A STUDY ON RESERVOIR FILLING PRACTICES FOR SMALL EARTHFILL DAMS

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
MIDDLE EAST TECHNICAL UNIVERSITY

BY

EGE CAN KURTER

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF MASTER OF SCIENCE
IN
CIVIL ENGINEERING

DECEMBER 2022

Approval of the thesis:

A STUDY ON RESERVOIR FILLING PRACTICES FOR SMALL EARTHFILL DAMS

submitted by **EGE CAN KURTER** in partial fulfillment of the requirements for the degree of **Master of Science in Civil Engineering Department, Middle East Technical University** by,

Prof. Dr. Halil Kalıpçılar
Dean, Graduate School of **Natural and Applied Sciences** ______________

Prof. Dr. Erdem Canbay
Head of Department, **Civil Engineering** ______________

Prof. Dr. A. Melih Yanmaz
Supervisor, **Civil Engineering, METU** ______________

Assoc. Prof. Dr. Melih Çalamak
Co-supervisor, **Civil Engineering, METU** ______________

Examining Committee Members:

Assoc. Prof. Dr. Onur Pekcan
Civil Engineering, METU ______________

Prof. Dr. A. Melih Yanmaz
Civil Engineering, METU ______________

Assoc. Prof. Dr. Melih Çalamak
Civil Engineering, METU ______________

Assist. Prof. Dr. Cüneyt Baykal
Civil Engineering, METU ______________

Assist. Prof. Dr. Meriç Yılmaz
Civil Engineering, Atılım University ______________

Date: 16.12.2022

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Name, Surname: Ege Can Kurter

Signature :

ABSTRACT

A STUDY ON RESERVOIR FILLING PRACTICES FOR SMALL EARTHFILL DAMS

Kurter, Ege Can
M.S., Department of Civil Engineering
Supervisor: Prof. Dr. A. Melih Yanmaz
Co-Supervisor: Assoc. Prof. Dr. Melih Çalamak

December 2022, 91 pages

The initial filling is considered to be the first safety test of the embankment followed after construction. The earth material acts significantly different during the first interaction with water. The majority of the failures occur during initial filling. The reasons of failures are mainly attributable to crack formation and internal erosion followed by piping. The filling is commonly practiced in multi-stages where intermediary holds are provided for monitoring purposes before the embankment is further loaded. This study focuses on the behavior of the embankment during first filling and aims to propose certain hold durations as well as the components affecting the durations. The finite element model is applied on a 20 m high earthfill dam using GeoStudio™ software. Coupled stress and pore-water pressure analyses are carried on in which the transient behavior of filling process is clearly identified. Hydraulic fracturing phenomenon is considered as the driving mode for crack and internal erosion formation. Furthermore, the study findings investigate the effect of uncontrolled filling rate on the durations of the holds. It is found that the initiation of hydraulic fracturing is more influenced from the initial conditions of the soil rather than a planned filling schedule. The earth material placed in dry conditions caused crack formation at the higher elevations of upstream side of the core. Additionally, the study proposed certain hold duration for intermediary hold up to a low pool level and revealed the positive effect of waiting at higher pool levels against hydraulic fracturing.

Keywords: Earthfill Dams, First Filling, Staged Filling, Hydraulic Fracturing, Coupled Stress-PWP Analysis

ÖZ

KÜÇÜK TOPRAK DOLGU BARAJLARDA HAZNE DOLUMU ÜZERİNE BİR ÇALIŞMA

Kurter, Ege Can
Yüksek Lisans, İnşaat Mühendisliği Bölümü
Tez Yöneticisi: Prof. Dr. A. Melih Yanmaz
Ortak Tez Yöneticisi: Doç. Dr. Melih Çalamak

Aralık 2022 , 91 sayfa

İlk su tutma, toprak dolgu barajların inşaatının tamamlanmasını takiben karşılaşacağı ilk sınavdır. Toprak dolgu malzemesi, suyla ilk temasında dikkate değer derecede farklı bir davranış sergiler. Baraj kazalarının büyük bir bölümü ilk su tutma esnasında meydana gelmektedir. Bu kazalar genellikle, çatlak oluşumu ve içsel erozyonu takiben borulanma sebeplidir. İlk su tutma genellikle çok aşamalı olarak gerçekleştirilir. Bu aşamalar arasında barajın daha yüksek kuvvetle yüklenmeden önce gözlemlenebilmesi için bekleme süreleri sağlanır. Bu çalışma toprak dolgu barajların ilk dolum sürecini inceleyip, aşamalar arasındaki bekleme sürelerini öne sürmeyi amaçlamıştır. GeoStudio™ yazılımını kullanılarak, sonlu elemanlar tekniğiyle 20 metre yüksekliğinde bir baraj modeli oluşturulmuştur. Bütünleşik gerilme ve boşluk suyu basıncı analizleri kullanılarak dolum sürecinde zamana bağlı değişimler gözlenmiştir. Çatlak oluşumu ve içsel erozyonun sebebi hidrolik çatlama olarak bilinmektedir. Bu çalışma ilk dolum esnasındaki hidrolik çatlama gözlenmesinde belirlenmiş bir dolum takvimindense, topraktaki başlangıç koşullarının daha çok etkili olduğunu ortaya koymuştur. Kuru koşullarda yerleştirilen toprak, ilk dolum esnasında orta dolgunun üst bölümlerinde çatlak oluşumuna yol açmaktadır. Çalışma buna ek olarak, bekleme sürelerinin uzunlukları üzerine önermeler yapmıştır. Alçak rezervuar seviyelerindeki bekleme sürelerini önermiş ve yüksek rezervuar seviyelerindeki artan bekleme sürelerinin hidrolik çatlamaya karşı olumlu etkisini göstermiştir. Ayrıca, bu çalışma kontrolsüz dolum hızlarının bekleme süreleri üzerindeki etkisini de ortaya çıkarmıştır.

Anahtar Kelimeler: Toprak Dolgu Barajlar, İlk Su Tutma, Aşamalı Dolum, Hidrolik Çatlama, Bütünleşik Gerilme-Boşluk Suyu Basıncı Analizi

To every single stardust I am composed of

ACKNOWLEDGMENTS

I would like to express my most grateful thoughts to my supervisor, Prof. Dr. A. Melih Yanmaz. He recognized the curiosity and ambition inside me, and guided me through my first steps into academia starting from my undergraduate years. The idea of choosing to start graduate school and find my life pursuit in learning more about water resources engineering clicked for me when I got the highest grade on the exam of his class. It will be thanks to his enthusiasm and energy for lifelong learning, that I have progressively become a better researcher.

I want to also thank my co-advisor Assoc. Prof. Dr. Melih Çalamak for his continuous support and supervision throughout the course of my research. He always greeted me with a positive attitude whenever I required guidance and when things were getting complicated for me. His undying curiousity has made me a better researcher and has taught me how to better implement and translate skills into research.

My deepest gratitude is to my partner Dorothy Rau. She has been the only person that I could emotionally lean on during endless hours of working for my graduate degree. It was a pleasure to share in the parallel steps of graduate education together.

To my dear family, thank you for teaching me how to be an individual in this strange world and always supporting me to walk my own path. I would not be the confident person I am today without the life lessons that I have learned from each of you. To my mother Sevgi Kurter, my father Hasan Derya Kurter and my brother Ahmet Eren Kurter, thank you.

To my friends and colleagues from METU, we have made a promise to each other to make this world a more equal, gentle, and better place. I have learned that my thoughts and beliefs are always worth standing behind, and I will never feel alone knowing that we have spread all around the world with the common goal of pursuing the same responsibility. It is my duty to continue carrying this flag and representing this embracing culture.

TABLE OF CONTENTS

LIST OF TABLES

TABLES

LIST OF FIGURES

FIGURES

LIST OF ABBREVIATIONS

DSI	Turkish State Hydraulic Works
USBR	United States Bureau of Reclamation
ICOLD	International Commission of Large Dams
USACE	United States Army Corps of Engineers
USCS	Unified Soil Classification System
ACER	Assistant Commissioner - Engineering and Research
IDNR	Indiana Department of Natural Resources
PWP	Pore Water Pressure
VWC	Volumetric Water Content
D/S	Downstream
U/S	Upstream

LIST OF SYMBOLS

c'	Effective Cohesion
D'	Effective Stress Stiffness Matrix
E	Elastic Modulus
g	Gravitational Acceleration
H	Total Energy Head
h	Pressure Head
i	Hydraulic Gradient
k	Element Mesh Size
K_h	Horizontal Hydraulic Conductivity
K_v	Vertical Hydraulic Conductivity
K_w	Isothermal Liquid Water Hydraulic Conductivity
M_v	Coefficient of Volume Compressibility
m_w	Slope of The Storage Curve
q	Specific Discharge
Q'	Boundary Flux
r	Filling Rate
r_{hh}	Hold-to-Height Ratio
S	Degree of Saturation
t	Time
u_a	Pore-Air Pressure
u_w	Pore-Water Pressure
z	Elevation Head
β_w	Isothermal Compressibility of Water

ε_p	Volumetric Strain
γ_w	Unit Weight of Water
ν	Poisson's Ratio
ϕ'	Effective Angle of Friction
ψ	Matric Suction
ρ_w	Density of Water
σ_3	Minimum Principal Stress
σ_x	Total Horizontal Stress
σ_y	Total Vertical Stress
τ_{xy}	Shear Stress
θ	Volumetric Water Content
Θ_r	Residual Volumetric Water Content
Θ_s	Saturated Volumetric Water Content

CHAPTER 1

INTRODUCTION

1.1 General

Earthfill dams are the most common and practical dam type to be constructed on various topographical features. The use of earth material as a barrier was human nature's first approach to store and regulate water resources. As our understanding of dam construction stepped ahead, the design of earthfill dams had been perfected. Zoned embankment with central clay zone is a well-known design. While the impervious core controls the seepage, the surrounding pervious shells provide the stability of the body. The most probable failure mode of these structures is internal erosion caused by cracking during the most crucial time of the dam's life, that is first filling of the reservoir (USBR, 2021). For this reason, dam engineers put their efforts to carefully plan and closely monitor the first filling and divide the filling into multi-stages where an intermediary hold/stoppage, i.e., waiting times, of water level is provided. These holds are intended to provide a sufficient time for dam to respond to changing water levels and for designers to closely monitor the internally installed instruments before loading the embankment with higher water levels. Although the filling rates are specified and limited for a rise in reservoir water level, recommendation on the duration of intermediate holds is the gap in the literature. Internal erosion is widely accepted to be occurred due to hydraulic fracturing of the core as a result of differential settlement and arching effect of the core. Since the compressibility of shell and core material are different from each other, the arching effect occurs when the core is hanged and carried between the shell zones and creates low stress areas beneath (Sherard, 1986). As the increasing water pressure coincides with the low stress zones, the hydraulic fracturing develops and followed by crack propagation with excessive leakage (Sherard, 1986). Current study put its scope into the first filling of a hypothetical dam and investigate its behavior to increasing water load. The author aimed to propose a certain hold duration considering hydraulic fracturing and differential settlement possibility during initial filling such that an additional waiting is not necessary with respect to dam's safety. A filling schedule in three stages recommended by Turkish State Hydraulic Works (DSI), is applied to a 20 m high finite-element dam model and dam behavior for different hold durations is studied. The study proposes waiting times for different elevations and provides a detailed look into dam's response to initial filling. The analyses are conducted using the package software GeoStudio™ 2021, that transient coupled stress - pore-water pressure (PWP) formulation are practiced considering consolidation effect on embankment. The study kept its scope into relatively small dams to idealize the filling schedule against unexpected floods and seasonal changes. In addition, the study investigates the behavior during intermediary holds only, it does not consider the full reservoir load at the end of first filling. The recommended waiting times and the followed approach is applicable and practical for design of earthfill dam's reservoir filling utilization.

1.2 Motivation of the Study

The scope of this study is to research the first filling, i.e., initial impoundment, practices of the earthfill dam's reservoirs. The history of catastrophic failures of the dam body during initial filling has made this subject very crucial for engineers and authorities. Foster et al. (2011) revealed that 49% of the internal erosion related failures occurred during first filling of the reservoir for embankments. For example, the well-known Teton Dam (USA) is failed completely during the first reservoir filling in 1976 due to piping caused by hydraulic fracturing (Widjaja et al., 1984). Therefore, water rise in the reservoir from end of construction to desired operation level should be well planned, controlled and closely monitored. It is unique for each reservoir to design a filling schedule depending on purpose of the reservoir, size and type of the dam, location, and hydrological features of the region. But to have a common approach, state agencies have certain decision/technical memorandum stating the filling rates and certain dam heights to hold water in order to monitor dam performance before further raising (USBR (2021); DSI (2014); IDNR (2001)). The purpose of the holds are to monitor and evaluate dam's performance before filling it up to higher levels. The dam is evaluated through the obtained data from the many monitoring devices placed on various locations on and inside the dam body. These data are compared to expected values from prior modeling or calculations. If the hold time is decided to be adequate and the dam is safe as it is designed to, the filling continues to higher levels. Since the duration of the water level holds play an important role on the design of the filling schedule, this study intends to contribute to the practices by defining the waiting time considering embankment's safety. Additionally, the effect of finite element method (FEM) coupled analysis would provide another inside look to dam's response to initial filling.

1.3 Research Objective

The study aims to model a hypothetical embankment dam prior to filling in order to simulate dam's behavior as the reservoir water level increases. The first filling schedule consists of filling rates as well as intermediary holds such that filling is done in stages. USBR (2021) defines these intermediate holds, or stoppage of filling, as time windows to monitor dam conditions before filling the reservoir up to higher water levels. In addition, there is a time lag between the deformation/stress response of the dam to reservoir water elevation rise; such that, the holds can provide the lag time in order to see the governing effects of previous elevation rise. Nobari and Duncan (1972) indicate a month-long lag for observing the expected deformation and pressure after the increased water elevation, which occurred during the impoundment of Cherry Valley Dam (USA). By performing first filling in stages, designers are allowed to evaluate dam's respond to different water level stages and time to grading the data more carefully before proceeding to next level (USBR, 2021). It is also stated that the holds should be appropriately placed into the schedule but no further detailed recommendation is provided answering at which level to put the holds. The hold periods are required during initial impoundment, after an existing dam had a major repair or encountered a severe flood which is followed by a reservoir drawdown to intermediary water levels. In the Turkish practice, the holds are placed at $H/2$ and

$3H/4$ of the reservoir filling, where H is the planned depth of water in the reservoir. This study is based on the given schedule where two intermediary stops are desired during initial impoundment.

It should be noted that as the reservoir size and dam height increase, the total filling duration proportionately increases and makes inflow conditions more complex due to expanded seasonal changes, flood, and earthquake probability. Therefore, the presented study kept its scope into relatively smaller dams that both flow conditions and stage separations are easier to be idealized. However, also for larger dams, the approach of the study fills the gap in the literature in a way that instantaneous instrumentation data can be interpreted accordingly to decide if the hold process is adequate. The study findings are applicable to new dams, existing dams experienced excessive inflow rate due to flood and existing dams having a remarkable repair. Likewise, the dam model presents the movements in zoned embankments, and scope into solely dam body's behavior, when foundation movements are extracted.

The study is aimed to find the answers of the following questions and propose a certain hold duration.

- How do the seepage, displacement, and stress change during first filling? Do they converge to certain values during intermediary holds?
- How does the initial moisture content of the earth material affect the possibility of a crack formation during first filling?
- What should be the duration of intermediary holds in the filling schedule in order to monitor the dam's performance before loading up to higher water levels?
- How does the uncontrolled filling rate affect the hold duration?
- What are differences on hold duration between reservoir filling up to a low and high pool level?

1.4 Contribution of the Thesis

To the best of author's knowledge, there is not a published work on multi-stage filling practices by considering the safety of dam with respect to cracking and internal erosion, which are the most severe problems during first impoundment. Staged filling practices are studied by each countries state agencies with reference to commonly accepted rules and experience, and performed under the consequent recommendations. However, there is no published work about the monitoring duration, i.e., the intermediary holds. The contractors prefer shorter total filling durations due to economical and operational reasons. However, there is a minimum hold period with respect to dam's safety such that the dam can be cautiously loaded further, such that both minor and catastrophic consequences are minimized. Since most probable failure reason during first filling is to have a crack initiation on the body, that can enlarge and cause piping by internal erosion, this thesis approaches to the problem through the possibility of hydraulic fracturing and differential settlement by inspecting their temporal and spatial variations in the dam body using a time-dependent simulation of the impoundment. The main result the thesis propose is how long the hold should be in a staged filling schedule. The study aims to improve the understanding of embankment's reaction against filling and quantify the hold period such that it goes one step further than rule of thumb. USACE (2004) explains the need of finite-element model to be established in order to identify expected key behaviors and observations which will be used to monitor the performance of the constructed dam. Therefore, in addition to the study findings of the key parameters of stresses-deformations and seepage behavior, the approach of the study could be an example for further applications during preliminary and final design in order to establish a reliable initial filling schedule.

1.5 Literature Review

The embankment failure connected with crack formation and internal erosion have been in the scope of many researchers. Investigators put hydraulic fracturing and differential settlement as the driving force for the aforementioned failure modes. The understanding of the hydraulic fracturing in the earth dams as well as its driving forces comes with Sherard's works on the phenomena (Sherard, 1953; Sherard, 1973; Sherard, 1986). Nobari and Duncan (1972) gathered the case studies related to occurrence of hydraulic fracturing and differential settlement during initial filling of an embankment. Many investigators attribute the occurrence with lack of moisture content during placement, narrow core geometry, differently compressible materials on zoning, inappropriate or lack of filter design and high hydraulic gradients. The hydraulic fracturing occurrence is first related to arching formation of the core by Löfquist (1957). Kjaernsli and Torblaa (1968) presented that open crack formation starts due to stress distribution in the core during initial reservoir filling. Settlement and resultant arching action in the core decrease the vertical stress over overburden pressure such that hydraulic fracturing occurs as the filling process continues. Initially, it was Kjaernsli and Torblaa (1968) to point out that excessive leakage was a result of hydraulic fracturing in Hyttejuvet Dam, Norway. Widjaja et al. (1984) conducted experimental and theoretical studies on different soil groups to inspect parameters effecting hydraulic fracturing. Their study concluded that the minor principal stress of the soil is the main parameter against hydraulic fracturing. The authors stated that water content and density of the soil to be compacted have a great influence on hydraulic fracturing because of their direct effect on initial pore pressure, strength and permeability.

Rashidi and Haeri (2017) inspected earth and rockfill dam behaviors during construction and initial filling via a case study of Gavoshan Dam, Iran. Authors conducted 2D finite difference method of numerical analysis on the dam model calibrated with the instrumentation data from the real dam using back-analysis. They investigated settlement-stresses and hydraulic fracturing possibility considering arching effect. Their findings were in agreement with the expectations of Nobari and Duncan (1972) for an embankment during initial filling.

Talukdar and Dey (2019) compiled the hydraulic failures in the history starting from 1950 and concluded that hydraulic fracturing of the central core as one of the most crucial influences on the zoned embankment's safety.

Talukdar and Dey (2022) inspected the formation of cracks in earthfill dams. Authors searched for the hydraulic fracturing and differential settlement phenomena as the causes for cracks. Their study questioned the effectiveness of drainage blanket in the case of incidental cracking in the core. The study found out that the drainage blanket does not have a major effect on reducing the hydraulic fracturing possibility in the core, but it is practical for channelizing the water toward the exit drain such that satisfactory dam performance is maintained until a repair is implemented. The finite element model on GeoStudio™ software is formulated as a coupled stress/PWP analysis. Their approach via GeoStudio™ using coupled analysis is validated with measured values from the case study.

Wang et al. (2020) inspected the effect of water level fluctuations of the reservoir on the stability of an earthfill dam under transient seepage. They conducted a probabilistic approach to stability analysis and investigated the effects of uncertainties in the soil parameters. This study revealed the significant effect of the rate of water level changes on the stability.

Ghanbari and Shams Rad (2013) conducted both experimental and finite-element analysis investigating hydraulic fracturing possibility. Their finite-element model reflected initial filling of the reservoir conditions. The study suggested that the hydraulic fracturing initiation is correlated with the minor principal stress of the soil. Therefore, the authors focused on lowest minimum principal stress variation and arching phenomena through transient analysis.

A remarkable work of inspecting hydraulic fracturing phenomena during initial filling was conducted by Eslami, Ghorbani, and Shahraini (2020). The authors conducted uncoupled seepage and stress-strain analysis, in this perspective the pore-water pressures and seepage quantities are calculated by a seepage analysis, such that stress-strain analysis are based on the conditions from it. The authors commented on the possibility of a fracture occurring based on if minimum principal stress is exceeded by pore-water pressure at the same height in the core. The study also compared different filling scenarios for different total filling duration. However, intermediate hold of water level during filling schedule is not provided. It should be noted that, although the authors tried to follow a coupled stress-PWP approach for better understanding, they met serious convergence problems for a coupled analysis such that uncoupled approach is chosen. It was stated that non-linear constitutive model of dam materials creates the convergence problems.

A review of hydraulic fracturing risk on embankment dams is provided by Tran et al. (2020). For the embankments encountered hydraulic fracturing, 78% of the incidents occurred during first filling of the reservoir, that is, 28 case studies out of 36. The authors accredit differential settlement and arching actions for internal stress redistribution, such that the minor principal stress is exceeded by the pore-water pressure at low stress zones. Study compares the differential settlement for different axes of the dam body and focused on the settlement action of impervious core and shell. By the statistics provided in the study, very narrow impervious core applications for zoned embankments caused hydraulic fracturing near the core such that the design of the core updated to have thicker core slopes.

CHAPTER 2

FIRST FILLING GUIDELINES

2.1 History of Dam Failures During First Filling

Initial filling of the reservoir is the most dangerous period in the lifetime of the dam (IDNR, 2001). Due to comprehensive experience of dam failures and/or repair requirements during initial impoundment of the reservoir following the construction of the embankment, each authority approaches this process by intensive care. The initial impoundment of the reservoir is the first test of dam's capability of stability and hydrological requirements. Due to the first interaction of water and the dam material, this process is considered as one of the most important event of the dam would ever face. State agency that is responsible for hydraulic works of each country has their own design memorandum that specifies at which schedule the reservoir should be raised from empty to full reservoir level. In early years of dam constructions, no detailed look is pointed to filling process; that used to be, filling the reservoir as the inflow comes without intermediary holds and specified rates, until the major failures occurred, and lessons learned. One of the most well-known failures is Teton Dam failure, located in Idaho, USA.

After the construction of the dam, it is normally a low-flow period in the river when the filling starts such that the dam has as much time as possible to be monitored and evaluated appropriately (USBR, 2021). It is compulsory that the outlet works and spillways are finished before the reservoir starts to impound such that reservoir filling rate could be controlled in case of an unexpected inflow (USBR, 2021; USBR, 1990) and/or emergency evacuation considering the hazard potential.

2.2 Different Approaches to Multi-Staged Filling Practices

The dam-specific basis practices are not common to be published and generalized since every dam shows site-specific features. However, some published work is available to give an insight and limit some filling properties. The author presents three well documented filling practices by Turkish State Hydraulics (DSI), USBR (see Section 2.6) and IDNR through this chapter.

DSI (2014), recommends a certain filling schedule for earthfill dams within their design standards. As on Fig. 2.1, the impoundment is planned in three steps. The filling rate is not specified for the lower half of the dam, and called this rate as uncontrolled filling. During this uncontrolled filling phase, the dam is being loaded by a relatively larger loads from its desired capacity; therefore, no specific rate is planned (See Section 2.6). However, higher rates result in higher hydraulic gradient that would cause piping initiation. Therefore, even an uncontrolled filling does not necessarily let every rate. In DSI specifications, the filling rate after the mid-height of the dam; called as controlled filling is restricted to 0.3 m/day for embankment dams. Outlet works are operated accordingly to maintain this rate constant. The intermediary holds are named as monitoring periods but there is no further explanations for this duration.

Division of Water at Indiana Department of Natural Resources (IDNR) presents a stage filling in three steps. By their typical filling schedule, pool level is divided into three. Although there is no information about the hold duration between different pools, filling rates are specified for each stage (IDNR, 2001). The filling rate is not restricted for the first 1/3 of the pool level. Rate is limited to be less than 0.6 m/day for the second 1/3 of pool level. For the final one third of the reservoir, filling rate restricted to 0.3 m/day. This schedule shows the same principals with other authorities', because the filling rate is limited to lower values as water level gets higher and the rate is uncontrolled for lower pool levels as in USBR (2021) and DSI (2014).

2.3 Cause of Failures

The failure of an earth or rockfill dam could happen by overtopping, slope failure, sliding, and internal erosion (Terzaghi, Peck, & Mesri, 1996). In the case of initial

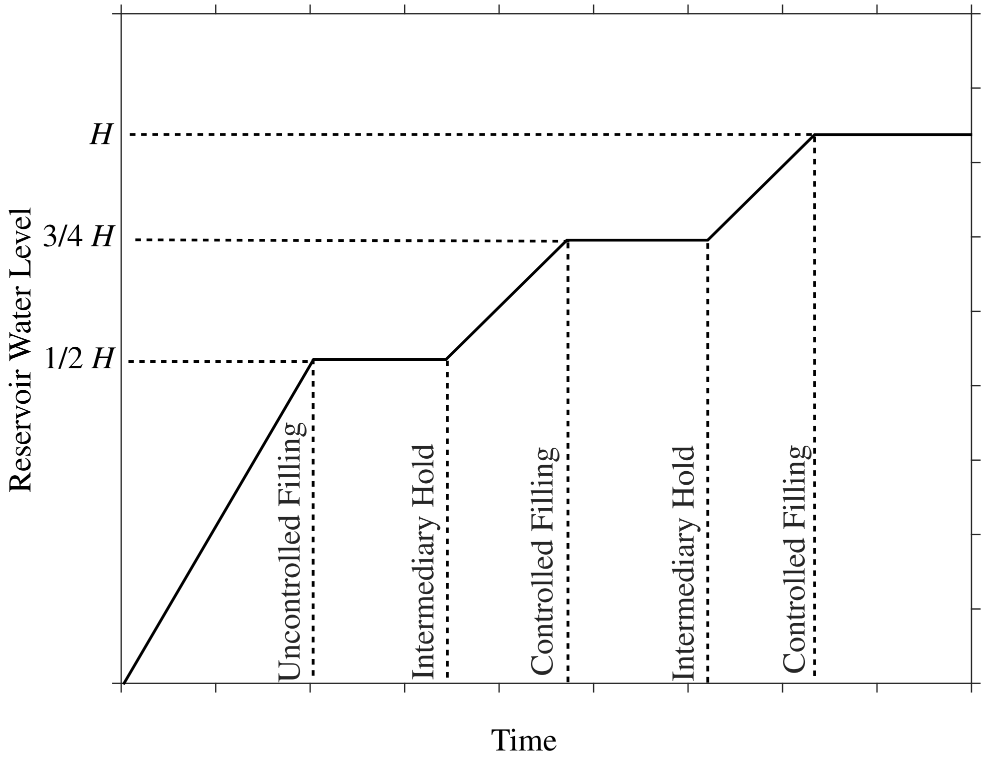


Figure 2.1: Turkish practice (DSI, 2014)

filling, a possible slope failure or internal erosion could lead to failure. The internal erosion is a serious concern for relatively impervious core zoned embankments (USBR, 2014b). Terzaghi et al. (1996) states the reasoning of the internal erosion in zoned embankments as pre-existing or recent cracks through the body. The authors also suggest differential settlements as the cause of crack formation.

This thesis, therefore, concerns with the occurrence of internal erosion and slope failure during reservoir impoundment by considering hydraulic fracturing and differential settlement.

2.3.1 Differential Settlement

Differential settlement between the shells and core is an acknowledged reason for crack formation as well as different crest shoulders' movements (USBR, 2014a; Sherard, 1986; USBR, 2014b). The well-known reasons are inadequate initial moisture content during placement and zoning the body with materials of different stiffness for differential cracking. Nobari and Duncan (1972) gather some the real-dam crack formation explanations by differential settlement during initial filling. Some of them are Cherry Valley Dam (USA) in 1963, Cougar Dam (USA) in 1966, Gepatsch Dam (Austria) in 1967 and Round Butte Dam (USA) in 1964. Differential settlement of shell and core zones are inspected in Chapter 5.

2.3.2 Hydraulic Fracturing

The hydraulic fracturing phenomenon is a common reasoning for crack formation, that is followed by internal erosion, piping. It is remarkably expected in zoned embankments and the historical record of the hydraulic fracturing mainly occurs during initial filling (Sherard, 1986). The phenomenon occurs when the sum of minor principal stress and the tensile strength of the soil is exceeded by the hydraulic pressures (USBR, 2014b; Lo & Kaniaru, 1990). The hydraulic fracturing is a recognized application for pressurized-water drilling in other industries, however, the hydraulic fracturing term in this study refers to pore-water pressure in the soil due to reservoir rise and cause fractures in the low stress zones in the core. The differential settlement induced arching effect in the core is considered to be the main reason for hydraulic fracturing; yet, erosive leaks through the central core without adequate filters and water filling into the already existing cracks in the core also causes the phenomenon (Sherard, 1986). The cracks due to improper design/application or shrinkage/swelling are expected for embankments during or after the construction. Normally, these open cracks cannot propagate through the core due to high internal compressive stresses (Sherard, 1986). Nevertheless, differential settlement (See subsection 2.3.1) and different stiffness material zoning induced stress transfer from core to shells (See subsection 2.3.3) reduces the internal compressive stress in the core which makes it vulnerable to crack propagation and hydraulic fracturing. The soil cannot show any opposition to water entry in this case. As the pore-water pressure increases with reservoir rise, continuing to filling would make the problem more severe. Therefore, if the low stress zones are identified through monitoring at intermediary holds, filling schedule could be revised and followed by proper precautions. High water pressure entrance to low stress zones induced by arching had an important role in the Teton Dam failure (USBR, 2014b). According to Nobari et al. (1973), the hydraulic fracturing likely start from a low effective stress point, then it propagates through the core rather than occurring all across the core suddenly.

The tensile stress of the soil has an additional preventive measure after the effective stress reduces to zero; however, tensile stress of the embankment soil is accepted not to be counted, conservatively. This assumption is confirmed by Nobari et al. (1973) from their conclusion after experimental studies. Therefore, when the effective stress reduces to zero, it is recognized as the failure. Penman (1986) states that the resistance of soil particles is dragged away by water interaction when the effective stress is under zero.

Kjaernsli and Torblaa (1968) investigated Hyttejuvet Dam in Norway, when an unexpected disastrous leakage was observed leading dam to failure, and determined hydraulic fracturing caused by the arching of core as the responsible.

It is not possible to model a hypothetical dam and point out a certain location of fracturing or a definite safety measure against it. However, this study is within the scope of observing the temporal and spatial variations of minor principal stress and vertical effective stress during reservoir filling in order to find out the possibility of hydraulic fracturing during a water hold.

2.3.3 Arching Effect

The arching ratio is the indicator of the arching effect between the core and shell zones of simple zoned embankments. The arching effect is one of the main reasons of hydraulic fracturing. The shell and core show contrasting settlement and strength capacity under the water load from reservoir filling. The total stress is transferred to shell from the cohesive core that makes core more lightweight and the earth pressure is reduced by arching (USBR, 2014b). When the smaller total stress coincides with higher pore-water pressures at the same elevation, the hydraulic fracturing occurs in the core. The stress transfer can be monitored via the installed instruments in the core if the observed pressures are somehow less than the expected embankment weight on top of it. Sherard (1986) states that the instrumented pressure in the core were less than 30% of the soil weight above during the construction of the John Martin Dam

(USA). Another important remark from Sherard (1986) for stress transfer indicator occurs in a Swedish dam when the actual settlement of crest was 1/12 of the expected settlement signaling arching. Hydraulic fracturing due to arching effect is observed in Hyttejuvet Dam (Norway) (Kjaernsli & Torblaa, 1968) and Balderhead Dam (England) (Nobari & Duncan, 1972) and many more. The arching effect is identified from the field data, which indicates how much of the load is carried on the core than its designed expectations. Therefore, the arching effect is not involved into the study on the hypothetical dam.

2.4 Monitoring and Instrumentation During First Filling

The role of instrumentation is crucial for the initial filling stage of a new dam since there is no historic performance data yet. This indicates a great potential of uncertainties on the embankment's performance during its first interaction with water. Therefore, the most intensive monitoring is required during this time, such that around-the-clock presence of experienced inspectors on site is necessary (USBR, 2014a). In addition to continuous visual inspections; readings of seepage (primary concern), stresses, settlement and deflection data from installed instruments are crucial to detect any possible crack formation (USBR, 2014a). ICOLD (1988) emphasizes the close and frequent monitoring of the embankment during first filling.

In terms of placement of the devices, it is hard to define minimum instrumentation requirements. The need for monitoring and instrumentation is again dam-specific basis. The general idea is to place them such that the key parameters of cracking, movement and seepage readings are sufficiently monitored. Both USBR (2021) and USBR (2014a) points out the high-value, low-cost approach while deciding the instrument's distribution on the body.

2.5 Recommendations on First Filling

All state agencies responsible from state hydraulics, have official documents consisting a section about initial filling, but mostly the information does not go further than a general look at the filling process. These documents consist of evacuation plan, hazard potential, monitoring frequency but no certain recommendation on a filling rate or a schedule where intermediary holds exist. It is, of course, a unique practice for each reservoir that how to fill it. Purpose of the dam, geometries and hydraulic features of the dam, type of material used, and hydrological features of the region differ too much such that the practice should be dam-specific. However, state agencies try to generalize this process as much as possible, in a such planned program having an adequate time for monitoring and performance evaluation of the dam. As a result, regulations depending on dam material is proposed.

The American practices has the most clear and specific recommendations relating to reservoir filling which is documented in USBR (2021) as First Filling Guidelines. Therefore, a review of it is given in this section. This document is formed by Assistant Commissioner - Engineering and Research (ACER) as Memorandum No. DES-2, "Reservoir Filling Criteria Preparation" initially, but was retired when ACER reorganizes as Technical Service Center (TSC). The Memorandum DES-2 is also assigned as the responsible for filling rate decisions by USBR (1990). Following intensive research on USBR and USACE manuals about first filling information, it should be mentioned that as a common point for all, there should be a decision/technical memorandum for impoundment referencing aforementioned DES-2 document for detailed information. However, the author has not yet to find this document in the open web platform. Therefore, USBR (2021) is considered as the final and the only document about specific information related to initial filling. Filling is divided into two parts in the document, filling lower half of the dam and the upper half.

First filling guidelines consisting of first filling control and evacuation ability of the dam, should be finished prior to reservoir filling. The intermediary holds of water levels are provided to give adequate time window of the dam performance and provide ample time in case of a problem to issue the problem and warn the public. Filling rates may be revised for filling upper parts of the reservoir if needed, depending on the monitoring data from installed instruments and other inspections during the intermediary holds.

According to USBR (2021), the filling has special requirements for general surveillance, reading and reporting instrumentation data as well as normal and emergency operating procedures. These are given by their necessary description as follows:

- *Onsite attendance:* 24-hour surveillance by trained observers, including operators and designers.
- *Visual observations:* In case of visible crack formation, seepage, slope instability and other evidence of abnormal functioning. The site should be adequately lit to permit night observations.
- *Reading of instruments:* At frequent intervals but may be continuous and instantaneous.
- *Reporting of monitoring:* The Division Chiefs shall be on alert and should be notified immediately on any abnormal conditions on the data from installed instruments. If the initial fill would be to a low pool (less than half of the dam's height), nominal surveillance and monitoring is acceptable. In case of a relatively high pool of filling is intended; extensive surveillance and monitoring is required. In general, it is vital to closely monitor the initial filling at critical elevations.
- *Normal operating procedures:* These operations are composed of maintaining normal predetermined operating plan, preserving filling rate and meeting project requirements.
- *Emergency procedures:* The outlined emergency plan; that is detailed in USBR (2021) is put into action.
- *Procedures to be followed after earthquakes:* Similar procedure that is used in existing dams in case of an earthquake will be considered during initial filling also.

2.6 Filling Rates

The rate of filling is one of the main components that affect embankment's safety during initial filling. The rate determination is normally based on the response of the embankment to increasing hydrostatic loading and typical ranges are 0.15 to 0.6 m/day (FEMA, 2005). The American practice of defining filling rates is located in USBR (2021). As stated before, by dam-specific basis the filling schedule is set and the major influences for determining a filling rate are listed with explanations as follows in USBR (2021):

1. *Purpose of the reservoir:* Whether a storage or a flood control the dam is designed, affect the filling rate. For storage purposed dams, normally, the extra amount from the downstream requirements is stored.

2. *Requirements for initiation of filling:* As previously mentioned, filling would generally begin during a low-flow period in order to allow as much time as possible for monitoring and evaluation purposes. When practical construction schedule and commitments limit the reservoir to be filled prior to completion of the outlet works, additional precautions should be taken (lower rate) into consideration. USBR (1990) confirms that initial filling plan should take into account completion of the second-stage construction when outlet works are used for diversion.

3. *Type of dam:* Dam type and material affect filling rate primarily. The first soil-water interaction (saturation) is an overly sensitive stage for the earth and rockfill dams since earth material acts differently during initial saturation. Therefore, slower rates are preferred such that the dam has enough time to response to saturation. But for concrete dams, filling rates are less restricted since both the dam and its foundation are relatively not sensitive to saturation.

4. *Geology and seismicity of the dam foundation and reservoir:* The filling rate is prone to be restricted due to physical properties of the geologic materials. These problems may be composed of excessive seepage, landslides in the reservoir and/or reservoir-induced seismicity.

5. *Hazard potential:* The hazard potential for the downstream is another factor for limiting filling rates. Adequate time should be provided for issuing a warning and the public to response.
6. *Hydrology (inflow):* Inflow may divided into seasonal baseflows, controlled inflows and flood flows (USBR, 2021) (USBR, 1990). Seasonal baseflow permit slow filling of the reservoir. Controlled inflow from an upstream reservoir would make ideal/desired filling rates possible to be established. Effect of unpredictable flood flows should be evaluated for each reservoir, in this scenario, higher filling rates are acceptable as long as flood is controlled. However, it may be also necessary to lower the reservoir after the flood event and place an intermediary water hold period to monitor dam's performance afterwards.
7. *Release provisions:* Downstream water and evacuation requirements govern the outlet work's capacity, which should be sufficiently enough to limit the filling rate.
8. *Design considerations:* Different-purpose design considerations also an influence for defining a filling rate. These may be additional holds or different rates depending on the location and topographic conditions of the reservoir and response time of instrumentation.

USBR (2021) presents filling rate recommendations which will be unique for each dam, however, the general recommendations also exist. The rates would be different for lower and upper reservoir elevation ranges. For embankment dams, the normal and common rate is given as 0.3 m/day in the ranges of less than 0.3 to 1 m/day. As it is stated before, the rate is crucially important for embankments, since earth material acts differently in case of first saturation.

Normally, filling rates are not specified for the lower half of the depth of the reservoir since the dam will only receive a fraction of its designed load (USBR, 2021). Naturally, filling is expected to be faster for the lower half, due to smaller storage to elevation relation. However, internal erosion and piping risk should be evaluated since higher seepage rates lead to higher hydraulic gradient.

For the upper portion, filling rate is limited to be less than 0.3 m/day for embankment dams in order to control hydraulic gradient and water load which decrease the possibility of internal erosion as well as allowing an ample time for monitoring and evaluation. Limiting the rate to 0.3 m/day is a common specification in the literature (DSI, 2014; USBR, 2011b; IDNR, 2001).

The outlet works play a vital role for defining filling rates since they will be used to control the reservoir by adjusting outflows. Therefore, their capacity as well as the rate of filling should be documented as a technical or decision memorandum in advance of impoundment. It should be noted that the outlet works are out of scope of this study, therefore accepted as appropriately constructed and meeting evacuation criteria enough not to govern for filling rate decision.

2.7 Necessary Parameters via Instrumentation

The instrumentation types to monitor different parameters in the dam body are suggested by USBR (2014a). Piezometers and observations wells are designed for seepage monitoring, total pressure cells for stress monitoring, internal vertical movement (IVM) devices for settlement monitoring, etc. For the current study model, continuous data of total/effective stresses, horizontal-vertical deflection and pore-water pressure are in consideration. The data of the stated parameters will be transformed into minimum principal stress in Chapter 5.

2.8 Previous Technical Reports Scoping into Initial Filling

The embankment behavior under first filling had been put into the subject of notable researches. Most remarkably, Nobari and Duncan (1972) combined previous investigations of different embankments during their initial filling in a technical report. Their outcome is well-grouped common behaviors of simple zoned embankments during first soil-water interaction. The four main behaviors that Nobari and Duncan (1972) pointed out, are illustrated as Figure 2.2.

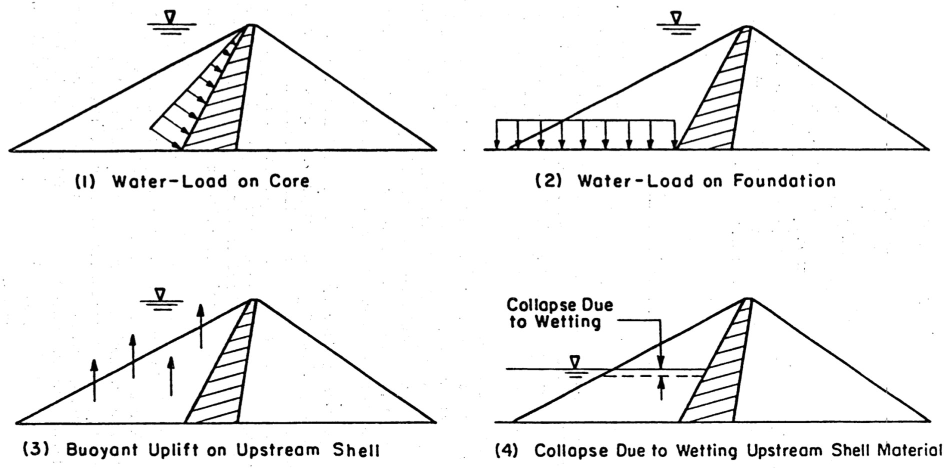


Figure 2.2: Embankment behavior (Nobari & Duncan, 1972)

These four different effects of a reservoir impoundment behind a zoned embankment are:

1. *The water load on the core zone:* This load pushes core to the downstream (D/S) and downward.
2. *The water load on the upstream (U/S) foundation:* This creates upstream and downward movements.
3. *The uplift forces on upstream shell:* Consequent upward movements in this zone.
4. *The collapse of U/S shell due to wetting:* Downward movement of this zone.

Nobari and Duncan (1972) denote possible various movements in dams depending on the magnitude of the given the four effects. In a general meaning, apart from seepage forces, stress changes and relative movements would form cracks during initial impoundment. Their investigation reveals that the water load causes downstream movement of the dam, especially in the higher water level stages. The same investigation came up with a clear conclusion about the importance of placing the fill in its optimum water content as the otherwise is the primary reason for cracking. The differential movements (Sherard, 1953) and hydraulic fracturing are counted as two reasons causing cracks. Kjaernsli and Torblaa (1968) pointed out it is hydraulic fracturing caused threatening cracks within embankments. When the total stress in the core is less than the water pressure at the same elevation, hydraulic fracturing occurs (Nobari & Duncan, 1972). The fracture may lead to open cracks; thus, leakage and internal erosion, through the core zone.

2.9 Optimum Moisture Content

The crucial effect of moisture content to earth material's dependence to crack formation had been noticed in early 1900's (Sherard, 1953). Sherard pointed out three properties of an earth material compacted without adequate water content. These are:

1. The relatively high permeability of dry material.
2. Higher impact from saturation, that cause remarkable settlement.
3. The stiffer and more brittle material.

Just as the upstream movement due to wetting of upstream shell described by Nobari and Duncan (1972), the upstream shell is pulled off from the brittle/stiff downstream shell and core in case of low moisture content. Sherard (1973) proposed that it was the material's low water content that drive upstream shell transverse crack on El Isiro Dam, Venezuela. For the dams that develop pore pressure during construction due consolidation, it is not expected for pore-water pressures to exceed minimum principal stresses, since principal stress in this case is too high (Sherard, 1986).

Penman and Charles (1981) and Penman (1986) recommended that the core should be placed by the wet of the optimum moisture content in order to generate high construction pore-water pressures that will prevent hydraulic fracturing.

CHAPTER 3

THEORY

3.1 Seepage Analysis

The spatial and temporal variations of pore water pressure and seepage throughout the domain lies on the famous Darcy's Law, published in 1856 by the French engineer Henri Darcy, where hydraulic gradient creates the water movement. The equation is:

$$q = -Ki \tag{3.1}$$

where q is the specific discharge (discharge per unit area), K is the hydraulic conductivity and i is the hydraulic gradient. The equation is modified for two-dimensional seepage problems (Papagianakis and Fredlund (1984); Geo-Slope Int. Ltd. (2012)) as follows:

$$\frac{\partial}{\partial_x}\left(K_x\frac{\partial H}{\partial x}\right)+\frac{\partial}{\partial_y}\left(K_y\frac{\partial H}{\partial y}\right)+Q' = \frac{\partial \theta}{\partial t} \tag{3.2}$$

where K_x and K_y are the hydraulic conductivities in x and y directions, H is the total head composed of pressure head (h) and elevation head (z), Q' is the boundary flux and θ is the volumetric water content (VWC) and t is the time. The rate of change of the soil storage is formed by the change of flow in x and y directions, and the external flux applied. Since there is no change of soil storage for steady-state conditions, Eq. 3.2 turns into following equation:

$$\frac{\partial}{\partial_x}\left(K_x\frac{\partial H}{\partial x}\right)+\frac{\partial}{\partial_y}\left(K_y\frac{\partial H}{\partial y}\right)+Q' = 0 \tag{3.3}$$

The change in soil storage, i.e., the volumetric water content, is a result of the changes in stress and soil properties. For both saturated and unsaturated conditions, the stress state is described by two state variables as in Fredlund and Morgenstern (1976) and Fredlund and Morgenstern (1977). These variables are $(\sigma - u_a)$ and $(u_a - u_w)$ where σ is total stress, u_a is the pore-air pressure, and u_w is the pore-water pressure. SEEP/W formulation assumes constant total stress; therefore, the loading and unloading effect cannot be seen. In addition, pore-air pressure remains constant at atmospheric pressure during transient processes. Therefore, the first term $(\sigma - u_a)$ has no effect on the soil storage change. The change is only related to the second term $(u_a - u_w)$, consequently, to u_w. In the light of the explanations of the terms, the change in volumetric water content reduces to the following equation (Eq. 3.4) and rearranged in terms of total head (H) and elevation as Eq. 3.5:

$$\partial\theta = m_w \partial u_w \tag{3.4}$$

$$\partial\theta = m_w \gamma_w \partial(H - y) \tag{3.5}$$

where m_w is the slope of the storage curve and y is the elevation. For a constant elevation, Eq. 3.2 turns into the following equation (Geo-Slope Int. Ltd., 2012):

$$\frac{\partial}{\partial_x}\left(K_x \frac{\partial H}{\partial x}\right) + \frac{\partial}{\partial_y}\left(K_y \frac{\partial H}{\partial y}\right) + Q' = m_w \gamma_w \frac{\partial H}{\partial t} \tag{3.6}$$

Eq. 3.6 is the governing partial differential equation on SEEP/W analysis where it is solved by finite element method. The well-known finite element method divides the domain into a number of interconnected elements; that is meshing. Between the defined boundary conditions, the finite element seepage equation (see Eq. 3.7) is solved for each element until this web of elements are all connected to each other for a consistent solution through the domain. It should be noted that, the differential equation is approximated as the finite element seepage equation by using Galerkin method of weighed residual (Geo-Slope Int. Ltd., 2012).

$$[K]\{H\} + [M]\{H\},t = \{Q\} \tag{3.7}$$

Eq. 3.7 for transient seepage analysis reduces to Eq. 3.8 due to the absence of time dependent head function for steady-state analysis.

$$[K]\{H\} = \{Q\} \tag{3.8}$$

where

K = element characteristic matrix

M= element mass matrix

Q = element applied flux vector

H = vector of nodal heads

SEEP/W is a well-known module of GeoStudio™ for both steady state and transient types water transfer analyses that is selected as the FEM solution to Darcy's Law. SEEP/W is successfully used in previous studies of Calamak and Yanmaz (2017), Calamak et al. (2018), Calamak et al. (2020), Calamak and Yanmaz (2018).

3.2 Hydraulic Conductivity of Unsaturated Soils

In the case of transient analyses, as the phreatic surface changes, the material can change its saturation level that is a function of hydraulic conductivity. Consequently, the need for hydraulic conductivity estimation for partially saturated or unsaturated material arises. Since the water content changes spatially and temporally, a function specifying the water content change with respect to different pressures of soil. This pressure is called as matric suction which is the difference between air (u_a) and water pressure (u_w) for unsaturated soil. GeoStudio™ allows user to define this function (soil-water characteristic curve) using well-known methods; such as, Fredlund and Xing (1994) and van Genuchten (1980) that both provides closed form equations. This study adopted van Genuchten (1980) method since its common practices through similar studies for both volumetric water content (See Eq. 3.9) and the consequent hydraulic conductivity function (Eq. 3.10) estimation. van Genuchten (1980) formulates unsaturated hydraulic conductivity relative to known saturated value and three curve fitting parameters as shown below. It should be noted that this approach is based on the Mualem (1976)'s theory.

$$\Theta_w = \Theta_r + \frac{\Theta_s - \Theta_r}{\left[1 + \left(\frac{\Psi}{a}\right)^n\right]^m} \tag{3.9}$$

$$K_w = K_s \frac{\left[1 - \left(a\Psi^{(n-1)}\right)\left(1 + \left(a\Psi^n\right)^{-m}\right)\right]^2}{\left(\left((1 + a\Psi)^n\right)^{\frac{m}{2}}\right)} \tag{3.10}$$

where:

$\Theta_w =$ the volumetric water content,

$\Theta_s =$ the saturated volumetric water content,

$\Theta_r =$ the residual volumetric water content,

$\Psi =$ the negative pore-water pressure (matric suction),

$K_s =$ saturated hydraulic conductivity,

$a, n, m =$ curve fitting parameters.

In order to illustrate the soil-water characteristic curve for a typical material, Figure 3.1 is provided from the work of Fredlund and Xing (1994).

The given theory is implemented into both SEEP/W and SIGMA/W modules for each material as a nonlinear VWC function that is used to estimate nonlinear hydraulic conductivity function for unsaturated soils.

3.3 Slope Stability Analysis

Limit Equilibrium Method (LEM) is preferred for slope failure theory that is dividing the domain into rigid material interslices. The LEM calculates two types of factor of safety resulted from moment equilibrium and horizontal force equilibrium for the critical slip surface as in the work of Spencer (1967). Even though there are different approaches to LEM, such as Morgenstern-Price, Bishop or Janbu, USBR (2011b)

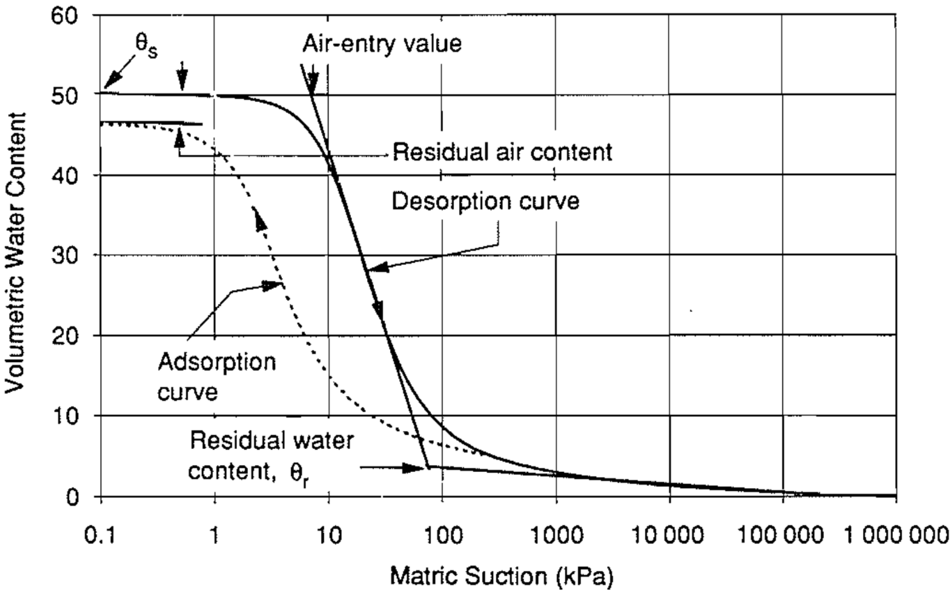


Figure 3.1: Soil-water characteristic curve (Fredlund & Xing, 1994)

recommends Spencer method to be used for calculations performed by LEM. Spencer method is an iterative procedure, changing the constant ratio between shear to normal force ratio between interslices until the same factor of safety results from both moment and force equilibrium. The interslice shear-normal ratio is the same between all slides.

The limit equilibrium factor of safety for moment equilibrium and force equilibrium given as follows:

$$F_m = \frac{\sum (c'\beta R + (N - u\beta) R \tan\phi')}{\sum Wx - \sum Nf \pm \sum Dd} \tag{3.11}$$

and,

$$F_f = \frac{\sum (c'\beta \cos\alpha + (N - u\beta) \tan\phi' \cos\alpha)}{\sum N \sin\alpha - \sum D \cos\omega} \tag{3.12}$$

where the terms are:

c'	=	effective cohesion
ϕ'	=	effective angle of friction
u	=	pore-water pressure
N	=	slice base normal force
W	=	slice weight
D	=	concentrated point load
β, R, x, f, d, ω	=	geometric parameters
α	=	inclination of slice base

The given limit equilibrium formulation computes F_m and F_f for a range of lambda (λ) values and plots factor of safety (FoS) versus λ graph. For Spencer method, factor of safety is determined when F_m and F_f curves cross. The factor of safety against slope failure is conducted using SLOPE/W package. The SLOPE/W is successfully implemented to slope stability analysis in the previous study of Calamak et al. (2020).

Trial slip surfaces are ranged between pre-specified entry exit points; that is, entry and exit technique in the software.

3.4 Stress-Strain Modeling

3.4.1 Finite Element Method

The stress-strain and seepage modeling in GeoStudio™ uses finite element method (FEM) as a numerical approach to partial differential equations (PDEs) solutions where the distribution of pressures and deformations varies both temporally and spatially. The governing equations often come from fundamental considerations of the physical environment. FEM discretizes the complex domain into a number of finite elements using defined geometry and material properties and tries to obtain a numerical solution across the domain between given boundary conditions.

3.4.2 Coupled Stress and Pore-Water Pressure Formulation

The theory of consolidation had been initiated by Biot (1941) by assuming an isotropic, linear elastic, small strains, incompressible pore-fluid and Darcian flow (Geo-Slope Int. Ltd., 2022). The later coupled formulation is provided by Dakshanamurthy, Fredlund, and Rahardjo (1984), that is followed by the finite element method solution of Wong, Fredlund, and Krahn (1998). For GeoStudio™ coupled consolidation analysis, the governing equations of stress-strain response and pore fluid transfer through the soil matrix are as follows, respectively:

$$\rho_w \left(\theta \beta_w \frac{\partial u_w}{\partial t} - S \frac{\partial \varepsilon_p}{\partial t} - m_w \frac{\partial u_w}{\partial t} \right) = \frac{\partial}{\partial y} \left[\frac{K_w}{g} \left(\frac{\partial u_w}{\partial y} + \rho_w g \frac{\partial y}{\partial y} \right) \right] \tag{3.13}$$

$$\{\delta\sigma\} = [D'] \{\delta\varepsilon\} + \{m\} \alpha \delta u_w \tag{3.14}$$

in which the terms are:

ρ_w = density of water
θ = volumetric water content
β_w = isothermal compressibility of water
u_w = pore-water pressure
S = degree of saturation
ε_p = volumetric strain
m_w= slope of VWC function
K_w= isothermal liquid water hydraulic conductivity
g = gravitational acceleration
D' = effective stress stiffness matrix
m = matrix to reflect isotropic water pressure
α = the coefficient between 0 and 1.0 (see Eq. 3.15)

The coefficient α, depending on degree of saturation, is assumed as equal to the effective saturation (van Genuchten, 1980):

$$\alpha = S_e = \frac{\theta_w - \theta_r}{\theta_s - \theta_r} \tag{3.15}$$

The mass conservation and equilibrium are achieved by solving Eq. 3.13 and Eq. 3.14 for each time step.

Coupled pore-water pressure and stress analysis, i.e., soil consolidation, is the remarkable feature of the GeoStudio™ software. Normally, stress-strain analyses are conducted independently from the soil volume change and considers only stress-deformation equations. Thanks to coupled analyses, stress-deformations and seepage dissipation equations are solved simultaneously; thus, while the stress change on soil contributes the seepage behavior accordingly and the change in pore-water pressure from seepage solutions does affect stress-deformation calculations such that the effective stress changes are determined (Geo-Slope Int. Ltd., 2013). The coupled formulation requires a simultaneous solution of three equations for each node; these are, two equilibrium (displacement) and one continuity (flow) equations resulting in both pore-water pressure and displacement changes. SIGMA/W module of GeoStudio™ is used for this purpose. The successful use of the coupled analysis on GeoStudio™ is implemented in the previous study of Talukdar and Dey (2022). It is worth mentioning that the minor principal stress conversion is manually conducted using Eq. 3.16.

$$\sigma_3 = \frac{\sigma_x + \sigma_y}{2} - \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2} \tag{3.16}$$

in which the terms are:

σ_3 =	minimum principal stress
σ_x =	total horizontal stress
σ_y =	total vertical stress
τ_{xy}=	shear stress

3.5 Solution Tools

The formulation and the theory behind each analysis with respect to their intended objective are detailed in this chapter. The study is conducted using different modules of GeoStudio™. SEEP/W, SLOPE/W and SIGMA/W modules are used for each specific purpose. It should be noted that the study is conducted on GeoStudio™ 2021.4 version 11.3.2.23783.

3.6 Convergence of Analysis Results

Numerical solution to governing FEM equation on GeoStudio™ is based on the iterative root finder. The governing equation for transient seepage analyses on SEEP/W, slope stability on SLOPE/W and transient coupled stress-PWP on SIGMA/W are presented in the earlier sections.

For SIGMA/W solutions, convergence scheme for stress is composed of Relative Displacements/Residual Loads with a tolerable error of 0.005 and tolerable error for stress update as 0.001. For water side of the equation, iteration comparison criteria has maximum pressure head difference as 0.005 m and iterating using under-relaxation criteria. The selected tolerable errors give minimum numerical noise and relatively fast solution as well as being in agreement with the similar published works that GeoStudio™ uses.

The convergence is obtained easily for SEEP/W and SLOPE/W analysis. However, achieving a successful convergence for transient coupled stress-PWP analysis takes an extra effort of trial and error between time step and mesh size selection. After severe convergence problems to coupled equation in the present study, the author had worked on finding a similar condition just like Courant and Péclet numbers; that is, the ratio of time step and mesh size approach for transfer equations, to arrange mesh size and time step correlation for successive results. However, no similar condition exist for coupled analysis. The author reached out to GeoStudio™ Support officials for their approval and confirmation on using transient coupled analysis for modeling the initial filling of a reservoir (K. Dompierre, personal communication, October 3, 2022). The convergence problem is solved by simplifying the material definitions.

van Genuchten's approach to hydraulic conductivity as a function of volumetric water content creates a need of solutions to non-linear equations. Therefore, author's approach is to simplify the generation of volumetric water content function by using sample functions of GeoStudio™ rather than Van Genuchten fitting parameters. It was, then, possible to solve the water transfer and force equilibrium equations simultaneously for each time step and have a transient solution.

It should be noted that the analyses are conducted on Windows 10 computer having a Intel Core i7 950 3.07 GHz processor with 16 GB ram. The average duration of each coupled analysis takes 10,324 seconds. Additionally, the numerical model of 20 m high embankment is discritized into 1,299 elements with 1406 node when a mesh size of k=1 m is selected.

3.7 Time and Mesh Discretization

The convergence to multiple non-linear equations requires a successive time step and mesh size correlation. There is no Courant and Péclet numbers for the solution of the coupled equation. GeoStudio™ manuals recommend trial and error for finding a matching mesh size and time step. After numerous trial and error processes, the successive result is achieved by choosing exponential increase in time step with and initial increment size of 0.05 days with global mesh size of 1 m. In detail, total duration of 180 days transient analysis is divided into number of 360 exponentially increasing time steps with an initial increment size of 0.05 days.

3.8 Modeling Basics in GeoStudio™

The package software GeoStudio 2021.4™ provides user-friendly modeling environment. The basic scheme of creating a model and obtaining a solution is listed in steps as:

1. Choosing the analysis type and the estimation method followed by convergence settings,

2. Sketching the domain by poly-lines and dividing into different regions for each different material,
3. Defining material properties,
4. Defining boundary conditions,
5. Discretization of the domain: meshing,
6. Calling the solver,
7. Checking the convergence reliability of the results
8. Plotting appropriate result data for better interpretations.

The following chapter, Chapter 4, consists of the application model on GeoStudio™ and the result data obtained from various analyses.

CHAPTER 4

APPLICATION STUDY

The earthfill dams are the most common and the oldest dam type (USBR, 1987; Terzaghi et al., 1996), since locally available and excavated natural materials require minimum processing for the construction (USBR, 2011a). A typical cross section of central core zoned embankment with a chimney drain is seen in Figure 4.1. A toe drain is also included in the design to model seepage dissipation out of the domain. The operation level of the reservoir, H, is selected to be at 17 m of the 20 m dam model using the regression analysis on the statistical database of embankments located in Turkey (Yanmaz, 2022). The typical first filling application of DSI is implemented to the model. It should be noted that dam on the fully load condition case are not considered in terms of hydraulic fracturing throughout the study, since the durations of intermediary holds during first filling are in the scope.

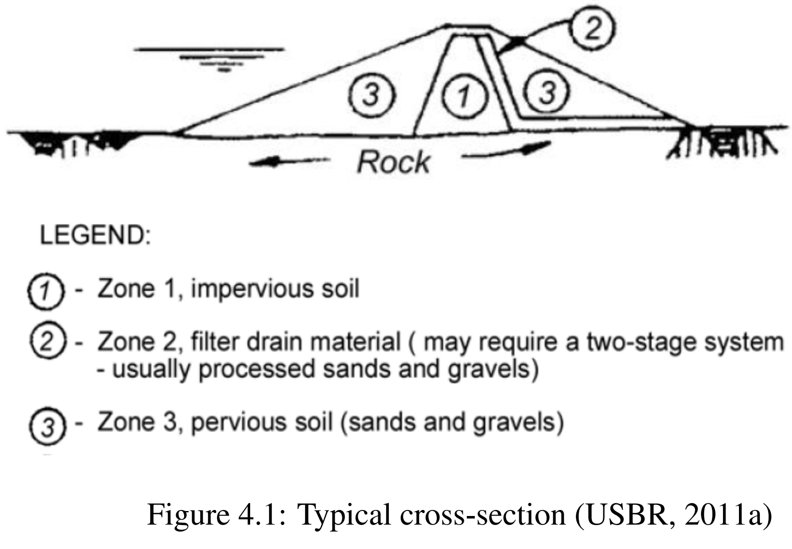


Figure 4.1: Typical cross-section (USBR, 2011a)

4.1 Assumptions

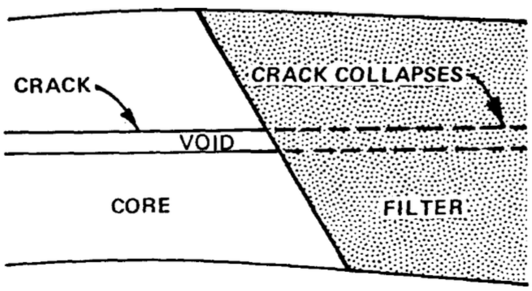
The study is based on several assumptions to simplify the problem and narrow the scope of the study. These are listed as follows:

- Construction of the dam is successfully finished prior to reservoir filling.
- The fill material is adequately compacted and placed in their optimum moisture content by appropriate construction of the lifts. There is no crack formation prior to filling.
- The filling starts at low-flow months and there is no unexpected flood during filling. The rate is constant between each intermediary holds by the properly sized and functioned outlet works.
- The foundation is impervious and rigid so that behavior of the dam body solely is under the scope.

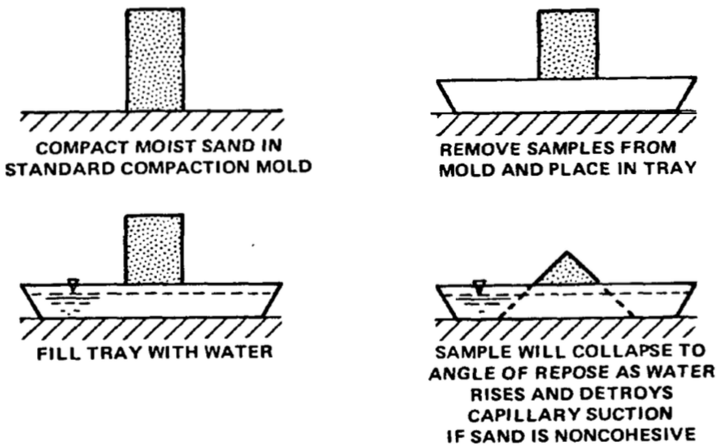
4.2 Application Model

4.2.1 The Zoning

The reservoir filling practices are conducted on a simple zoned, central impervious core earthfill dam with a chimney drain. Zoning is commonly preferred since it provides adequate strength, seepage and cracking controlling (USBR, 2012). Cracking during initial filling occurs on simple zoned embankments more frequently due to different behaviors of different zone materials. Due to its popularity for being a research subject as the experiences of Nobari and Duncan (1972) and Sherard (1986), the application model is chosen to be zoned embankment accordingly. However, the propagation of the cracks are restricted thanks to filter zones and heavy soil compression (USACE, 1986). The self-healing by the collapse of the non-cohesive filter that prevents crack propagation coming from downstream of the core is illustrated by USACE (1986) and is shown below as Figure 4.2. This illustration clearly shows the importance of the filter material to prevent internal erosion. Both upstream and downstream sides of the cores are considered to be properly filtered such that the particle mitigation is prohibited. However, the filters are not a part of the model since their effect on the dam's behavior in terms of seepage and stress quantities are negligible, but their contribution is related to particle mitigation stoppage. In order to work on a real-life like model and generalize the results for alike dams, the literature; that is, the recommendations on an embankment design are reviewed broadly to create a proper hypothetical model. Although most of the recommendations and design manuals for different countries' practices for embankment design are considered, properties mainly recommended by USBR and USACE are followed in this study. Among other recommendations on various sources, USBR manuals are prioritized for selecting the parameter values due to their complementary and inclusive information/experience on the dam design criteria and experimental research on specifically compacted properties of earth materials. A chimney drain, that ensure seepage control from the core to downstream shell (USACE, 2004; Singh & Varshney, 1995), is included in the model and extended through top of the impervious core (USBR, 2012). The chimney drain creates a clear path for seepage until the drain located at the toe, such that downstream face is kept dry against slope failure.



a. Self-healing (by collapse) of filter



b. Laboratory test for ability of filter material to self-heal (by collapse)

Figure 4.2: Self-healing by collapse of filter (USACE, 1986)

The impervious core zone is composed of clay (CL) with a medium to high dry strength ((Terzaghi et al., 1996)). It should be noted that Ghanbari and Shams Rad (2013) points out CL material is the most susceptible one to hydraulic fracturing for a core material. The pervious shell material is composed of well-graded sand (SW - gravelly). The drain material consists of processed gravels and sand (GP) which ensure good drainage (USBR, 1998). The selection of the materials should be in coherence with other zones as well. Therefore, the combination of materials for the shell and core zones is implemented from USBR (2012) and USBR (1987).

4.2.2 Model Geometry

The model geometry is formed in the light of design manuals and state practices. The hypothetical model is 20 m high simple zoned embankment. The main model is chosen to be 20 m high such that the total filling duration is relatively short, in order to satisfy the assumption that filling is initiated during low-flow period and no unexpected flood occurred throughout. The symmetrical core slopes are designed to be 0.5H:1V (Bilgi, 1990), where this selection also has an effect on the effectiveness and geometry of the chimney drain (FEMA, 2011). Also it should be noted that, a relatively narrow core, which is more prone to cracking and hydraulic fracturing (Sherard, 1986), is preferred in the model. The top width of the dam is determined to be 9 m from (Senturk, 1988) and greater than minimum requirement of 8 m (USACE, 2004). Chimney drain thickness is accepted to be 2 m for satisfactory performance and greater than the minimum limit of 1.4 m (FEMA, 2011). Calamak et al. (2018) revealed that the thickness of the chimney drain does not change the pore-water pressures inside the dam. The slopes of the embankment are selected by the specifications of USBR (1987); that is, 3H:1V for upstream slope and 2.5H:1V for downstream slope considering shell and core zone's materials. The model geometry is presented in Figure 4.3.

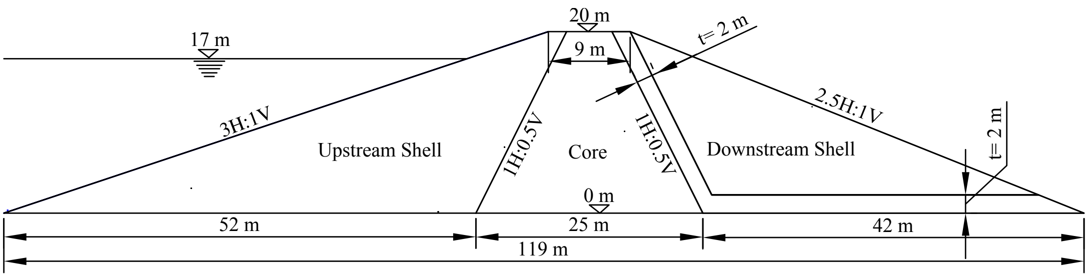


Figure 4.3: Model geometry

4.2.3 Material Properties

The representative properties of the hypothetical model are:

- Hydraulic conductivity (K_h) values are selected from USBR (2014b) for compacted shell, core and washed drain materials. For well graded, gravelly sand shell material 8.34 m/day is assigned between the interval of 2.10 to 20.90 m/-day. 0.00752 m/day for the clay core and 125.26 m/day for washed drain material gravel and sand. These values are also included in the relative recommended intervals from these sources: Bowles (1996), Rawls et al. (1982).
- Saturated water content (θ_s) and residual water content (θ_r) are selected from Carsel and Parrish (1988) as mean values from their descriptive statistics.
- Anisotropy values are assigned from USBR (2014b), originally complied by USBR (1987). These values are given considering field compacted conditions. The manual suggests values between 4 to 9 for standard placed shell and core materials. Since anisotropy increases with higher water contents during placement, a ratio of 5 for shell and 9 for core are selected considering their initial water content. Drain material is accepted as isotropic (USBR, 2014b) due to lack of compaction effort and thicker lifts during placement. It is worth bearing in mind, USACE (n.d.) suggests anisotropy of 9 due to compaction, while checking steady-state slope stability.
- Coefficient of volume compressibility (M_v) parameter is calculated using Eq. 4.1 as suggested by Carter and Bentley (2016).

$$M_v = \frac{1}{E}\frac{(1+v)(1-2v)}{(1-v)} \tag{4.1}$$

- Compacted unit weight (γ_s), effective cohesion (c') and angle of friction (ϕ') parameters are the average engineering values provided by USBR (2011b) based on their extensive data with earth material placed in appropriate conditions. Optimum water content is also presented by USBR (2011b) from the average value of proper laboratory tests. The activation pore-water pressures are manually assigned by the defined VWC content to each material corresponding to their initial water content that represents initial pore-water pressure of the material.

- Elastic modulus (E) and Poisson's ratio (ν) are assigned in the interval that Bowles (1996) presents as representative numbers.

The values are more dependable since they are based on extensive experience of USBR on earth materials and their compacted use in embankment dams. Table 4.1 shows the selected values.

Table 4.1: Material properties

	$Unit$	$Shell(SW)$	$Core(CL)$	$Drain(GP)$
Hydraulic conductivity, K_h	m/day	8.34	7.52×10^{-3}	125.26
Saturated water content, θ_s	m^3/m^3	0.43	0.38	0.194
Anisotropy	K_v/K_h	0.2	0.11	1
Vol. compressibility coeff., M_v	$1/kPa$	1×10^{-6}	1×10^{-5}	1×10^{-5}
Compacted unit weight, γ_s	kN/m^3	19.81	16.73	20
Cohesion, c'	kPa	10	71	0
Angle of friction, ϕ'	$degrees$	37.4	25.1	41.4
Elastic modulus, E	kPa	80,000	50,000	90,000
Poisson's ratio, ν		0.35	0.4	0.1
Optimum water content	$percent$	9.1	16.7	11.4
Activation pressure	kPa	10	308	0.9

4.2.4 Boundary Conditions

The FEM connects the model domain between predefined boundary conditions. The type of boundary conditions changes according to the governing laws of physics for that specific analysis. In addition to initial stress-deformation conditions coming from in-situ analysis, initial pore-water pressure and seepage quantities comes from initial steady state empty reservoir analysis to coupled stress-PWP analysis. The stress/strain boundary conditions are listed for coupled analyses:

- No deformation bottom line. This condition assumes foundation as rigid and reflects no movement or deformations from the bottom of the dam.
- Initially zero water level to no-account any water load from the reservoir during in-situ analyses.
- Transient water level on upstream slope. This will attribute water elevation on the upstream slope as reservoir rises. This condition is a time-dependent input the filling schedule.

The hydraulic boundary conditions are:

- Initially zero water level on the upstream slope reflecting empty reservoir.
- Transient water level on upstream slope. This will attribute water pressure of the upstream slope as reservoir rises such that seepage starts. This condition is time-dependent and inputs the filling schedule.
- Zero pressure on toe drain to model seepage dissipation would result there.
- Zero flux at downstream slope to model potential seepage line.
- No-flow line to bottom boundary of the dam to account impervious foundation.

4.2.5 Hypothetical Model's General Safety Checks

The hypothetical model should be tested in accordance with general design considerations such that it is feasible to conduct further studies on an appropriate design against slope failure. These checks are composed of slope stability analyses of both upstream and downstream shells at the steady state for different scenarios of after construction/empty and full reservoir conditions. It is expected to achieve critical factor of safeties that are higher than minimum required that suggested by USBR.

Upstream failure during static loading conditions are very unlikely; on the other hand, during rapid drawdown or after construction, failure is potential (USBR, 2012). When transient analyses representing temporal variation of upstream slope failure factor safety are considered; it can be seen that, starting from the unfavorable stage of empty reservoir, factor of safety would lead to increase as the reservoir is filled since water load on the upstream contributes to slope's stability. Therefore, for the scope of this study, upstream slope stability is not a concern but an additional behavioral information caused by reservoir filling. In case of downstream slope failure possibility, the downstream does not feel the effect of water rise in the reservoir; that is, factor safety is constant through the filling. This is because there is no water interaction to downstream shell thanks to chimney drain; thus, no water weight affects the LEM. Therefore, in addition to upstream being safer, possibility of downstream slope failure does not increase due to filling.

4.2.6 Instrumentation Placement on Dam Body

For the parameters aforementioned in Section 2.7, location of the readings from the application model are placed in such a way that they reflect the placement of the installed instruments in real dams. Figure 4.4 shows the sections considered. The section A-A is crucially important since water seepage through core will initiate and most probable location of hydraulic fracturing formation. Section B-B gives the horizontal variety of the key parameters at second hold level. Section C-C, at mid-height, is a common section of interest among other researchers also for differential settlement monitoring and checking the variations of parameters in horizontal axis.

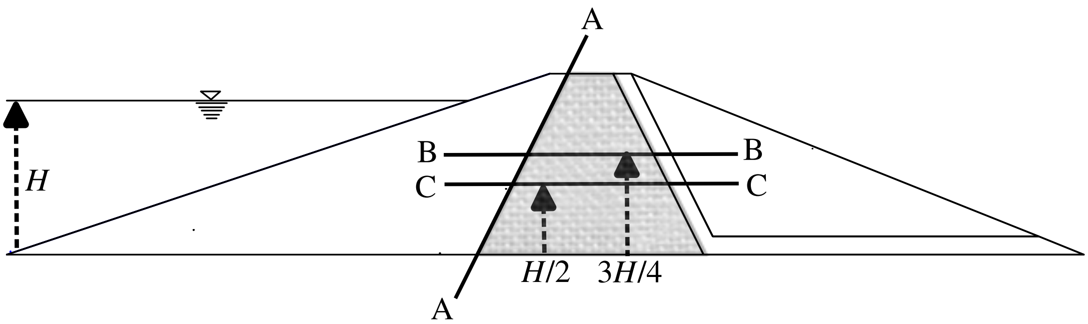


Figure 4.4: Sections of interest

4.3 Slope Stability

For slope stability inspection of the hypothetical model, upstream and downstream slopes are analyzed using SLOPE/W software. The LEM factor safety requirements for different loading scenarios including the references are given in Section 4.4. The end-of-construction (empty reservoir) and steady-state seepage conditions (full reservoir) should be analyzed for general slope stability check (USBR, 2011b), additionally factor of safety to slope failure of upstream face is computed for each time step by the stress output from the coupled analysis. Even though embankments with clay-core is not expected to reach steady-state phreatic surface for a long time, USBR (2011b) recommends it to be examined as an accepted practice.

4.4 Factor of Safety Criteria

Factor of safety criteria does not necessarily ensure safety but gives a relative degree of stability. The recommended values are as follows:

- Steady-state seepage condition: factor of safety is limited minimum to 1.5, such that the uncertainties in material strength and pressure conditions are considered (USBR, 2011b).
- End-of-construction condition: minimum factor of safety of 1.3 is supposed to be met when using effective stress parameters (USBR, 2011b).

For all slope stability analyses, effective stress parameters, considering drained conditions, are defined (USBR, 2011b) throughout this study. For the drained conditions, where pore-water pressures are controlled by the hydraulic boundary conditions, Duncan et al. (2014) recommends analyses to be conducted under total unit weights, effective shear strength parameters and pore-water pressures from hydrostatic water levels.

4.5 Modeling The Hypothetical Dam

Since there is no field data from the readings of installed instruments for back-analysis of model calibration, all the hypothetical model parameters are assigned in accordance to the design recommendations in the literature. Thus, the model is aimed to reflect a new dam preliminary analysis for conducting a filling schedule. As USBR (2014b) suggests, the hypothetical dam does not search the best answer but it aims at predicting the range of potential behavior. Therefore, this study should be considered as a reflector of a typical zoned dam's expected response to initial filling. The success of model calibration is analyzed by previous real dam experiences and engineering judgement. It should be noted that, the current study findings can be a topic of further probabilistic research for more accurate and sensitive results.

4.6 Initial Uncontrolled Filling Rate

The reservoir inflow and corresponding water level data from a number of real-dams are inspected in this section. For the uncontrolled filling part up to a low pool level is considered for different dams in order to have a common sense for the rate to be chosen for the study. The approximate initial filling rates for earth/rockfill dams are:

- 0.6 m/day for Medicine Creek Dam (Holtz & Hilf, 1961).
- 0.75 m/day for Cherry Valley Dam (Nobari & Duncan, 1972).
- 0.6 m/day for Round Butte Dam (Nobari & Duncan, 1972).
- 0.45 m/day for Cougar Dam (Nobari & Duncan, 1972).
- 1.0 m/day for Oroville Dam (Nobari & Duncan, 1972).
- 1.0 m/day for El Infiernillo Dam (Nobari & Duncan, 1972).

It should be noted that, all the given dam examples are located in USA, except El Infiernillo Dam which is located in Mexico. In the light of the previous experiences, a constant initial water rise rate of 0.6 m/day until 50% of the embankment height is implemented into the applied filling schedule to the model.

CHAPTER 5

DISCUSSION OF RESULTS

The followings are the result for a 20 m high embankment dam. This section is composed of the initial verification of the model's slope stability, determination of the first and second hold duration in the first filling schedule, sensitivity analysis of the results, describing the dam's general behavior during initial filling by the determined schedule. In addition, the verification of the results by past experiences in the literature and engineering judgement, is provided.

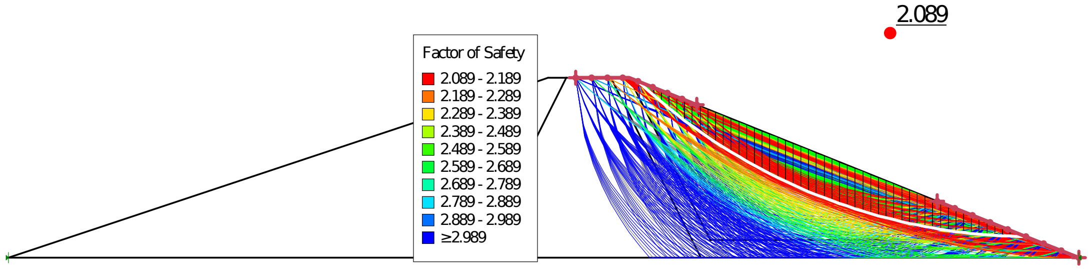
5.1 Slope Stability Verification

It is verified if the hypothetical model's slope failure tendency is within the recommended limits such that the design is proper enough for further analyses. For this purpose, upstream and downstream slopes of the model are subjected to stability analysis. Two different scenarios, i.e., end-of-construction and steady-state full operation level reservoir are implemented.

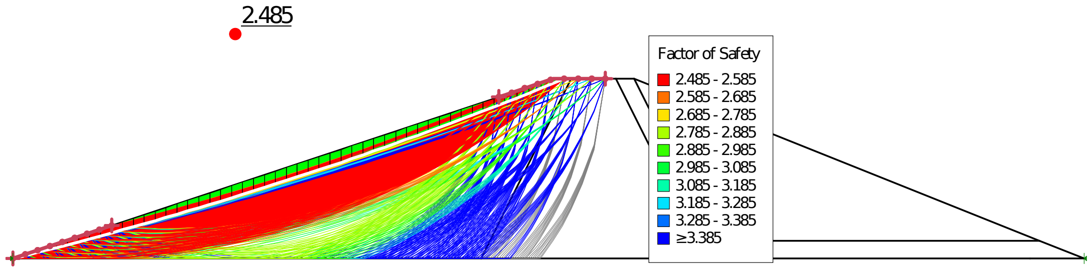
As can be seen from Figure 5.1, the model is adequately designed in terms of slope stability. The critical downstream FoS of 2.089 (Figure 5.1a) and upstream FoS of 2.485 (Figure 5.1b) both exceed minimum limits suggested by USBR (2011b).

5.2 Determination of Intermediary Hold Durations

As aforementioned, this study mainly questions the durations of intermediary holds of water level. For this purpose, the filling schedule with varying hold durations is applied to upstream boundary of the dam model. A 180-day duration, transient coupled



(a) Full reservoir condition - downstream slope



(b) End-of-construction condition - upstream slope

Figure 5.1: Slope stability analyses

analyses are accepted to be moderate and conducted in order to see how the behavior of the dam body varies with changing waiting times. By five days of increments, nine different waiting times from five to 45 days are implemented separately. At first, hold at mid-height, $H/2$, is subjected to be analyzed. During this analysis, hold at $3H/4$ is kept constant to 50 days since it has no effect on the prior filling and upper level rates are arranged to 0.3 m/day. The varying filling schedule can be seen in Figure 5.2. According to the schedule, the reservoir water level starts to increase by a rate of 0.6 m/day until the height of $H/2$, 8.5 m at 14$^{\text{th}}$ day. At this time, intermediary level hold starts and lasts for five days of increments. This is followed by continuing the water level rise again until the height of $3H/4$ by a rate of 0.3 m/day. The effect of filling on the dam body is examined for each alternative at the end of the hold. Then the filling starts again. The data comparison is extracted from Section A-A in Figure 4.4 where hydraulic fracturing criteria is prone to occur. It should be noted that, at this time, the water starts to seep through the core. The variations of pore-water pressure, total vertical stress, effective vertical stress, horizontal and vertical displacement with respect to height of nodes on Section A-A (Figure 4.4) are plotted in order to compare different waiting times.

5.2.1 Determination of the duration of intermediary hold at H/2 elevation

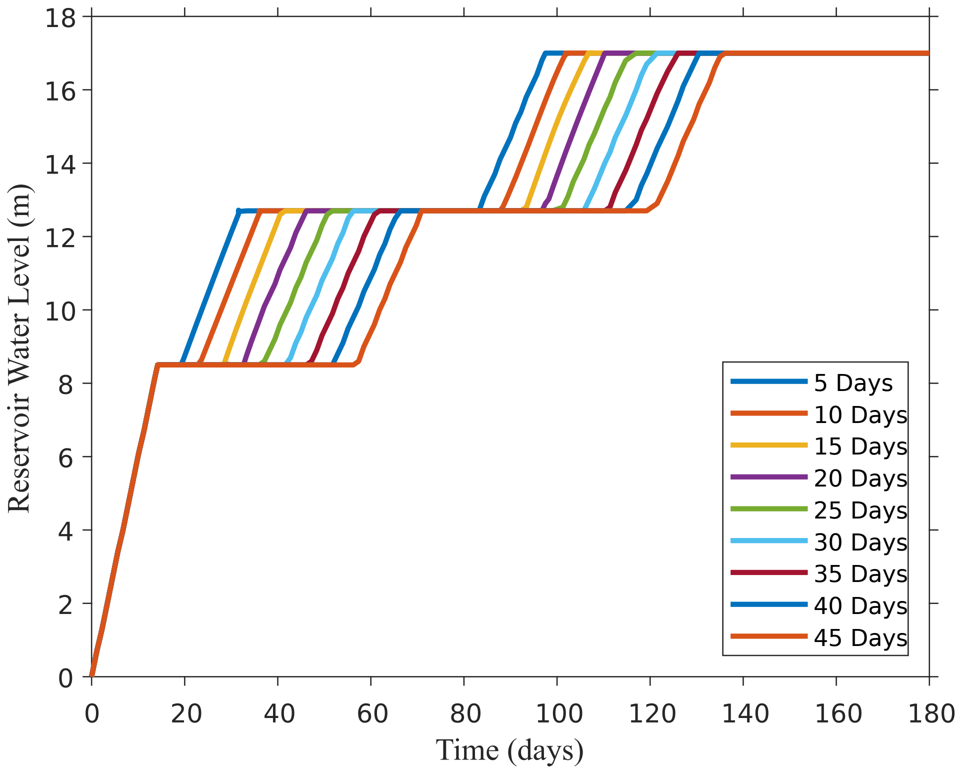


Figure 5.2: Applied boundary condition of filling schedule

The effective stress variations on Figure 5.3 show a clear behavior of converging as the waiting process continues. The difference between each five days of increments gets smaller for longer hold durations. Normally, effective stress variation over the height of the dam is expected to be increasing as it gets deeper in elevation since the soil height above increases with depth. However, around the $H/2$ water level, the effective stress makes a jump to much lower values. This is due to the fact that the seepage arrives to the upstream side of the core that lead to increase in pore-water pressure. Since there is a lag between the reservoir level rise and its effects occur on the core, the effective stress gets lower as the hold continues. The change is more rapid in the beginning of the hold but converges to certain values after 20 days of waiting. It is seen that; it still continues to decrease if a longer waiting is provided. However, the effect of waiting is relatively smaller than the decrease on effective stress.

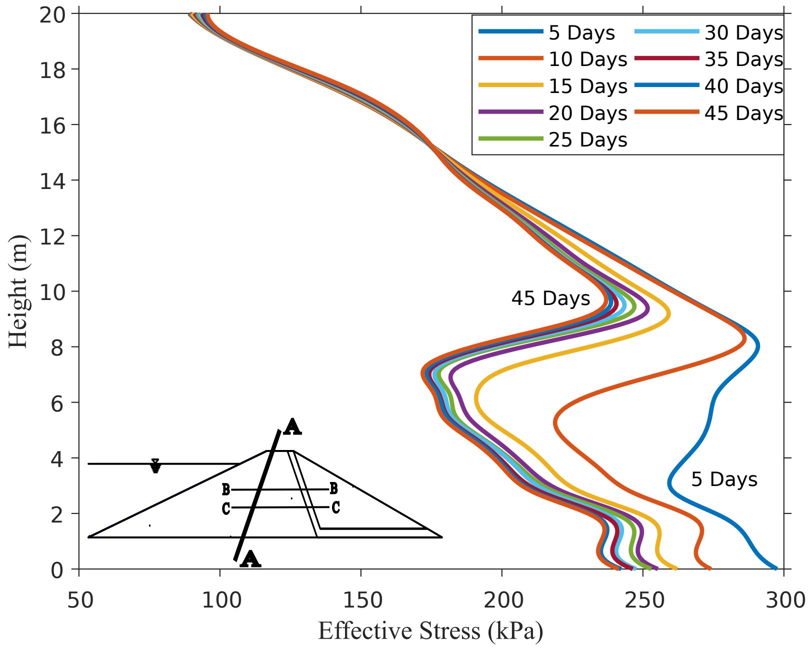


Figure 5.3: Effective stress variations for different first hold durations on section A-A

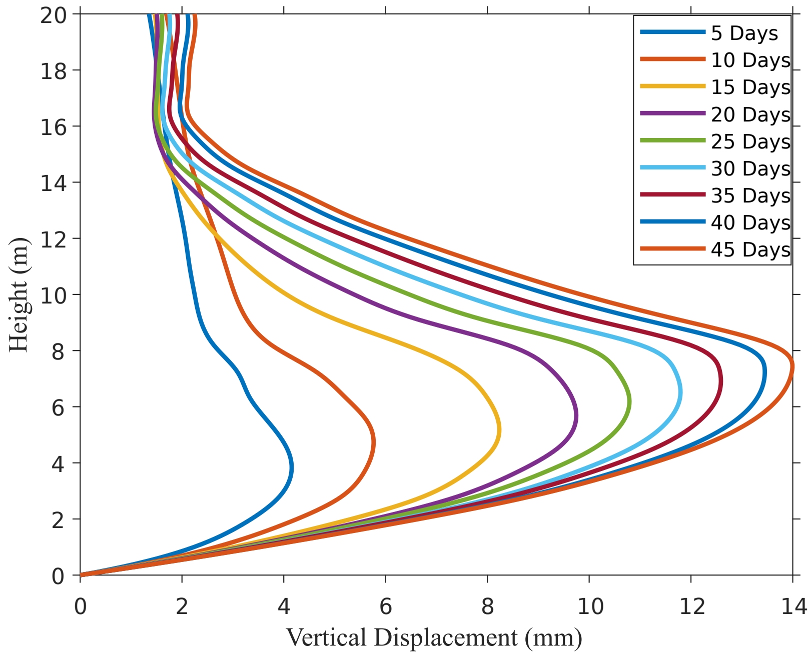


Figure 5.4: Vertical displacement variations for different first hold durations on section A-A

Figure 5.4 shows the vertical displacement variation over the dam's height at the end of each hold level for different waiting times. It should be emphasized that the displacement values are positive indicating an upward movement. This is due to water entrance to the dam body that creates uplift. In a field measurement, the vertical displacement graph is also affected by the consolidation settlement of the body over a long time from the construction from the self-weight. Since the magnitude of self-weight settlement is relatively larger than the uplift displacement, the uplift effect is harder to be emphasized. However, the FEM model of the current study is based on the assumption that the soil had been already settled due to its self-weight after the construction, and only displacement from water entrance is considered. The displacement over the height, increases as the water hold continues. However, the effect of waiting decreases for the incremental hold duration. The maximum displacement occurs at higher elevations until the height of $H/2$ where the water level is kept constant. It is not feasible to expect vertical displacement to converge to certain values so that this parameter can also govern waiting time proposal before the reservoir is loaded to higher water levels. But it is helpful in a way that the difference on displacement between each increment of waiting times does get less significant after 20 days. Afterwards, the difference stayed constant between further increments. In addition, even the maximum displacement at this stage is not significant for the current configuration of the model since it is only 0.07% of the dam height. In general, a safe design is limited settlement to be less than 1% of the embankment height.

The horizontal displacement variation is presented in Figure 5.5. The figure indicates the soil movement to the upstream side for the lower elevations in the core. The movement direction is the opposite for the higher elevations, to the downstream side, and the maximum deflection occurs at the $H/2$ water level. The maximum value shifts to higher heights as waiting continues; this is because the water level reaches up to the $H/2$ in the shell also. It can be said that the rate of displacement tends to decrease after each increment of waiting times.

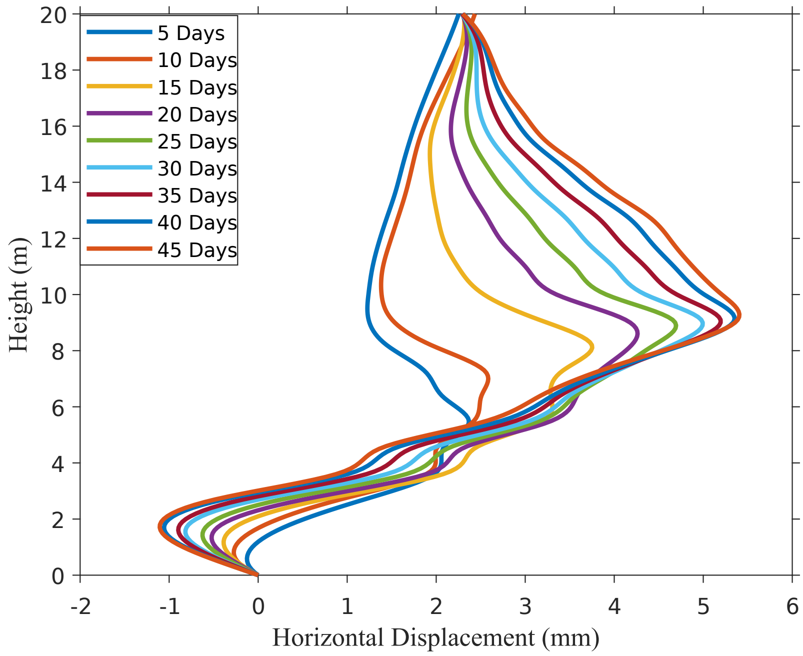


Figure 5.5: Horizontal displacement variations for different first hold durations on section A-A

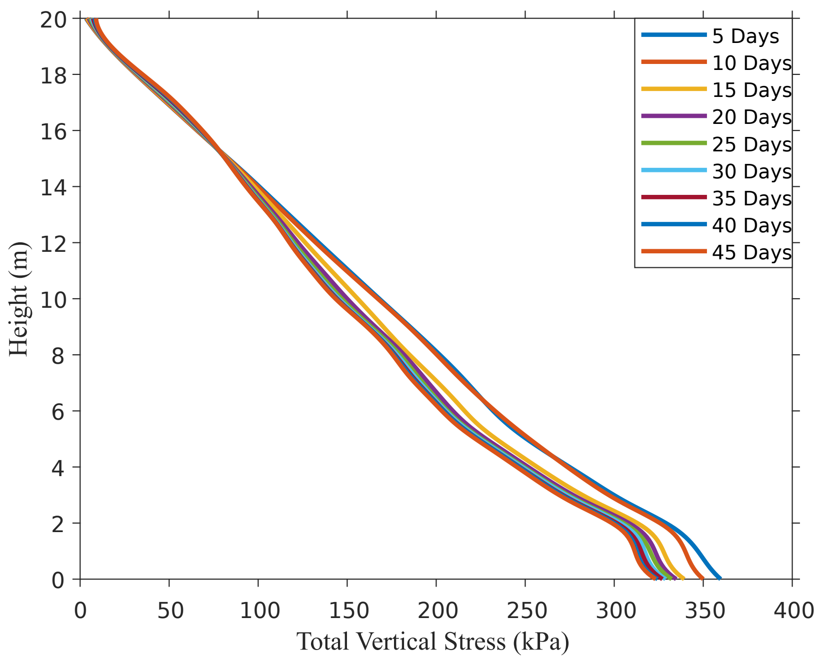


Figure 5.6: Total vertical stress variations for different first hold durations on section A-A

Figure 5.6 shows almost no change in total vertical stress at the upstream face of the core for waiting time increments after 20 days. It slightly decreases at the bottom parts due to the water entrance.

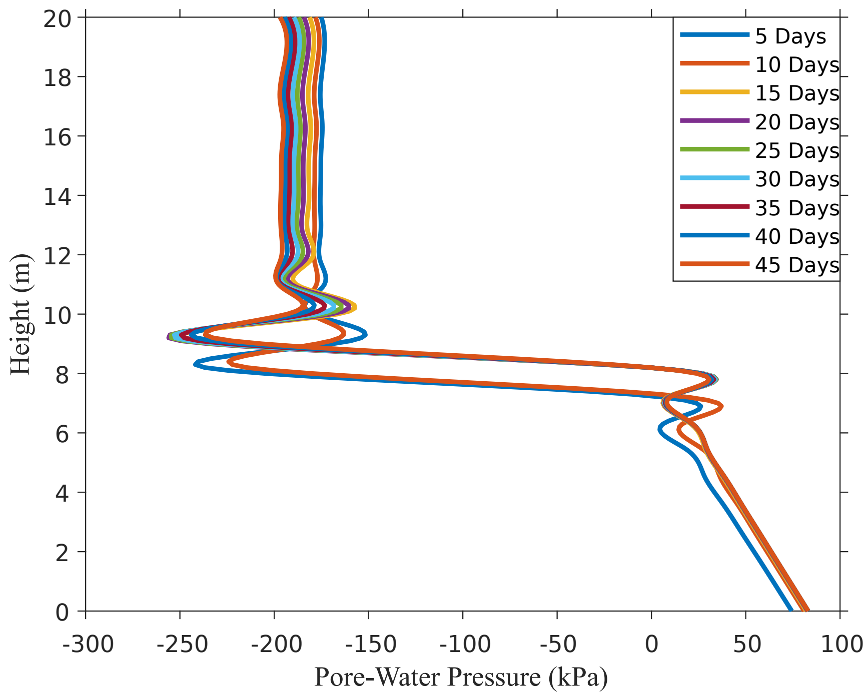


Figure 5.7: Pore-water pressure variations for different first hold durations on section A-A

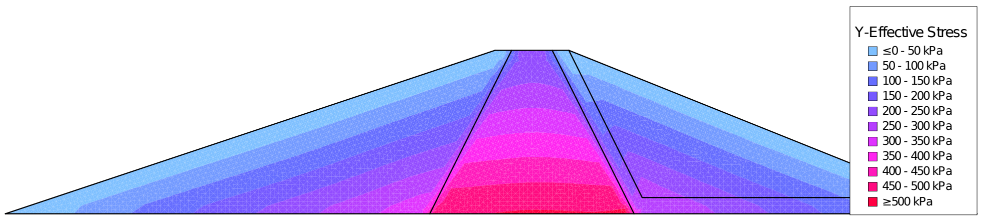


Figure 5.8: Initially induced effective stresses due to water content

Figure 5.7 shows a convergence of the change after 20 days. The nodes at the upper part of the reservoir does not feel the filling as their values are kept at their initial activation pressure values. However, as seepage reaches to and below $H/2$ level, the pore-water pressure increases to positive values. A well-known linear increase with depth is obtained for the nodes located under $H/2$. It should be noted that the activation pressure of -310 kPa of the core and -10 kPa in the shell is linearly interpolated in the transition layer between the zones and therefore, the values from Section A-A are not necessarily -310 or -10 kPa. This transition is clarified in Figure 5.8.

In order to observe the horizontal variations of the parameters, a horizontal section C-C in Figure 4.4 is preferred. The horizontal differences of the parameters for different zones play a crucial role for observing differential settlement. The section C-C is located at $H/2$ level due to the existence of the maximum values aforementioned in previous figures where vertical variations were indicated. The dotted line indicates the center line axis of the dam.

Effective stress differences between incremental waiting times is plotted on Figure 5.9. It is clearly visible how effective stress changes between the shell and core material, due to their initial water content and unit weight contrasts. It is again acceptable to say that the effect of different waiting times on horizontal effective stress is not significant except the transition zone, approximately 10 m away from the center line of the dam towards the reservoir. In addition, the effective stress decreased to 25% at -10 m, resulted from water intrusion. However, this decrease converges after the 20 days hold and does not decrease further in case of a longer monitoring. Therefore, this figure is a good indicator of an expected effective stress reduction on the upstream boundary of the core zone for longer waiting times, that should be taken into consideration during practical applications that the stress tends to decrease before converging. This information is valuable for on-site operation engineers such that it is not feasible to continue further filling before this convergence occurs.

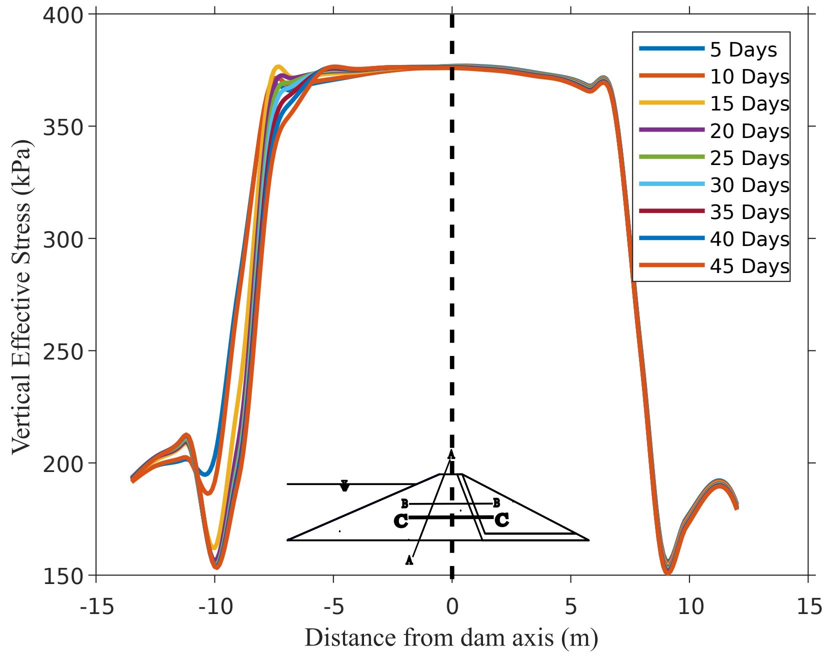


Figure 5.9: Effective stress variations for different first hold durations on section C-C

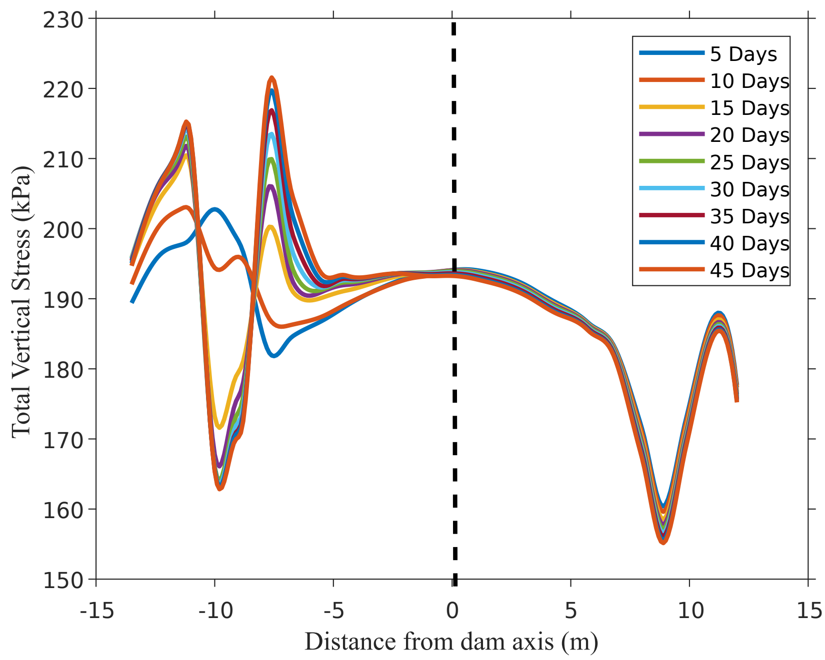


Figure 5.10: Total vertical stress variations for different first hold durations on section C-C

The horizontal variation in total vertical stress is interesting in terms of the stress variations for different zones and presented in Figure 5.10. Almost constant vertical effective stress values around the dam center line is disturbed from the upstream side by the arrival of water. An increase just inside the upstream side of the core and a decrease at -10 m progressively continue as waiting time increases. The decreasing part is prone to convergence at 165 kPa at day 20 and does not further decrease in case of a longer monitoring.

The minimum principal stress does not significantly vary for different hold duration. The slight changes between increments, around 10 kPa, are converged after 15 days of monitoring (See Figure 5.11) due to the water entrainment.

Figure 5.12 clearly shows the possible low principal stress location at -10 m, that is the upstream face of the core, which is the most expected place for hydraulic fracturing to occur. This is where principal stress decreases at first after hold starts, but starts to increase for longer waiting times until it converges at 20 days of waiting. A further increase after 20 days is not significant. Therefore, the reservoir can safely be subjected to further increasing water levels in terms of hydraulic fracturing possibility. It is crucial that if filling continues without the proper hold, the hydraulic fracturing possibility is quite high. Because the minimum principal stress first decreases to its minimum before converging to increasing values, such that the hold is necessary in order not to load the body with even further loads in this susceptible time zone. However, the dam is still safe against fracturing because the pore-water pressure at the same elevation is lower than the minimum value of principal stress as shown in Figure 5.12. It should be noted that, even though the downstream side of the core has lower principal stresses, hydraulic fracturing is not possible because there is no water intrusion to that zone yet.

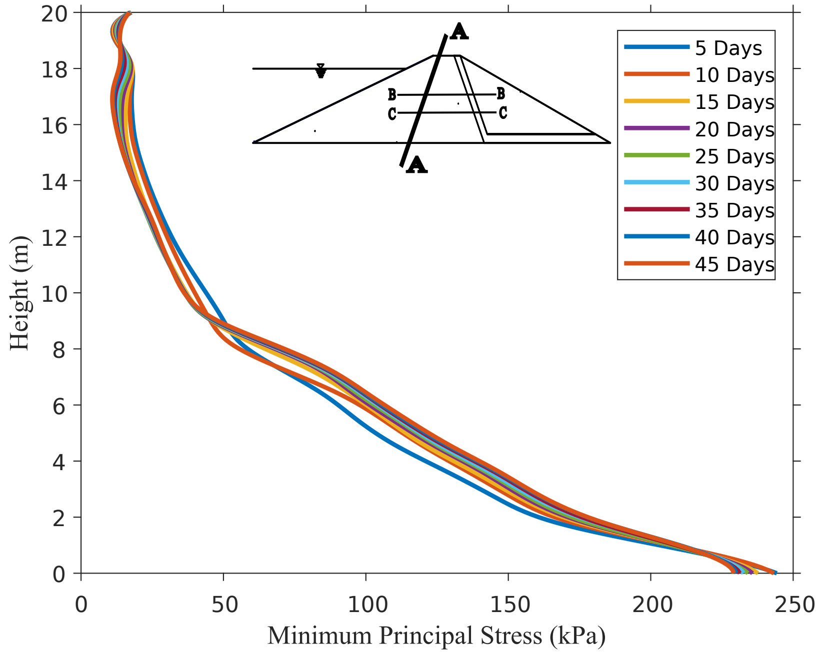


Figure 5.11: Minimum principal stress variations for different first hold durations on section A-A

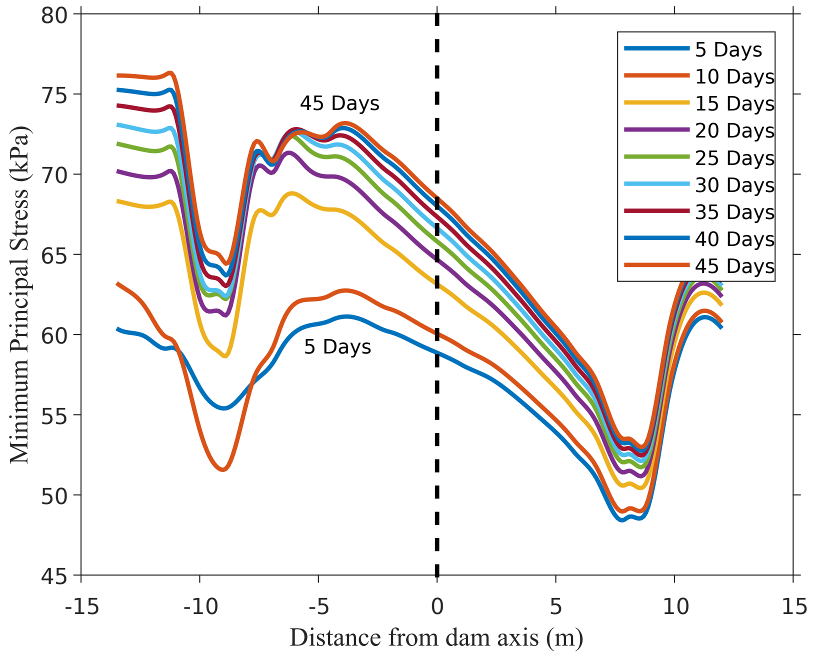


Figure 5.12: Minimum principal stress variations for different first hold durations on section C-C

In the light of the aforementioned figures, the acceptable variations show a common convergence at 20 days of monitoring and a further waiting is not necessary in terms of hydraulic fracturing possibility. Therefore, the hold duration is suggested to be 20 days at $H/2$ for further loading of the reservoir.

5.2.2 Determination of the duration of intermediary hold at 3H/4 elevation

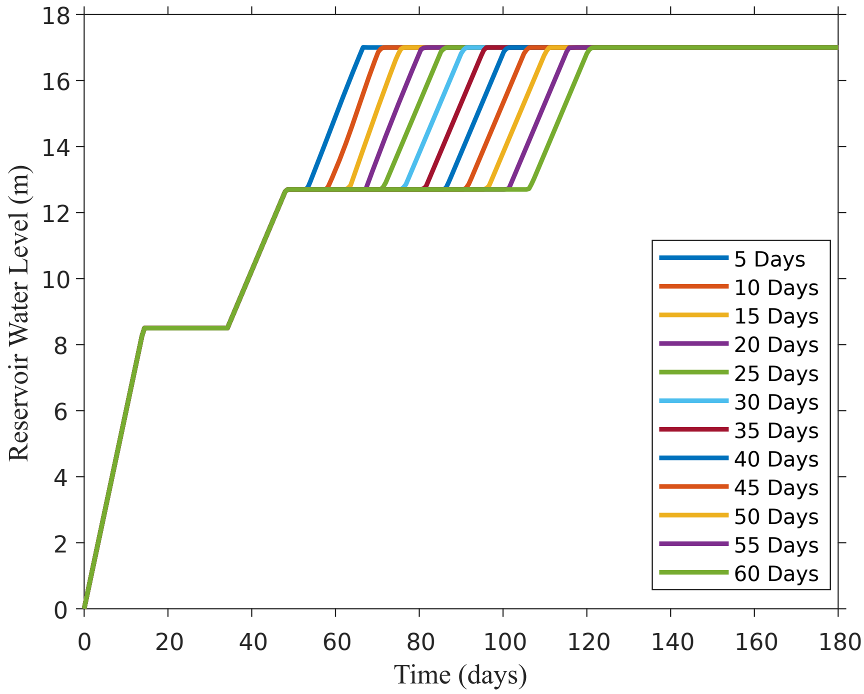


Figure 5.13: Applied boundary condition of filling schedule

A similar approach is followed for the determination of the duration of second hold. This time twelve different alternatives of hold duration at $3H/4$ starting from five days to 60 days with five days of increments. From Section 5.2, the minimum recommended hold duration of 20 days at $H/2$ is implemented for low pool filling. The filling rate for the current rise is 0.3 m/day and takes 14 days and followed by the second hold alternatives. The time of readings from the provided graphs is right after the end of hold duration in order to see its results to the dam's behavior. This part of filling is more hazardous to dam body in terms of hydraulic fracturing occurrence because the reservoir is loaded by a higher pool level and the seepage through the core increased due to higher hydraulic gradient through the dam. The following figures are obtained from the data located to Section A-A in Figure 4.4. It should be noted that the movement of seepage is mainly through the core during the second hold. The upstream shell quickly reacts to water level changes and phreatic surface is raised simultaneously. The implemented filling schedule is provided in Figure 5.13

The effective stress variations for the second hold is presented in Figure 5.14. This time, the variations are mainly located at $3H/4$ on the section and tend to converge after 45 days of hold. After the 45 days of hold, the effective stress converges to its ultimate value where further monitoring does not have any effect. Emphasizing the variation over the height, the nodes under the phreatic surface has lower effective stresses, while the upper located nodes do have greater effective stresses even though the weight of the soil above is relatively small.

Figure 5.15 shows the upward movements over the height of the dam. The displacement continues to increase just like the first hold. The displacement has its main propagation between $H/2$ and $3H/4$. Moreover, the maximum displacement slightly shifts upwards as waiting time increases. With respect to waiting times alternatives, the differences between the curves are bigger for first time increments and reaches up to an almost constant propagation after 30 days of hold.

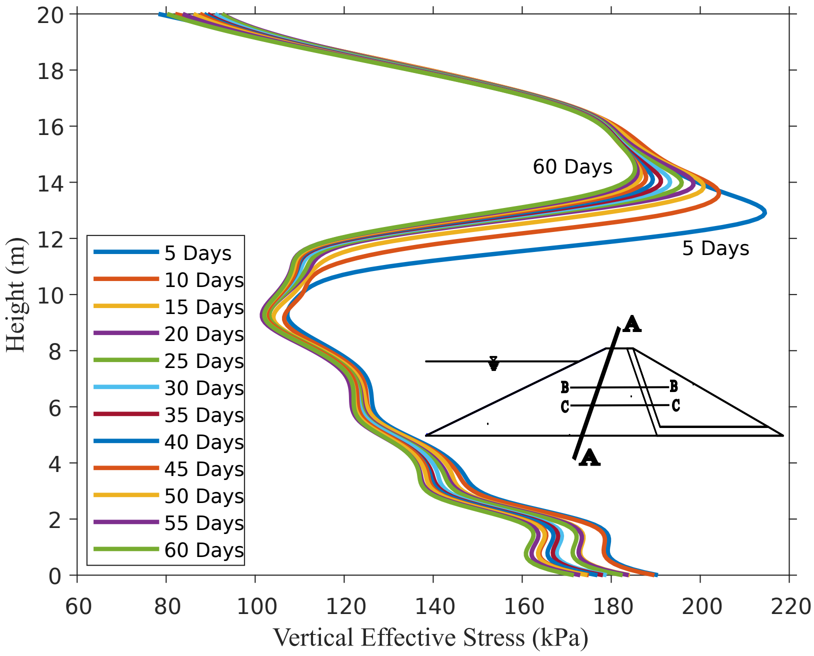


Figure 5.14: Effective stress variations for different second hold durations on section A-A

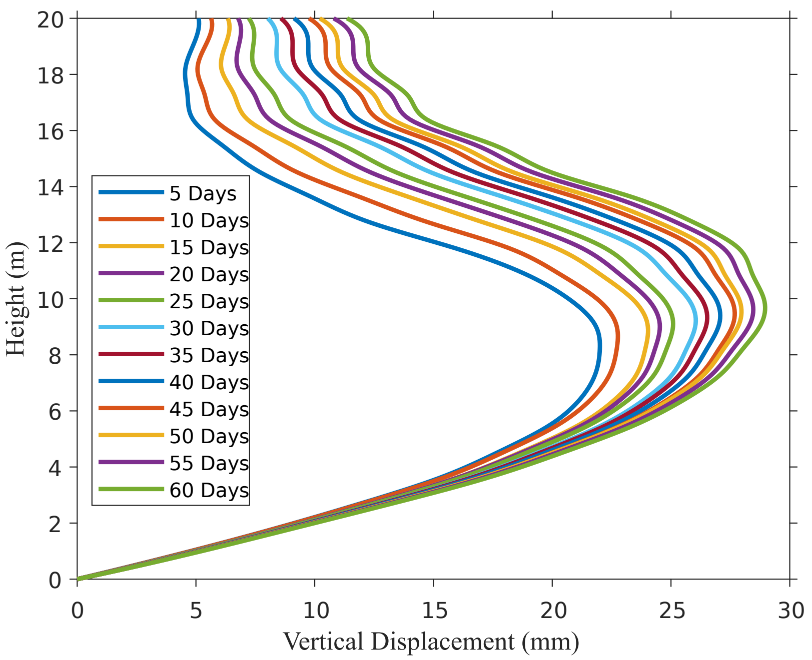


Figure 5.15: Vertical displacement variations for different second hold durations on section A-A

Figure 5.16 shows the horizontal displacement for different nodes at Section A-A (See Figure 4.4). The horizontal movement of the upstream side of the core is to the right and has the maximum displacement value at $3H/4$ where the current phreatic surface is located. The magnitudes of displacements are relatively insignificant compared to the dimensions of the body. This small movement is a consequence of the water weight on the upstream shell pushing the dam towards the downstream side. Different waiting time alternatives only affect the upper part of the body and has no significant effect under the phreatic surface. This may be interpreted as the upper heights catch the movement of the lower side slowly. The difference between the alternatives gets negligible after 30 days of monitoring.

The total stress variation almost steadily increases with the depth as presented in Figure 5.17. Since the phreatic surface is stationary over the Section A-A in Figure 4.4, there is no effect of waiting times. As expected, similar behavior is observed for minimum principal stress as it is presented in Figure 5.18.

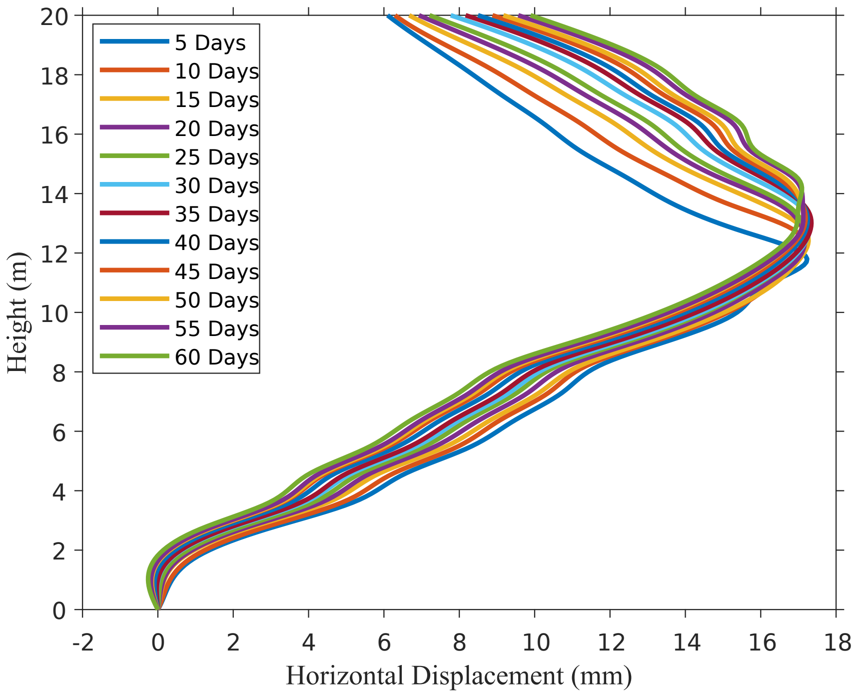


Figure 5.16: Horizontal displacement variations for different second hold durations on section A-A

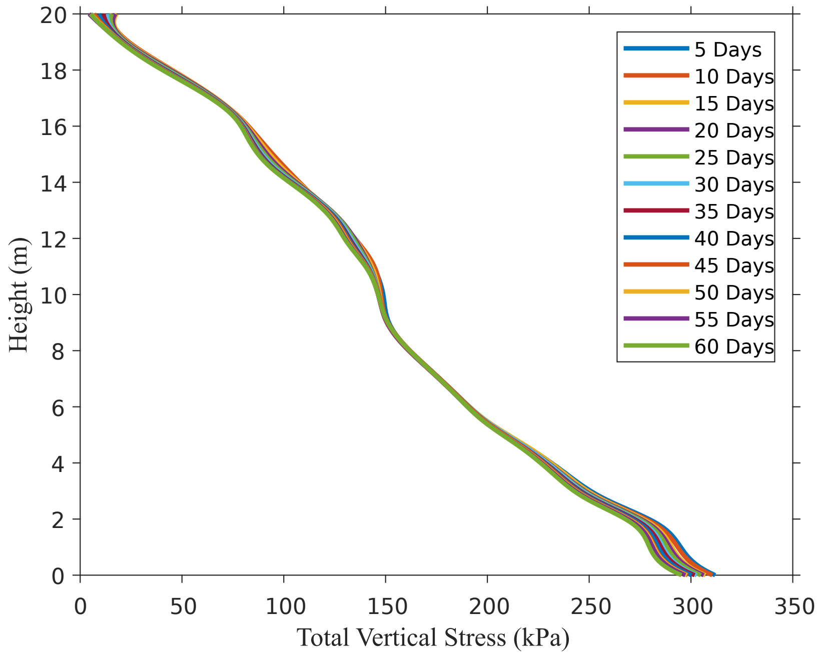


Figure 5.17: Total vertical stress variations for different second hold durations on section A-A

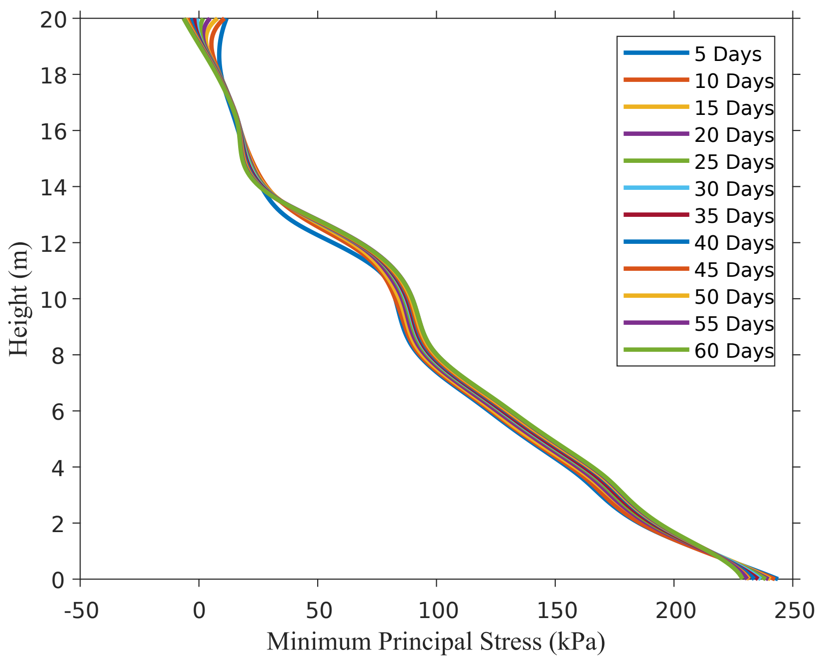


Figure 5.18: Minimum principal stress variations for different second hold durations on section A-A

Pore-water pressure distribution (Figure 5.19) over the height for second hold is similar to the one for the first hold. The reaction to water raising is relatively fast coming after 5 days of waiting. Negative pressures due to initial moisture content turn into positive as the phreatic surface reaches to higher level of $3H/4$.

The effect of the raise is inspected through the horizontal Section C-C in Figure 4.4 in order to see the propagation of the seepage. Figure 5.20 shows the decrease in effective stress by the presence of phreatic surface for different waiting times. The decrease is almost constant for different alternatives and shows no significant behavior for any specific alternative. It can be understood that the different waiting times only enable further propagation of the effective stress decrease. It should be noted that the minimum value, that is prone to hydraulic fracturing, is still located at the upstream face of the core and is not affected from different waiting times. Therefore, for this case it can be said that the different alternatives of waiting times have no significant effect on the hydraulic fracturing occurrence since their effect is limited to the near-central axis where the effective stresses are still relatively higher than the transition zone.

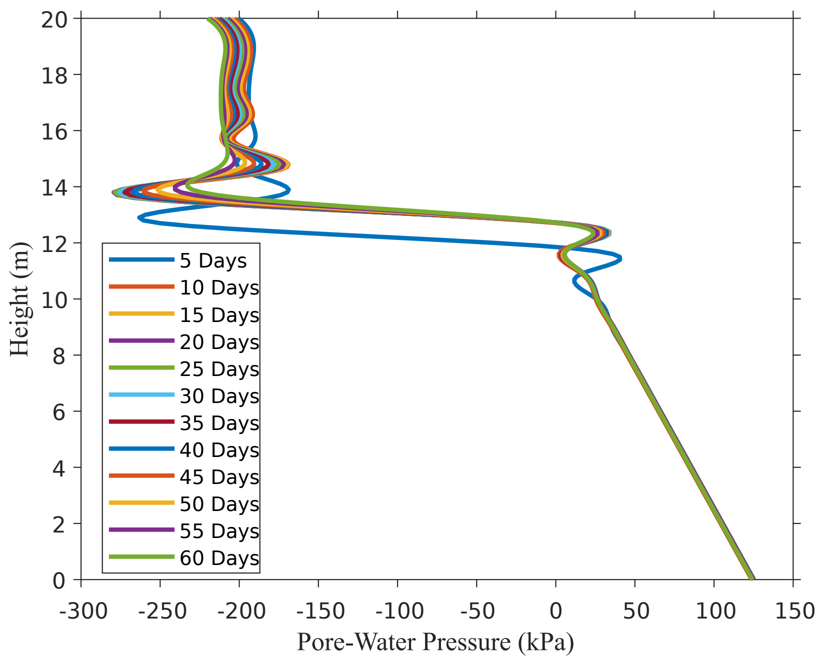


Figure 5.19: Pore-water pressure variations for different second hold durations on section A-A

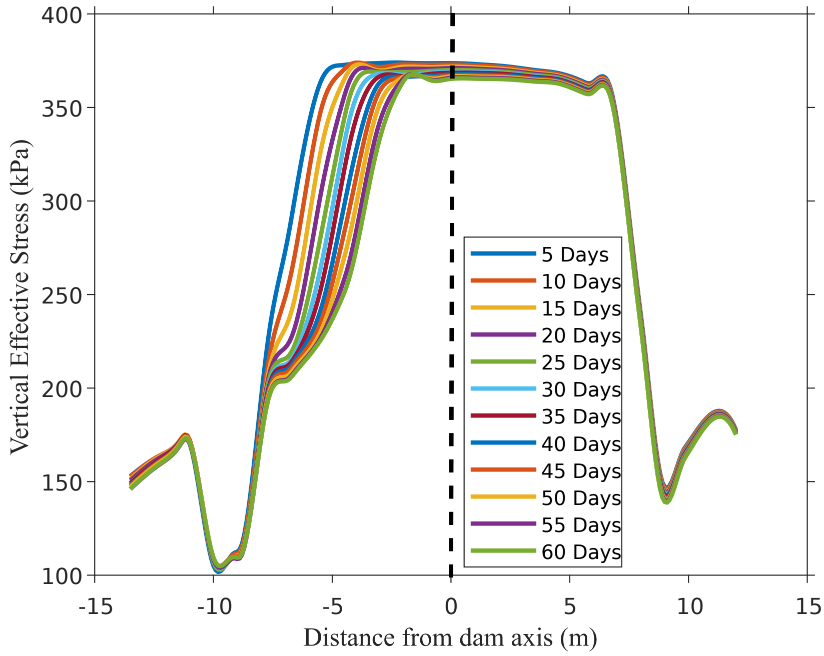


Figure 5.20: Effective stress variations for different second hold durations on section C-C

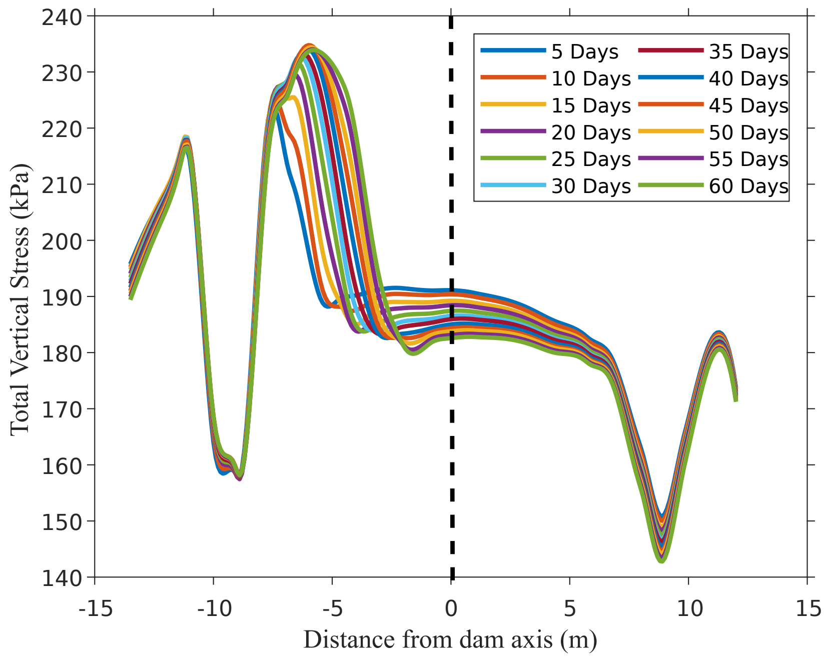


Figure 5.21: Total vertical stress variations for different second hold durations on section C-C

Figure 5.21 presents the variation of total vertical stress with respect to the different horizontal locations. The presence of phreatic surface decreases the total stress and pushes it propagate to the right side of the core. For the different alternatives of waiting time, the change between the curves gets almost constant after 30 days of hold. The minimum values are still located on the upper face of the core where the hold has no effect on it while the pore-water pressure is present. The downstream side of the core has no significant meaning with respect to hydraulic fracturing since there is no pore-water pressure in that zone yet.

The vertical displacement variation is presented in Figure 5.22. The differential movement between the core and shell is clearly visible. The core material that has lower elastic modulus, tends to move upward more than the stiffer shell material. It is not possible to say that this difference of approximately 30 mm would definitely cause a crack formation in the transition zone but surely leads to differential movement that may cause the crack. In terms of different hold alternatives, the difference between the curves starts to converge after 30 days of hold and propagates steadily afterwards. The differential movement between the maximum and minimum points does not change with waiting durations because both points tend to move upward by the same velocity.

Since hydraulic fracturing initiates when minimum principal stress is exceeded by pore-water pressure at the same height, the horizontal variation of pore-water pressure is inspected at $H/2$. Figure 5.23 shows the change in pore-water pressure as seepage propagates through the core. The maximum change between the curves occurs after 30 days of monitoring and only propagation of seepage continues afterwards. Only positive values are located on the upstream side of the core and still relatively small with respect to the total stresses.

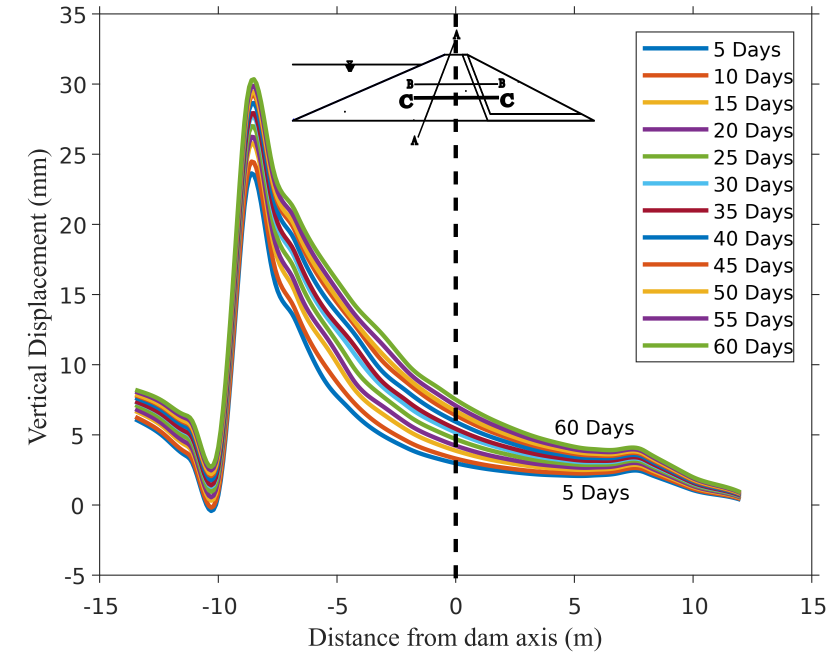


Figure 5.22: Vertical displacement variations for different second hold durations on section C-C

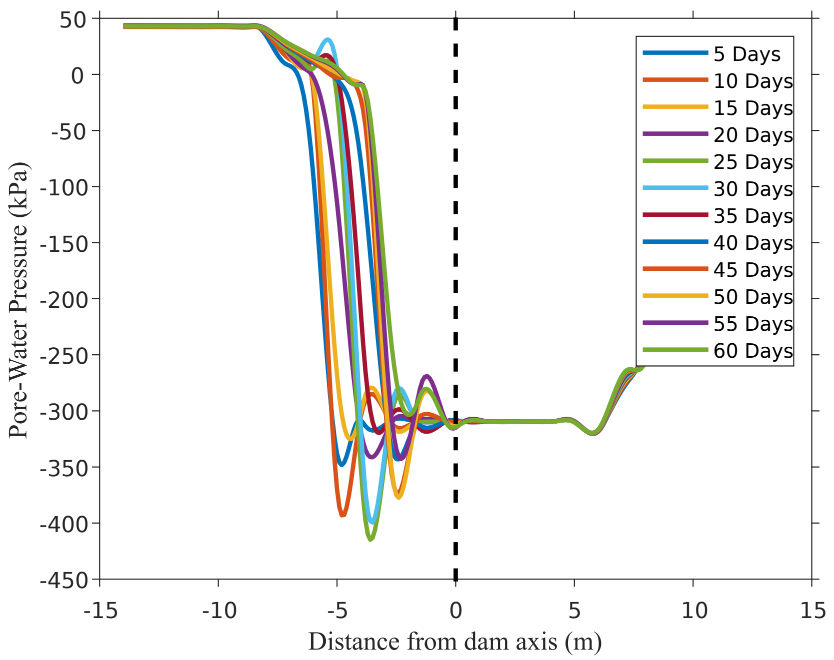


Figure 5.23: Pore-water pressure variations for different second hold durations on section C-C

Figure 5.24 shows an increase in minimum principal stress at the upstream side of the core as waiting time increases. The increase tends to a constant value after 30 days of waiting time. From this figure, it should be said that any waiting period after the second filling has an improving effect on minimum principal stress that is the main parameter against hydraulic fracturing. However, for the current model, hydraulic fracturing is still not the issue since the pore-water pressures at the same elevation presented in Figure 5.23 are much less than the minimum principal stresses. Therefore, initial conditions after the construction play much greater role on the hydraulic fracturing formation than strategically planning the filling schedule for this current hold.

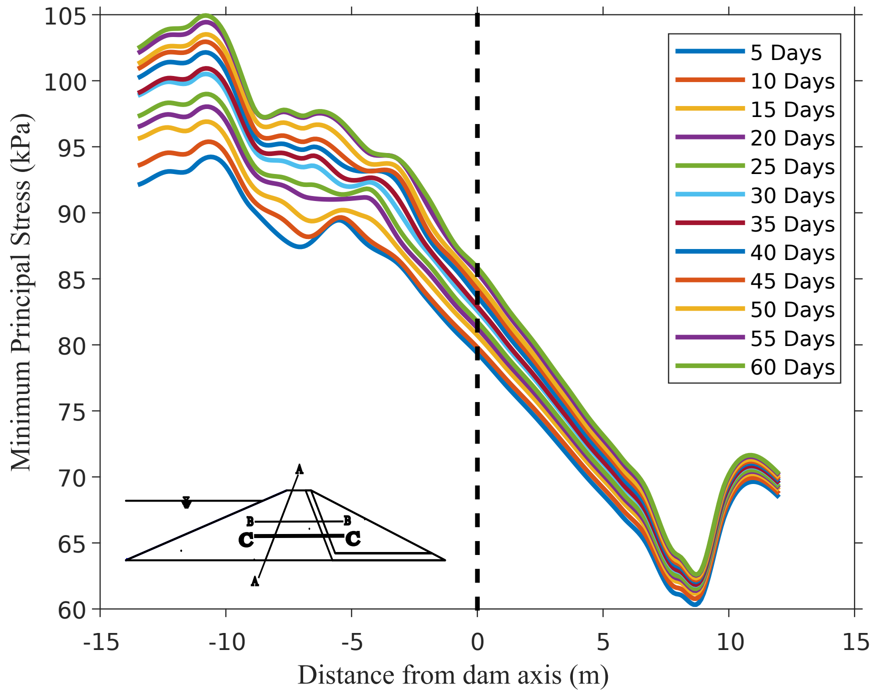


Figure 5.24: Minimum principal stress variations for different second hold durations on section C-C

The horizontal Section B-B (See Figure 4.4) located at the $3H/4$ elevation, is an indicator showing the effect of waiting durations. Minimum principal stress increases when longer holds are provided. Figure 5.25 points out the increase in minimum principal stress on the upstream side of the core due to the existence of the phreatic surface, while it tends to decrease near the central axis of the dam where phreatic surface does not arrive yet.

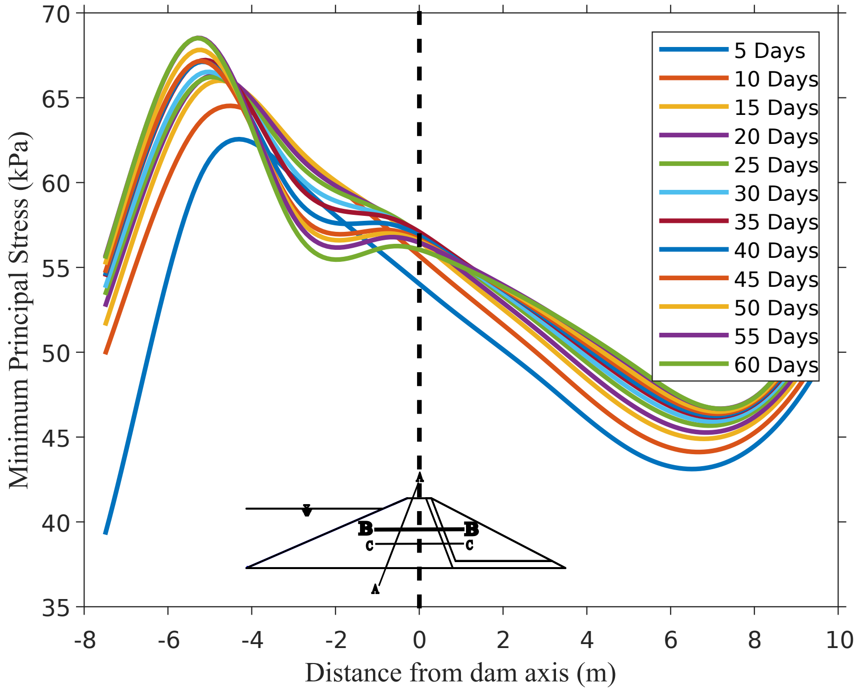


Figure 5.25: Minimum principal stress variations for different second hold durations on section B-B

In the light of the presented data from the different sections in the dam body, the increasing hold time has a positive effect against hydraulic fracturing occurrence in the core zone since it always tends to increase minimum principal stresses. However, a proposal of minimum required second hold duration is hard to be provided by this study, since the behavior of the principal stress only propagates for longer durations for which the values are already greater than the pore-water pressure at the same height. However, if inadequate minimum principal stresses are observed in the site conditions, the longer the provided holds, the more precautions against the hydraulic fracturing. The propagation of the phreatic surface is very slow in the core due to the low hydraulic conductivity and this, by itself, gives an ample time to react against the rise of reservoir water level. Therefore, different hold durations do not create a major difference in the core since the magnitude of durations are not significant considering to phreatic surface propagation in the core. In order to see an actual convergence in the core after the second hold, much longer durations of holds should be provided which is not suitable for a practical filling schedule. In this configuration, the seepage does not even reach to the central axis of the dam. However, this nongoverning and safe condition is observed due to even lower hydraulic conductivities introduced by the placement under optimum water content. Therefore, in case of a lower initial activation pore-water pressures, the higher hydraulic conductivity would lead to faster seepage propagation and create more susceptible conditions in the core against hydraulic fracturing. For this specific case, second hold duration of 50 days is considered to be enough, conservatively. Because there is no further significant change in the data located in the critical sections in the case of longer monitoring.

5.3 Sensitivity Analyses

5.3.1 Effect of Meshing

The effect of meshing on the results of finite element analysis is compared for three different mesh sizes. GeoStudio™ manuals (Geo-Slope Int. Ltd., 2012) recommends the most simple mesh style composed of quads and triangles. The software has an automatic mesh creator according to user's preference. The domain is discretized into global element size, k, of 1 m, 1.2 m and 1.5 m meshes in order to see how this selection affect the results. Since the coupled stress-PWP analysis solves both mass conservation and force equilibrium equations simultaneously, one parameter of seepage quantity and one parameter of stress/displacement quantity are inspected for this purpose. Pore-water pressure from a representative node from the upstream side of the core, Node 1 in Figure 5.26, and horizontal upstream side crest deflection are selected for this comparison of mesh size. The variations of these two parameters are presented in Figure 5.27.

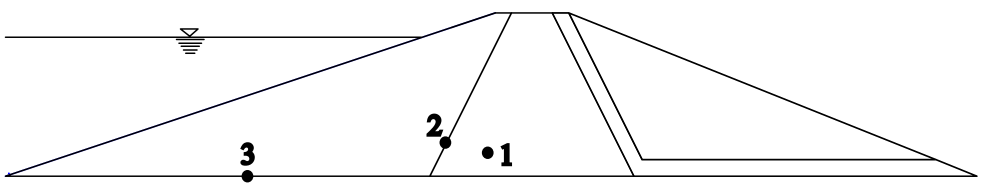
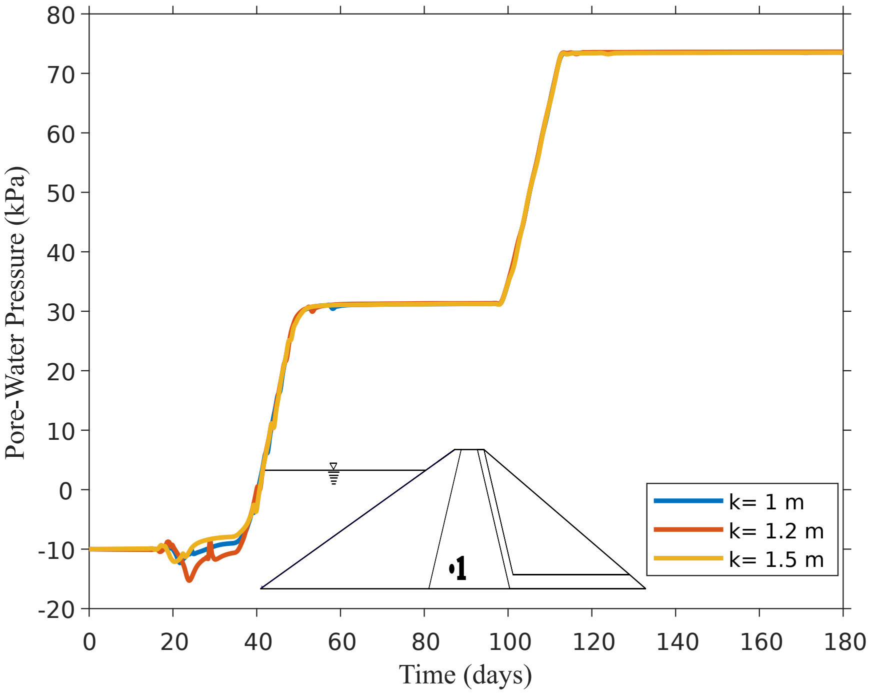
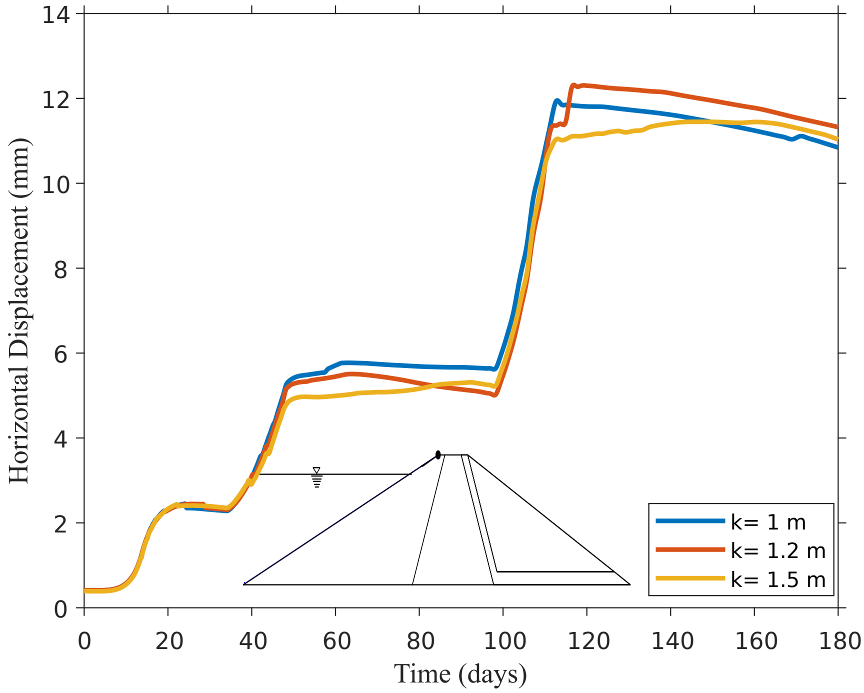


Figure 5.26: Representative nodes



(a) Temporal variation of pore-water pressure at upstream shell on node 1



(b) Temporal variation of horizontal displacement of upstream crest

Figure 5.27: Mesh sensitivity

Seepage quantities for different mesh sizes agree with each other with a relatively small numerical noise between day 20 and day 40 in Figure 5.27a. The horizontal movement of the crest shows the same behavior for each mesh size in Figure 5.27b. However, converged values after 50 days are slightly differed from each other. This is understandable if the exponential time step is considered. As aforementioned, the coupled analysis convergence is overly sensitive to time step and mesh correlation. Since time step increases exponentially, the effect of mesh size points out more clearly. However, this much of difference is acceptable for the current study since the behavior is exactly the same for different mesh selections. In addition, the general behavior of the embankment against filling is in the scope and the aim is not to provide the most accurate result. Therefore, a global element mesh size of 1 m is selected throughout the study with an appropriate convergence and minimum numerical noise.

5.3.2 Effect of Uncontrolled Rate

Since design manuals do not specify the uncontrolled filling up to a low pool, this study assumed to use rate of, r=0.6 m/day for water level rise rate up to $H/2$ level since this value is highly expected and understandable for embankments. However, in case of different inflow conditions and site-specific features (different storage-elevation relation), four different filling rates of 0.4 to 1 m/day with 0.2 m/day increments are inspected. Only the filling duration up to $H/2$ are different for this approach, that the first hold is 20 days, the second is 50 days and the upper reservoir filling rates are 0.3 m/day. It should be noted that, the rate is commonly restricted due to limiting seepage rate that may initiate internal erosion, piping. However, for the first hold conditions, seepage is mainly propagating in the upstream shell which is permeable. Therefore, the phreatic surface is sensitive to higher and lower filling rates. Nonetheless, the effect of uncontrolled filling rate has negligible effects on the seepage in the core. Figure 5.28 shows the behavior in terms of effective stress at Section A-A and elevation $H/2$, where the phreatic surface locates during the first hold. It can be seen that the behavior is the same for all alternatives. However, the convergence is achieved faster for higher rates and most necessarily, tends to the same values. It should be considered that the difference between the curves of each filling rate decreases as it gets closer to 1 m/day.

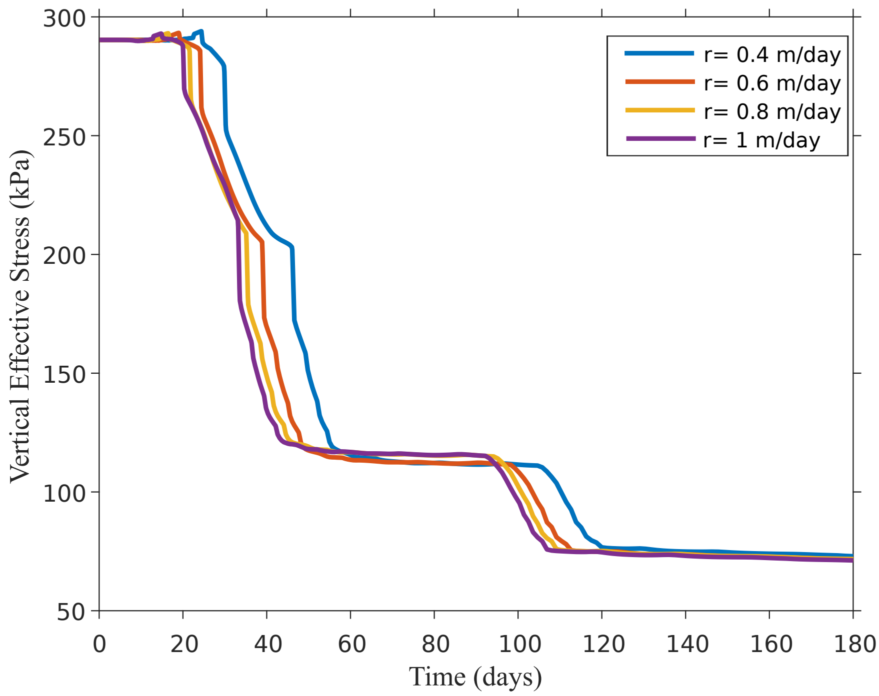
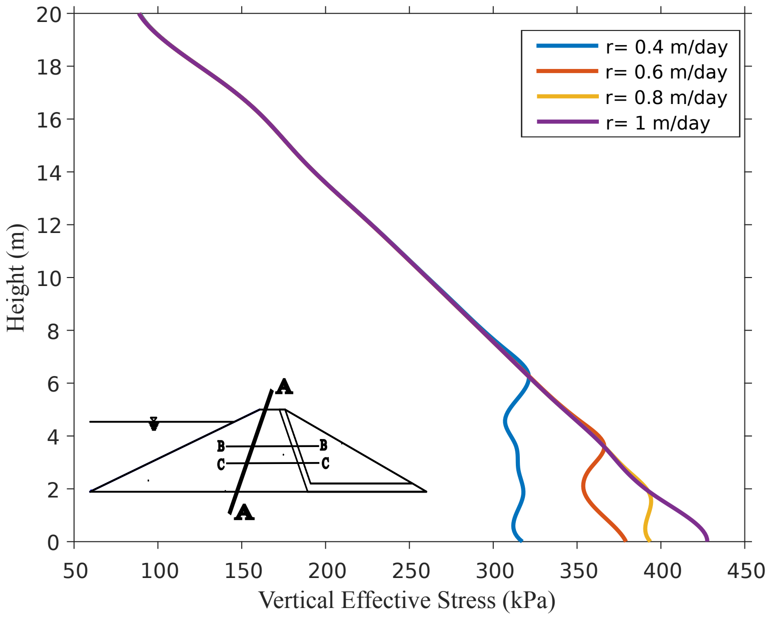
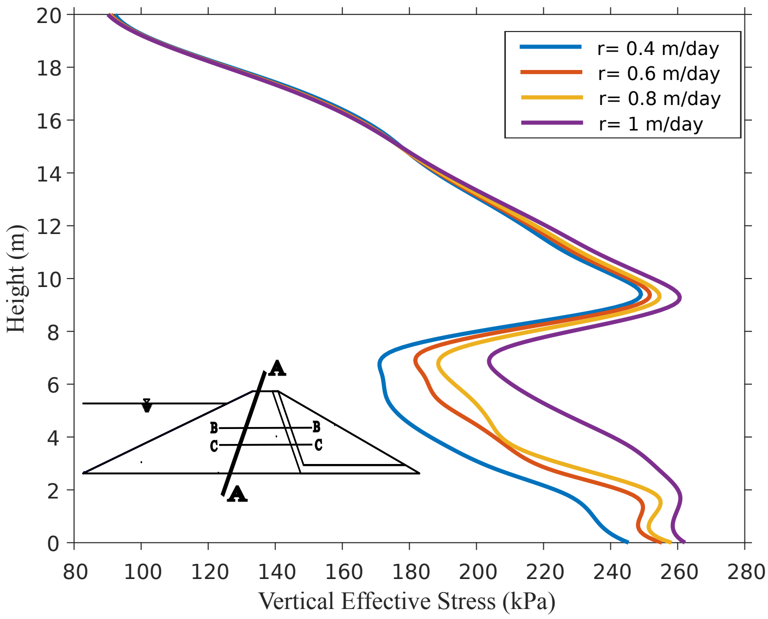


Figure 5.28: Effective stress on section A-A at elevation $H/2$

The alternatives are compared with respect to height at Section A-A in Figure 4.4 at the end of each filling that is, day 21.25 for 0.4 m/day, day 14.15 for 0.6 m/day, day 10.62 for 0.8 m/day and day 8.5 for 1 m/day. The effect of the first hold of 20 days is also inspected in order to see the effect of uncontrolled rate on first hold duration. These are presented in Figure 5.29a and Figure 5.29b, respectively. The readings are extracted from the end of the first hold that is uncontrolled filling duration additional to 20 days of first hold that is, day 41.25 for 0.4 m/day, day 34.15 for 0.6 m/day, day 30.62 for 0.8 m/day and day 28.5 for 1 m/day. Figure 5.29a indicates the seepage arrival to the core zone for different rates. While phreatic surface reaches only to the bottom part of the core zone for 1 m/day rate, for 0.4 m/day reaches up to its stationary level of $H/2$ that is the hold elevation. Figure 5.29b shows the effective stress values at the end of the first hold. During the first hold, it is expected to see a decrease in effective stress as time passes (See Figure 5.3). Since the minimum effective stress curve is obtained by 0.4 m/day which is closer to the convergence among other rates, it had adequate hold for convergence. However, for the rate of 1 m/day case, there is still some time for its convergence, therefore, a longer monitoring is required. This results in a conclusion that faster rates need longer hold time for convergence than slower term. However, since the filling time is shorter for faster rates, the overall time for low pool filling in addition to first hold duration is still shorter than slower rates.



(a) Effective stress variation at the end of uncontrolled filling at section A-A

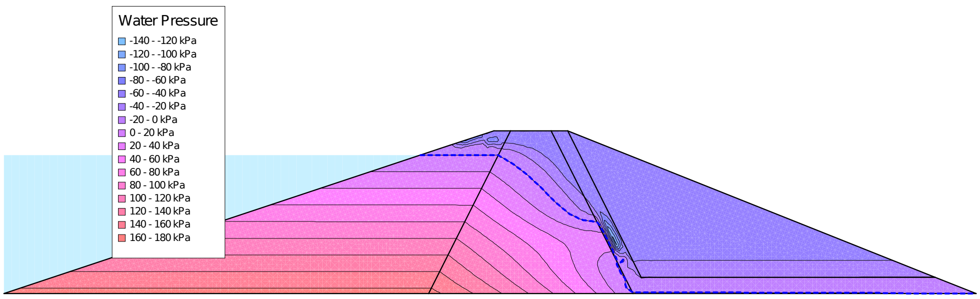


(b) Effective stress variation at the end of 20 days hold at section A-A

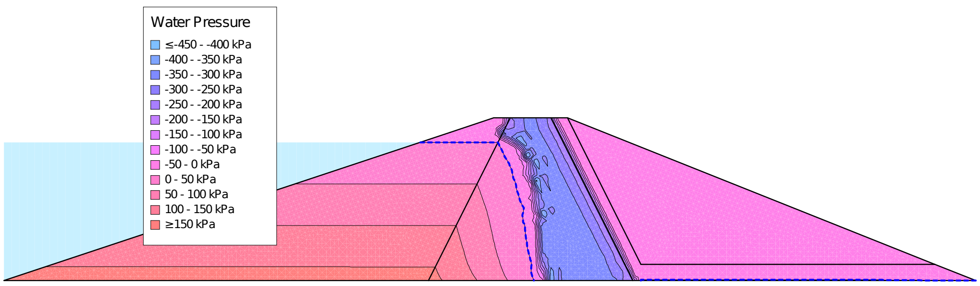
Figure 5.29: Effect of different filling rates

5.3.3 Effect of Moisture Content During Material Placement

The effect of moisture content of a material placement is a crucial factor for a proper design. At previous sections, it is mentioned that the water content during the construction placement plays an important role on crack formation during first filling. The priorly generated pressures and resultant lower conductivities are the engineering measures against internal erosion. This study emphasizes the role of water content. USBR (2014b) recommends the optimum water content for each earth material. The recommended values of the water content are marked on the VWC function of the material to note corresponding negative pore-water pressure. Later, these negative pore-water pressure values are assigned to each material as their initial activation pressure reflecting their placed conditions.



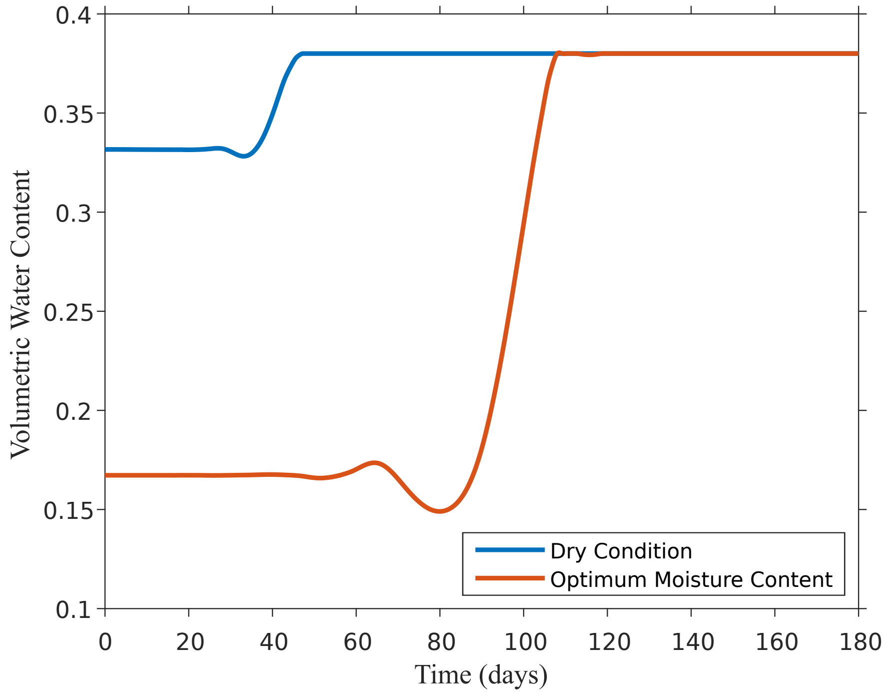
(a) Pore-water pressure contours at day 180 for dry initial conditions



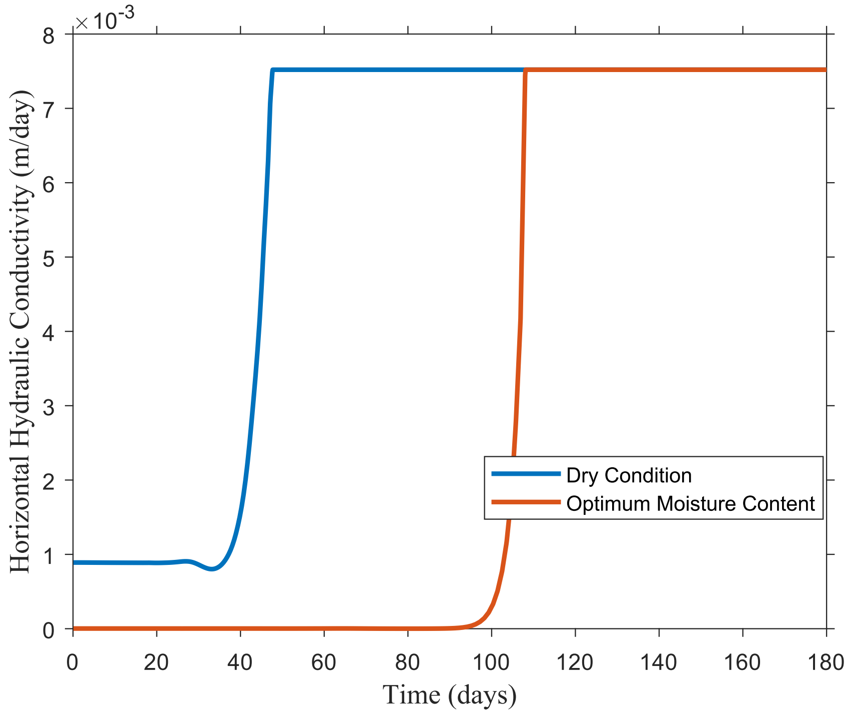
(b) Pore-water pressure contours at day 180 for optimum moisture content initial conditions

Figure 5.30: Phreatic surface at the analysis end for different initial conditions

In this application, negative initial pressure induced soil has much lower hydraulic conductivity that makes first filling much longer in total. The long lag time between the initiation of saturated hydraulic conductivity value limits the seepage propagation in the core in a such way that the phreatic surface difference between the alternatives are quite different from each other, as shown in Figure 5.30.



(a) Temporal variation of volumetric water content for clay material



(b) Temporal variation of horizontal hydraulic conductivity of clay material

Figure 5.31: Effect of placement water content

From the Node 1 (See Figure 5.26), located in upstream side of the core, representative VWC and corresponding hydraulic conductivity variation is provided as Figure 5.31a and Figure 5.31b, respectively. The temporal variation of hydraulic conductivity is responsible for the speed of seepage propagation. Figure 5.32 shows the variation of minimum principal stress at the end of the first hold between the initial conditions of dry and optimum moisture content. It can be seen that, the effective stress curve has already converged if the conditions are dry. This is expected since the seepage is much faster due to higher hydraulic conductivity. However, the crack formation near the crest is highly expected since the soil is loaded by its tensile strength. Therefore, this figure clearly shows how initial conditions are very important measures against crack formation during first filling.

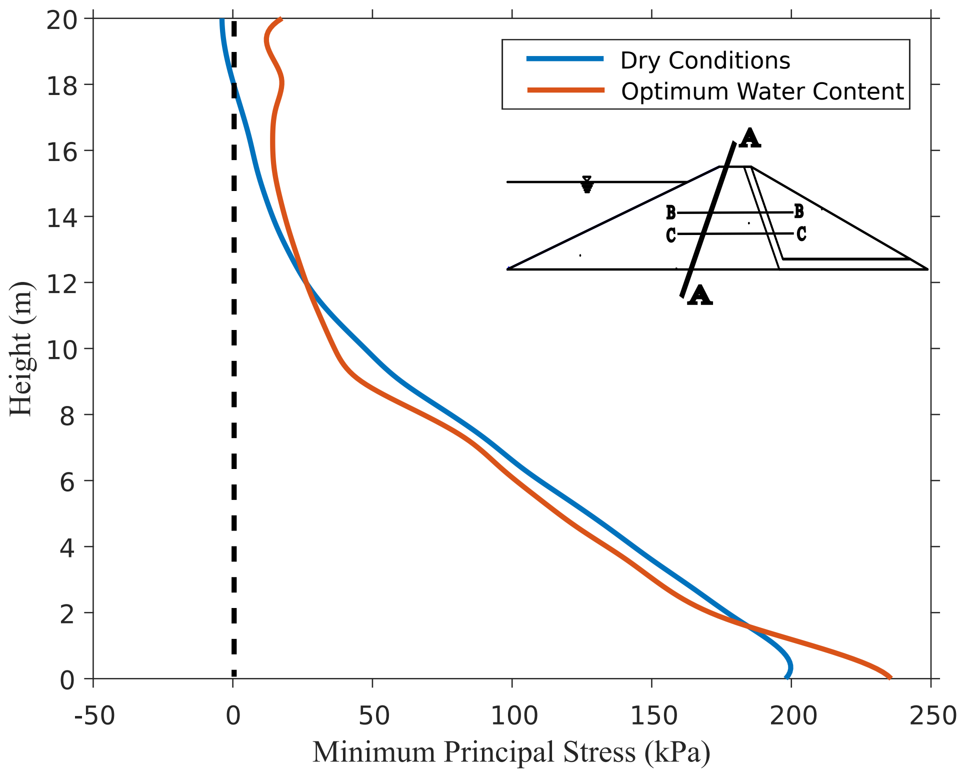


Figure 5.32: Effective stress on section A-A at the end of first hold for different initial conditions

5.3.4 Effect of Material Properties

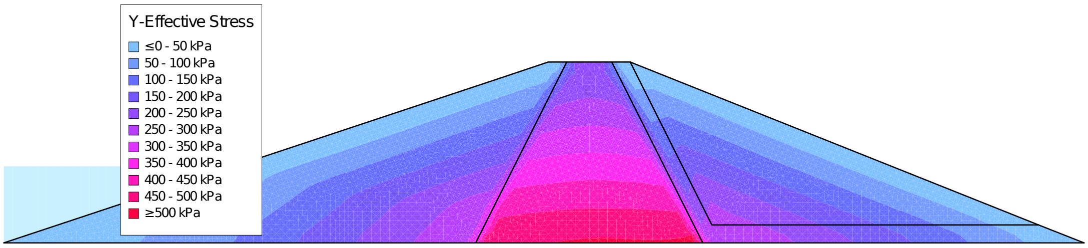
The results are highly sensitive to saturated and unsaturated properties of the shell and core material, because, the key parameters for different hold durations are mainly affected from the propagation time of the seepage. These are unsaturated hydraulic conductivity and nonlinear function of VWC. This study is based on the sample VWC functions for each material from GeoStudio's database, since only the simplification of the functions resulted in convergent solutions. As Section 5.3.3 reveals, the higher hydraulic conductivity of the core resulted from initially dry placement, led to a faster seepage propagation and formed hydraulic fracturing in the upstream side of the core. Therefore, in the case of materials with higher hydraulic conductivities, hold durations are expected to be lower. The proposed hold durations are based on the assigned saturated hydraulic conductivity, VWC function and placement water content. This approach is sufficient for the purpose of the study, that is obtaining the hydraulic fracturing possibility on the upstream side of the core for a hypothetical model. However, for the real applications, seepage properties of the fill materials should be carefully implemented to finite element models in preliminary and final design of the filling schedule in order to decide on exact hold durations. On the other hand, stress/strain properties of materials have negligible effect on hold durations.

5.4 General Behavior of The Dam With The Proposed Filling Schedule

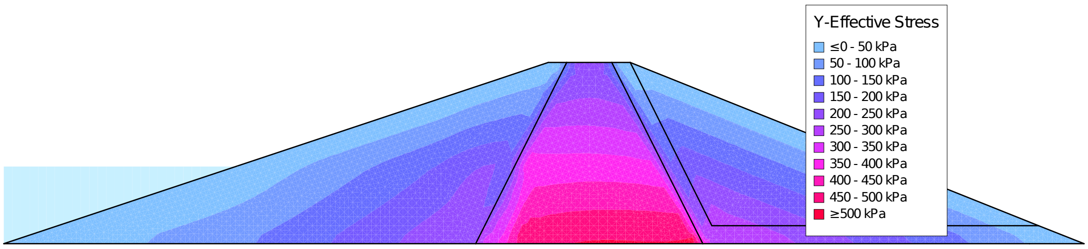
In the light of previous sections and accepted filling schedule of 20 days of first hold and 50 days of second hold, the complete initial filling process is investigated in this section. Effective stress distribution in the dam body starts with relatively higher values. This is due to the placement water content induced negative pore-water pressures. The core layer, especially, has more than 300 kPa effective stresses (Figure 5.33a). The reduction in the effective stress starts with the reservoir filling (Figure 5.33b).

The upstream shell is more susceptible to this reduction over the time as phreatic surface moves into the core. When the water level in the reservoir reaches up to $3H/4$ level, the effective stress reduction reaches to critical levels. Figure 5.33c shows the reduced blue area just near the upstream boundary of the core. This indicates that the crack formation possibility increases between $3H/4$ and $H/2$ levels. The crack formation occurs when effective stress reduced below zero. Fortunately, by the initially induced high effective stresses, the stress does not reach under zero. It should be noted that, it the material were placed in dry conditions, crack occurrence would be highly possible. Eventually, the reduction in the stress propagates into the core zone.

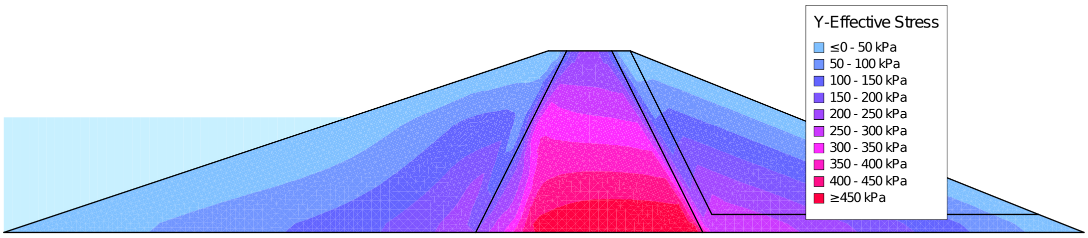
Vertical displacement variation in the dam body is presented in Figure 5.34. The red colored upward movement follows the phreatic surface and has its maximum value at the upstream slope of the dam after first stage of filling is done as shown in Figure 5.34a. When the first hold ended, the vertical displacement is spread uniformly inside the upstream shell (See Figure 5.34b) and set its focus on to the upstream side of the core (See Figure 5.34c). Eventually, higher displacements are distributed inside the core zone after the second hold ended. The magnitudes of the displacements are in the accepted limits that is under 1%.



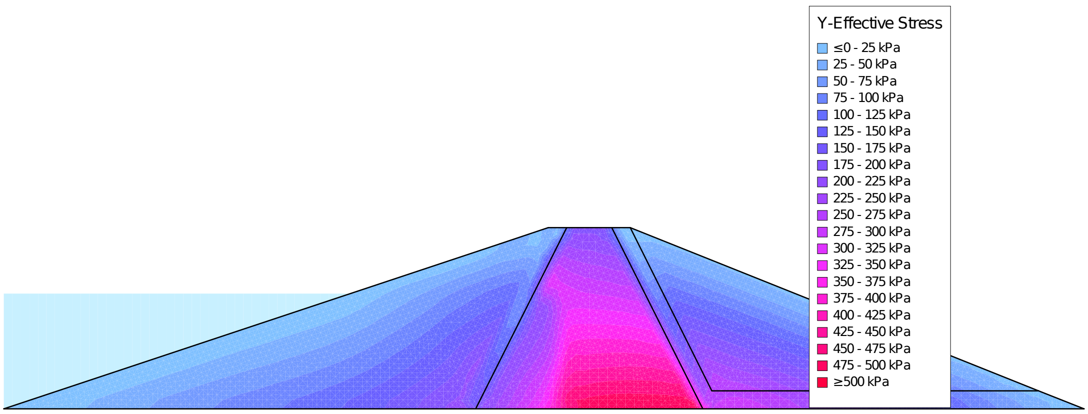
(a) Uncontrolled filling ended - day 14



(b) First hold ended - day 34

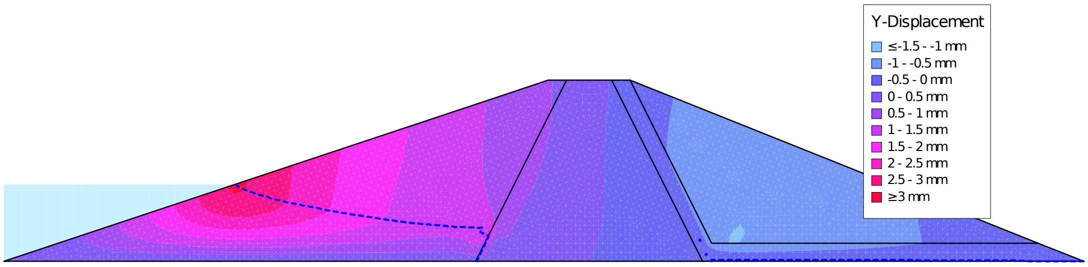


(c) Second filling ended - day 48

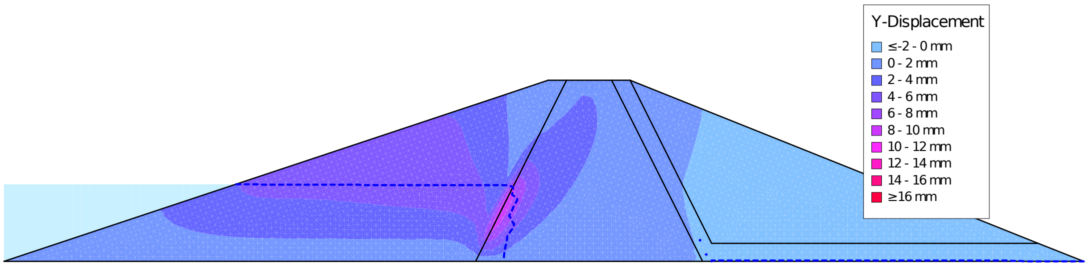


(d) Second hold ended - day 98

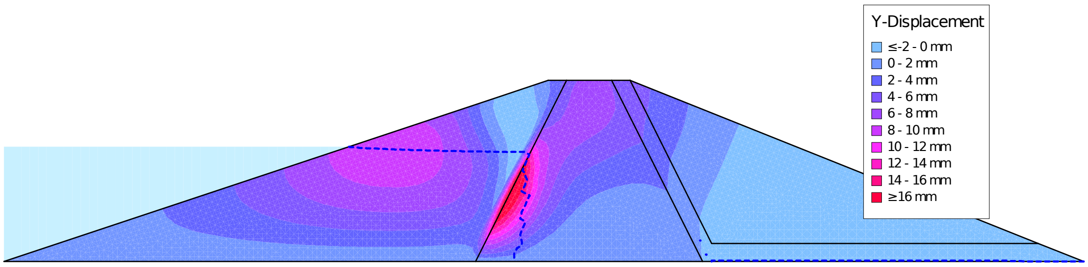
Figure 5.33: Effective stress distribution



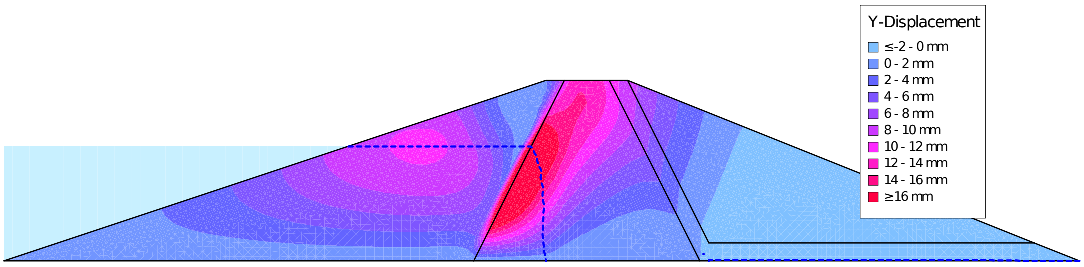
(a) Uncontrolled filling ended - day 14



(b) First hold ended - day 34



(c) Second filling ended - day 48



(d) Second hold ended - day 98

Figure 5.34: Vertical displacement distribution

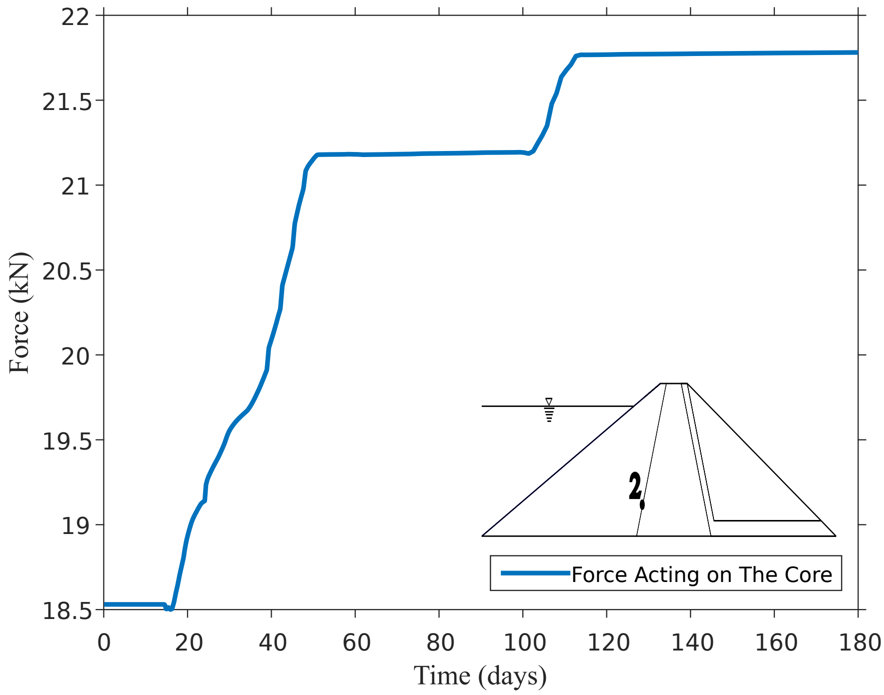
5.5 Verification of Results

The verification of the results is based on the past experiences in the literature, expected behavior of the embankment and engineering judgement. The verification is most commonly established on the field monitoring data of case studies for finite element models using back-analysis. However, since this study focuses on the general behavior and approach the problem with the use of a hypothetical model, a calibration using the field data is not presented.

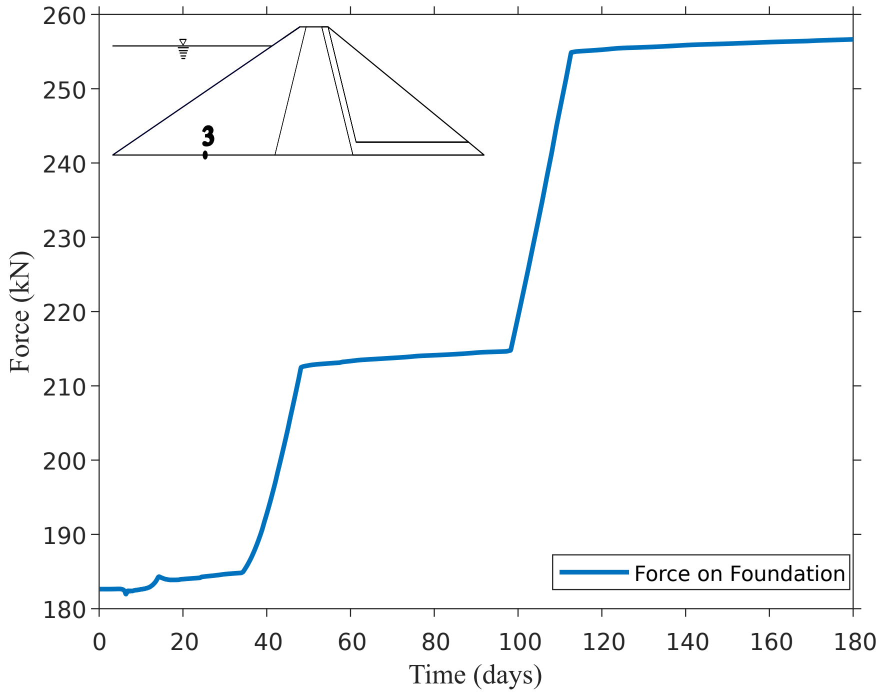
The findings of the study are compared to the actual field practices. The first hold duration of 20 days at 8.5 m and 50 days of second hold at 12.5 m are reasonable for small dams, which equal to hold-to-height ratio (r_{hh}) of 2.35 and 4, respectively for the low and high pool levels. Similar first and second hold durations are presented inside the compilation of initial filling data in the work of Nobari and Duncan (1972). For the dams with different heights and low/high pool levels, hold duration to pool level height ratio is preferred for comparison. These are approximately; 20 days of hold at 9 m high low pool level (r_{hh}=2.22) for Medicine Creek Dam (USA), 60 days of high pool hold at 30 m pool level (r_{hh}=2) for Cherry Valley Dam (USA), 45 days of second hold at 30 m height (r_{hh}=1.5) for Cougar Dam (USA), 100 days of first hold at 50 m height (r_{hh}=2) for Round Butte Dam (USA), which gives a similar height-to-hold ratio.

The four expected behaviors (See Figure 2.2) of Nobari and Duncan (1972) are clearly observed for the current model. At first, the expected water load on the core is experienced as water level rises in the reservoir and presented in Figure 5.35a by the data from Node 2 (See Figure 5.26). The second effect of increasing water load on foundation is shown in Figure 5.35b. A representative node is selected as Node 3 for foundation effects.

The third effect of uplift on the upstream shell is noticeable in the model's behavior (See Figure 5.34) as well as the immediate upward movement (See Figure 5.34a). In addition, the fourth effect of collapsed settlement is remarkable in the model (See Figure 5.34c) as the collapse occur near the boundary between upstream shell and core as the phreatic surface becomes horizontal. It would be clearer if the uplift effect is less emphasized.



(a) Total force acting on the core at node 2



(b) Total force acting on the foundation at node 3

Figure 5.35: Boundary forces

CHAPTER 6

CONCLUSION

6.1 Summary and Conclusions

This thesis puts a light into to the first filling application on a new small earthfill dam. The aim is to present the behavior of dam and propose the durations of intermediary holds in a multi-staged filling using the recommended Turkish practice of first filling schedule. The expected failure reasons suggested on the literature are slope failure and internal erosion due to cracking. Therefore, this study focused on the slope stability and driving phenomena of internal erosion, that is hydraulic fracturing. Fracturing formation is based on the approach that is the fracturing occurs when the minor principal stress is exceeded by the pore-water pressure at the same elevation. The instrumentation readings are extracted from the concerned sections in the dam body, that is the upstream face of the core zone vulnerable to hydraulic fracturing. The study findings were derived specifically for 20 m high zoned embankment dam as follows:

- The study suggests the 20 days long hold duration for low pool level, i.e, $H/2$ when uncontrolled filling rate is limited to 0.6 m/day. It is also revealed that the longer the hold duration at high pool level, the safer the dam against hydraulic fracturing. Therefore, an acceptable duration of 50 days is found to be adequate for high pool levels, i.e, $3H/4$.
- The results of the study reveal the most possible location of crack formation during initial filling, that is the upstream core between the height of $H/2$ and $3H/4$.

- The initial conditions of the earth material are crucially important against crack formation during initial filling. The study concluded that the use of optimum moisture content is a more effective measurement than the well-planning of the filling schedule. The determination of filling schedule plays a critical role on controlling the magnitude of seepage and the load from the reservoir, considering the time-lag between the reservoir level and its effect in the core body.

- The compaction water content of the earth material during construction generates high pore-water pressures that would prevent hydraulic fracturing. This is clearly presented in the current study in terms of minimum principal stress comparison between dry and optimum water content conditions. The dry material is prone to crack formation at the upstream boundary of the core since the minimum principal stress values are partially negative. The increased effective stress values resulting from negative pore-water pressures, had made the embankment resilient to crack formation. The initial high effective stresses eliminate the risk of reducing below zero, that accepted as the crack initiation.

- Another important effect of the compaction water content is the reduced hydraulic conductivity due to negative pore-water pressures. This, by itself, gives an ample time to reservoir during initial filling. In the optimum wet condition, it is not expected to see seepage propagation in the central axis of the dam even when the total filling process ends. Therefore, a proper convergence of parameters during the high pool hold comes much later than the total filling duration.

- The filling rate to the low pool levels, that is uncontrolled filling, has considerable effects on filling. The study finds that the faster rates need longer holds for reaching to convergence compared to slower rates. However, when both filling and hold durations are taken into consideration together, the overall duration of filling schedule is reduced for faster rates. It should be noted that the rate limitations are recommended by considering leakage and piping occurrence due to higher seepage velocities. Therefore, even though relatively faster rates does not create considerable contribution to hydraulic fracturing possibility, higher seepage velocities should be taken into account.

- Differential settlement between the core and the upstream shell is visible through the study findings. The study found that waiting times does not have considerable effect on decreasing the differential settlement. Its occurrence depends on the initial conditions of the earth material and the selection of zoning materials with varying elastic modulus.

6.2 Contribution to Practical Applications

The study provides practical findings that would be considered during the preliminary design of embankments. The expected behavior of the embankment, use of coupled stress-PWP analyses, inspection of fragile sections in the dam body would be taken into account for design engineers.

Secondly, the information is valuable for on-site engineers that has an access to instrumentation data of the embankment during initial filling. Throughout the thesis, the convergence trend of the key parameters as well as their duration is inspected. Engineers may use this information to manipulate the actual dam data in order to understand when to expect convergence from their propagating data. Especially for the holds at low pool levels, the minor principal stress behavior over the time is very crucial for preventing hydraulic fracturing. The initially decreasing principal stress at the upstream boundary of the core should be carefully watched until the increasing trend begins, in order to provide adequate time before loading the dam by even higher loads.

6.3 Suggested Future Work

The following topics may be further studied:

- An optimization study for selection of intermediary water levels is valuable to decide at which elevation the holds should be provided.
- Wider sensitivity analyses of the effect of shell slopes and material properties on hold durations can be conducted since the upstream shell dimensions and material plays considerable role on the water propagation until the core.

- Earthquake effect to the results may be examined.
- Probabilistic analysis of first filling should be considered for the sake of a complete investigation.
- The same approach would be used for existing dam having a remarkable repair or for an existing dam encountered a major flood.
- The effect of slope and thickness of the core would be studied in detail against hydraulic fracturing occurrence since narrow core zones are more susceptible to fracturing.
- The effect of dam height on the filling schedule can be investigated.
- The effect of staged construction prior to filling would also be taken into consideration since the consolidation of the earth material plays an important role for the initial pressure distribution across the dam body.

REFERENCES

Bilgi, V. (1990). *Toprak ve Kaya Dolgu Barajların Projelendirme Kriterleri*. The General Directorate of State Hydraulic Works of Turkey (DSI).

Biot, M. A. (1941). General theory of three-dimensional consolidation. *Journal of Applied Physics*, *12*(2), 155–164. doi: 10.1063/1.1712886

Bowles, J. E. (1996). *Foundation Analysis and Design Fifth Edition* (Fifth Edition ed.). The McGraw-Hill Companies, Inc.

Calamak, M., Kilic, Y., & Yanmaz, A. M. (2020). On the Stability of Moderate Height Berm-Type Earthen Dams: The Hancagiz Dam Example. *Geotechnical and Geological Engineering*, *38*(4), 4169–4179. doi: 10.1007/S10706-020-01286-6

Calamak, M., & Yanmaz, A. M. (2017). Uncertainty Quantification of Transient Unsaturated Seepage through Embankment Dams. *International Journal of Geomechanics*, *17*(6). doi: 10.1061/(ASCE)GM.1943-5622.0000823

Calamak, M., & Yanmaz, A. M. (2018). Assessment of Core-Filter Configuration Performance of Rock-Fill Dams under Uncertainties. *International Journal of Geomechanics*, *18*(4), 06018006. doi: 10.1061/(ASCE)GM.1943-5622.0001114

Calamak, M., Yilmaz, A. N., & Yanmaz, A. M. (2018). Performance Evaluation of Internal Drains of Earthen Dams. *Journal of Performance of Constructed Facilities*, *32*(6), 04018085. doi: 10.1061/(ASCE)CF.1943-5509.0001232

Carsel, R. F., & Parrish, R. S. (1988). Developing joint probability distributions of soil water retention characteristics. *Water Resources Research*, *24*(5), 755–769. doi: 10.1029/WR024I005P00755

Carter, M., & Bentley, S. P. (2016). *Soil Properties and their Correlations: Second Edition* (2nd ed.). John Wiley & Sons, Ltd. doi: 10.1002/9781119130888

Dakshanamurthy, V., Fredlund, D., & Rahardjo, H. (1984). Coupled Three- Dimensional Consolidation Theory of Unsaturated Porous Media. *Fifth International Conference on Expansive Soils*(84/3), 99–103.

DSI. (2014). *Baraj Hidrolik Yapılar Tasarım Rehberi* (Tech. Rep.). Ankara.

Duncan, J. M., Wright, S. G., & Brandon, T. L. (2014). Soil Strength and Slope Stability, 2nd Edition. *John Wiley & Sons*, 336.

Eslami, A., Ghorbani, A., & Shahraini, S. V. (2020). Health monitoring and dynamic analysis of an earth-fill dam at the stage of first impounding: case study—Siahoo dam of Iran. *Journal of Civil Structural Health Monitoring*, *10*(3), 425–442. doi: 10.1007/s13349-020-00393-5

FEMA. (2005). *Conduits through Embankment Dams* (Tech. Rep.). Federal Emergency Management Agency.

FEMA. (2011). *Filters for Embankment Dams Best Practices for Design and Construction* (Tech. Rep.). Federal Emergency Management Agency.

Foster, M., Fell, R., & Spannagle, M. (2011). The statistics of embankment dam failures and accidents. *https://doi.org/10.1139/t00-030*, *37*(5), 1000–1024. doi: 10.1139/T00-030

Fredlund, D. G., & Morgenstern, N. R. (1976). Constitutive relations for volume change in unsaturated soils. *https://doi.org/10.1139/t76-029*, *13*(3), 261–276. doi: 10.1139/T76-029

Fredlund, D. G., & Morgenstern, N. R. (1977). Stress State Variables for Unsaturated Soils. *Journal of the Geotechnical Engineering Division*, *103*(5), 447–466. doi: 10.1061/AJGEB6.0000423

Fredlund, D. G., & Xing, A. (1994). Equations for the soil-water characteristic curve. *Canadian Geotechnical Journal*, *31*(4), 521–532. doi: 10.1139/T94-061

Geo-Slope Int. Ltd. (2012). *Seepage Modeling with SEEP/W* (July 2012 Edition ed.). Calgary.

Geo-Slope Int. Ltd. (2013). *Stress-Deformation Modeling with SIGMA/W Edition* (Tech. Rep.).

Geo-Slope Int. Ltd. (2022). *Stress-Strain Modeling with GeoStudio* (Tech. Rep.).

Ghanbari, A., & Shams Rad, S. (2013). Development of an empirical criterion for predicting the hydraulic fracturing in the core of earth dams. *Acta Geotechnica*, *10*(2), 243–254. doi: 10.1007/s11440-013-0263-2

Holtz, W., & Hilf, J. (1961). *Settlement of Soil Foundations Due to Saturation* (Tech. Rep.).

ICOLD. (1988). *Dam Surveillance Guide Bulletin 158* (Tech. Rep.).

IDNR. (2001). *General Guidelines for New Dams and Improvements to Existing Dams* (Tech. Rep.). Indianapolis: Department of Natural Resources, Division of Water, Indiana.

Kjaernsli, B., & Torblaa, I. (1968). *Leakage through horizontal cracks in the core of Hyttejuvet darn* (Tech. Rep.). Norwegian Geotechnical Institute.

Lo, K., & Kaniaru, K. (1990). Hydraulic fracture in earth and rock-fill dams a. *Canadian Geotechnical Journal*, *27*, 496–506. doi: 10.1139/t90-064

Löfquist, B. (1957). in Discussion on cracking in earth dams. In *Proceedings of the 4th international conference of soil mechanics and foundation engineering* (pp. 261–262). London.

Mualem, Y. (1976). A new model for predicting the hydraulic conductivity of unsaturated porous media. *Water Resources Research*, *12*(3), 513–522. doi: 10.1029/WR012I003P00513

Nobari, E. S., & Duncan, J. (1972). *Effect of Reservoir Filling on Stresses and Movement in Earth and Rockfill Dams. A Report of an Investigation.* (Tech. Rep.). College of Engineering University of California Berkeley.

Nobari, E. S., Lee, K. L., & Duncan, J. M. (1973). *Hydraulic Fracturing in Zoned Earth and Rockfill Dams* (Tech. Rep.). USACE.

Papagianakis, A. T., & Fredlund, D. G. (1984). A steady state model for flow in saturated-unsaturated soils. *Canadian Geotechnical Journal*, *21*(3), 419–430.

Penman, A. (1986). On the embankment dam. *Géotechnique*.

Penman, A., & Charles, J. A. (1981). Assessing the risk of hydraulic fracture in dam cores. In *Proceedings of the 10th international conference on soil mechanics and foundation engineering* (pp. 457–462). Stockholm, Sweden.

Rashidi, M., & Haeri, S. M. (2017). Evaluation of behaviors of earth and rockfill dams during construction and initial impounding using instrumentation data and numerical modeling. *Journal of Rock Mechanics and Geotechnical Engineering*, *9*(4), 709–725. doi: 10.1016/J.JRMGE.2016.12.003

Rawls, W. J., Brakensiek, D. L., & Saxton, K. E. (1982). Estimation of Soil Water Properties. *Transactions of the ASAE*.

Senturk, F. (1988). *Barajların Projelendirilmesinde Hidrolik Esaslar* . The State Hydraulic Works of Turkey (DSI).

Sherard, J. L. (1953). *Influence of soil properties and construction methods on the* *performance of homogeneous earth dams.* (Tech. Rep.). USBR.

Sherard, J. L. (1973). Embankment Dam Cracking. In *Embankment-dam engineering* (Vol. Casagrande, pp. 324–328). New York: John Wiley.

Sherard, J. L. (1986). Hydraulic fracturing in embankment dams. *Journal of Geotechnical Engineering*, *112*(10), 905–927. doi: 10.1061/(ASCE)0733-9410(1986)112:10(905)

Singh, B., & Varshney, R. S. (1995). *Engineering For Embankment Dams.*

Talukdar, P., & Dey, A. (2019). Hydraulic failures of earthen dams and embankments. *Innovative Infrastructure Solutions*, *4*(1). doi: 10.1007/s41062-019-0229-9

Talukdar, P., & Dey, A. (2022). Influence of Drainage Blanket on the Sustenance of an Earthen Dam with a Cracked Central Core. *Journal of The Institution of Engineers (India): Series A*, *103*(1), 215–225. doi: 10.1007/s40030-021-00588-z

Terzaghi, K., Peck, R. B., & Mesri, G. (1996). *Soil Mechanics in Engineering Practice* (3rd ed.). John Wiley & Sons.

Tran, D. Q., Nishimura, S., Senge, M., & Nishiyama, T. (2020). Risk of Embankment Dam Failure from Viewpoint of Hydraulic Fracturing: Statistics, Mechanism, and Measures. *Reviews in Agricultural Science*, *8*, 216–229. doi: 10.7831/RAS.8.0{_}216

USACE. (n.d.). *Recommended Guidelines for Safety Inspection of Dams* (Tech. Rep.). United States. Army. Corps of Engineers.

USACE. (1986). *Engineering and Design: Seepage Analysis and Control for Dams* (Tech. Rep.).

USACE. (2004). *General Design and Construction Considerations for Earth and Rock-Fill Dams* (Tech. Rep.).

USBR. (1987). *Design of Small Dams* (3rd ed.). United States Government Printing Office, Denver, Colorado.

USBR. (1990). *ACER Technical Memorandum No.3: Criteria and Guidelines for Evacuating Storage Reservoirs and Sizing Low-Level Outlet Works* (Tech. Rep.).

USBR. (1998). *Earth Manual* (3rd ed.).

USBR. (2011a). *Embankment Dams Chapter 1: General Design Standards: In Design Standards No. 13* (Tech. Rep.).

USBR. (2011b). *Embankment Dams Chapter 4: Static Stability Analysis: In Design Standards No. 13* (Tech. Rep.).

USBR. (2012). *Embankment Dams Chapter 2: Embankment Design: In Design Standards No. 13* (Tech. Rep.). Washington D.C..

USBR. (2014a). *Embankment Dams Chapter 11: Instrumentation and Monitoring: In Design Standards No. 13* (Tech. Rep.).

USBR. (2014b). *Embankment Dams Chapter 8: Seepage: In Design Standards No. 13* (Tech. Rep.).

USBR. (2021). *Appurtenant Structures for Dams (Spillways and Outlet Works) Chapter 4: General Outlet Works Design Considerations: In Design Standards No. 14* (Tech. Rep.).

van Genuchten, M. T. (1980). A Closed-form Equation for Predicting the Hydraulic Conductivity of Unsaturated Soils. *Soil Science Society of America Journal*, *44*(5), 892–898. doi: 10.2136/SSSAJ1980.03615995004400050002X

Wang, L., Wu, C., Gu, X., Liu, H., Mei, G., & Zhang, W. (2020). Probabilistic stability analysis of earth dam slope under transient seepage using multivariate adaptive regression splines. *Bulletin of Engineering Geology and the Environment*, *79*(6), 2763–2775. doi: 10.1007/s10064-020-01730-0

Widjaja, H., Duncan, J. M., & Seed, H. B. (1984). *Scale and time effects in hydraulic fracturing* (Tech. Rep.).

Wong, T. T., Fredlund, D. G., & Krahn, J. (1998). A numerical study of coupled consolidation in unsaturated soils. *undefined*, *35*(6), 926–937. doi: 10.1139/T98-065

Yanmaz, A. M. (2022). *Applied Water Resources Engineering* (6th ed.). Nobel Academic Publications.