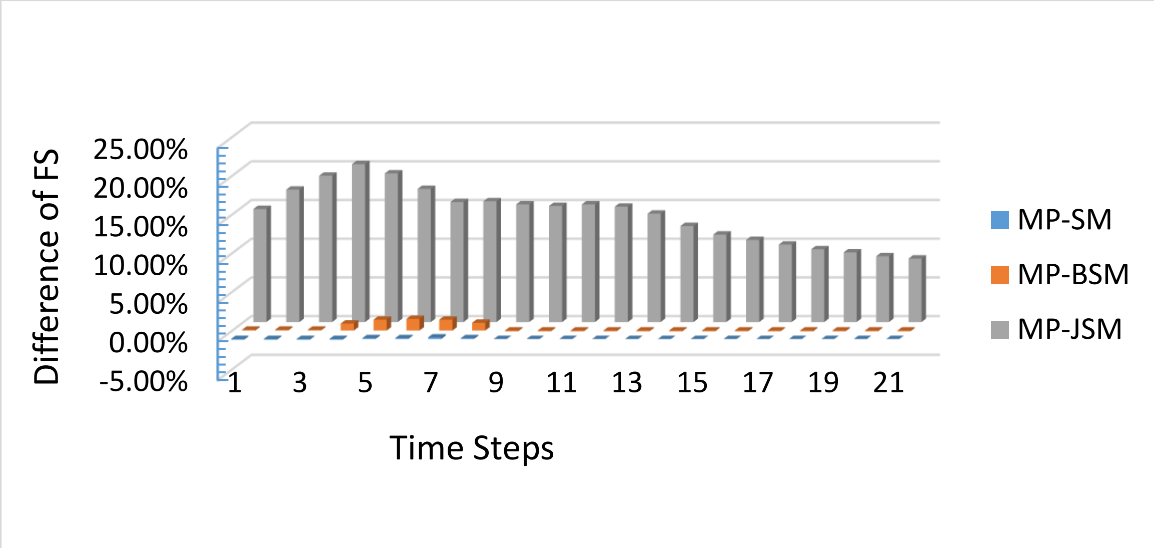


Figure 4.22: Difference of factor of safety versus time for slice methods

The JSM shows the largest variation on the upper side, ranging from 8-20%. Upon comparison with the results calculated by the MP method. As in the previous discussion, Bishop simplified method computes consistent FS with minor variations (±1%) on the higher and lower side, and the reason has already been discussed above. Similarly, Spencer method also shows minor variations (0.2%) on the higher side and (-0.1%) on the lower side. However, both Spencer and MP methods result in exactly the same FS. This indicates that both methods compute the FS with the same accuracy. In addition the lower FS in Spencer method identified by itself means that the method is able to search for the CSS more accurately than MPM. Nevertheless, the marginal variations in the FS show that both methods are equally good among the LE methods even for the individual critical shear surface analysis.

CHAPTER-5

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The seepage and stability analysis has been done using the professional version of the popular geotechnical software GeoStudio.

1. Two fundamental types of seepage analysis: steady state and transient (rapid and slow drawdown) were analyzed using SEEP/W software. The result were shown for malka wakana earth and rock fill embankment dam
2. The total flux discharge through the malka wakana earth and rock fill embankment dam continuously reducing with increasing time of drawdown and the exit gradients are less than 1.0 for different water level, therefore the dam is stable under this condition.
3. The slope stability analysis result shows that the slope is potentially stable throughout steady state and transient state or for rapid drawdown and slow drawdown analysis.
4. The factor of safety for steady state stability condition is analysed for downstream slope and the result in minimum factor of safety is 1.985 which is greater than the minimum FS requirement for downstream slope stability under steady state condition (>1.5) (USACE, 2004). Therefore, the slope is stable under this condition.
5. Factor of safety increases as flux discharge decreases and beyond the minimum factor of safety requirement for upstream slope under drawdown condition (1.958 and 1.613 > 1.3) (USACE, 2004 and USBR, 2011) which indicates the dam is extremely stable throughout drawdown of the reservoir.
6. The rapid drawdown of reservoir more affect the slope stability than slow drawdown.
7. The simplified Bishop (BS), Morgenstern-Price (M-P) and Spencer methods yield in most cases identical FS for circular shear surface without any external loads on the slopes. However, the simplified Janbu (JS) method was underestimate the FS from 8-20% for the CSS obtained by this method.

5.2 Recommendation

Since the study of this paper work is not supported by the overall required documents, identifying the main problem of the dam is so stiff. Therefore, further study focusing on the following points are recommended.

1. Identification on the source of hydraulic failure needs an extensive back analysis with help of frequent field visit.
2. The study area has exposed for hydraulic failure. Therefore, for further detail investigation the amount of seepage should be measured.
3. The observed excess seepage and erosion of the downstream face of the dam have no effect on the design. Actually, the dam needs the maintenance due to its service lifetime.
4. From the current status of the dam excessive seepage observed. However, the dam should have to be grouted effectively to control excessive seepage.
5. The downstream face of the dam have no berm structure. Therefore, it is recommended that the berm should include in maintenance plan to control the erosion of the face.
6. The recommended remedial measures to address the problem are based on literature review, before the implantation it needs detail analysis with the experienced professional.
7. The slow drawdown of the reservoir was recommended if the modification of the structure were needed.
8. The Morgenstern-Price method is recommended to use in any kind of shear resistant analysis since it is satisfy the moment and force equilibrium and considering both shear and normal interslice forces and allows for a variety of user selected interslice force function.

The studies of 3D-slope stability analysis show better FS than 2D-analysis (Duncan, 1996). Therefore, such studies not only increase the FS and optimize the design. Within these perspectives, further study on the following area is recommended:

9. 3D-slope stability analysis are recommended to compare the FS obtained from 2D-analysis.

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APPENDICES

Appendix A

Volumetric water content and Hydraulic conductivity functions for embankment materials

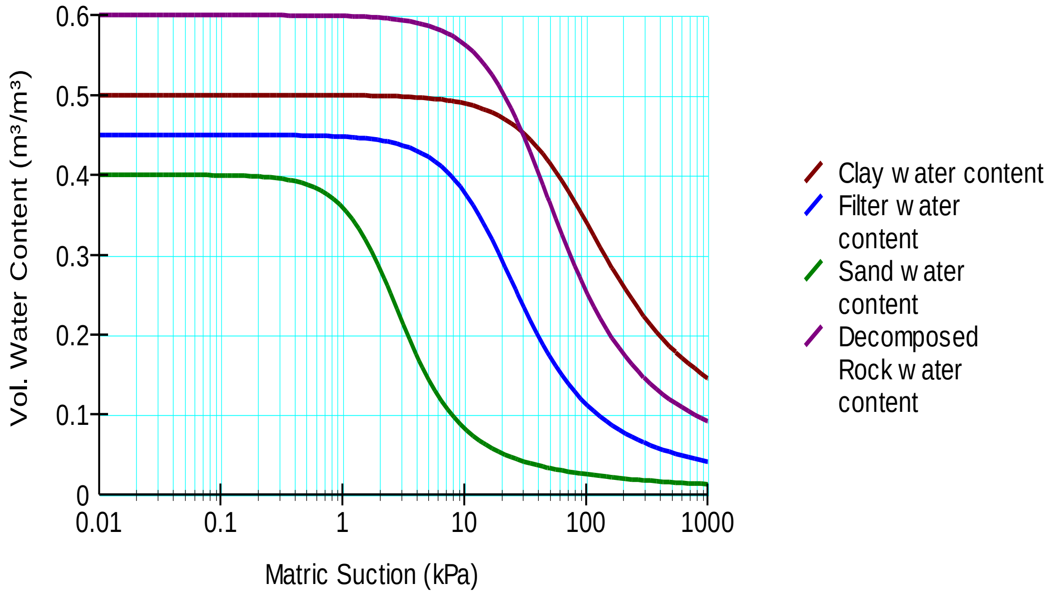

Figure A.1: Volumetric water content functions for embankment materials

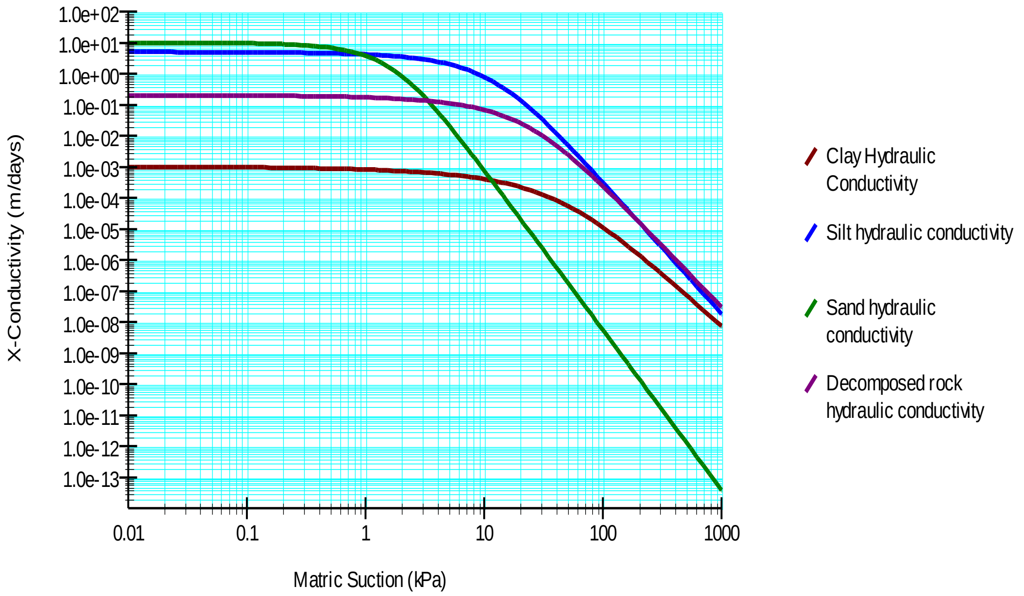

Figure A.2: Hydraulic conductivity function for embankment materials

Appendix B:

Transient seepage and slope stability results

Table B.1: Flow through the dam cross section for rapid drawdown

Time Period	Total Flux (*$10^{-3}m^3$/day)
0	22.00
7.2 hours	15.30
16.8 hours	13.60
1 day and 4 hours	12.10
1 day and 19 hours	10.70
2 days and 12 hours	9.30
3 days and7 hours	8.20
4 days and 9 hours	7.10
5 days and 19 hours	6.00
7 days and 9 hours	5.10
9 days and 12 hours	4.30
12 days 2 hours	3.50
15 days and 4 hours	2.80
19 days and 4 hours	2.00
24 days	1.50
30 days	1.10
37 days and 12 hours	0.90
46 days and 19 hours	0.70
58 days and 4 hours	0.50
72 days and 9 hours	0.40
90 days	0.30

Table B.2: Factor of safety for all rapid draw down time steps

Time	M-Price	Spencer	Bishop	Janbu	USBR Standard
Initial	2.468	2.469	2.468	2.320	
7.2 hours	1.613	1.614	1.607	1.50	1.3
16.8 hours	1.749	1.750	1.745	1.629	1.3
1 day and 4 hours	1.861	1.861	1.856	1.731	1.3
1 day and 19 hours	1.961	1.961	1.955	1.819	1.3
2 days and 12 hours	2.043	2.043	2.043	1.900	1.3
3 days and 7 hours	2.103	2.103	2.102	1.947	1.3
4 days and 9 hours	2.148	2.149	2.148	1.987	1.3
5 days and 19 hours	2.183	2.183	2.182	2.026	1.3
7 days and 9 hours	2.214	2.215	2.213	2.057	1.3
9 days and 12 hours	2.237	2.238	2.237	2.083	1.3
12 days 2 hours	2.259	2.259	2.259	2.110	1.3
15 days and 4 hours	2.274	2.274	2.275	2.138	1.3
19 days and 4 hours	2.283	2.283	2.284	2.160	1.3
24 days	2.287	2.287	2.288	2.171	1.3
30 days	2.288	2.288	2.290	2.179	1.3
37 days and 12 hours	2.289	2.289	2.290	2.185	1.3
46 days and 19 hours	2.289	2.289	2.290	2.189	1.3
58 days and 4 hours	2.289	2.289	2.290	2.193	1.3
72 days and 9 hours	2.289	2.289	2.290	2.197	1.3
90 days	2.289	2.289	2.290	2.200	1.3

Table B.3: Flow through the dam for different time under slow drawdown

Time Period	Total Flux (*10^{-3}m3/day)
0	22.00
7.2 hours	19.80
16.8 hours	17.30
1 day and 4 hours	14.50
1 day and 19 hours	12.20
2 days and 12 hours	10.50
3 days and7 hours	9.00
4 days and 9 hours	7.70
5 days and 19 hours	6.50
7 days and 9 hours	5.50
9days and 126hr	4.60
12 days 2 hours	3.70
15 days and 4 hours	2.70
19days and 4 hours	2.10
24 days	1.60
30 days	1.20
37 days and 12 hours	0.9
46 days and 19 hours	0.7
58 days and 4 hours	0.6
72 days and 9 hours	0.4
90 days	0.3

Table B.4: Factor of safety for all slow drawdown time steps

Time	Morgenstern-Price	Spencer	Bishop	Janbu	USACE Standard
Initial	2.47	2.471	2.469	2.324	1.3
7.2 hours	2.387	2.388	2.386	2.216	1.3
16.8 hours	2.296	2.297	2.295	2.107	1.3
1 day and 4 hours	2.196	2.197	2.187	1.992	1.3
1 day and 19 hours	2.090	2.089	2.076	1.898	1.3
2 days and 12 hours	2.001	2.000	1.986	1.829	1.3
3 days and7 hours	1.958	1.956	1.944	1.803	1.3
4 days and 9 hours	2.027	2.026	2.017	1.871	1.3
5 days and 19 hours	2.109	2.109	2.110	1.957	1.3
7 days and 9 hours	2.157	2.157	2.158	2.007	1.3
9 days and 126 hours	2.200	2.200	2.201	2.048	1.3
12 days 2 hours	2.229	2.229	2.230	2.080	1.3
15 days and 4 hours	2.251	2.251	2.252	2.111	1.3
19 days and 4 hours	2.267	2.267	2.268	2.143	1.3
24 days	2.275	2.275	2.276	2.162	1.3
30 days	2.278	2.278	2.279	2.172	1.3
37 days and 12 hours	2.279	2.279	2.280	2.179	1.3
46 days and 19 hours	2.279	2.279	2.280	2.185	1.3
58 days and 4 hours	2.279	2.279	2.280	2.189	1.3
72 days and 9 hours	2.279	2.279	2.280	2.194	1.3
90 days	2.279	2.279	2.280	2.197	1.3

Appendix C

1-Steady-State Seepage Analysis

File Information

Title: Cross section of Malka Wakana Dam

Created By: Hassen Hussien

Last Edited By: Hassen Hussien

Revision Number: 762

File Version: 8.1

Tool Version: 8.11.1.7283

Date: 11/12/2017

Time: 8:59:44 AM

File Name: Correct cross section of Wakana.gsz

Directory: C:\Users\TOSHIBA\Desktop\Geo-studeo\

Last Solved Date: 11/12/2017

Last Solved Time: 8:59:46 AM

Project Settings

Length (L) Units: meters

Time (t) Units: Days

Force (F) Units: kN

Pressure (p) Units: kPa

Mass (M) Units: g

Mass Flux Units: g/days

Unit Weight of Water: 9.807 kN/m³

View: 2D

Element Thickness: 1

Analysis Settings

1-Steady-State Seepage

Kind: SEEP/W

Method: Steady-State

Settings

Include Air Flow: No

Control

Apply Runoff: Yes

Convergence

Maximum Number of Iterations: 500

Minimum Pressure Head Difference: 0.005

Significant Digits: 2

Max # of Reviews: 10

Hydraulic Under-Relaxation Criteria

Under-Relaxation Initial Rate: 1

Under-Relaxation Min. Rate: 0.1

Under-Relaxation Reduction Rate: 0.65

Under-Relaxation Iterations: 10

Equation Solver: Parallel Direct

Time

Starting Time: 0 days

Duration: 0 days

Ending Time: 0 days

Materials

Core

Model: Saturated / Unsaturated

Hydraulic

K-Function:Clay Hydraulic Conductivity

Ky'/Kx' Ratio: 1

Rotation: 0 °

Vol. WC. Function: Clay water content

Filter

Model: Saturated / Unsaturated

Hydraulic

K-Function: Silt hydraulic conductivity

Ky'/Kx' Ratio: 1

Rotation: 0 °

Vol. WC. Function: Filter water content

Shell

Model: Saturated / Unsaturated

Hydraulic

K-Function: Sand hydraulic conductivity

Ky'/Kx' Ratio: 1

Rotation: 0 °

Vol. WC. Function: Sand water content

Decomposed Rock

Model: Saturated / Unsaturated

Hydraulic

K-Function: Decomposed rock hydraulic conductivity

Ky'/Kx' Ratio: 1

Rotation: 0 °

Vol. WC. Function: Decomposed Rock water content

Fractured rock

Model: Saturated / Unsaturated

Hydraulic

K-Function: Fractured Rock Hydraulic conductivity

Ky'/Kx' Ratio: 1

Rotation: 0 °

Vol. WC. Function: Fractured rock water content

Bed Rock

Model: Saturated / Unsaturated

Hydraulic

K-Function:

Bed rock hydraulic conductivity

Ky'/Kx' Ratio: 1

Rotation: 0 °

Vol. WC. Function: Intact rock water content

Toe Drain

Model: Saturated Only

Hydraulic

K-Sat: 10 m/days

Ky'/Kx' Ratio: 1

Rotation: 0 °

Volumetric Water Content: 1 m^3/m^3

Mv: 0 /kPa

Concrete

Model: (none)

Grout Curtain

Model: Saturated / Unsaturated

Hydraulic

K-Function:

Grout Curtain Hydraulic Conductivity

Ky'/Kx' Ratio: 1

Rotation: 0 °

Vol. WC. Function: Grout Curtain

Boundary Conditions

Zero Pressure

Type: Pressure Head 0

Review: No

Upstream face

Type: Head (H) 18

Review: No

Flux Sections

Flux Section 1

Coordinates

Coordinate: (53, -1) m

Coordinate: (54, 15) m

K Functions

Clay Hydraulic Conductivity

Model: Hyd K Data Point Function

Function: X-Conductivity vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

K-Saturation: 0.001

Data Points: Matric Suction (kPa),

X-Conductivity (m/days)

Points

	X (m)	Y (m)
Point 1	5	10
Point 2	30	20
Point 3	33	20
Point 4	55	10
Point 5	43	10
Point 6	32	19.5
Point 7	31	19.5
Point 8	18.5	10
Point 9	36	10
Point 10	35.5	9
Point 11	35	9
Point 12	34.5	9
Point 13	34.5	9.5
Point 14	33	9.5
Point 15	32	19
Point 16	31	19
Point 17	30	9.5
Point 18	29	9.5
Point 19	29	9
Point 20	28.5	9
Point 21	28	9
Point 22	27	10
Point 23	35	7.5
Point 24	32.5	7.5
Point 25	32.5	7
Point 26	32.5	5
Point 27	32.5	4.5
Point 28	30.5	4.5
Point 29	30.5	5
Point 30	30.5	7
Point 31	30.5	7.5
Point 32	28.5	7.5
Point 33	0	7
Point 34	0	10
Point 35	60	7
Point 36	0	5
Point 37	60	5
Point 38	0	0
Point 39	60	0
Point 40	27.5	19
Point 41	60	10
Point 42	60	9
Point 43	43	9
Point 44	25	18
Point 45	31	20

Lines

	Start Point	End Point	Hydraulic Boundary
Line 1	3	4	
Line 2	4	5	Zero Pressure
Line 3	5	6	
Line 4	6	7	
Line 5	7	8	
Line 6	8	1	
Line 7	9	10	
Line 8	10	11	
Line 9	11	12	
Line 10	12	13	
Line 11	13	14	
Line 12	14	15	
Line 13	15	16	
Line 14	16	17	
Line 15	17	18	
Line 16	18	19	
Line 17	19	20	
Line 18	20	21	
Line 19	21	22	
Line 20	22	8	
Line 21	14	17	
Line 22	12	19	
Line 23	11	23	
Line 24	23	24	
Line 25	24	25	
Line 26	25	26	
Line 27	26	27	
Line 28	27	28	
Line 29	28	29	
Line 30	29	30	
Line 31	30	31	
Line 32	31	32	
Line 33	32	20	
Line 34	33	34	
Line 35	34	1	
Line 36	30	33	
Line 37	35	25	
Line 38	36	33	
Line 39	29	36	
Line 40	35	37	
Line 41	37	26	
Line 42	38	36	
Line 43	37	39	
Line 44	39	38	
Line 45	40	2	
Line 46	9	5	
Line 47	4	41	Zero Pressure
Line 48	41	42	
Line 49	42	43	
Line 50	43	5	
Line 51	42	35	
Line 52	44	40	
Line 53	2	45	
Line 54	45	3	
Line 55	1	44	Upstream face

Regions

	Material	Points	Area (m^2)
Region 1	Shell	1,44,40,2,45,3,4,5,6,7,8	143.88
Region 2	Core	17,16,15,14	19
Region 3	Concrete	19,18,17,14,13,12	2.75
Region 4	Grout Curtain	20,19,12,11,23,24,25,26,27,28,29,30,31,32	15.75
Region 5	Decomposed Rock	33,34,1,8,22,21,20,32,31,30	85.5
Region 6	Fractured rock	36,33,30,29	61
Region 7	Fractured rock	26,25,35,37	55
Region 8	Bed Rock	38,36,29,28,27,26,37,39	299
Region 9	Filter	8,7,6,5,9,10,11,12,13,14,15,16,17,18,19,20,21,22	107.63
Region 10	Toe Drain	5,4,41,42,43	17
Region 11	Decomposed Rock	25,24,23,11,10,9,5,43,42,35	