SAFETY ANALYSIS OF EMBANKMENT DAMS AND ALTERNATIVE ECONOMICAL DESIGN

by

SEFA YILDIRIM

(Under the Direction of Jason Christian)

ABSTRACT

This research evaluates cost effective rehabilitation options for an existing earthen dam located in Grayson, Georgia. The case study dam did not meet Georgia dam safety requirements and therefore required an engineered intervention minimizing risk for downstream receptors. The solution implemented included a roller compacted concrete lining on the dam face, which increased stability enough to meet regulatory standards. This work focuses on evaluating alternative design options to determine an effective and cost efficient solution for similar dam rehabilitation. Using numeric modeling, we evaluated the existing dam configuration to confirm that it did not meet dam safety standards. We evaluated the implemented rehabilitation design to show if it improved reliability of the dam structure to meet regulatory standards. Finally, we evaluated an additional design option to determine if other intervention designs would be as robust. An economic evaluation of expected construction costs for engineering design options led to a recommendations for rehabilitation of similar earthen embankment dams.

INDEX WORDS: Dam safety, embankment dams, geotechnical analysis

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ECONOMICAL DESIGN

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DEDICATION

Dedicated to my dear parents and sisters…

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I would like to thank all of my committee members for their help and to Golder Associates Inc. for providing us all the geotechnical data.

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INTRODUCTION

General

There are many reasons for construction of new dams including recreation, flood control, water storage, irrigation, mine tailing, electrical generation and navigation (FEMA, 2016; Evans, 2000). Although dams are integral to meeting specific societal needs, they can wreak havoc on infrastructure and social networks when they fail. Dam failures can cause death and significant amount of property and environmental damage. It is not known how many dam failures have occurred worldwide, but there have been hundreds of dam failures in the USA history alone, including a documented 173 dam failures from 2005 to June 2013 ("Association of State Dam Safety Officials", n.d.).

Dam failures and resulting release of impounded water cause much larger floods compared to snowmelt or rainfall floods (Costa, 1985). Many have lost life or property because of these disasters all over the world, making dam safety analyses very important. In response, jurisdictional authorities have provided criteria or procedures for dam design, construction, inspection and operation to prevent dam failures, and all dams in Georgia must meet these requirements (Georgia Safe Dams Program, 2015). Additionally maintenance of the dams should be done continuously as long as the dams are in use. A continuous schedule of inspection can help engineers predict pending failures of dams. Studies show that early recognition of potential dam failure and prompt warnings can decrease the number of fatalities by a factor of 19 as compared to unexpected and sudden failures (Costa, 1985).

Geotechnical analyses are one of the most important criteria for dam safety. They should be performed not only for new dams but also for dams that have to be rehabilitated for any reason (Georgia Safe Dams Program, 2015). This study evaluates cost effective rehabilitation options for an embankment dam and uses The Big Haynes Creek Watershed Dam No. 3 Dam (H-3) located near Grayson, Georgia as a case study. Safety analyses are based on the State of Georgia dam safety requirements. H-3 was constructed in 1963 as an earthen embankment dam for flood control purpose and is currently owned, operated and maintained by the Gwinnett County. Since its construction, forty new residential structures have been built downstream of the dam, and they are considered to be at risk of damage from any possible failure of H-3. Geotechnical analysis performed in 2008 of the H-3 dam showed that it did not meet all the State of Georgia dam safety requirements. Specifically, upstream slope stability was inadequate to meet the requirements under rapid drawdown conditions in the reservoir. Gwinnett County considered H-3 as a high hazard dam that may cause loss of human life. Therefore, an engineering intervention and rehabilitation was required to improve its stability.

Golder Associates Inc. (Golder) is a private engineering company with specialty in dam construction, rehabilitation that was founded in Toronto, Canada in 1960. It is a global company that provides consulting, design, and construction services for civil infrastructures projects (Golder Associates, 2017). Golder was retained by Gwinnett

County for engineering analysis and dam rehabilitation design, and they provided all the geotechnical data for this study.

Scope of the Study

The main purposes of this study are to consider the importance of dam safety and to introduce optimal rehabilitation approaches for embankment dams. Dam stability is characterized under the different loads such as static and seismic loads or different condition such as rapid drawdown of the reservoir. Rapid drawdown occurs when reservoir water is removed quickly, and it may create excess negative pore pressure and decrease the upstream slope stability. Defining the conditions where the dam has insufficient stability and taking the measure of these specific conditions can lead to identification of the most effective rehabilitation effort. A typical renovation of the H-3 dam is considered as a case study in this work.

Chapter 2 introduces societal benefits of dams, classification of dams based on structure and design, the most common dam reasons for failure, measures for these failure types, and some historical dam failures and the resulting consequences. Chapter 3 presents the geotechnical analysis of H-3. A geotechnical stability modeling program (Slide) is used for steady-state seepage stability and slope stability analysis. Under all geotechnical analysis conditions, the magnitude of resisting forces against failure divided by the magnitude of driving forces is called the factor of safety (FS). The higher the factor of safety, the safer the construction is. Factor of safety is estimated by the Slide software for embankment dams and chapter 3 considers three different designs of H-3 dam, their geotechnical suitability and probable rehabilitation construction costs. Chapter 4 summarizes all geotechnical and rehabilitation cost analyses performed. Finally, Chapter 5 discusses the advantages and disadvantages of the proposed alternative design and the most appropriate rehabilitation works with different factor of safety values.

CHAPTER 2

LITERATURE REVIEW

Dams can be classified by their constructed purpose or type of structure and design. Because the structure and design of dams differ depending on their purpose, their vulnerabilities to different failure mechanisms also vary. This chapter will present common purposes of dams, different dam designs, and common dam failure mechanisms. It is important to study past dam failures and investigate the reasons for these failures in order to prevent future ones. Therefore, several historical dam failures will also be mentioned in the following sub-sections.

Classification of Dams Based on Constructed Purposes

Dams can be constructed for many purposes such as recreation, flood control, water source or navigation. Some dams might have single construction purpose while others might have multiple construction purposes (Evans, 2000). Figure 1 presents the different purposes of dams, and shows that recreation is the most common reason for construction of dams, accounting for approximately 38% of dam structures built. They are followed by flood control, fire and farm ponds and irrigation purposes.

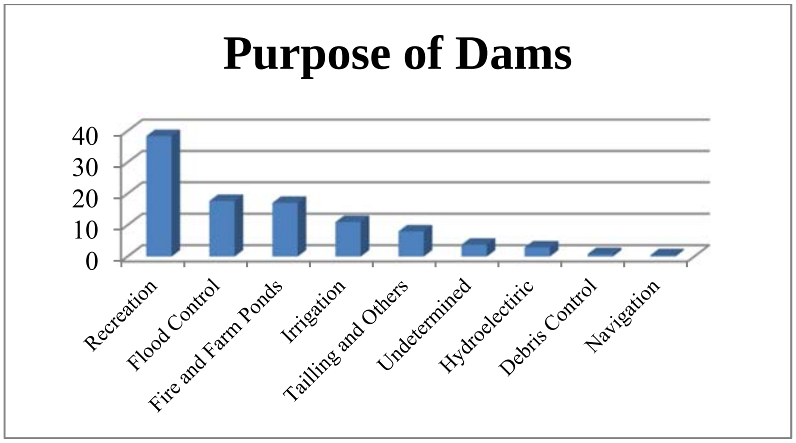

Figure 1: Reasons for dam construction (Redrawn from (FEMA, 2016))

Recreation / Navigation

Dams create reservoirs which commonly used for recreational purposes such as fishing, boating, skiing, camping and having picnic (FEMA, 2016) and there are many examples of this kind of dams in the world. A survey showed that 41% of the participants visit the reservoirs because they find such areas safe and suitable for all family members (Tekccedil et al., 2010). Dams can also be used for navigation as there are many dams dedicated to navigation in the world. To manage the water in dams used for navigation, the water level can be adjusted by keeping canals filled during dry seasons and lowering the water during wet seasons ("Dams and Reservoirs", n.d.).

Flood Control

The impacts of flood control dams on river hydrology are significant (Magilligan and Nislow, 2005). Flood control dams function by impound excess flowing water in upstream reservoirs, therefore protecting downstream receptors from excess runoff events. This water can be released over time if needed or can be kept in the reservoir indefinitely. Being able to control flowing water prevents possible damages of uncontrolled water on downstream properties (FEMA, 2016).

Studies of past catastrophic dam failures show that flood control is crucial after a dam failure. These failures release a significant amount of water that can destroy other dams on the downstream river segment potentially leading to cascading failures and significant loss of life and property. The Massive Banqiao and Shimantan dam failures are unfortunate examples of lack of flood control. These two dams, located in Henan, China, collapsed after a heavy rainfall in 1975. Massive amounts of released water produced dozens of small dam failures and as a result of this tragedy, eighty-five thousand people lost their lives and millions more lost their homes (Si, 1998). This dam failure is the biggest dam failure in the world history (Peng and Zhang, 2012).

Water Storage (Fire & Farm Ponds)

Dams are beneficial structures for water storage to be used for industrial, municipal, and agricultural purposes (FEMA, 2016). These dams are actively managed, so if an extreme rainfall is expected, storage water can be released under control before the rain starts. During heavy rains, more water can be stored giving some flood control benefit when actively managed. These structures can therefore decrease the negative impact of heavy rainfall on society and stored water can later be used when it is needed (ICOLD, n.d.).

Irrigation

Dams and reservoirs are often used to provide water to irrigate land. Irrigation is vitally important to supply food for around 15% of the world's population. Because of the increasing demand for food, providing irrigated land available for growing crops will become even more important. Therefore irrigation dams are one of the most effective ways to provide water for the food production purpose ("Dams and Reservoirs", n.d.).

Many of the single-purpose dams are irrigation dams, and they account for 38% of dams worldwide (ICOLD, n.d.). Approximately 277 million hectares of land are irrigated in the world. It is expected that in 2025, 80% of the food will be provided by irrigated lands. This expectation requires more dams for irrigation (ICOLD, n.d.). Impounded reservoir water is a good source for irrigating cropland in the USA as well, as ten percent of the cropland is irrigated - providing job opportunities for thousands of Americans (FEMA, 2016). On the other hand, there are some negative impacts of irrigation on human health. Irrigation projects facilitate a change in the frequency and transmission dynamics of malaria, so constructing more dams for irrigation purposes might increase the risk of malaria (Keiser, et al., 2005). A number of irrigation dams should be removed and people should not be allowed to live around the dams that remain.

Electrical Generation

One of the most useful benefits of dams is electricity generated in hydro-electric power plants. Hydropower dams can harvest gravitational potential energy to create electricity, and provide inexpensive and clean sources of renewable energy (Lindström and Grani, 2012). Most of hydro-power plants have capacities up to several hundred megawatts (ICOLD, n.d.). Only a few of them can reach the capacity of ten thousand megawatts, providing electricity for millions of people. In the world, 675,000 megawatts of electricity is provided by hydro-power plants each year producing 2.3 trillion kilowatts per hour. This amount fulfills 24% of all the world electricity needs. Extreme examples of hydro-power reliance include Norway and The Domestic Republic Congo (generating

99% of the local power demand) and Brazil (generating 91% of its electricity from hydropower plants) (ICOLD, n.d.). The largest hydropower producer in the world is Canada providing an estimated 310 billion kilowatt-hours of power each year, which is followed by the USA generation 255 billion kilowatt-hours of power each year ("Facts About Hydropower", 2017).

Providing hydro-power without using any fossil fuels is one of the most beneficial social uses of dams (Schiermeier et al., 2008). Hydro-power is a good source of renewable energy as it provides energy without contributing to global warming, air pollution, acid rains or ozone depletion (FEMA, 2016). Dams provide the number one source of renewable energy, providing 90% of the total renewable energy of the world (ICOLD, n.d.).

Mine Tailing / Debris Control

Tailings which are also called mine dumps are the materials consisting of milled rock and process wastes produced in the mine processing plant (Engels, 2012). Mine tailing dams allow mining as they are built with the purpose of obtaining coal and minerals. In the United States alone, there are more than 1300 mine tailing dams. Dams can also supply great protection for the environment by retaining of dangerous material and sediment (FEMA, 2016). The debris control dams trap sediments carried by flood control and debris flows to help mitigating the risk of debris pollution.

Considering all of these purposes for dams, the most effective structural materials and designs should be chosen. The next section will introduce the most common types of dam structures and design.

Classification of Dams Based on Structure and Design

Dams are designed following strict standards to ensure that any failure or collapse of dam does not occur. Engineering judgment is very important for design of dams in terms of proposing effective solutions to potential dam failures. Despite the specific requirements which must be met by all constructed dams, available options for dam design vary greatly (Georgia Safe Dams Program, 2015). Different types of dams are designed to accomplish different purposes and different reasons can cause to fail of these dams. This section discusses the most common types of dams highlighting their strengths and weaknesses. Based on a dam's design and construction material, they can be simply classified as gravity, arch, buttress or embankment dams. Figure 2 shows typical cross-sections of dam type. Embankment dams and concrete dams are the most common types of dam (Prasiddha, 2015).

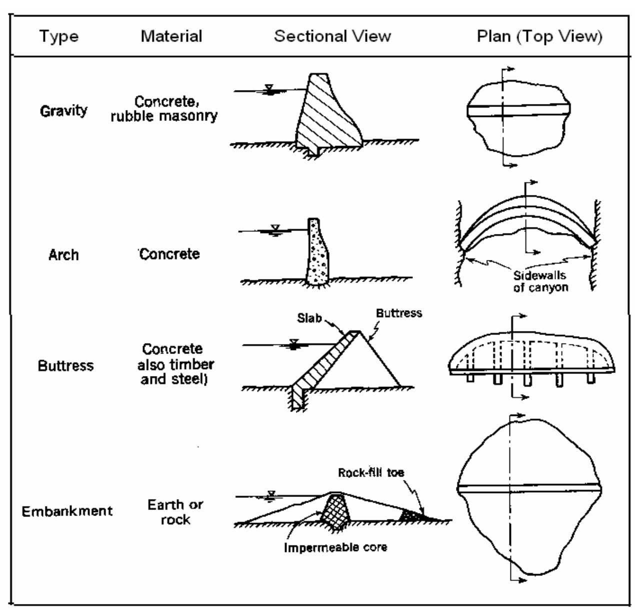

Figure 2: Typical cross-sections of each type of dams (adapted from (Prasiddha, 2015))

Gravity Dams

The most common purpose of gravity dams is holding a massive volume of water within the reservoir. They are generally made of concrete or stone masonry. Because of these construction materials, the weight of gravity dams are very high, allowing them to resist horizontal forces coming from impounded water. Because of their weights, they have to be built on a solid foundation (Prasiddha, 2015).

Embankment Dams

Embankment dams are the most common type of dams in the world. The main purpose of these dams is impounding and diverting water. Natural materials, generally soil and rock, can be used for the construction of embankment dams, and they can be constructed on both soil and rock foundations (Imbrogno, 2014). They are classified as earthen and rock fill embankment dams (Narita, 2000).

Soil materials are more permeable than concrete. Therefore, water movement through the dam is easier in embankment dams compared to concrete dams. Rock fragments and boulders can be used for rock-fill dams. There should be a membrane on the rock-fill at the upstream side or an earth core in the middle of the crest to reduce water movement. Cement and asphalt concrete are common membrane material. Additionally, a proper filter should be provided between earth core and rock-fill otherwise soil particle movement cannot be precluded (Stephens, 2010).

Arch Dams

Arch dams are curved in their plan and are convex toward the upstream side. They transfer most of the water pressure to the abutments by arch action. Therefore, they are good choices for narrow canyons. Arch dams may have single or double curvature in the vertical plane.

Buttress Dams

Buttress dams are made from concrete or masonry similar to gravity dams. The triangular shaped walls called buttresses support their watertight upstream side. The buttresses which are spaced at intervals on the dam's downstream side resist the force of the reservoir water to protect dams to be pushed over by the water. This dam design was inspired by the idea of the gravity dam, however buttress dam uses far less material than gravity dam because of the additionally internal structure provided by the buttress supports. Similar to gravity dams, they are suitable to build in both narrow and wide valleys. They also must be constructed on sound rock ("Dams and Reservoirs", n.d.).

Common Reasons for Dam Failure

Dam failures began when the first dams was built. There have been more than 2,000 documented dam failures in world history (Costa, 1985; Jansen, 1980). There are many reasons for dam failure such as overtopping, internal erosion-piping, structural failure, poor designs, sabotage, inadequate maintenance and foundation settlement (Cheng, Yen and Tang 1982; Baecher et al., 1980).

The following sections describe the most common reasons for dam failure, their specific failure mechanisms, and recommendations to avoid these failures. Additionally, specific historical dam failures will be described to better understand the failure mechanism, the effects of dam failure on society, and to take lessons from past failures.

Overtopping Failure

Overtopping flow is the most common reason for dam failures (Morán and Toledo, 2011). Forty-one percent of dam failures have occurred due to overtopping, generally seen in embankment dams (Singh, 1996; Imbrogno, 2014). Overtopping leads to erosion of the crest and downstream face, which leads to steeper slopes, which leads to slope failure, leading to discharge of impounded water. Although not common as in the past, it is still one of the biggest concerns for dam failure (Tingsanchali and Chinnarasri, 2001). Main reasons for overtopping are reservoir water level increases and wave action. Strong wind, a landslide, or an earthquake can trigger wave action (Cheng, Yen and Tang, 1982). A large amount of settlement in the foundation can also cause loos of freeboard and increase the possibility of overtopping failure (Imbrogno, 2014).

Case History of Overtopping Failure

South Fork Dam (Johnstown Dam) was located near South Fork River, near Pennsylvania. The failure of this dam is one of the worst failures in the history of the United States (Frank, 1988). It was a rock-fill earthen dam, constructed for water supply. The failure occurred on May 31, 1889, after a heavy rainfall. As a result of the failure, more than 2,200 people died, most in the downstream city of Johnstown (Coleman et al., 2016; Frank, 1988).

The Taum Sauk Dam was a rock-fill dam, located in Reynold County, Missouri USA. It was constructed between 1960 and 1963 for water storage and failed on December 14, 2005. Only 6-7 minutes of overflowing water caused the failure. As a result of the failure; five people were injured and significant downstream damage occurred (Rogers et al., 2005).

The Walnut Grove Dam was a rock fill dam located near Wickenburg, Arizona. It was constructed between 1986 and 1987 for irrigational and mining purposes. Three days of strong precipitation in the Bradshaw Mountains raised the reservoir water level. The spillway was not big enough to prevent the actual water level increase. Eventually, the dam failed at 2:00 AM on February 21, 1890. As a result of the failure; more than 70 people died (Rogers, 2009).

Many of these catastrophes could be minimized or eliminated with proper engineering design. The following measures should be considered to avoid overtopping dam failure (Duricic, 2014).

- Spillways should not be constructed in faults or slide prone areas.
- Outlets should have sufficient capacity to release water during a flood. Use of inadequate spillway capacity to decrease the cost of construction should be avoided (Marcello et al., 2009).
- Backfill around the conduits should be well compacted to prevent settlement.
- Functioning of gates, valves and conduits have to be tested to assess their behavior during a flood.
- Higher possibility of overtopping failure should be considered for embankment dams. The downstream slope of embankment dams should provide sufficient resistance to erosion.

Seepage Failure

Seepage is the percolation of impounded water through a dam or its foundation. The velocity and quantity of seepage has to be controlled, otherwise, movement of the impounded water forces small soil particles to move and internal erosion starts Internal erosion starts from the downstream at the exit point of seepage and continues persistently towards the upstream of the dam where seepage began. Eventually, this continuous internal erosion creates a preferential flow path, leading to void in the dam structure. This process is known as piping or undermining ("Dam Safety: Earth Dam Failures", 1994, 1994; Sandhu, 2015).

Piping is one of the main reasons for dam failure in embankment dams as half of embankment dam failures have occurred due to piping (Berrones et al., 2010). It can be identified either by increased seepage flow or the color of the discharged water ("Dam Safety: Earth Dam Failures", 1994). Most piping failures occur rapidly, within 12 hours after leakage is observed (Wang, 2016).

Case History of Seepage Failures

The Teton dam, located in Idaho was constructed between 1974 and 1976. The dam was an embankment dam 123 m in height, 945 m in length, with a total embankment volume of 7.6 x 10^6 m^3 and core volume of 4.0 x 10^6 m^3. The dam failed during the first filling of the reservoir when it was at 87% of full volume on 5 June 1976 (Marcello et al., 2009). Two days before the failure, 0.38 m^3/sec of seepage was observed, and a day before the failure 0.76 m^3 per meter of seepage was observed. However, no unusual condition was reported after the slope investigation. A worker observed dirty seepage water at 8 a.m. on 5 June when seepage flow rate reached 25 m^3/sec. The seepage flow rate increased rapidly and an eroded hole in the dam was observed at 11:20 am, with complete failure occurring at 11:55 am. As a result of the failure fourteen people died and approximate property damage was 400 million dollars (Seed and Duncan, 1981).

Kelly Barnes Dam was a 122 meters long embankment dam, located in Georgia. The dam width was 6.1 meters at the crest and the maximum section height of the dam was 12.2 meters. The dam failed after maximum lake level of 348 meters was reached due to heavy rainfall with a volume of impounded water at about 764555 m^3. The Dam failed at midnight on November 6, 1977 and killed thirty-nine people. As a result of the failure; two college buildings, many houses and vehicles were damaged (Sanders and Sauer, 1979).

Many seepage dam failures could be minimized or eliminated with some precautionary measures and proper engineer design. Following measures are recommended by Casagrande (1968) to avoid the seepage failure (Berrones et al., 2010; Casagrande, 1968).

- Constructed materials should be chosen properly such as material which has low hydraulic conductivity;
- During construction, material homogeneity should be ensured;
- Between coarse and fine materials transition zones should be provided;
- Designed filters should be used to reduce water movement.

Structural Failures

Structural failures can occur in the embankment or the appurtenances such as spillways and lake drains. Cracking, settlement and slope sloughing are the explicit signs of failure. Lake level should be decreased, and people around the dam should be warned when signs of impending failure are observed. Structural failures interact with other reasons for failure. For instance; excessive seepage can decrease the stability of the soil and lead to structural failure. Also, structural failure may cause piping ("Dam Safety: Earth Dam Failures", 1994).

Proper Design Standards

The average operating lifetime of dams is 50 years (Imbrogno, 2014). However, history suggest that 31% of dam failures have occurred during the construction or the first five years of the dam’s life (Regan 2010).

Case History of Design Flaw Failure

The Bouzey dam was located near Epinal France. It was constructed in1881 and failed on the 27th of April 1895 (Smith, 1994). More than 100 people were killed and many properties were damaged. It was a 528 meters long masonry gravity dam 22 meters in height. The thickness of the dam was 11.3 meters at the base and 4 meters at the crest (Marcello, 2009).

First, leakage of 0.057 m^3/sec appeared. The leakage increased over the following year, and part of the dam slipped down approximately 380 millimeters. This event created even more leakage but no measures were taken. One and a half years later the reservoir was drained and a 93 meters horizontal break was observed. Masonry block with puddle clay was built along the crack and the base thickness extended outward and downward. That modification increased the foundation and water pressure on the masonry. Also, no modification to the top of the dam was made. When the reservoir was filled up with water, defects in the crest were ignored. A couple of months later, the top part of the dam broke down and complete failure occurred (Marcello, 2009).

The previous discussion revealed that many of these failures can be minimized or eliminated with proper design and construction standards. The next chapter will describe a case study of H-3 dam introducing an alternative cost effective design ensuring the regulatory requirements of the State of Georgia.

CHAPTER 3

ANALYSIS OF CASE STUDY EMBANKMENT DAM

General

The Big Haynes Creek Watershed Dam No. 3 (H-3) was constructed in 1963 for flood control purpose and it is located near Grayson, Georgia. It was the one of the 14 watershed structures which were maintained and operated by Gwinnett County. Although no failure was observed, Gwinnett County retained the Golder Associates Inc. to perform hydrology and hydraulics (H&H) studies for H-3. Studies showed that forty structures on the downstream side of the dam would be affected by the possible failure of H-3. Therefore, H-3 was considered to have high hazard potential. Geotechnical analysis of the dam was performed by Golder and the dam did not meet the State of Georgia dam safety requirements. Gwinnett County then retained Golder for rehabilitation design of H-3 in 2008. As part of this rehabilitation design of H-3, 3.35 vertical meters of earthen material was removed and replaced with 0.31 meters gravel drain and 1.52 meters roller compacted concrete (RCC). The original 3:1 horizontal: vertical side slope was not changed and the entire downstream slope was covered by RCC as shown in Figure 12. After the implemented rehabilitation design, the H-3 was considered a both flood control and a recreational purpose dam (Golder Associates Inc, 2009).

The design of H-3 dam which is constructed in 1963 will be called original design; the design implemented by Golder will be called improved design and the design proposed as a cost effective rehabilitation will be called alternative design in this work.

Methods

The engineering analysis tool "Slide" is a slope stability analysis software used to evaluate slope stability of embankment and earth dams with all types of soil and rock slopes. It is capable of performing finite element groundwater seepage analysis which is fully integrated with the slope stability analysis (Slide, 2017). Slide (Version 6.003) is used in this work to analyze seepage and slope stability. Slide evaluates the pore pressure effects from based on a groundwater seepage analysis (Slide, 2010a).

Limit Equilibrium Analysis

Limit equilibrium analysis is a simple and accurate method to calculate slope stability. This method divides slope into slices and applies force or/and moment equations (Matthews et al., 2014). Based upon normal and shear forces, there are several limit equilibrium methods, with the difference between these methods being how shear and normal forces are determined (Aryal, 2006). Bishop's, Spencer's, Morgenstern-Price's or Janbu's methods can be used for earth dam slope stability analyses (Guidelines for Permitting a Category I Dam, 2015). The Morgenstern-Price method was applied to calculate slope stability of H-3.

Quantifying strength parameters of the soil is one of the most important parts of the stability analysis (Swenty, 1989). The Mohr-Coulomb expression can be used for all limit equilibrium analyses to find shear strength (Aryal, 2006).

$$\bar{\tau} = \bar{c} + (\sigma - u) \tan \bar{\Phi} \qquad \text{Equation 1}$$

Where:

$\bar{\tau}$ = effective shear stress on the surface at failure (kN/m^2);

$\bar{c}$ = cohesion intercept based on effective stresses (kN/m^2);

σ = total normal stress acting on the failure surface (kN/m^2);

u = pore water pressure (kN/m^2);

$\bar{\Phi}$ = friction angle based on effective stresses (deg);

And,

$$\tau = c + \sigma \tan \Phi \qquad \text{Equation 2}$$

Where:

τ= shear stress on the surface at failure (kN/m^2);

c= cohesion intercept (kN/m^2);

σ= total normal stress acting on the failure surface (kN/m^2);

Φ= friction angle (deg).

Factor of safety (FS) is a general output of limit equilibrium analysis. It can be calculated as a ratio of resisting forces to driving forces (Swenty, 1989; Eberhardt, 2015).

Morgenstern-Price Method

The Morgenstern-Price method satisfies both moment and force equilibriums, and it is commonly used in engineering practice. Figure 2 shows the considered forces of this method for an interslice. T is a function of an arbitrary function of $f(x)$ as it is shown in Equation 3. The relationship between normal force (N) (kN) and interslice forces (T, E) (kN) are shown in Equation 4 and Equation 5 (Aryal, 2006; Nash, 1987).

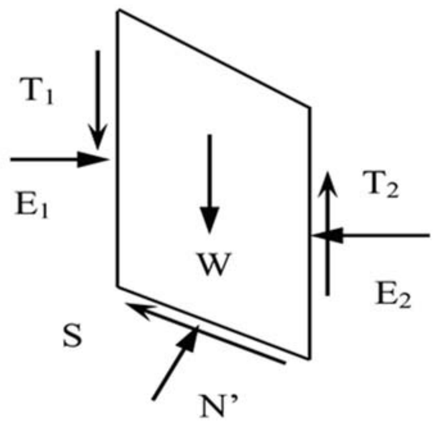

Figure 3: The forces for an interslice considered for Morgenstern-price method (adapted from (Aryal, 2006)).

$$T = f(x)\lambda E \qquad \text{Equation 3}$$

Where:

$f(x)$= interslice force function that varies along the slip surface (unitless);

λ= scale factor of the assumed function (unitless).

$$N = \frac{1}{m_\alpha}\{W - (T_2 - T_1) - \frac{1}{FS}(\overline{c}l - ul\tan\overline{\Phi})\sin\alpha\} \qquad \text{Equation 4}$$

$$T = \tan\alpha_t E - \frac{dE}{dx}h_t \qquad \text{Equation 5}$$

Where:

$$m_\alpha = \cos\alpha\,(1 + \tan\alpha\frac{\tan\overline{\Phi}}{FS}) \qquad \text{Equation 6}$$

Where:

$\tan\alpha_t$= slope of the line of thrust;

h_t= height from the mid-point of the slice base to dE (m);

l= slice base length (m);

α= inclination of slip surface at the middle of the slice (deg).

FS_f *and* FS_m (unitless) represent factor of safety for force and moment equilibrium, respectively. The Morgenstern-Price method considers both force and moment equilibrium and evaluates FS for each type of equilibrium. Equations for FS_f and FS_m are shown in Equations 7 and 8, respectively.

$$FS_f = \frac{\sum[\{\bar{c}l+(N-ul)\tan\bar{\Phi}\}\sec\alpha]}{\sum\{W-(T_2-T_1)\}\tan\alpha+\sum(E_2-E_1)}$$ Equation 7

$$FS_m = \frac{\sum(\bar{c}l+(N-ul)\tan\bar{\Phi})}{\sum W\sin\alpha}$$ Equation 8

The Morgenstern-Price method calculates Equation 7 and 8 with a different value of FS until FS_f is equal to FS_m (Aryal, 2006; Nash, 1987).

Rapid Drawdown Analysis

The B-bar method in Slide was used to calculate FS for a rapid drawdown condition. Slide uses the finite element method to evaluate the pore pressure in the dam. If reservoir water is removed quickly, it may create excess negative pore pressure. The change in pore pressure is shown in Equation 9. The sum of the initial pore pressure and the excess pore pressure gives the final change in pore pressure at any point (Slide, 2010b).

$$\Delta u = \bar{B}\Delta\sigma_V$$ Equation 9

Where:

Δu= change in pore pressure (kPa);

$\bar{B}$= overall pore pressure coefficient for a material (unitless);

$\Delta\sigma_V$= vertical stress (kPa).

Material Properties of H-3

Embankment Fill

The original dam consisted of a central core and shell material. The core zone and shell were specified as plastic to non-plastic silt (ML) and composed of silty sand (SM) respectively. Golder took samples from the crest and downstream slope of the dam in February 2008. Based on these samples, the soil texture class of most of the embankment was found to be silty sand. Golder did not observe any distinction between the central core and shell material.

The core material had fine to medium sand texture and it was extended to 10.5 meters deep below the crest. The water content of core material was estimated between 9.6 and 34.5%. Approximately 45.7% of the materials passed a No. 200 sieve and was considered as fine materials. The dry unit weight of the embankment material was 16.76 kN/m^3 and average moisture content was found 16.2%. Table 1 shows the geotechnical parameters of the embankment fill in the H-3.

γ_{moist} (kN/m^3)	Total Shear Strength		Effective Shear Strength		K_h (m/sec)	K_v (m/sec)
	Φ (deg)	c (kN/m^2)	$\bar{\Phi}$ (deg)	$\bar{c}$ (kN/m^2)		
18.85	20	23.94	32	9.58	2.9×10^{-7}	1×10^{-7}

Table 1: Geotechnical parameters of the embankment fill

Where:

γ_{moist}= moist unit weight (kN/m^3)

K_h= horizontal hydraulic conductivity (m/sec)

K_v= vertical hydraulic conductivity (m/sec)

Alluvial Soils

A small amount of alluvial soil was found at the base of the dam on the downstream side. They were described as silty fine sand texture. The average water content of alluvial soils was 22.7%.

Residual Soils

Residual soils were found under the alluvial soil and at the base of the entire dam on top of the partially weathered rock (PWR). They were described as silty sand (SM) and well-graded sand (SW) textures. The average water content of the residual soil was estimated as 24% in H-3.

Partially Weathered Rock (PWR)

The PWR consisted of saprolite. It is defined as a weathered crystalline rock. The saprolite was described as fine to coarse sand texture with rock fragments. It was located at the top of the bedrock and under the embankment fill, alluvial and residual soils in the dam.

Bedrock

The bedrock at H-3 was typical of the Piedmont Physiographic Province. It consisted of moderately weathered to fresh, weak to strong granitic gneiss with moderate foliation.

Table 2 shows the geotechnical parameters of alluvial soils, residual soils, partially weathered rock (PWR) and bedrock.

Materials	γ_{moist} (kN/m^3)	Effective Shear Strength		K_h (m/sec)	K_v (m/sec)
		φ' (deg)	c' (kN/m^2)		
Alluvial soils	18.85	30	0	1×10^{-6}	4.9×10^{-5}
Residual soils	18.85	34	0	1×10^{-6}	1×10^{-7}
PWR	18.85	35	0	1×10^{-6}	1×10^{-7}
Bedrock	21.21	40	31.12	1×10^{-6}	1×10^{-7}

Table 2: Geotechnical parameters of the partially weathered rock, bedrock, alluvial and residual soil in the H-3 dam.

Geotechnical Analysis of H-3

Steady state seepage analysis (SSS) and slope stability analyses for original, improved and alternative dam designs are described in this section. Based on the Georgia Rules of Dam Safety minimum FS requirements, the design was considered safe or unsafe. Table 3 shows the State of Georgia dam safety requirements.

	Static Loading	Seismic Loading	Rapid Drawdown
The State of Georgia Dam Safety Requirements	1.5	1.1	1.3

Table 3: State of Georgia dam safety requirements

According to The Georgia Safe Dams Act and Rules for Dam Safety, the following calculations should be performed (Guidelines for Permitting a Category I Dam, 2015).

- Seepage analysis;
- Factor of safety of the downstream slope under steady state condition, considering normal and maximum pool level elevations in the reservoir;

- Upstream slope stability analysis under the condition of rapid drawdown;
- Downstream slope stability with seismic loading.

Water can seep through the dam or its foundation from the upstream side to the downstream side. Water movement has to be controlled and monitored. The main purpose of seepage analysis is predicting seepage behavior in the embankment and foundation (Guidelines for permitting a category I dam, 2015). Velocity and flow rate depend on the horizontal and vertical permeability of the materials and the reservoir water level. The phreatic surface is the top of the saturated zone in the dam (Swenty, 1989).

Original Dam Geotechnical Analyses

The original design height was approximately 8.5 meters and it had 3:1 horizontal: vertical slope at both the downstream and upstream side.

Steady state seepage (SSS) analysis was performed to predict seepage in the embankment and foundation. Figure 3 and Figure 4 show the results for normal and maximum pool water elevations, respectively, for original dam design.

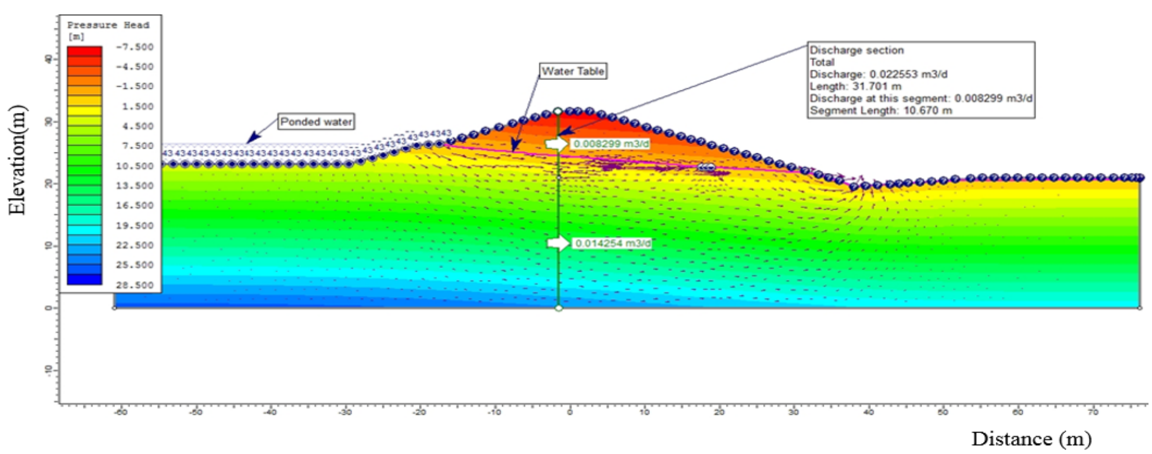

Figure 4: SSS Analysis of original dam at normal pool water level elevation

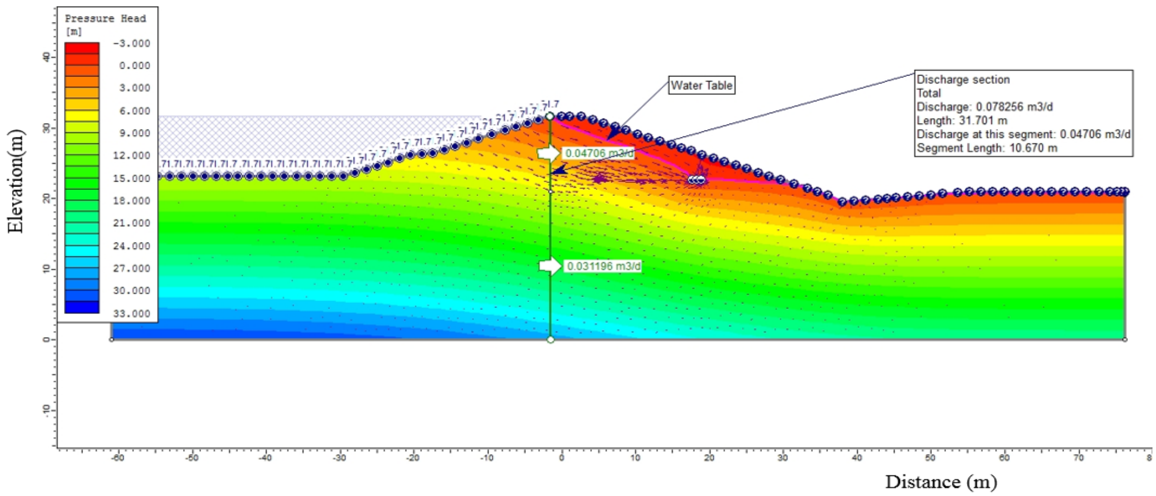

Figure 5: SSS Analysis of original dam at maximum pool water level elevation

The scale at the left side of Figure 3 and 4 represents the pressure head distribution over the dam. Arrows in the embankment and foundation show the water flow directions. The amount of seepage water through the dam and its foundation is shown over green discharge sections.

The water table is labeled. It was for the following slope stability analysis for normal and maximum pool water elevation. The limit equilibrium method of the Morgenstern-Price method (Equation 7 and 8) was used for slope stability analysis. Figures 5 and 6 represent the slope stability analysis of the original H-3 dam with normal and maximum pool level elevations.

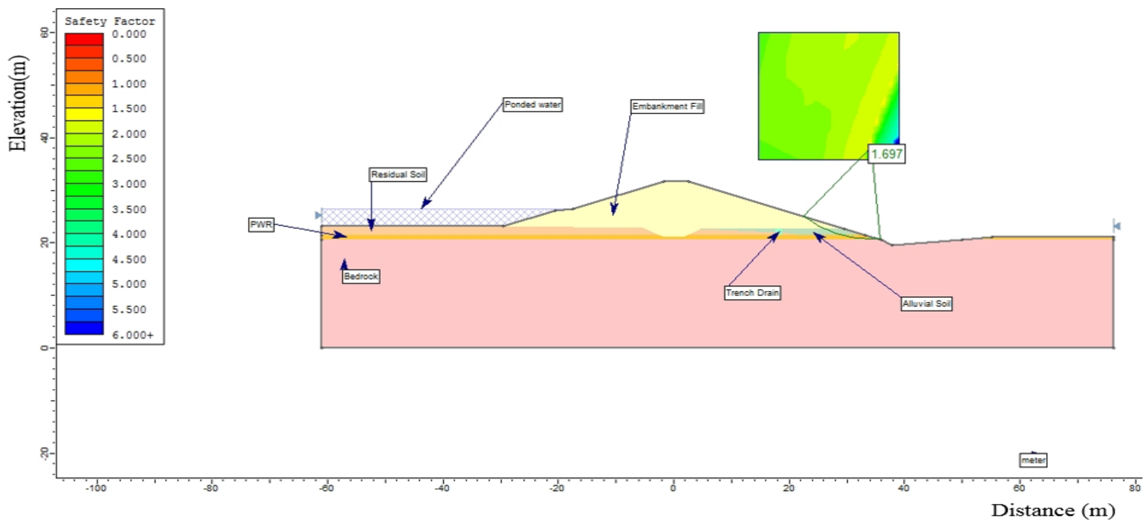

Figure 6: Slope stability analysis of original dam at normal pool water level elevation

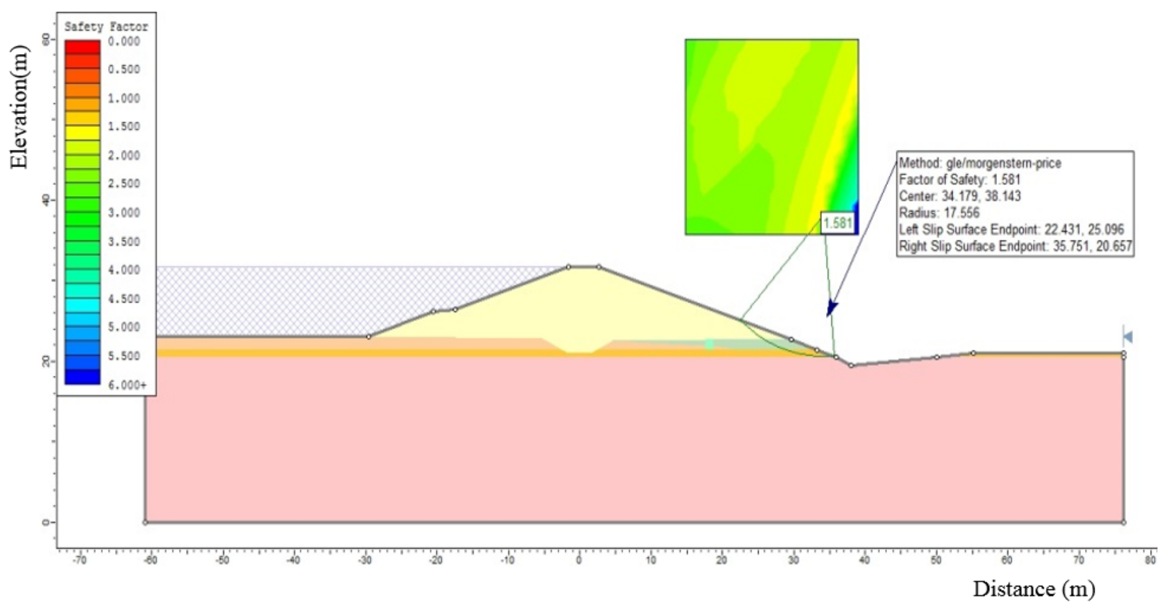

Figure 7: Slope stability analysis of original dam at maximum pool water level

Slip surface and slip centers can be seen in Figure 5 and Figure 6. The rectangular box on the right side contains the slip surface center points. Different colors in the box represent different factors of safety for the slip surface. The factor of safety scale is shown at the left side of Figures 5 and 6. The slip surface with the highest potential of failure has the minimum factor of safety and it is called the global minimum. Specific information about the global minimum is also shown in Figures 5 and 6.

As a result of these slope stability analyses, factors of safety of 1.697 and 1.581 were found with normal and maximum pool water level elevations, respectively. According to Georgia Rules for Dam Safety criteria in Table 6, the minimum factor of safety should be higher than 1.5 under static loading. Therefore, slope stability of original dam met the required criteria under static loading.

Corresponding to a peak ground acceleration with a 2% probability of exceedance in 50 years at the site, the seismic load was chosen as 0.11g (Golder Associates Inc 2009; Frankel et al., 2002). Slide software calculates the seismic force as shown below.

$$\text{Seismic Force} = [\text{Slide Weight}] \times [\text{Seismic Load Coefficient}] \qquad \text{Equation 10}$$

The seismic load was added to the system as a pseudo-static load. Figure 7 shows the steady state seepage analysis and slope stability analysis with the seismic load.

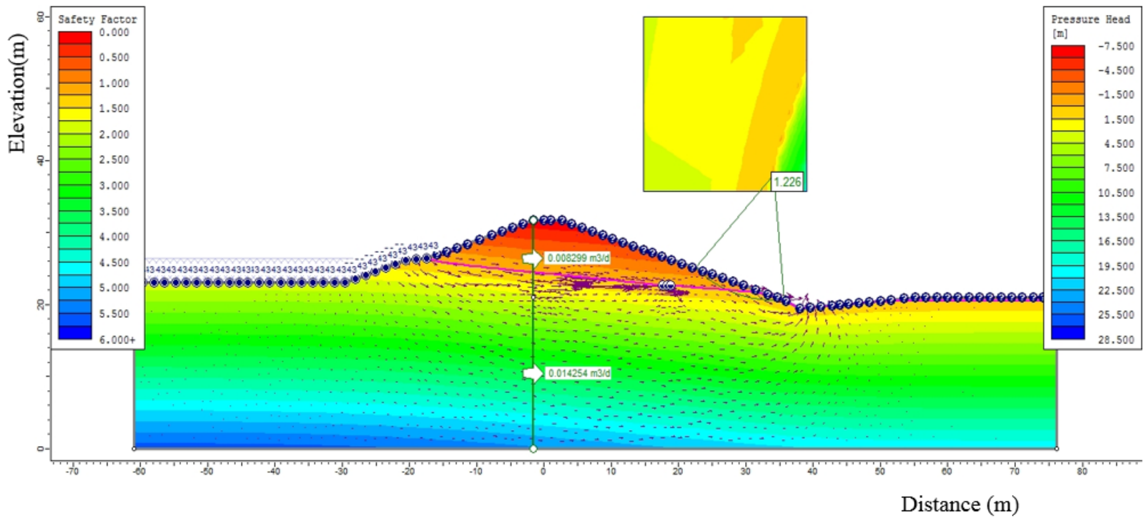

Figure 8: SSS and slope stability analysis of original dam with seismic loading

The result of this analysis showed a factor of safety of 1.226 for slope stability under the seismic loading. According to Georgia Rules of Dam Safety criteria, the factor of safety should be 1.1 or higher for such conditions. Therefore, the original dam was considered safe for seismic condition.

The B-bar method (Equation 10) in Slide is used to evaluate upstream slope stability for a rapid drawdown condition, as shown in in Figure 8.

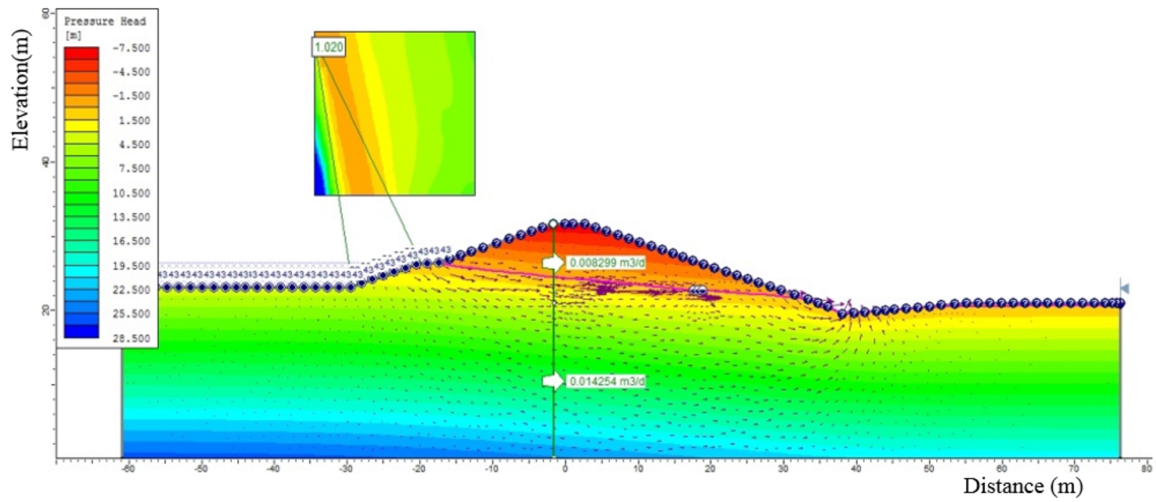

Figure 9: Slope stability analysis of original dam for rapid drawdown condition

The result of this analysis showed a factor of safety for rapid drawdown condition of 1.02, which is less than Georgia Rules of Dam Safety criteria of 1.3. Therefore, the original dam upstream slope was not safe under rapid drawdown conditions.

Improved Dam Geotechnical Analyses

The original H-3 dam did not satisfy all geotechnical requirements. To mitigate this deficiency, a renovation design specified removing approximately 3.4 vertical meters of embankment fill on the downstream face to be replaced with 1.5 meters roller compacted concrete, concrete sand and crushed stone. The final slope of the embankment profile surface was not changed.

Following the same geotechnical analysis procedures to evaluate the stability of the original dam, a new analysis was performed for the improved condition. This effort began with evaluating seepage through the dam and its foundation. Figure 9 and Figure 10 represents the SSS analysis for normal and maximum lake water level elevations.

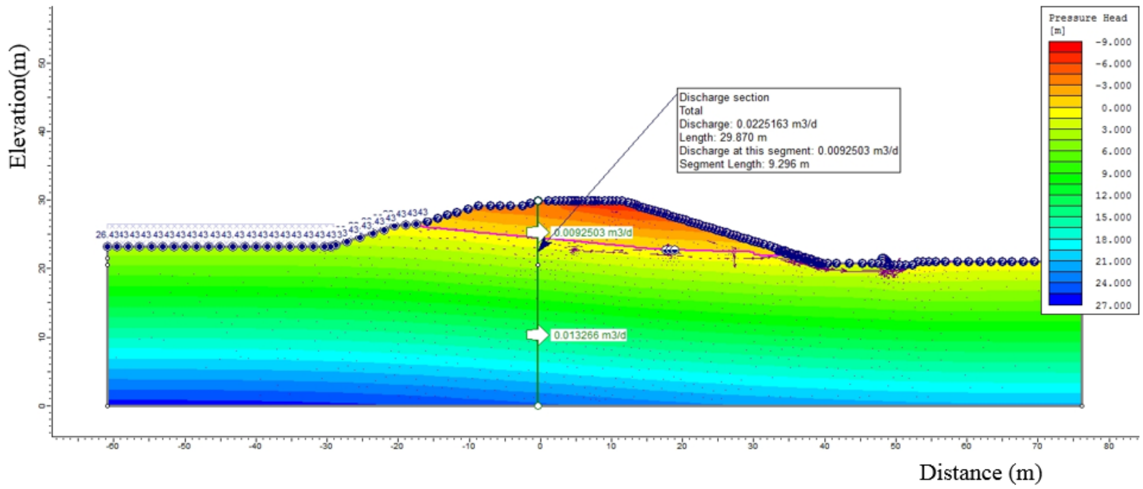

Figure 10: SSS analysis of improved dam at normal pool water level elevation

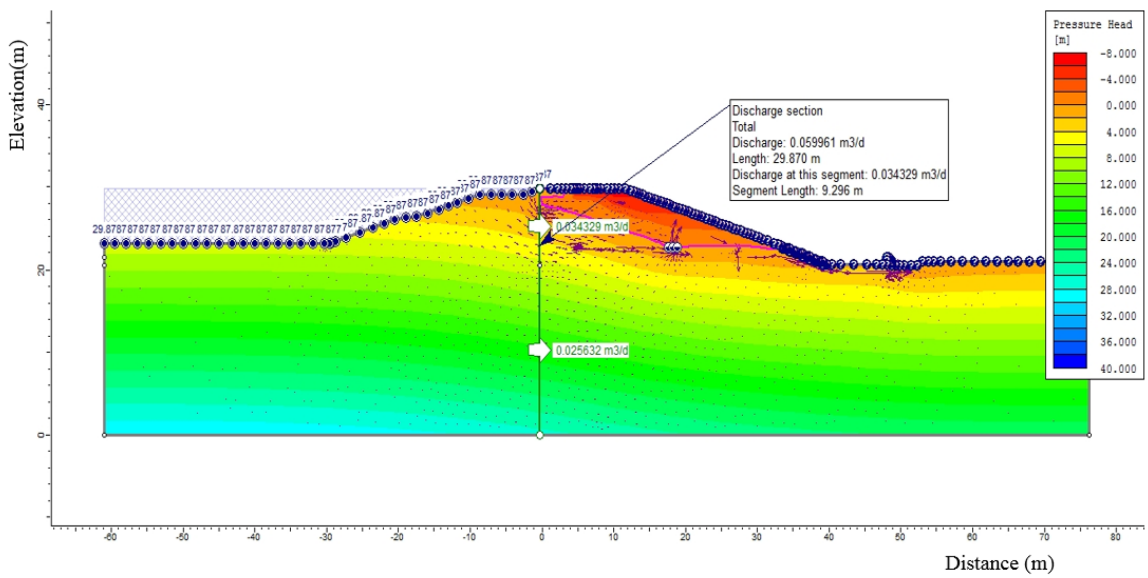

Figure 11: SSS analysis of improved dam at maximum pool water level elevation

Water tables were found for normal and maximum pool elevations. They were used to calculate slope stability for all geotechnical conditions. Figures 11 and 12 represent the slope stability analyses at the normal and maximum pool water elevations, respectively.

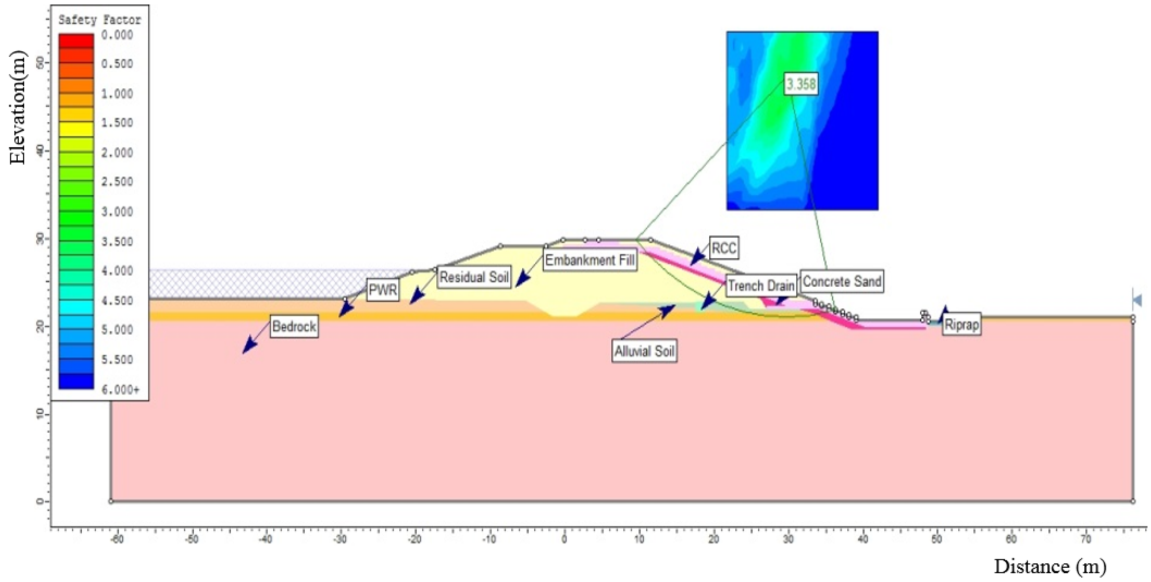

Figure 12: Slope stability analysis of improved dam at normal pool water elevation

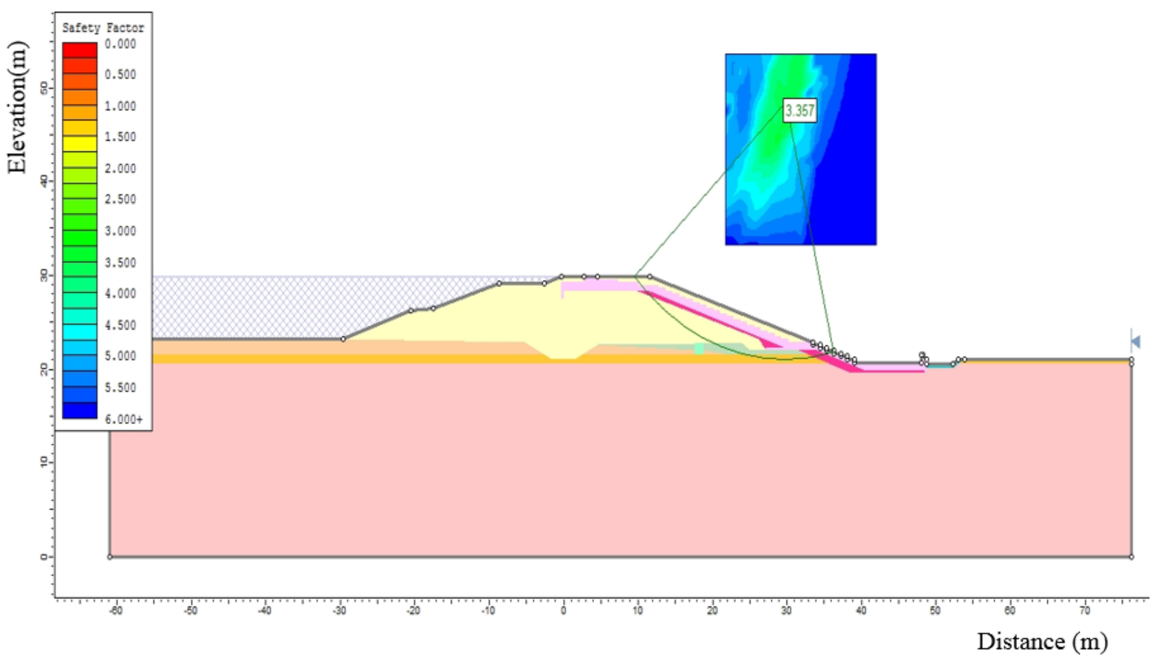

Figure 13: Slope stability analysis of improved dam at maximum pool water elevation

This revised analysis showed estimated factors of safety of 3.358 and 3.357 at normal and maximum pool water level elevations. According to Georgia the Rules for Dam Safety criteria, the factor of safety should be 1.5 or higher under static loading. Therefore, the slope stability of the improved dam met regulatory requirements under static loading.

Using the same seismic coefficient of 0.11g for the improved dam calculations, a seismic loading analysis was performed. Figure 13 represents the slope stability and seepage analysis of the improved dam with seismic loading.

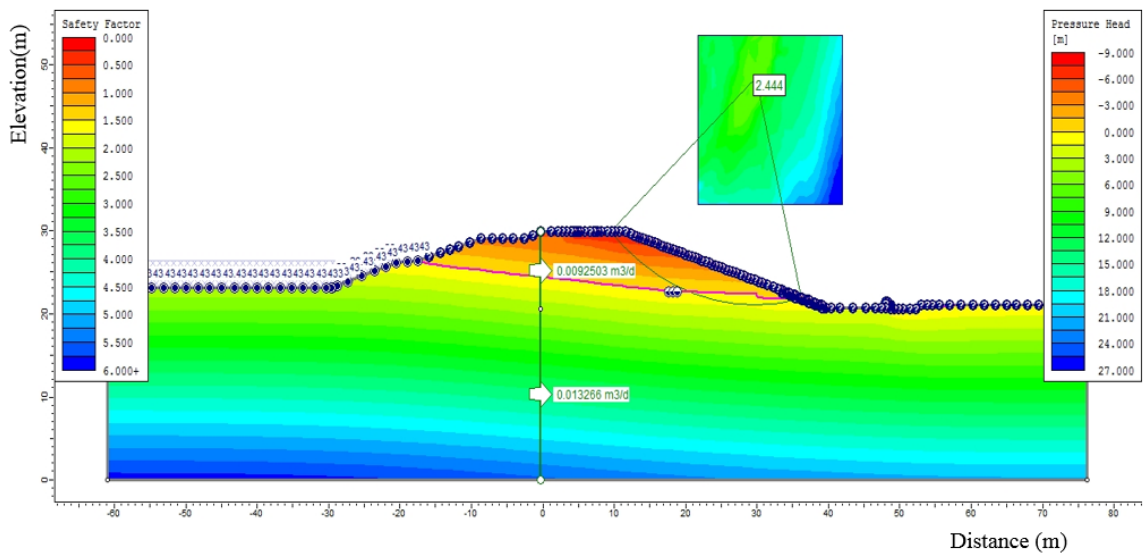

Figure 14: SSS and slope stability analysis of improved dam with seismic loading

Results showed a factor of safety of 2.444 for slope stability analysis under seismic loading. According to Georgia Rules of Dam Safety criteria, the factor of safety should be 1.1 or higher under this loading. Therefore, the improved dam met regulatory requirements for seismic condition.

Figure 14 shows the upstream slope stability for a rapid drawdown condition. According to Georgia Rules of Dam Safety criteria, the factor of safety should be 1.3 or higher for the upstream slope stability under the condition of rapid drawdown.

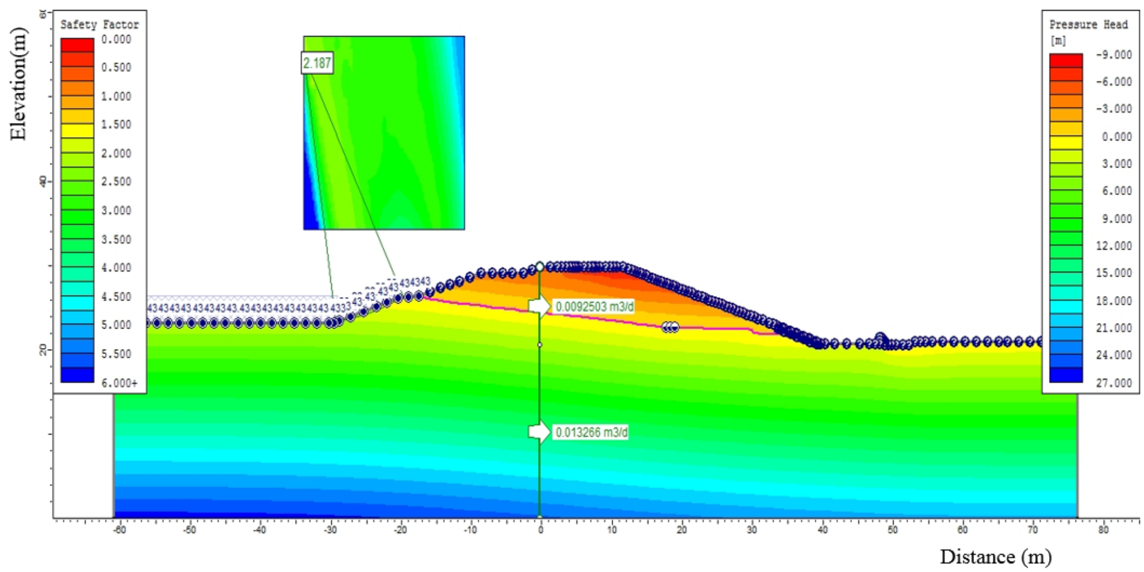

Figure 15: Slope stability analysis of improved dam for rapid drawdown condition

This revised analysis showed a factor of safety estimate of 2.187 for a rapid drawdown condition. It is higher than the minimum required value of 1.3. Therefore, the improved dam met regulatory requirements for the condition of a rapid drawdown.

The improved design decreased the height of the dam 1.53 meters and made overtopping more likely compared to the original dam design. However, increasing the stability of the dam by using RCC decreased the negative impact of overtopping. The RCC works like a spillway to discharge reservoir water when the water level elevation passes the lowest height of the dam. This design is known as a chute spillway (Rice and Kem, 1996). All results of the required geotechnical analyses showed that the implemented rehabilitation design improved the reliability of the original dam structure significantly to meet regulatory requirements, and the improved design also provided extra spillway capacity.

Alternative Proposed Dam Geotechnical Analyses

Geotechnical analyses of the original dam showed that only the upstream slope stability under a rapid drawdown condition was insufficient. To increase the factor of safety for the rapid drawdown condition to meet the regulatory requirements in a more economical way, an alternative dam design was considered. Decreasing the angle of the original dam upstream slope was chosen as an alternative rehabilitation approach for this dam. Figures 15 and 16 show the geometry and SSS analysis for the alternative dam design at the normal and maximum pool water level elevation, respectively.

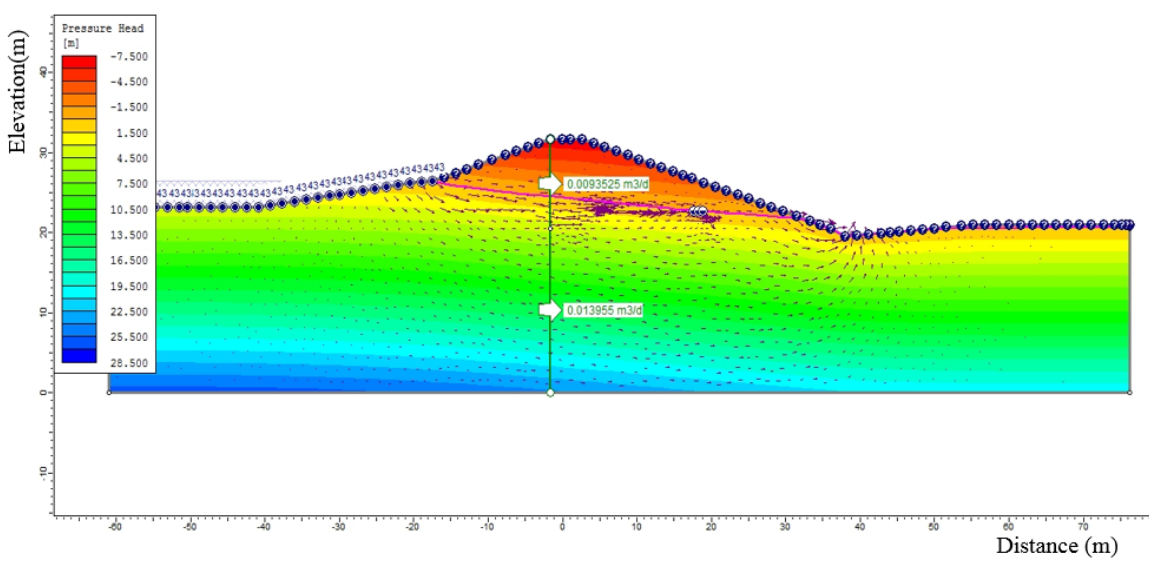

Figure 16: SSS analysis of alternative dam at normal pool water level elevation

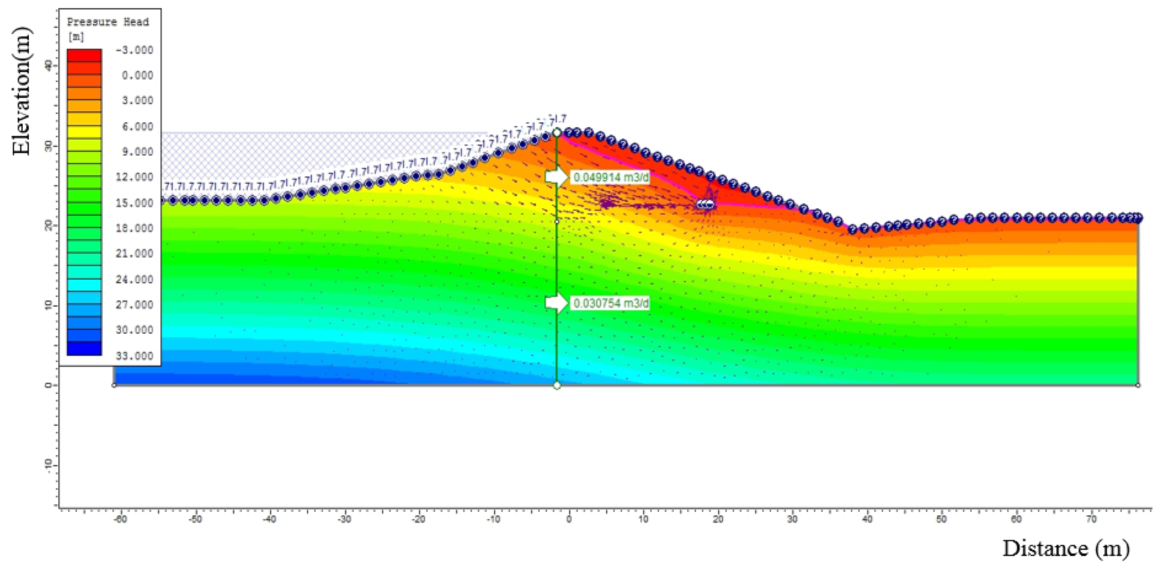

Figure 17: SSS analysis of alternative dam at maximum pool water level elevation

Comparing the seepage analysis of original and alternative dam designs, it was found that changing the upstream slope angle did not change the pressure head distribution, the amount of discharged water, or the water table.

The same procedures that applied for the original and improved dam designs to calculate slope stability was followed for alternative dam design. The calculated water tables from SSS analyses were used for slope stability analyses. Figures 16 and 17 show the slope stability analyses at normal and maximum pool water elevations.

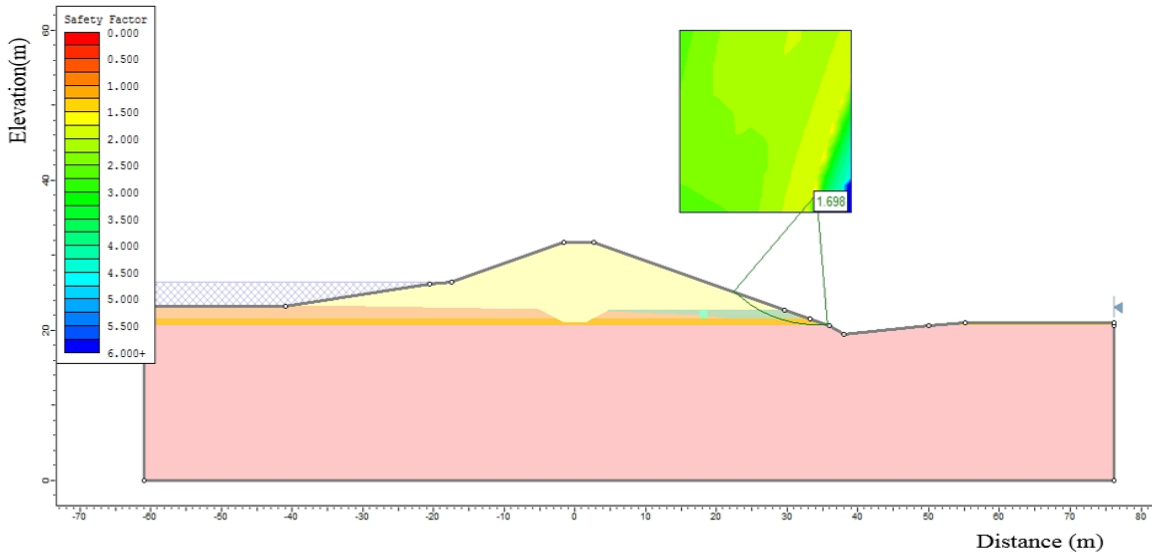

Figure 18: Slope stability analysis of alternative dam at normal pool water level elevation

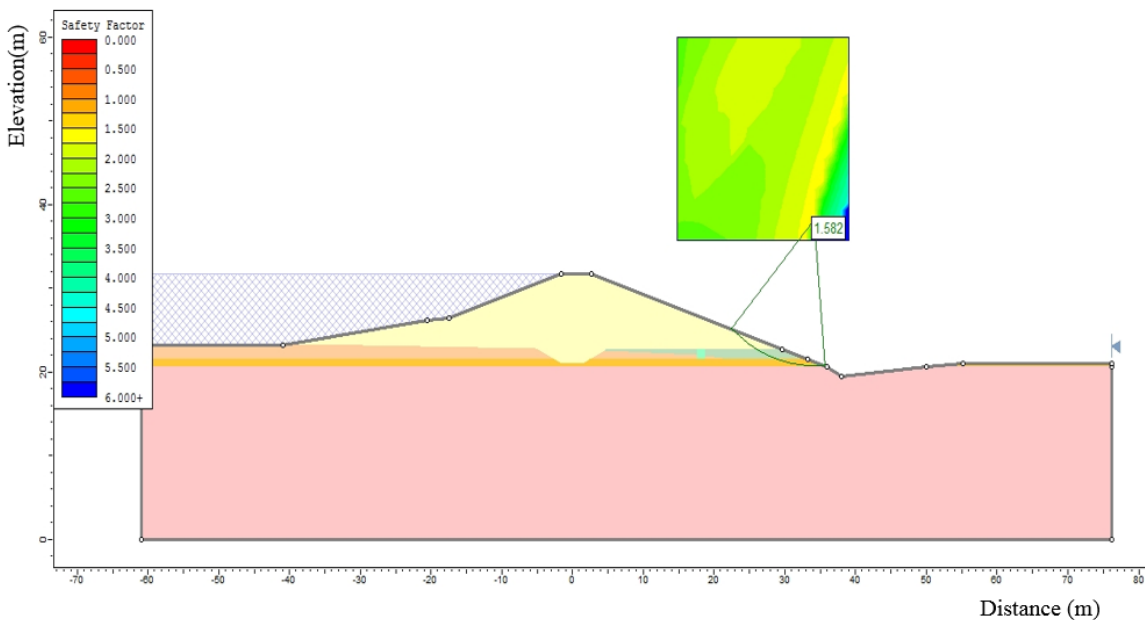

Figure 19: Slope stability analysis of alternative dam at maximum pool water level elevation

Comparing the alternative dam and original dam stability, the alternative design did not increase the slope stability of the original dam, but it still satisfied the Georgia Rules of Dam Safety requirements. Using a seismic coefficient of 0.11g for the alternative dam calculations, a seismic loading analysis was performed. Figure 18 shows the seismic analysis of the alternative dam.

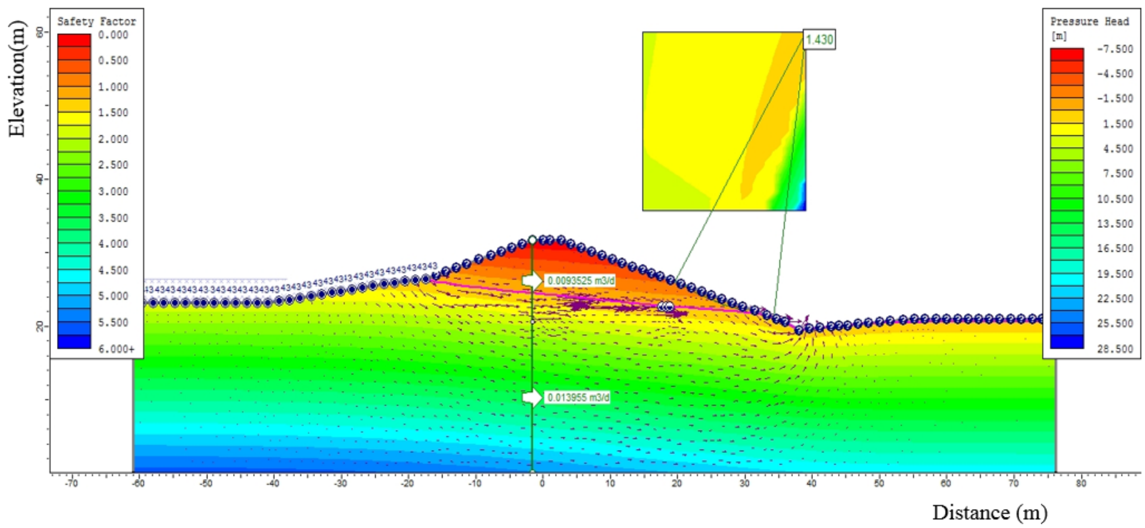

Figure 20: SSS and slope stability analysis of alternative dam with seismic loading

Results showed a factor of safety of 1.430 for the slope stability analysis under seismic loading, compared to 1.226 in the original dam. It can be seen that the alternative design also increased the dam stability under seismic loading. Therefore, this alternative design can also be used for rehabilitation of an embankment dam which does not meet the required factor of safety under seismic loading.

Finally, upstream slope stability of the alternative dam for the condition of rapid drawdown was analyzed as shown in Figure 20.

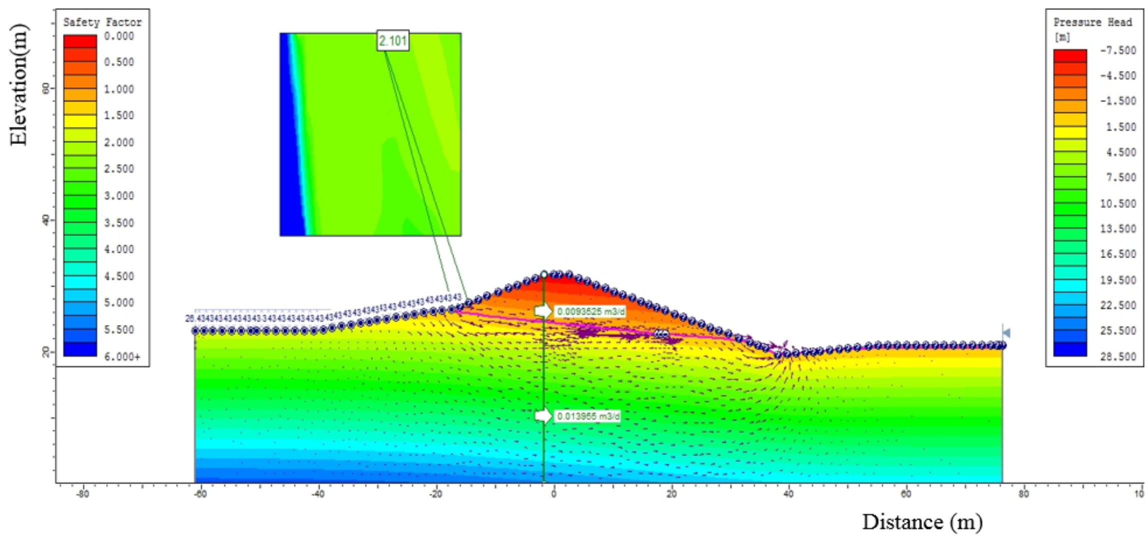

Figure 21: Slope stability analysis of alternative dam for rapid drawdown condition

The factor of safety was 1.02 for the original dam and was 2.101 for the alternative design as shown in Figure 20. It was higher than the required value of 1.3. The alternative design met all the geotechnical regulatory standards of the State of Georgia.

Economical Analysis of Improved and Alternative Dam Designs

The purpose of the alternative design was to propose a cost effective rehabilitation design for the original dam design to improve the factor of safety and meet the State of Georgia dam safety requirements. Geotechnical analyses showed that the alternative design improved the stability and met regulatory standards. This section shows the probable cost analyses of implementing the rehabilitation and alternative design, as shown in Tables 4 and 5 respectively.

				Opinion of Probable Cost	
Item No.	**Article or Service**	**Quantity**	**Unit**	**Unit Price**	**Amount**
8-001	Mobilization and Demobilization	1	LS	XXXX	$ 250,000
2-001	Clearing and Grubbing	3.0	AC	$ 7,500.00	$ 22,500
2-002	Mulch Grinding and Composting	1	LS	XXXX	$ 12,000
3-001	Structure Removal	1	LS	XXXX	$ 20,000
52K-001	Soil Erosion and Sediment Control	1	LS	XXXX	$ 100,000
52K-002	Temporary Sediment Barriers (Silt Fence, Type C)	3400	LF	$ 5.00	$ 17,000
52K-003	Temporary Sediment Barriers (Hay Bales)	400	LF	$ 3.50	$ 1,400
6-001	Permanent Vegetative Cover	3.0	AC	$ 4,000.00	$ 12,000
6-002	Temporary Vegetative Cover	3.0	AC	$ 2,000.00	$ 6,000
6-003	Erosion Control Matting	500	SY	$ 5.00	$ 2,500
6-004	Erosion Control Matting, Flexterra FGM	3.0	AC	$10,000.00	$ 30,000
6-005	Sodding	10,000	SF	$ 1.00	$ 10,000
6-006	Temporary Irrigation	1	LS	XXXX	$ 25,000
7-001	Construction Surveys	1	LS	XXXX	$ 50,000
7-002	Construction Quantity Verification	1	LS	XXXX	$ 30,000
9-001	Traffic Control	1	LS	XXXX	$ 30,000
9-002	Parking Lot Restoration	1	LS	XXXX	$ 70,000
11-001	Removal of Water	1	LS	XXXX	$ 100,000
21-001	Excavation, Common	14,500	CY	$ 7.00	$ 101,500
21-002	Excavation, Rock	400	CY	$ 150.00	$ 60,000
23-001	Earthfill, On-site Use	8,500	CY	$ 7.00	$ 59,500
23-002	Earthfill, Off-Site Disposal	6,400	CY	$ 8.00	$ 51,200
24-001	Drainfill, Fine	1,100	TON	$ 40.00	$ 44,000
24-002	Drainfill, Coarse	1,950	TON	$ 40.00	$ 78,000
24-003	Drainfill, Surge Stone	850	TON	$ 35.00	$ 29,750
24-004	Drainfill, Graded Aggregate Base	3,500	TON	$ 25.00	$ 87,500
26-001	Topsoiling	14,500	SY	$ 1.50	$ 21,750
26-002	Composted Mulch Additive	14,500	SY	$ 0.50	$ 7,250
31-001	Miscellaneous Concrete, Class 4000	400	CY	$ 500.00	$ 200,000
31-002	Concrete Retaining Wall, 0 to 5 ft.	5	LF	$ 450.00	$ 2,250
31-003	Concrete Retaining Wall, 5 to 10 ft.	40	LF	$ 500.00	$ 20,000
31-004	Concrete Retaining Wall, 10 to 12.5 ft.	85	LF	$ 525.00	$ 44,625
31-005	Concrete Retaining Wall, 12.5 to 15 ft.	25	LF	$ 550.00	$ 13,750
31-006	Concrete Retaining Wall, 15 to 17.5 ft.	20	LF	$ 600.00	$ 12,000
31-007	Concrete Retaining Wall, 17.5 to 20 ft.	5	LF	$ 700.00	$ 3,500
36-001	Roller Compacted Concrete, Mix, Convey, & Place	4,000	CY	$ 150.00	$ 600,000
45-001	Plastic Pipe, 6-inch Schedule 80 PVC, Slotted	350	LF	$ 40.00	$ 14,000
45-002	Plastic Pipe, 6-inch Schedule 80 PVC, Solid	200	LF	$ 30.00	$ 6,000
45-003	Plastic Pipe, 18-inch HDPE drainage pipe	260	LF	$ 20.00	$ 5,200
61-001	Rock Riprap, Type 1	300	TON	$ 65.00	$ 19,500
61-001	Rock Riprap, Type 3	150	TON	$ 45.00	$ 6,750
63-001	Treatment of Rock Surfaces	1,000	SY	$ 20.00	$ 20,000
81-001	Principal Spillway Riser Rehabilitation	1	LS	XXXX	$ 15,000
91-001	Fencing	1,500	LF	$ 10.00	$ 15,000
93-001	Identification Plaques or Markers	1	LS	XXXX	$ 2,500
95-001	Geotextile, Non-woven	3,500	SY	$ 3.00	$ 10,500
96-001	Field Office	1	LS	XXXX	$ 10,000
			TOTAL		**$2,349,425.00**

Table 4: Probable rehabilitation cost of implemented design

Item No.	Article or Service	Quantity	Unit	Opinion of Probable Cost Unit Price	 Amount
8-001	Mobilization and Demobilization	1	LS	XXXX	$ 250,000
2-001	Clearing and Grubbing	3.0	AC	$ 7,500.00	$ 22,500
2-002	Mulch Grinding and Composting	1	LS	XXXX	$ 12,000
3-001	Structure Removal	1	LS	XXXX	$ 20,000
52K-001	Soil Erosion and Sediment Control	1	LS	XXXX	$ 100,000
52K-002	Temporary Sediment Barriers (Silt Fence, Type C)	3400	LF	$ 5.00	$ 17,000
52K-003	Temporary Sediment Barriers (Hay Bales)	400	LF	$ 3.50	$ 1,400
6-001	Permanent Vegetative Cover	3.0	AC	$ 4,000.00	$ 12,000
6-002	Temporary Vegetative Cover	3.0	AC	$ 2,000.00	$ 6,000
6-003	Erosion Control Matting	500	SY	$ 5.00	$ 2,500
6-004	Erosion Control Matting, Flexterra FGM	3.0	AC	$ 10,000.00	$ 30,000
6-005	Sodding	10,000	SF	$ 1.00	$ 10,000
6-006	Temporary Irrigation	1	LS	XXXX	$ 25,000
7-001	Construction Surveys	1	LS	XXXX	$ 50,000
7-002	Construction Quantity Verification	1	LS	XXXX	$ 30,000
9-001	Traffic Control	1	LS	XXXX	$ 30,000
9-002	Parking Lot Restoration	1	LS	XXXX	$ 70,000
11-001	Removal of Water	1	LS	XXXX	$ 200,000
21-001	Excavation, Common	226	CY	$ 7.00	$ 1,582
23-001	Earthfill, On-site Use	1,400	CY	$ 7.00	$ 9,800
23-002	Earthfill, Off-Site Disposal	226	CY	$ 8.00	$ 1,808
91-001	Fencing	1,500	LF	$ 10.00	$ 15,000
93-001	Identification Plaques or Markers	1	LS	XXXX	$ 2,500
95-001	Geotextile, Non-woven	3,500	SY	$ 3.00	$ 10,500
96-001	Field Office	1	LS	XXXX	$ 10,000
			TOTAL		$939,590.00

Table 5: Probable rehabilitation cost of alternative design

Table 4 shows that, the probable cost for the improved design was $2,349,425. It was $939,590 for alternative design as shown in Table 5. Therefore, the alternative rehabilitation design satisfied the purpose of both improvement of the original dam stability to meet the State of Georgia dam safety requirements and cost effectiveness.

CHAPTER 4

CONCLUSION

The purpose of this study was to propose alternative cost-effective approaches to rehabilitate embankment dams. The H-3 dam was used as a case study. The slope stability analysis software, the Slide, was used to evaluate steady state seepage through the dam and its foundation for the original dam design. The water table in the dam was found using following slope stability evaluations. Slope stability analyses were performed under static loading, seismic loading, and for the condition of a rapid drawdown. The factor of safety was not sufficient for the rapid drawdown condition in the original dam design. Then, the same analyses were performed for the implemented improved dam design. It was seen that the improved dam satisfied all geotechnical requirements and stability was much higher than original dam under all conditions. Finally, alternative dam design with a lower angle of upstream slope was proposed and analyzed. These analyses showed that the dam met all of the State of Georgia geotechnical dam safety requirements. The probable cost analyses of the implemented and alternative designs were evaluated and the alternative designs were found cost effective.

Factor of safety values for the three different dam designs and different stability conditions and the State of Georgia regulatory dam safety requirements are summarized in Table 6. Table 7 shows the final probable cost of improved and alternative dam designs.

	Original Design	Improved Design	Alternative Design	Regulatory Standards
Normal Pool Level Elevation	1.697	3.358	1.698	1.5
Maximum Pool Level Elevation	1.581	3.357	1.582	1.5
Seismic Loading	1.226	2.444	1.43	1.1
Rapid Drawdown Condition	1.02	1.689	2.101	1.3

Table 6: Summary of safety design evaluation

	Improved Dam Design	Alternative Dam Design
Probable Rehabilitation Cost	$2,349,425	$939,590

Table 7: Summary of probable cost evaluation

CHAPTER 5

DISCUSSION

General

Cost effective rehabilitation for an embankment dam H-3 was studied in this work. The original dam did not meet Georgia dam safety requirements. Using limit equilibrium analysis, the original dam configuration was evaluated to confirm that it did not meet dam safety standards. Factor of safety of 1.697 and 1.581 with static loading for normal and maximum pool water elevations, respectively, were found. Considering the Georgia dam safety requirements, the original dam design met the dam safety standards for static loading. A factor of safety of 1.226 under seismic loading also met the regulatory standards of 1.1. The factor of safety for a rapid drawdown was found to be 1.02, which did not meet the regulatory standards of 1.1. Therefore, it was confirmed that the original dam did not meet all dam safety standards.

Then, evaluation of the implemented rehabilitation design was performed to determine if it improved the reliability of the dam structure and met regulatory standards. Factors of safety of 3.358 and 3.357 were found under static loading for normal and maximum pool water elevations, respectively. They both met the dam safety standards of 1.5 for static loading. The factor of safety was 2.444 under seismic loading and it also met the regulatory standards of 1.1. A factor of safety for the condition of a rapid drawdown was estimated as 2.187 which met the regulatory standards of 1.1. It was confirmed that the improved dam met all geotechnical dam safety standards. The implemented rehabilitation design increased the original dam stability significantly for all geotechnical conditions.

An alternative design was proposed as an additional design option to determine if other intervention designs would be as robust. The rehabilitation approach for the alternative design was to increase the factor of safety for the specific condition that the original dam did not meet Georgia dam safety requirements, instead of increasing the entire dam stability. According to the geotechnical analyses, the only factor of safety that did not meet the Georgia dam safety requirements was the upstream slope stability for a rapid drawdown condition. Considering the configuration of the original dam, it was decided that decreasing the original dam upstream slope angle with earthen fill could increase the upstream slope stability. Avoiding extra excavation cost and using relatively less expensive material than RCC decreased the construction cost.

Finally, slope stability analyses of the alternative design were evaluated. Factors of safety of 1.698 and 1.582 were estimated under static loading for normal and maximum pool water elevations, respectively. Compared to the original dam factor of safety values under static loading, the alternative design did not improve the reliability of the dam structure. The factor of safety was 1.43 under seismic loading and it met the regulatory standards of 1.1. Comparing the original dam and alternative design, the alternative design improved the factor of safety from 1.226 to 1.43 the factor of safety for the condition of a rapid drawdown was estimated as 2.101 which met the regulatory standards of 1.1. The alternative dam design met all Georgia dam safety requirements. Comparing the cost of implemented rehabilitation design and alternative design, the alternative dam design decreased the cost from \$2,349,425 to \$939,590. Therefore, the alternative design was found cost effective and safe as a rehabilitation design for an embankment dam.

<u>The Pros and Cons of Alternative Design</u>

- Alternative design is more cost effective than the improved design because it uses less expensive materials for rehabilitation work. Also it requires less excavation work.
- Considering the configuration of both dams, the improved dam design has a lower height than alternative dam. The possibility of overtopping for the improved dam is higher than the alternative design. Therefore, alternative dam design is safer for recreational purposes.
- Alternative dam design does not improve the stability for static loading, but improved dam stability for rapid drawdown significantly.
- Alternative design does not have any protection against piping. RCC in the improved dam design has very low permeability and it can prevent piping.
- Alternative design does not offer any additional spillway to prevent overtopping failure and it decreases the available reservoir capacity by using embankment fill on the upstream side. Implemented rehabilitation design with RCC works as chute emergency spillway. The low height of the dam makes overtopping flow likely but it decreases the risk of possible overtopping failure with a high factor of safety for downstream slope stability. Flowing water over a wide crest decreases the velocity of flowing water (Rice and Kem, 1996).
- Alternative dam design is typical earthen dam. The common failure of earthen dams is piping. Alternative dam design does not have any measure for piping.

- Stability of the dam mostly relies on RCC in the improved dam design. Any failure of RCC will be much faster and harder to be recognized compared to piping.
- Original H-3 dam was constructed in 1963. Considering a dam average life expectancy of 50 years (Imbrogno, 2014), it might be better to apply such rehabilitation that improve entire dam stability.

Recommendations for Rehabilitation of Similar Earthen Embankment Dams

The evaluation of an embankment dam using geotechnical safety analyses were performed and alternative safe and cost effective design based on Georgia dam safety requirements was proposed in this study. Requirements for different states and countries may vary. Therefore, dams should be designed or modified based on appropriate requirements.

It is important to emphasize that geotechnical analyses are not the only criteria for dam safety analyses in deciding final design. The results of hydrology, hydraulics and geotechnical analyses should all be taken into consideration for the final design.

In such case downstream slope stability is not sufficient according to related requirements; decreasing downstream slope angle, improving or adding filter material or modification with RCC can increase the slope stability.

In such case an embankment dam does not meet the requirements for seismic loading or rapid drawdown condition; the alternative dam design proposed in this paper can be applied to the dam to improve stability.

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