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STRESS AND DEFORMATION ANALYSIS OF GENALE DAWA III HYDROPOWER CONCRETE FACED ROCK FILL DAM, ETHIOPIA

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STRESS AND DEFORMATION ANALYSIS OF GENALE DAWA III HYDROPOWER CONCRETE FACED ROCK FILL DAM, ETHIOPIA

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Certification

This is to certify that the thesis prepared by Tegegn Filfilu, entitled: Stress and Deformation Analysis of Genale Dawa III hydropower Concrete Faced Rock fill dam, Ethiopia, submitted in partial fulfillment of the degree of Masters of Science Civil and Environmental Engineering (Major Dam Engineering) complies with the regulation of the university and meets the accepted standards with respect to originality and quality.

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ABSTRACT

The deformation of the dam is a serious problem for the faced rockfill dam, so it is very important to analyze the stress and deformation of the concrete faced rockfill dam. Genale DawaIII hydropower CFRD is the dam in Ethiopia in which the study is conducted which aims to analyze the maximum stress and deformation. The finite element software Midas GTS NX is used to establish the model to analyze the stress and deformation of the Genale Dawa III hydropower concrete face rockfill dam. The stress and deformation characteristics of the concrete faced rockfill dam at the normal water storage condition, designed flood level condition, and normal water storage condition during the earthquake are obtained through analysis. Settlements, horizontal displacements, and principal stresses are evaluated for the above conditions, and these results are compared with each other in detail. The result shows that the maximum horizontal displacement is 60cm at the crest and vertical displacement 103cm occurs in normal water pressure with earthquake conditions. The computed maximum tensile stress at the concrete facing was found to be 1.94MPa, which is greater than the tensile strength of concrete (1.67MPa). This indicates that the tensile capacity of concrete is insufficient in the linear-elastic range, and hence non-linear analysis is required to ensure that no collapse would occur during the dam's life span. Additionally, the computed maximum compressive stress is 42.12MPa, which is significantly greater than the concrete's compressive strength (25.3MPa), justifying the need for additional studies using a non-linear analysis approach. The maximum major and minor principal stresses are located near the perimetric joint or on the concrete facing at the heel of the dam.

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ACRONYMS AND ABBREVIATIONS

CFRD	Concrete face rockfill dam
ECRD	Earth core rockfill dam
GD3	Genale Dawa Dam III
USBR	United States Bureau of Reclamation
ICOLD	International Commission on Large Dams
USCOLD	United States Committee on Large Dams
USSD	United States Society on Dams
USACE	United States Army Corps of Engineers
EEP	Ethiopian Electric Power
MCE	Maximum Credible Earthquake
MDE	Maximum Design Earthquake
SEE	Safety Evaluation Earthquake
OBE	Operating Basis Earthquake
PSHA	Probabilistic Seismic Hazard Analysis
PGA	Peak Ground Acceleration
MPa	Mega Pascal
GPa	Giga Pascal
MW	Megawatt
PEER	Pacific Earthquake Engineering Research Center
PEER-NGA	Pacific Earthquake Engineering Research
NGA-West2	Next Generation Attenuation Relationships for Western US
PGA	Peak Ground Acceleration
PGHA	Peak Ground Horizontal Acceleration
GSI	Geological Strength Index
EARS	East African Rift System (EARS)
PMF	Probable Maximum Flood
FEM	Finite Element Method
FEA	Finite Element Analysis
2-D	Two-dimensional
3-D	Three-dimensional
U/S	Upstream
D/S	Downstream

SYMBOLS

g	Acceleration due to gravitational force (9.81m/s^2).
ag	Seismic ground acceleration
C	Cohesion
E	Modulus of Elasticity
ε	Strain
γ	Unit Weight
σ_1	Major Principal Stress
σ_3	Minor Principal Stress
σ	Normal stress
Φ	Angle of internal friction
ν	Poisson's ratio
τ	Shear stress
Sa	Spectral Acceleration

1. INTRODUCTION

1.1 Background

The first type of dam ever utilized in the world was an embankment dam, which was made of earthen materials. It is also among the most popular and advanced dam types in use today. Earth-core rockfill dams (ECRD) and concrete-faced rockfill dams (CFRD) are the two primary types of dams made of rockfill. Since the 18th century, dams have been built using a rock-fill method. A concrete face that is impermeable was constructed on the upstream end of the Concrete-Faced Rockfill Dam (CFRD), which has a rock fill as its body structure (M. Saberi et al 2013). The first one of this kind was built in California in 1856.

Due to the concrete-faced rockfill dam's high environmental application, straightforward construction, reasonably low cost, and good seismic performance, it has been employed extensively around the world (Runying Wang, Keping Yu 2021). The benefits include fewer settlement concerns due to the use of compacted rock fill; increased overall dam stability since water pressure acts on the upstream face; and, if properly designed and built, no pore water pressure develops in the rock fill zone. It is considered that well-compacted CFRD has good resistance to seismic loading based on several characteristics, including the satisfactory prior performance of comparable dams. Despite numerical analyses indicating that these dams are as safe as ordinary embankment dams, their behavior when subjected to strong seismic loading remains unknown [A. O. Sfriso 2008].

Experience has shown that the idea of constructing concrete-faced rockfill dams is both economical and safe. In comparison to concrete dams and rockfill dams with an impermeable core, CFRD is less expensive to construct. The fact that the rock fill may be removed from the site makes the CFRD effective in terms of economy [Cruz, et al., 2009]. Another benefit is that grouting for the CFRD can be carried out parallel with the building of the embankment, which can shorten the construction process. If there is not any suitable soil for the core nearby, CFRD may be advantageous to other embankment dams. A CFRD can have higher slopes than a dam with an earth core, which will require less fill and result in lower construction costs [Fell, et al.2005].

The primary issue with the safety of CFRDs is the deformation of the concrete face. Monitoring the deformation of the dams is crucial to ensure that the deformation of the concrete face dam does not exceed the critical limit. Unexpected or very massive deformations may be a sign that the dam or its foundation may have issues. Another reason to monitor dam deformations is the need for a better understanding of fundamental design principles, stress-deformation characteristics, and geotechnical properties of soil and rock fill. This reason is less urgent but could have significant long-term implications for the engineering profession [Anna Szostak-Chrzanowski, Michel Massiéra, Nianwu Deng]. It is crucial to examine the stress deformation of the concrete face rockfill dam since dam deformation is a significant issue for face rockfill dams [Runying Wang and Keping Yu 2021].

The focus of this study is on the post-construction stress and deformations of a CFRD caused by water impounding and creep of the rock fill material. This study aims to analyze the stress and deformation of the Genale Dawa III hydropower Concrete-Faced Rockfill dam using FEM.

1.2 Statement of Problem

The primary concern with CFRDs is the cracking of the face slabs, which results in leakage and further damage to the rockfill body and water loss. In the study of a CFRD, it is crucial to estimate the rock fill settlements and face slab deflections because these are directly related to stresses on the concrete facing. Shear and tensile movements are also caused at the parametric joint. With time, these movements continue. They might be sufficient to crack or open joints, causing leaks. It is critical to examine the stress-deformation of the concrete-faced rockfill dam since dam deformation is a significant issue for concrete-faced rockfill dams.

When a CFRD dam is being built, deformations begin to happen. These deformations are brought on by the creep of the material as well as an increase in effective stresses brought on by the development of successive layers of earth material. Deformations of the foundation, stress transfer between the different zones of the dam, and other elements may also have an impact on deformations. The pressure of the water during the initial filling of the reservoir might cause significant shifts in the dam's crest and body once construction is complete.

When un-loading or re-loading to stress levels less than those previously encountered (for example, due to reservoir level variations), the rock fill modulus is very high and the resulting deformation is limited. On initial wetting, collapse deformation can result in relatively large deformations. The concrete slab distorts due to the weight of the water and the deformations of the dam's rock fill. Because the settlements caused by the earthquake are significantly greater than those experienced under normal operation circumstances, the face slabs of CFRD are subject to damage under large seismic loads. The concrete slab serves as an impervious barrier, and any cracks that form would enable water to seep into the dam's rock fill, weakening or maybe destroying the structure.

1.3 Research Questions

- Which areas of the Genale Dawa III hydropower CFRD cross-section are most susceptible to stress and deformation under empty reservoir condition, normal operating conditions, peak water level condition and seismic loads?
- What is the magnitude of the total displacements at the plinth, perimeter joint, and face slab of the Genale Dawa III hydropower CFRD under a maximum Design earthquake (MDE)?
- To what extent does the Genale Dawa III hydropower CFRD deform elastically and plastically under shaking induced by an MDE?

1.4 General Objective

To determine the stress and deformation analyses of the Genale Dawa III hydropower Concrete Faced Rockfill dam using finite element analysis (FEA) and to ensure the safety of the project Concrete Faced Rockfill dams.

1.4.1 Specific Objectives

The research will be able to:

- Assess the value of the stress and deformation throughout the Dam cross-section using finite element analysis (FEA) for the safety of Concrete Faced Rockfill dams.
- Evaluate the deflection of the concrete slab during the underling of the rockfill body using FEA and assess its impact on the structural integrity of the dam.
- Estimate the settlement of the dam due to seismic waves of magnitude 7.0 and frequency 1 Hz using FEA for the safety of Concrete Faced Rockfill dams.

1.5 Scope of work

To analyze the stress and deformation of the Genale Dawa III hydropower concrete face rockfill dam at the post-construction stage using finite element analysis (FEA). The Geometry of the dam, Reservoir water level, Earthquake loading input parameters, and Stress distribution in the dam cross-section, Vertical and horizontal deformation of the dam, deflection of the concrete slab and stress of the concrete face and slab output parameters.

1.6 Organization of the Study

This study is structured as follows. Chapter 1 presents a brief introduction to the overall background information about Concrete face rockfill dams and the scope, objectives, research questions, and thesis outline of the study are presented. Chapter 2: presents detailed literature reviews conducted for the research focusing on the deformation behavior, stress distribution, engineering materials, and analysis method. Chapter 3: presents the methods and materials used in conducting this study. It describes the brief description of the study area, the materials and data collection, the development of structural analysis modeling, and the analysis methodology. Chapter 4: presents the analysis (inputs and outputs), the salient results, and discussions of this study. Chapter 5: summarizes the thesis conclusions and recommendations drawn from this study.

2. LITERATURE REVIEW

2.1 Concrete-Faced Rockfill Dams

Concrete-faced rockfill dams (CFRDs) is a type of dam that is constructed with a concrete face slab on the waterside and a rockfill embankment on the downstream side. CFRDS is one of the most common types of dams in the world, and they are used for a variety of applications, including water supply, hydropower generation, and flood control. Concrete-Faced Rock Fill Dam (CFRD) is a type of embankment dam that is built with compacted rockfill in layers or lifts and covered with concrete slabs at the upstream face to provide a watertight barrier. The body of the dam is often divided into zones identified by numbers and letters, depending on particle size, material type, and purpose [, Juan E. Quiroz and Mehdi Modares 2014].

2.2 Embankment Deformations

2.2.1 Causes of Deformations

Because of volumetric changes, lateral spreading, or shear displacements between the embankment and foundation components, embankment deformations take place under static stress. Volumetric changes happen because of either the dilatation of soil elements subjected to shear or an increase in normal stresses on a soil element, which results in a decrease in void volume. When the materials adapt to the stress conditions imposed by building the embankment and operating the reservoir, lateral spreading and shear displacements are caused by the squeezing, distorting, and localized shear failures of material parts. The dissipation of excess pore pressures and the emergence of steady-state seepage conditions both affect how quickly these deformations take place (DS-13(9)-17, 2011).

2.2.2 Effects of Deformations

The main effects of deformations include loss of freeboard, harm to ancillary structures within or upon the dam, a loss of confidence in the dam due to its appearance of swayback, cracking of the embankment (most harmful to the impervious core), the development of localized zones susceptible to hydraulic fracturing, and instrumentation failure.

The consequences of deformation are frequently avoided by designing features based on experien ce obtained from examining the historical performance of existing dams, without the need for co mplex analyses. Simple "rules of guidelines" and/or basic settlement calculations to calculate the amount of overbuild or camber to install on top of a dam and settlement estimates for appurtenant structures produce satisfactory results in the majority of circumstances. Detailed attention to embankment zoning and foundation contouring can reduce differential settlements, lowering the risk of core cracking or the establishment of hydraulic fracturing zones.

For any large or hazardous dam, the designer should expect that some core cracking is unavoidable, and filters and drains must be built into the design to limit seepage and prevent material displacement. When the "rule of thumb" and/or basic settlement calculation approach reveals excessive design requirements, the possibility for cost savings is the deciding factor for doing additional analysis.

2.3 Deformation Behavior of CFRDs

CFRDs can undergo a variety of deformations, including elastic deformation, plastic deformation, and creep. Elastic deformation is the temporary deformation that occurs when a load is applied to the dam. Plastic deformation is the permanent deformation that occurs when the load is removed. Creep is the slow deformation that occurs over time under sustained load.

Several factors, including the properties of the concrete and rockfill materials, the geometry of the dam, and the loading conditions, influence the deformation behavior of CFRDs. For example, CFRDS with stiffer concrete face slabs will experience less deformation than CFRDS with more flexible concrete face slabs.

During construction, impoundment, and reservoir fluctuations, changes in effective stress lead to deformations in embankment dams. Because of buoyancy effects, the vertical and horizontal effective strains reduce during the initial impoundment. Reduced effective stresses reduce the compressive strength of the saturated rock fill material, primarily on the upstream toe's outer surface. Wetting deformation, which happens when rockfill materials are not sufficiently compacted, can cause significant deformation because saturated rockfill loses its shear strength [Pooya Vahdati, 2014].

To achieve minimal distortion, the dam shell should be filled with sturdy, unweathered, well-graded rock and compacted in layers. Dam engineers are required to address two major geological and physicmechanical characteristics of rockfill materials: grading and compressibility. The higher the density of the inserted material, the better graded the material. Dams made of well-graded material have higher compressibility moduli, resulting in less rock smashing and settlement within the structure. The compressibility modulus describes the capacity of rock fill to contract in volume under external force because of grain breakage, rearrangement, and compaction. Strength, permeability, and water resistance are other crucial characteristics of rock fill material to take into account for design. Long-term displacements in rockfills that have been properly compacted are typically less than 0.2% of the embankment height, or between 100 and 300mm [Pooya Vahdati, 2014], whereas long-term displacements in fills that have been improperly compacted are between 1% to 1.6% of the dam height.

Construction methods, such as compaction parameters, thickness, and sprinkler water; material behavior, such as rockfill lithology, intact rockfill strength, rockfill size distribution, void ratio, fine grain content, and particle breakage; and load and boundary conditions, such as reservoir filling, water level fluctuations, rainfall, and earthquakes; are some of the factors that have an impact on CFRD deformation behaviors. Fell et al. found that the values of crest settlement and face slab deflection could differ by one or two orders of magnitude depending on the affecting factors. The other variables, such as foundation quality, rockfill strength, and operation time, are discrete, but the dam height and valley shape factor are continuous variables (Yanlong Li et al., 2021).

A CFRD's performance is assessed based on the concrete face's vertical and horizontal deflections, creep behavior, leakage rate, and other factors. The deformability of the supporting zone and rock fill directly affects the behavior of concrete face slabs.

2.3.1 Vertical Deformation of CFRDs

Settlements (vertical displacements) in rockfill dams are brought on by the foundation and fill being compacted by the reservoir pressure and the weight of the rock. The majority of the total settlement of rock fill happens during construction. The progressive realignment of the rock grain structure is what leads to settlements after building. Although there are few contact surfaces between individual rock grains, there exist substantial pressures. When the rock grains crush at the site of contact or slip relative to one another, a significant deformation may happen right away after construction [Pooya Vahdati, 2014].

The lowest third of the concrete face results in a bulging deformation that places tensile loads on the concrete slabs and the largest settlements are often seen at mid-height. The contours of the valley and the kind of rock used to build the embankment have an impact on the vertical deformation of a CFRD. Lower compressibility moduli lead to higher settlements, while CFRDs erected in broad valleys deform vertically more than those built in tighter valleys [Cruz et al. 2010].

For CFRDs built on hard rock, the differential movements at the plinth, perimeter joint, and face slab are brought on by the settlement of just the rock fill. The crest settlement and the varied movements at the plinth, perimeteric joint, and face slab of a CFRD built on compressible deposits are thought to be caused by the sum of the foundation's settlement and the rock fill’s settlement. Since alluvium often consists of a mixture of clays, silts, sands, and gravel, it may be predicted that the consolidation of the clays and silts will begin only after building, whereas the consolidation of the rock fill will begin quickly after construction. Therefore, a current CFRD on bedrock with known or "finished" deformations is placed on thick, weak alluvium in a finite element model to explore the behavior of a CFRD on weak alluvium. Afterward, it would be possible to evaluate the compressible alluvium's contribution to the overall projected deformations (post-construction). The measured settlements of the rock fill are considered to have previously taken the "valley shape factor" into account [M Kamper, and S Shinde, 2018].

Even under standard operating conditions, the researchers have seen creep deformation of rock fills in the body of high CFRDs. Rock fill and concrete faces may deform differently and inconsistently because of creep, which could lead to the separation of these two structural components. Without the support of rock fill, the weight and external loads on concrete-face panels may produce cracks, which negatively affects the performance and fatigue life of CFRDs [Xinjie et al 2020].

Maximum crest settlement is the largest accumulated settlement at the dam body's crest at the time of measurement, according to Yanlong Li et al. numerous measured data show that crest settling usually takes place in the middle of the dam's crest and gradually gets smaller as it gets closer to the dam's sides. Maximum crest settling is mostly made up of reservoir filling deformation and time-dependent deformation. The impoundment considerably enhances the maximum crest settlement and speeds up the crest settlement process. According to Gikas and Sakellariou's (2008) research, 60% of crest settlement in rockfill dams took place before the impoundment was fully constructed. In the first few years following the impoundment's completion, CFRDs' maximum crest settlement rate extremely decreased, according to Sherard and Cooke's research from 1987. Hunter (2003) asserts that after six years of impoundment, the crest settling of CFRDs tended to a final value. The statistical ranges for maximum crest settlement, maximum internal settlement, and maximum face slab deflection based on the references that are currently available are shown, with the majority of CFRDs having maximum crest settlement values in the 0.1% – 0.3% H range, where H stands for dam height. Additionally, the dam on the alluvium foundation has a maximum crest settlement that is typically 0.05%H higher than the dam on the rock foundation.

The maximum internal settlement of a dam after dam construction is represented by the parameter maximum internal settlement, according to Yanlong Li et al. Maximum In the dam section's middle, internal settlement is highly valued. During the construction phase, the typical CFRD's highest internal settlement value appears at half the height of the dam, but the maximum deformation caused by reservoir filling appears in the middle of the upstream face and gradually decreases in the downstream direction. During the construction phase, the maximum internal settlement value increases quickly, and the majority of the total internal settlement occurs before the water storage process is finished. Even if the settlement rate drastically decreases, the settlement grows over time following impoundment. The statistical results show that the greatest internal settlement value of most CFRDs is between 0.05 and 1.0%H under various situations. The dam on an alluvium foundation has a maximum internal settlement value that is, on average, 0.12%H higher than the dam on a rock foundation.

2.3.2 Horizontal Displacements of CFRDs

Horizontal displacements during construction are normally toward the upstream and downstream faces as expected for a conventional embankment construction, depending on how quickly the rock fill is being produced. The application of hydraulic pressure causes the embankment to deflect horizontally toward the downstream during impoundment; following the impoundment phase, the horizontal deformations also enter a condition of creep with slower deformation rates. By providing precise reference locations by sealing and attaching monitoring devices, monitoring horizontal displacements has the disadvantage of lowering measurement accuracy [Robin Fell et al., 2015].

2.3.3 Leakage through CFRDs

Leakage through CFRDs is crucial for efficiency, although it is frequently not as crucial for stability. Even stated leakage rates of up to thousands of liters per second did not result in instability. There are certain CFRDs where the rates increased in the years after the impoundment, even though leakage rates often peak during impoundment periods. Leakage rates for CFRDs typically range from 30 to 400 l/s, depending on the particulars of the project. In the literature, it was stated that rates more than 800-2000 l/s called for an investigation and corrective action. Solutions that are frequently selected include sand-mortar combinations, fine material placement, and strengthening the rock fill.

2.3.4 Face Slab Deflection of CFRDs

When the proper bond with the rock fill is ensured, how much a face slab of a CFRD deflects largely depends on the behavior of the rock fill. As is typical in dams, movements during the impoundment period open the perimeter joint. In addition, creep behavior is more important in the face slab than in the rock fill, especially if work is paused or completed in slow-moving phases [Robin Fell et al., 2015]. The flexibility of the rock fill in comparison to the stiffness of the concrete membrane directly affects how face slabs behave. The entire dam body deforms, followed by the deformation of the concrete slabs, causing enormous stresses in the concrete slabs. Additionally, the strain on the slabs during impoundment increases shear transfer between the concrete and the rock fill underneath it, adding to the strains [Mehdi Modares and Juan E. Quiroz, 2014].

The deformability of the rock fill and the supporting zone has a direct impact on how concrete face slabs behave. Mid-height is often where the largest settlements are observed, and the lowest third of the concrete face creates a bulging distortion that exerts tensile pressure on the slabs [Marquez and Pinto, 2005]. The rock fill can move towards the dam's center thanks to the valley's tridimensional effect, which may cause the rock fill to experience additional tensile strains and cause the slabs at the abutments to drag. The primary cause of the pulling force on the concrete slabs in both slope and horizontal directions is the deformation of the rock fill. The normal pressure on the slabs during impoundment causes an increase in friction resistance at the interface with the rock-fill body, causing the concrete membrane to bend and perhaps develop cracks.

Problems with concrete face rockfill dams recently have demonstrated the limitations of the current empirical practice. The main issue is the growth of fractures at the concrete face, which affects the structure's ability to keep moisture out. The severe distortion at the dam's upstream face is what causes these fissures to emerge. The mechanical behavior, or compressibility, of the rock fill material, is what causes this deformation [Xinjie Zhou, et al 2020].

Dam deformation primarily affects how the face slab deforms. Yanlong Li et al. 2021 define maximum face slab deflection as the largest face slab deflection at the time of measurement. A D-type distribution can be used to explain the face slab deflection during construction. The maximum value is based on the observed deflection deformation of numerous dam examples, and it appears in the upper and lower portions of the face slab or a B-type distribution. Due to the action of water pressure during reservoir filling, the face slab greatly deforms in the center. The face slab deflection quickly increased as the reservoir filled, as was found by Fitzpatrick et al. in

1985. The impoundment stage is when the majority of face slab deflections take place. The rate of face slab deflection substantially decreases during reservoir filling. The statistical ranges show that the highest face slab deflection value is typically less than 0.40%H, and in more than half of the cases, it is less than or equal to 0.2%H. The alluvium foundation's maximum face slab deflection value is roughly 0.08%H higher than the rock foundation's maximum face slab deflection value.

2.3.5 Concrete Slab Cracking

In rock-fill dams, the concrete face at the top plays the job of sealing, and any cracking or damage to this covering can cause water to flow into the reservoir, disrupting the dam's operation. Consequently, one of the most significant issues with concrete-face rockfill dams is concrete face cracking [Sediq V et al. 2018].

Under excessive deflection, the concrete face will crack, worsening the seepage from the dam. It has been reported that numerous CFRDs have had problems with cracks in the center of the face slab in the past [Cristian et al. 2011]. The main causes of face cracks are shrinkage, stress, temperature stress, and structural stress brought on by dam deformation, with structural stress being the most frequent. In a concrete-face rock fill dam, a substantial deformation can cause the face, which is a component of the anti-seepage body, to deflect, which can result in face cracking. When the cracks penetrate and link with one another, this causes serious leaking. When water seeps, tiny particles move with it, greatly endangering the stability and security of the dam [Binpeng Zhou, et al. 2021].

The two main categories of cracks that occur on the concrete face of rock-fill dams are macro cracks and micro-cracks with a random distribution. Micro cracks affect the compression and tensile strengths of concrete, while macro cracks cause anisotropy in the mechanical properties of concrete. Micro cracks are created during the cracking process, and these micro-cracks then grow and perforate to become macro cracks, which ultimately cause failure.Concrete-faced cracking incidences on some already-built dams have been useful for understanding the mechanics underlying CFRD in these cases.

Table 2-1. Precedent CFRDs with cracking [Ma and Cao 2007]

CFRD	**Issue**	**Cause**
Aguamilpa	Concrete facing cracking	Rock fill deformability
Tianshengqiao 1	Horizontal cracking	Construction sequence

Xingó	Slabs cracking	Sharp geometry of the left abutment and zone 3c material deformability
Itá	Slabs cracking	Rock fill deformability
Itapebi	Cracks parallel to the plinth	Foundation geometry
Bara Grande	Concrete facing cracking	Joint failures
Compos Novos	Concrete facing cracking	Rock fill deformability
Mohale	Compression joint rupture	Rock fill deformability

Compressive failures caused by concrete slab cracking that was seen on multiple CFRDs included reinforcement buckling, slab hiving, and significant concrete spalling. When the compressive demand is greater than the concrete slab's capability, this form of failure results. The design efforts are therefore concentrated on reducing the growth of these compressive stresses to reduce the likelihood of slab cracking. Settlements during impoundment are related to the formation of these compressive stresses and failures that have been seen [Juan E. et al. 2014].

2.3.6 Abnormal deformation

By comparing the shape, material types, placement techniques, and foundation condition to the typical deformation patterns in comparable embankment types, aberrant deformation behavior is identified. These differences are located using patterns in the direction and rate of deformation, patterns in the amplitude of deformations, and surface monitoring of the crest and shoulders. While there is no significant issue with the embankment's overall stability, there is only one location of the embankment where aberrant behavior is present. A large increase in settlement rate at the crest or upstream slope, plastic displacements within a short time after the first filling, and a shift in displacement direction are examples of potentially abnormal deformation patterns [Pooya Vahdati, 2014].

2.4 Deformation Behavior of Concrete Face Rockfill Dams under Earthquake Loading

Concrete Face Rockfill Dams (CFRDs) are an innovative type of dam structure that combines the advantages of both earth and concrete dams. These dams consist of a compacted rockfill with a concrete facing on the upstream side. The structural characteristics of CFRDs make them highly suitable for a variety of geological and topographical conditions. According to Modares and Quiroz (2016), CFRDs are designed to resist the forces generated by hydrostatic pressure, seismic events, and other external loads. The rockfill core provides stability and resistance against sliding, while the concrete facing enhances the impermeability and stability of the dam. The interaction between the rockfill and the concrete facing creates a composite structure that can withstand significant loads and deformations. The concrete facing also acts as a protective barrier against erosion and wave action. Moreover, CFRDs can be built using locally available materials, making them cost-effective and environmentally friendly. Overall, the structural characteristics of CFRDs make them an attractive option for dam construction, particularly in regions with challenging geological conditions. (Modares and Quiroz 2016)

Concrete Face Rockfill Dams (CFRDs) are commonly used structures for water storage and energy generation. However, their behavior during seismic events is of great concern due to the potential risks of dam failure and catastrophic consequences. A study conducted by Wen et al. (2018) aimed to understand the behavior of CFRDs under seismic loading. The authors conducted a series of numerical simulations and laboratory tests to investigate the dynamic response of CFRDs. The results revealed that during seismic events, the behavior of CFRDs is influenced by various factors, including the type of rockfill material, the degree of compaction, and the geometry of the dam. The study found that CFRDs with a higher degree of compaction exhibited better resistance to seismic loading, as they demonstrated less deformation and lower pore pressures. Additionally, the researchers observed that the presence of a concrete face on the dam significantly improved its seismic performance by providing additional stability and reducing the risk of internal erosion. These findings highlight the importance of considering factors such as compaction and concrete facing in the design and construction of CFRDs to ensure their stability during seismic events (Wen et al.).

Concrete Face Rockfill Dams (CFRDs) are a popular choice for constructing large-scale dams due to their cost-effectiveness and ability to withstand seismic loading. However, the deformation behavior of CFRDs under earthquake loading is influenced by various factors. According to Wen, Li, and Chai (2021), one key factor is the dynamic properties of the rockfill material. The stiffness and strength of the rockfill directly affect the overall stability and deformation characteristics of the dam during seismic events. Additionally, the presence of weak zones within the rockfill can lead to differential settlement and uneven deformation, further compromising the dam's structural integrity. Another significant factor is the interaction between the concrete face slab and the rockfill. The interface properties between these two materials, such as the bonding strength and frictional resistance, play a critical role in controlling the deformation behavior of the dam. A weak or deteriorated interface can result in sliding or separation between the face slab and the rockfill, leading to excessive deformation and potential failure. Furthermore, the seismicity of the region and the characteristics of the earthquake events, such as magnitude and frequency content, also influence the deformation behavior of CFRDs. Stronger and more frequent seismic events can subject the dam to higher levels of dynamic loading, increasing the likelihood of deformation and damage. Therefore, understanding and considering these factors are essential for the design and construction of CFRDs to ensure their stability and performance under earthquake loading (Wen, Li, & Chai).

2.4.1 CFRDs under Earthquake Ground Motion

CFRDs have previously been regarded to be fundamentally stable under seismic ground motion due to some distinguishing features. Sherard and Cook (1987) claim that since earthquake events do not result in the creation of additional pore water pressure in the CFRD embankment and since the shear strength of the compacted rock fill is high, these dams are inherently resistant to ground vibrations. However, some research indicates that settlements, slope displacement, and densification of the rock fill may happen because of earthquakes. Furthermore, the propensity of the concrete face to fracture due to the deformation of the rock fill under stress may promote flow in the downstream direction (M. Saberi, C. et al. 2013).

The performance of the concrete slab during earthquakes determines the seismic performance of a CFRD. The friction or welded contact at the concrete slab-rock fill interface, one of the most significant factors affecting dam behavior, is typically taken into consideration. Additionally, recent studies have looked into the reaction of rock fill in a CFRD and found that hydrodynamic pressure on the concrete face slab may result in major problems [Murat Emre Kartal 2017]. Uddin and Gazetas (1995) reported from their computational analysis of a 100 m CFRD subjected to strong seismic shaking (PGA = 0.60g) that the face slab is subjected to exceptionally high axial stresses but low bending moment and shear force. The majority of the slab exhibits tensile stresses that are much greater than the concrete's tensile strength. As a result, serious cracking of the concrete and breaking of the construction joints are unavoidable. Keep in mind that the plane-strain assumption holds for dams in very wide valleys, which are the subject of these results.

2.5 Deformation Analysis for CFRDs

Yanlong Li et al. 2021 claim that the construction, storage, and use of CFRDs all result in deformation. The two most frequent types of rockfill deformation are settlement and horizontal displacement. The majority of settlement deformations result from self-weight during the CFRD development stage since a dam is continuously filled. A dam may experience creep deformation because of rockfill fragmentation, rearrangement, stress release, adjustment, and transfer during reservoir filling. The nearby rockfill's deformation has a significant impact on how the face slab deforms. When a dam is subjected to the effect of a water load, it tends to settle and sag downstream. Maximum crest settlement, maximum internal settlement, and maximum face slab deflection are the three most significant deformation metrics that are frequently used to assess the deformation behaviors of CFRDs. The typical CFRD deformation patterns of our CFRDs are shown in Figure 2-4.

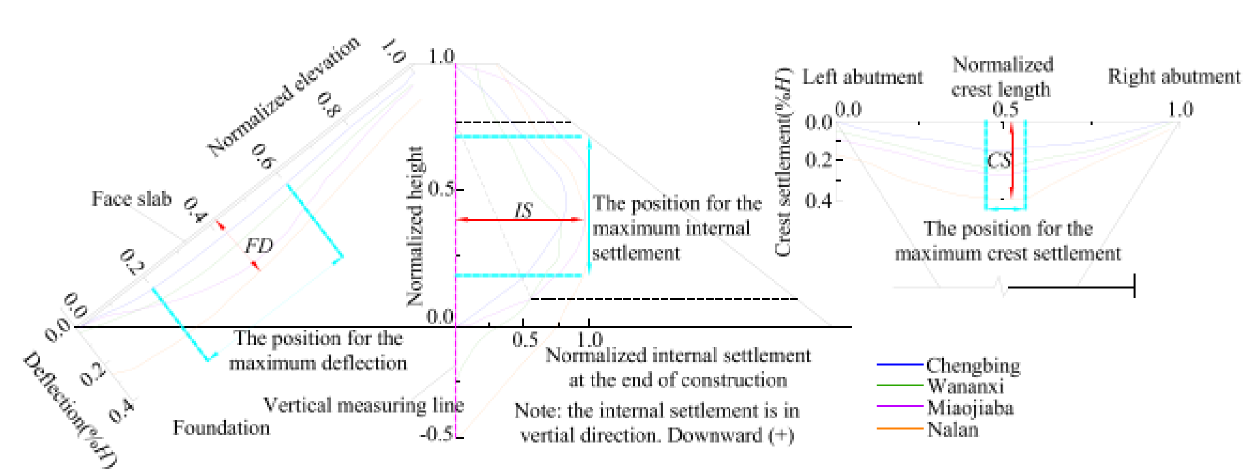

Figure 2-1 Typical CFRD deformation pattern of four CFRDs [Yanlong Li et al. 2021]

2.5.1 Construction stage

At the point where the rock fill body and the face slab meet, there is a cushion and transition layer, and the top of the cushion layer is coated with materials like asphalt that reduce friction. Due to the influence of water pressure at the reservoir water level, settlement, and deformation in the interior of the rock fill mass, the internal stress of the concrete slab will fluctuate throughout the advanced construction of the first-stage face slab and reservoir water storage.

The first-stage face slab will begin to warp owing to water pressure because the temporary construction seam level is below the first-stage water storage level and the face slab is now only supported by the dam body. The first-stage face slab will start to warp owing to water pressure and dam body settlement at the reservoir level because the temporary construction seam level is below the first-stage water storage level and the face slab is currently only supported by the dam body. As the face slab tilts higher over the water's surface, the cushion layer and face slab separate.

2.5.2 Impoundment stage

The researchers discovered that reservoir filling causes the most catastrophic damage, such as large horizontal and vertical movements in a dam. Such large movements occur for two main causes. The first is the dam's water loads, and the second is the softening and weakening of the fill material as a result of wetting. Because of the large variations in stresses at the wet areas of the dam, softening, and wetting are frequently considered in earth dams [Yeşim Sema Ünsever 2007].

Reservoir impoundment is one of the important factors to consider while evaluating dam behavior. Since the majority of post-construction deformations occur during this time. Rising water levels cause an increase in settlements. Significant settlements may cause cracks in the concrete slab, and leakage issues may result from cracks in the concrete slab. As a result of the normal pressure placed on the slabs during impoundment, there is more friction resistance at the concrete membrane's interface with the rock fill body, which might cause crack development.

Before concrete is poured, if the Filling stage face slab is unable to successfully prevent the internal stress and other negative impacts of the first-stage face slab, the internal stress of the first-stage face slab will be redistributed once more after the second-stage slab pouring and second-stage water storage, adversely affecting the first-stage face slab [Kongzhong Hu, et al 2018]. The primary problem during the impounding stage is the development of cracks in the concrete face. Three types of crack patterns can be distinguished based on the observations made in the situations previously stated.

i. Vertical compression cracks in the concrete face's top center region. Compression may eventually cause the slabs in the center to jolt. High compressive stresses cause these fissures.
ii. Horizontal cracks may develop in the lowest third of the concrete face,
iii. A few other diagonal cracks that are roughly orientated to the abutments' inclination have been seen close to them. The opening of the perimeter joint is another crucial issue.

Following is an explanation for the occurrence of vertical cracking: When water pressure initially affects concrete slabs, it is then transferred to the rock fill. Because of the weight placed over the rock fill, a settlement (i.e., vertical deformation) results. Because there are abutments, the vertical movement causes a subsequent horizontal movement. Since the concrete slabs and rock fill are in contact, the horizontal movement near the abutments causes shear forces at the rock fill-slab interface that cause the slabs to move horizontally. Finally, the identical dragging effect working at both abutments but in the opposite meaning causes large compressive stresses at the center of the concrete slabs. The slabs' bending, as seen in the Figure below, may have caused the horizontal tensile cracks. In any event, the waterproofing is lost once the concrete face fractures [Cristian Julian Nieto Gamboa 2011].

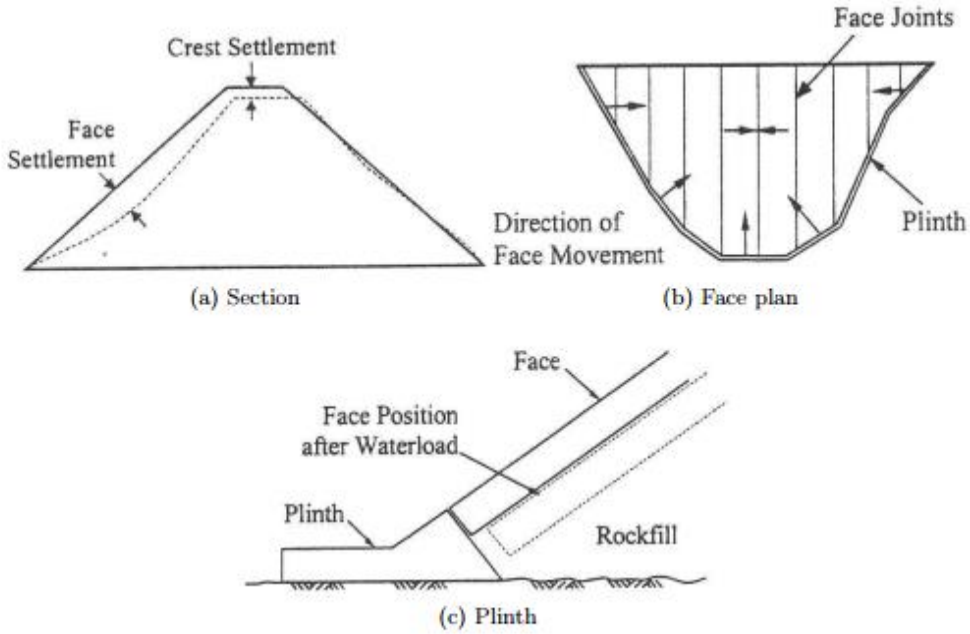

Figure 2-2. Schema of deformation of a CFRD in 2D under the action of water load

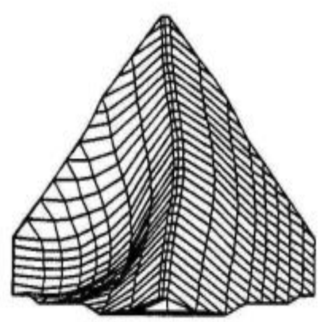

Figure 2-3. 3D schema of deformation of a CFRD due to impounding

2.5.3 Post construction stage

Concrete face rockfill dams (CFRDs) may distort after they are built for a variety of reasons, including gradual mechanical and hydro-chemical weathering of the rock fill material, internal erosion driven by seepage, earthquakes, and frequent fluctuations in the reservoir's water table. The latter causes the solid's hardness to deteriorate, which in turn causes grain fragmentation and rheological deformations. Different environmental factors can start the weathering process in a complex way, and it greatly affects long-term deformations. A change in moisture content for rock-fill materials that are sensitive to moisture might speed up the loss of solid hardness. Rheological deformations are essentially related to the degree of compaction, the geological conditions of the dam foundation, the seepage behavior, local sealing faults, and other design characteristics of the specific CFRD [Mohammadkeya et al. 2017].

2.6 Stress Distribution of Concrete Face Rockfill Dam

The stress distribution in CFRDS is complex and varies depending on the geometry of the dam, the reservoir water level, and the earthquake loading (if applicable). In general, the concrete face slab experiences the highest stresses, while the rockfill embankment experiences the lowest stresses.

The stress distribution in CFRDS can be determined using analytical methods or numerical methods. Analytical methods are relatively simple to use, but they can only be used for simple dam geometries and loading conditions. Numerical methods, such as finite element analysis (FEA), are more complex to use, but they can be used to analyze CFRDS with complex geometries and loading conditions.

According to Wen et al. (2018), the stress distribution in concrete face rockfill dams is influenced by various factors, including the dam's geometry, material properties, and loading conditions. The authors conducted numerical simulations using a three-dimensional finite element method to analyze the stress distribution patterns in a concrete face rockfill dam. Their findings revealed that the stress distribution in such dams is highly complex and can be categorized into three main zones: the face stress zone, the rockfill stress zone, and the core stress zone. In the face stress zone, the stress is primarily concentrated near the dam face, with the maximum stress occurring at the interface between the concrete face and the rockfill. In the rockfill stress zone, the stress gradually decreases with depth, reaching a minimum at the base of the dam. Finally, in the core stress zone, the stress distribution is relatively uniform, with a slight increase towards the base.

According to Hunter and Fell (2003), the geometry of the dam is one of the main variables that affect the distribution of stress. The dam's dimensions and shape can influence the quantity and distribution of stresses within the dam body. For instance, a dam with a steep face slope may encounter greater strains towards the crest because of the heavier fill material that lies on top of it. As the concrete face interacts with the underlying rockfill, the presence of a concrete face on the dam may also contribute extra strains. The characteristics of the materials used to build dams also affect how stress is distributed. The modulus of elasticity and Poisson's ratio of the concrete and rockfill can affect how stresses are transmitted through the dam, with stiffer materials generally leading to more uniform stress distribution. The placement and compaction of the rockfill material also play a role in stress distribution, as irregularities and voids in the fill can lead to localized stress concentrations. Finally, external factors such as water pressure and seismic loads can significantly influence stress distribution in concrete face rockfill dams. Overall, a comprehensive understanding of the various factors that influence stress distribution is essential for ensuring the safe and efficient design and operation of these dams (Hunter and Fell 2003).

Faces upstream and downstream are tensed and compressed, respectively. Because the values are higher than the tensile strength of concrete, it is feasible for transverse cracks to form in the face. The tensile strength of concrete may be exceeded by tensile strains on the upstream face. The possibility of an increase in compressive stresses and a reduction in compressive strength exists if the face experiences high compressive stresses because of static force action (M P Sainov 2020).

2.6.1 Principal Stress of CFRD

According to Hu et al. (2018), the principal stress analysis involves determining the maximum and minimum principal stresses within the dam structure. This analysis helps to understand the stress distribution and identify potential failure mechanisms. The principal stress in concrete face rockfill dams is influenced by various factors. According to Khalid, Singh, Nayak, and Jain (1990), one of the primary factors that affect the principal stress in such dams is the height of the dam. As the height of the dam increases, the stress on the dam structure also increases due to the additional weight of the rockfill material. This can lead to higher principal stresses in the dam, especially at the base where the load is the highest. Another factor that influences the principal stress in the dams is the angle of internal friction of the rockfill material. The angle of internal friction determines the resistance of the material to shearing forces and a higher angle of internal friction results in a higher resistance to deformation. Consequently, the principal stress in the dam will be lower if the rockfill material has a higher angle of internal friction. Additionally, the placement and compaction of the rockfill material can also affect the principal stress in concrete face rockfill dams. Proper placement and compaction techniques ensure that the material is uniformly distributed, reducing the potential for differential settlement and stress concentration. In conclusion, the principal stress in concrete face rockfill dams is influenced by factors such as the height of the dam, the angle of internal friction of the rockfill material, and the placement and compaction of the material (Khalid et al. 1990).

2.6.2 Shear Stress of CFRD

According to Hu, Chen, and Wang (2018), one of the critical factors influencing the stability of CFRDs is the shear stress exerted on the interface between the rockfill and the concrete face. Shear stress is the force per unit area that acts parallel to the interface, and it plays a significant role in determining the integrity and performance of the dam. The magnitude and distribution of shear stress can affect the potential for sliding, cracking, and deformation in the dam structure.

Factors affecting shear stress in concrete face rockfill dams can be categorized into two main groups: material-related factors and design-related factors. Material-related factors include the properties of the rockfill and the concrete facing. The rockfill properties, such as particle size distribution, shape, and angularity, significantly influence the shear stress behavior in the dam. The presence of fine particles in the rockfill can lead to increased shear stress due to increased interlocking between particles. On the other hand, the concrete-facing properties, such as strength and stiffness, play a crucial role in resisting shear stress. A stronger and stiffer facing can effectively transfer the shear stresses to the rockfill, reducing the potential for failure. Design-related factors include the dam geometry and the loading conditions. The geometry of the dam, such as the slope angle and the height-to-base width ratio, affects the distribution of shear stresses along the dam. A steeper slope angle or a higher height-to-base width ratio can result in higher shear stresses near the dam's base. Loading conditions, such as the water level and the seismic activity, also influence the shear stress behavior. An increase in water level can increase the hydrostatic pressure and, consequently, the shear stress. Similarly, seismic activity can induce dynamic loads that can significantly affect the shear stress distribution in the dam. In conclusion, a comprehensive understanding of the factors affecting shear stress in concrete face rockfill dams is crucial for the safe and efficient design and construction of these structures (Leps).

2.7 Interface – Concrete Face Rockfill dams

In the design and analysis of CFR dams, it is also essential to consider the interface between the concrete slab and the rockfill as well as the joints running vertically and horizontally between the concrete slab and the plinth's outer perimeter walls. In CFRD numerical research, precise modeling of these components is essential (M. Saberi1, et al. 2013). Numerical approaches can be used to estimate the deformation and stress distributions in the concrete face slab, where the behavior of the interface between the concrete face slab and the cushion layer is crucial. The prediction of displacement and stress distribution around the interface can differ depending on the treatment method used for the contact (Xiao-xiang Qian et al. 2013). In the past thirty years, the interface element, thin layer element, and contact analysis methods have all been suggested as three numerical approaches for simulating the displacement leap along the interface. The Goodman joint element method served as an inspiration for the interface element method. A zero-thickness interface constitutive model was presented as the main idea. This constitutive model may be rigid-plastic, elastic, or elastic-plastic.

A constitutive model of the behavior at the soil-structure interface has become a hot topic in numerical analysis. Ideal constitutive models, nonlinear elasticity models, and elastoplasticity and damage models are the three different forms of soil-structure interface constitutive models.

Ideal models include the rigid plasticity model and the elastic-ideal plasticity model, both of which could represent strength using the Mohr-Coulomb criterion. Regardless of the contact pressure stress, sliding is defined as the shear stress approaching an equivalent shear stress limit, max, established using the Mohr-Coulomb criterion. Numerous numerical studies, both static and dynamic, have used this model extensively (M. Saberi1, et al. 2013).

2.8 Analysis of Concrete Face Rockfill Dams

2.8.1 Static Analysis of Concrete Face Rockfill Dams

Static analysis is a crucial aspect in the design and evaluation of concrete face rockfill dams (CFRDs). The static analysis of CFRDs involves the assessment of various factors such as stability, stress distribution, seepage, and deformation.

One of the key considerations in the static analysis of CFRDs is the determination of the internal forces and stresses within the dam structure. This is typically done with numerical modeling techniques and finite element analysis, which allow engineers to simulate the behavior of the dam under various loading conditions. By analyzing the stress distribution, we can ensure that the dam structure can withstand the forces imposed by the reservoir water and external loads.

Furthermore, deformation analysis is an essential aspect of the static analysis of CFRDs. It involves assessing the dam's behavior under various loading conditions, including changes in water levels or seismic events. By evaluating the deformations, engineers can ensure that, the dam's structural integrity is maintained and that any potential issues, such as excessive settlements or deformations, are addressed. In conclusion, static analysis plays a fundamental role in the design and evaluation of concrete face rockfill dams. Through the assessment of stability, stress distribution, seepage, and deformation, engineers can ensure the safety and reliability of these crucial structures.

2.8.2 Dynamic Analysis of concrete face rockfill dams

A dynamic analysis of concrete face rockfill dams involves a comprehensive assessment of the structural behavior and response of these unique engineering phenomena under various dynamic loading conditions. A concrete face rockfill dam is a type of dam that combines the strength and stability of a concrete face with the flexibility and resilience of rockfill materials. This combination allows these dams to withstand extreme forces such as earthquake-induced ground motions, reservoir-induced seepage forces, and blast loads.

In a dynamic analysis, we carefully evaluate the performance of the dam under these dynamic loading conditions to ensure its structural integrity and safety. This analysis typically involves complex numerical simulations and computational models that take into account the material properties, geometry, and boundary conditions of the dam. By using advanced analysis techniques such as finite element analysis and dynamic response spectrum analysis, we can accurately predict the behavior of the dam and identify potential vulnerabilities.

One key aspect of dynamic analysis is the assessment of the dam's response to seismic forces. Earthquakes can exert significant dynamic loads on the dam structure, leading to ground shaking, soil liquefaction, and potential failure. Analyzing the seismic hazard at the dam site, considering parameters such as magnitude, distance, and frequency content of potential earthquakes, is used to determine the expected ground motion and its effect on the dam.

Another important consideration in dynamic analysis is the evaluation of reservoir-induced forces. These forces result from the water pressure exerted on the dam face due to the reservoir's fluctuating water level that examines the hydrodynamic effects of water pressure on the dam face, taking into account factors such as wave propagation, water level fluctuations, and damping effects.

Furthermore, the dynamic analysis also encompasses the assessment of blast loads on concrete-faced rockfill dams. These loads can occur due to various reasons, including deliberate detonation for construction purposes or accidental explosions. The analysis involves evaluating the impact of blast waves on the dam structure, considering parameters such as blast intensity, distance, and duration. Understanding the dynamic response of the dam to blast loads can be designed with appropriate measures to mitigate potential damage and ensure the safety of the structure.

2.8.2.1 Seismic Design Criteria

As well as the proper techniques for conducting dynamic evaluations for OBE and SEE ground motions, the seismic design criteria for dams with various risk classifications are presented. The pseudo-static method, which was previously utilized for all seismic evaluations but is now regarded as outmoded and unnecessary, will not be applied to huge dams situated in seismic zones. Modern seismic stress and deformation evaluations are performed using linear dynamic studies, and dynamic stability analyses of embankment dams are carried out using rigid body analyses, such as the Newmark sliding block approach. For example, thin impermeable membranes on dams need a higher level of accuracy in deformation analysis because they might be subject to dam deformations [Martin Wieland, 2019]

The inertia effects of the dam body, which are equal to mass times ground acceleration, are the seismic activities taken into account in the stress and sliding stability studies. The hydrodynamic pressure acting on the vertical upstream face of a dam is derived from the analytical solution of the pressure acting on a rigid vertical wall of a two-dimensional semi-infinite reservoir of constant depth subjected to harmonic horizontal ground acceleration. The dam body's inertia effects, which are equal to mass times ground acceleration, are also included in the stress and sliding stability analyses. The static study of a dam, which disregarded dynamic factors like Eigen frequencies, mode shapes, and damping, was made possible by the seismic analysis's assumption that these two horizontal earthquake stresses were independent of time. This method is hence called pseudo-static analysis. Nearly irrelevant to the seismic risk at a dam site, which was typically unknown because dams are generally situated in remote places, it was standard practice to apply a seismic coefficient of 0.1, equivalent to a horizontal ground acceleration of 0.1 g [Martin Wieland, 2019]. The analytically determined hydrodynamic pressure acting on a stiff vertical wall of a two-dimensional semi-infinite reservoir of constant depth subjected to harmonic horizontal ground acceleration. This pressure is then applied to the vertical upstream face of a dam. The static study of a dam, which disregarded dynamic factors like Eigen frequencies, mode shapes, and damping, was made possible by the seismic analysis's assumption that these two horizontal earthquake stresses were independent of time. This method is hence called pseudo-static analysis. Using a seismic coefficient of 0.1, which corresponds to a horizontal ground acceleration of 0.1 g, was standard procedure almost independent of the seismic risk at a dam site, which is typically unknown because dams are commonly built in remote areas [Martin Wieland, 2019].

As a result, the design and safety assessment of substantial existing dams must take into account the following earthquake levels:

Operating Basis Earthquake (OBE)

Throughout the dam's lifespan, an OBE is conceivable. There must be no loss of service or harm. Within the service life of 100 years, there is a 50% chance that it will take place. According to ICOLD's 2016 estimate, the return time will last 145 years. The OBE ground motion parameters are calculated using a probabilistic seismic hazard analysis (PSHA). It is possible to calculate the average values of the ground motion properties of the OBE.

Safety Assessment Seismic Event (SEE)

The SEE is the amount of earthquake ground motion that a dam must be able to withstand before a reservoir starts to overflow uncontrollably. The SEE serves as the governing earthquake ground motion for the safety assessment and seismic design of the dam, as well as for the safety-relevant components (gates and valves of spillways and bottom outlets, motors, emergency power supply, hydraulic pistons, etc.) that must operate after the SEE to control the reservoir's water level.

Maximum Design Earthquake (MDE)

The maximum ground motion for which a structure is constructed or assessed is known as the MDE. The project must meet the linked performance criteria without suffering fatalities or catastrophic failures like an uncontrolled reservoir discharge, albeit serious damage or financial loss may be permitted. The MDE and MCE share the same important properties. For all other attributes, the minimum MDE is determined using a PSHA that is guided by the findings of a site-specific deterministic seismic hazard assessment (DSHA) and is an event with a 10% probability of exceeding 100 years (average return period of 950 years).

2.9 Engineering Properties of Rockfill Material

The two main engineering materials used in CFRDs are concrete and rockfill. The concrete face slab is typically made with high-strength concrete that is resistant to erosion and cracking. The rockfill embankment is typically made with a well-graded mixture of gravel, sand, and cobbles.

The properties of the concrete and rockfill materials have a significant impact on the stress and deformation behavior of CFRDS. For example, CFRDs with more stiff concrete face slabs will experience less deformation than CFRDS with less stiff concrete face slabs.

CFRD construction materials include sedimentary rock, igneous rock, and metamorphic rock. Density, specific gravity, void ratio, compression strength, tensile strength, softening coefficient, and other indicators of the original rock of rock fill material are important. Rock fill material is classed as hard, moderate, or soft based on the saturated unconfined compression strength of the original rock. Hard rock fill material has a saturated uniaxial compressive strength of original rock greater than 80 MPa, medium strength rockfill material has strength between 30-80 MPa, and soft rock fill material has strength less than 30 MPa. The main engineering qualities of rockfill materials include stress and deformation, gradation, compressibility, and strength.

Blasting is the most common method of extracting rock-fill material. As a result, the gradation of rock fill material is mostly determined by the blasting method, the structure of the rock mass, and the formation of fissures and joints within the rock mass. The gradation curve of rockfill material usually presents continuous distribution. It is considered satisfactory gradation when the uniformity coefficient (Cu) of the rock fill grain is greater than 15 [Paulo T, Cruz, et al].

Another feature of rock fill material gradation is its variability. The most significant influence on t he variation of rock fill gradation is breaking during compaction, which is primarily determined b y rock strength and compaction energy. The gradation of rock fill material will alter in direct prop ortion to the engineering qualities of rock fill.

Rockfill particles are polyhedral in form. The general compressibility of rock fill material is prima rily determined by particle rearrangement, although it is also influenced by rock density, gradatio n, and other factors. Compacted rock fill typically has a high density and a low vacancy ratio. It h as a rather modest compressibility. The high stress level in rockfill for high CFRDs can produce secondary breakage of rockfill particles and the rearrangement of the fragmented particles can cause creep deformation.

When rock fill is wet, it deforms further due to the softening and breaking of the particle edge as well as the shifting and arranging of rock fill particles produced by the lubricating effect of water. The wetting deformation is directly connected to the rock characteristics. Deformation would be lessened if the density of the rock fill was high or the original water content was high. The compressibility of rock fill material is proportional to its grade. The compressibility of the same rock with different gradations will be completely different. When density increases, the compression modulus of a rock fill increases dramatically.

A granular material composed of hard particles is known as "rockfill." The effects of sliding friction and particle intercalation are included in the shear strength of rock fill. Both the effects of particle breaking and the effect of shearing dilation influence particle interlocking. Both linear and non-linear equations are frequently used to express the shear strength of rockfill materials.

2.9.1 Modulus of Elasticity

When analyzing the mechanical behavior of the rockfill materials used in concrete face rockfill dams, the modulus of elasticity is a critical factor. Because it shows how well a material can endure deformation under stress, it is very significant. The ratio of stress to strain within the elastic range is the definition of the modulus of elasticity given by Xu, Zou, and Liu (2012). The measurement of this parameter is essential in assessing the stability and performance of rockfill materials in dam construction. Various techniques have been employed to determine the modulus of elasticity in rockfill materials. These techniques include laboratory tests, such as uniaxial compression tests and triaxial compression tests, as well as field measurements using instrumentation. The laboratory tests provide valuable insights into the mechanical properties of rockfill materials, while field measurements offer a more realistic representation of the material's behavior under actual dam conditions. According to Marschi, Chan, and Seed (1972), the modulus of elasticity is particularly important in evaluating the settlement behavior of rockfill structures, such as dams and embankments. Additionally, the modulus of elasticity is also used to determine the stress-strain behavior of rockfill materials under different loading conditions. This information is essential for accurately estimating the deformation and stability of rockfill structures. Therefore, a thorough understanding of the modulus of elasticity is crucial for engineers working with rockfill materials, as it directly influences the performance and safety of these structures (Marschi et al., 1972). Depending on the rock, the grade of the rockfill, the layer thickness, and other variables, the vertical moduli measured in concrete face rockfill dams have ranged from 30 to 130 MPa (USSD 2011).

2.9.2 Poison's Ratio

For assessing the mechanical behavior of rockfill materials used in dam construction, the Poison's ratio is an essential criterion. An essential factor that describes the deformation behavior of materials is the Poisson's ratio, which is the ratio of lateral strain to axial strain. The poison's ratio, described by Honkanadavar and Sharma (2016) as the ratio of lateral strain to axial strain, sheds light on the material's capacity to deform under various loading scenarios. Understanding the significance of the Poison ratio is essential in designing and constructing safe and stable dams. For instance, a high poison ratio indicates that the material is highly compressible and exhibits significant lateral expansion when subjected to axial loads. On the other hand, a low poison ratio suggests that the material is less compressible and exhibits limited lateral deformation. By considering the poison's ratio, engineers can assess the potential for deformations and instability in the rockfill material, helping to ensure the durable safety and performance of the dam structure.

2.10 Numerical Analysis of CFRD

The design of a CFRD is gradually shifting away from depending entirely on an engineer's judgment and towards being guided by theoretical studies and lab field testing research. Earlier CFRD analysis was mostly analyses that used linear elastic models. Non-linear analysis has gained popularity in latest years, with the finite element method (FEM) or the finite difference approach serving as the primary analytical method [Paulo T, Cruz, et al.].

The most crucial elements for dam safety and performance in concrete-faced rockfill dams are the stress and deformation characteristics of the rock fill and the concrete face slab. The number of CFRDs has increased recently, along with the topography and geological features at the dam site, creating more challenges for theoretical modeling and the analytical technique of CFRD numerical analysis. How to accurately predict the deformation tendency for high CFRDs and how to improve the design and stress status of the concrete face slab are the key challenges in CFRD design.

The design of CFRDs can benefit greatly from numerical analysis. It makes it possible to analyze how CFRDs respond to stress and deformation under various circumstances and to forecast potential deformation patterns or failure mechanisms. The outcomes of numerous research can offer helpful recommendations for structural design and dam construction.

Non-linear elastic theory, elastoplastic theory, and visco-elastoplastic theory are actual theories on the Constitutive mode. Non-linear elastic and elastoplastic constitutive models are frequently utilized for rockfill material in CFRD numerical analysis. Rockfill material has a very nonlinear Stress-Strain relationship. The constitutive model used for the numerical study of CFRDs must accurately depict this nonlinear relationship. Additionally, the stress distribution on the concrete face will be affected by changes in rockfill volume during shearing [Paulo T, Cruz, et al].

Furthermore, the elastoplastic models with the use of multiple yield surfaces and the non-associated flow rule are theoretically appropriate for the correct evaluation of the volume changes of rockfill material under shear stress, including volume reduction and volume dilatation. In terms of testing, determining parameters, and computational approaches, these models still have certain issues now. An alternative to the elastoplastic model is a non-linear elastic model, such as Duncan-Chang's hyperbola model (E-B), which is more useful and adaptable.

2.10.1 Elastic Model

The elastic model is a mathematical framework used to describe the behavior of materials under stress and strain. It is based on the concept that when a material is subjected to external forces, it undergoes deformation, but once the forces are removed, it returns to its original shape and size. This model assumes that the material is perfectly elastic, meaning that it can withstand deformation without permanent damage or energy loss. In the elastic model, strain is the measure of the material's deformation, while stress is the force applied per unit area. The elastic modulus, a measure of a material's resistance to deformation, describes the relationship between stress and strain. The elastic moduli of many types of materials vary, and this characteristic governs their elasticity.

The elastic model is widely used in engineering and materials science to predict the behavior of structures and design components that can withstand various loads and stresses. By understanding the elastic properties of materials, one can ensure the safety and durability of structures, optimize designs, and prevent failure or deformation under normal or extreme conditions.

The linear model, which posits a linear relationship between the stress and strain determined by the Hooke law, is the fundamental material model. The required information is as follows: Unit weight of the soil (γ), Poisson's ratio (v), and elastic modulus (E)

As shown in the following Figure, the Hooke law uses the Young modulus E (modulus of elasticity) to characterize the linear dependence of stress on strain in a one-dimensional situation. The linear model provides a linear variation of displacements in this framework as a function of applied loads.

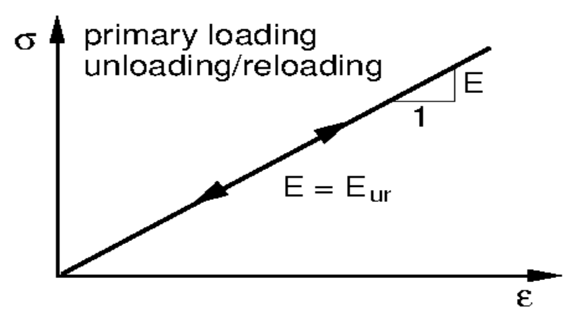

Figure 2-4 Stress-strain curve of a linear elastic material

2.11 Analysis Methods for CFRDS

There are varieties of methods that can be used to analyze CFRDS. The most common methods are analytical methods and numerical methods.

Analytical methods are relatively simple to use, but they can only be used for simple dam geometries and loading conditions.

Numerical methods, such as finite element analysis (FEA), are more complex to use, but they can be used to analyze CFRDs with complex geometries and loading conditions.

2.11.1 Non-Destructive Testing (NDT) Methods for CFRDS

NDT methods can be used to monitor the stress and deformation behavior of CFRDs without damaging the dam. Some of the most common NDT methods used for CFRDs include:

- Surface rebound hammer testing
- Ultrasonic testing
- Acoustic emission testing

NDT methods can be used to detect cracks, delamination's, and other defects in the concrete face slab and rockfill embankment. This information can then be used to assess the condition of the damn and to identify any potential problems.

2.12 Challenges of Implementing FEA Models for CFRDS

FEA is a powerful tool for analyzing the stress and deformation behavior of CFRDS, but it can be difficult to implement accurately. Some of the challenges of FEA modeling for CFRDS include:

- Modeling the complex interaction between the concrete face slab and the rockfill embankment
- Modeling the effects of cracking and delamination's in the concrete face slab
- Modeling the effects of earthquake loading

3. MATERIALS AND METHODS

3.1 Description of the study area

Genale Dawa Hydropower Project (GD3 HPP) is located in a tropical rainforest region at 5°38′ N and 39°43′ E in the south of Ethiopia. The project area is about 400km away from the capital, Addis Ababa, about 200km away from Kenya, and more than 100km away from Somalia. In Figure 3-1 below, the projects' location is depicted.

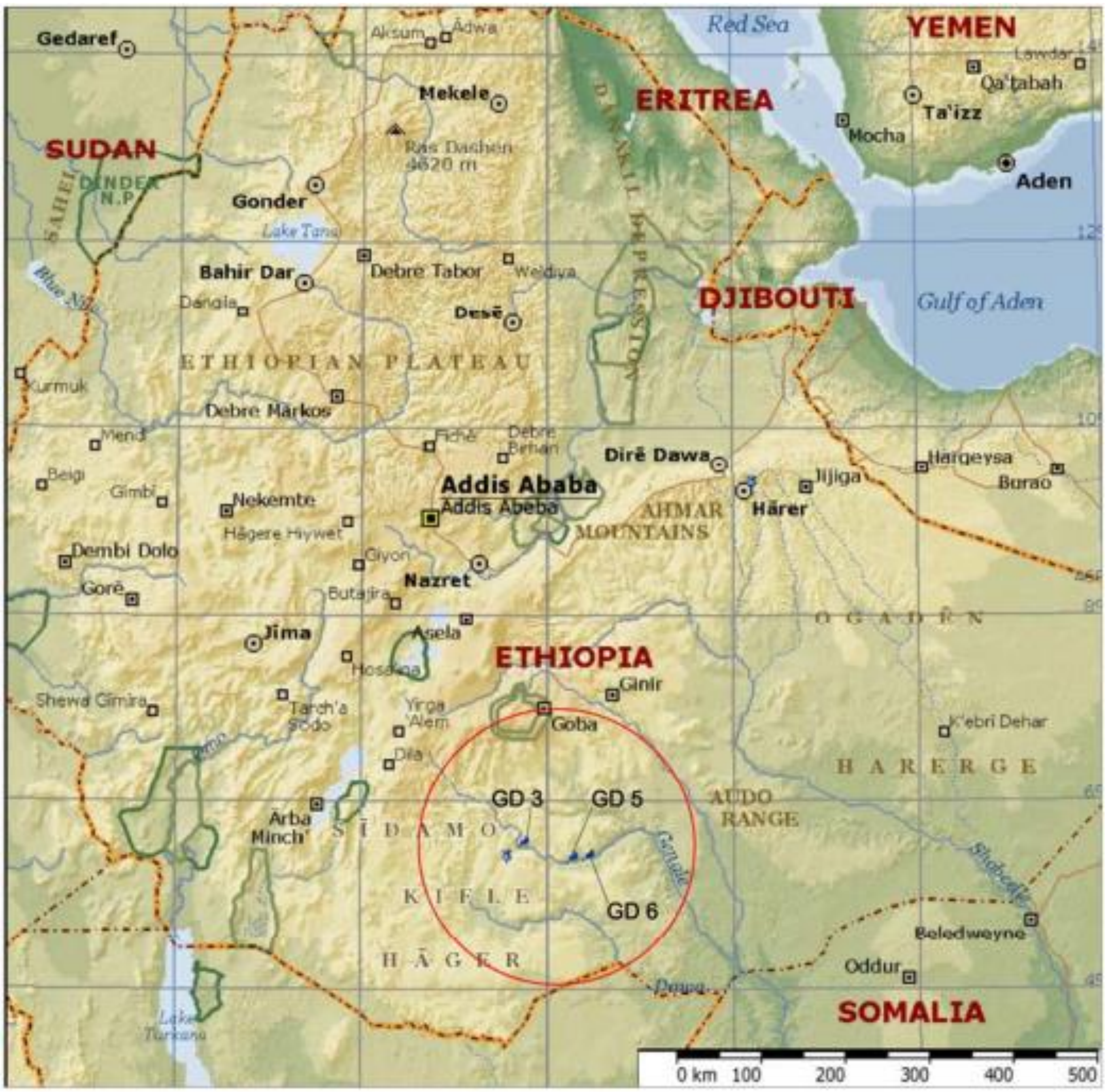

Figure 3-1 Location of the Genale Hydropower Projects GD-3, GD-5 and GD-6 [Genale GD-6 Hydropower Project Feasibility Study, 2009]

The project area has rocky terrain and valleys. The basic topography generally reflects the high and low flow of the Genale River in the northwest and southeast, respectively. The region's greatest height difference is 780 meters. From the reservoir and dam area to the powerhouse, the majority of the mountains' highest points on both banks are between 1,200m and 1,600m above sea level. In the area of the dam, the river valley's lowest point is 1,010 meters above sea level. While the tailrace outlet, which is located downstream of the powerhouse, is situated in a river valley with a bottom elevation of approximately 845 meters above sea level, the headrace elevation of the water transportation and power generation system traverses high mountains.

The dam will be erected in a valley surrounded by symmetrical mountains on either side. The terrain slopes between 30° and 35°, the mountain top elevation on both banks is over 1,200 m above sea level, and the dam's axis is V-shaped. The riverbed's water surface height at the dam axis is nearly 1,015 meters higher than sea level at the gully bottom, which is roughly 1,008 meters higher. With a water depth of 4 to 10 meters during the dry season, the river channel at the dam site is 14 meters long and 30 meters wide.

The dam's crest spans roughly 400 meters. Along the river, the topography for the dam's foundation is comparatively steep. With an average slope of 1.4%, the water level descends from the riverbed plinth to the dam toe by about 4.5m.

3.1.1 Geology in Project Area

The dam site area has a single rock type, late Precambrian grey to grayish white medium-grained to medium-coarse-grained biotite granite. The rock is compacted and very hard. The surface overburden consists of eluvium, colluvium, proluvium, and alluvium.

Eluvium: Eluvium is typically found on gentle slopes on both banks. It is made up of silty clay interspersed with rock blocks and pieces and ranges in thickness from 0.2 to 1.5 meters.

Colluvium: Colluviums are made up of rock blocks, boulders, and silt, and are primarily found on bank slopes and riverbeds, particularly close to the right side of the riverbed. Their thickness is typically less than 4 meters, but some of their boulders have diameters of more than 20 meters.

Proluvium: River proluvium is primarily found on soft riverbank shoals and at the bottom of river valleys. It is made up of sand, mud, and rock blocks, and its thickness on shoals is typically less than 3 meters and more than 4 meters at the bottom of river channels, respectively.

Alluvium: Alluvium is also made up of sand, mud, and rock blocks, and is typically found in floodplains and river channels. It is also unstable and can be eroded.

3.2 Project Description

3.2.1 Layout of Dam Body

The concrete-faced rockfill dam is a water retention dam with a crest elevation of 1125m, a maximum height of 110m, and an axis length of 401m. A wave wail with an elevation of 1126.20m is provided upstream of the dam crest to protect it from erosion and overtopping, and a masonry retaining wall (with a height of 4m) is provided downstream to minimize the risk of erosion and scouring. The beach spillway is adjacent to the left dam head and is designed to be efficient at discharging floodwaters, even in regions with a high risk of flooding. The dam crest width is 8.0m thick, which is adequate for both structural and operational needs but could be increased to provide more space for operation and maintenance activities, especially if the dam is expected to be subject to heavy traffic.

The slope ratio of the upstream dam slope is 1:1.40, and that of the downstream dam slope is 1:1.35/1.4. These slope ratios are appropriate for the given geological conditions and will help to ensure the stability of the dam. One 2.0m-wide berm will be set in places (on the downstream dam slope) with elevations of 1100m, 1070m, and 1040m to provide access for inspection and maintenance. The overall slope ratio downstream is about 1:1.45, which is relatively steep, but the presence of the berms will help to reduce the risk of erosion.

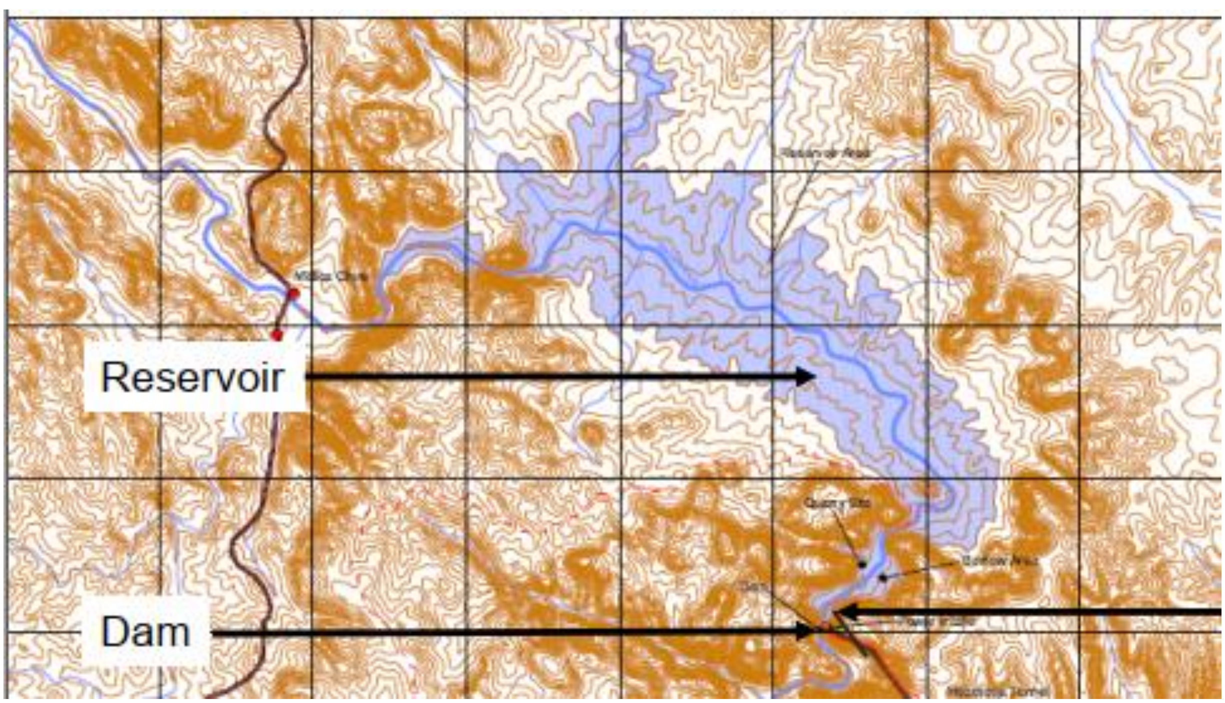

Figure 3-2. Scheme layout [EEP March 2017]

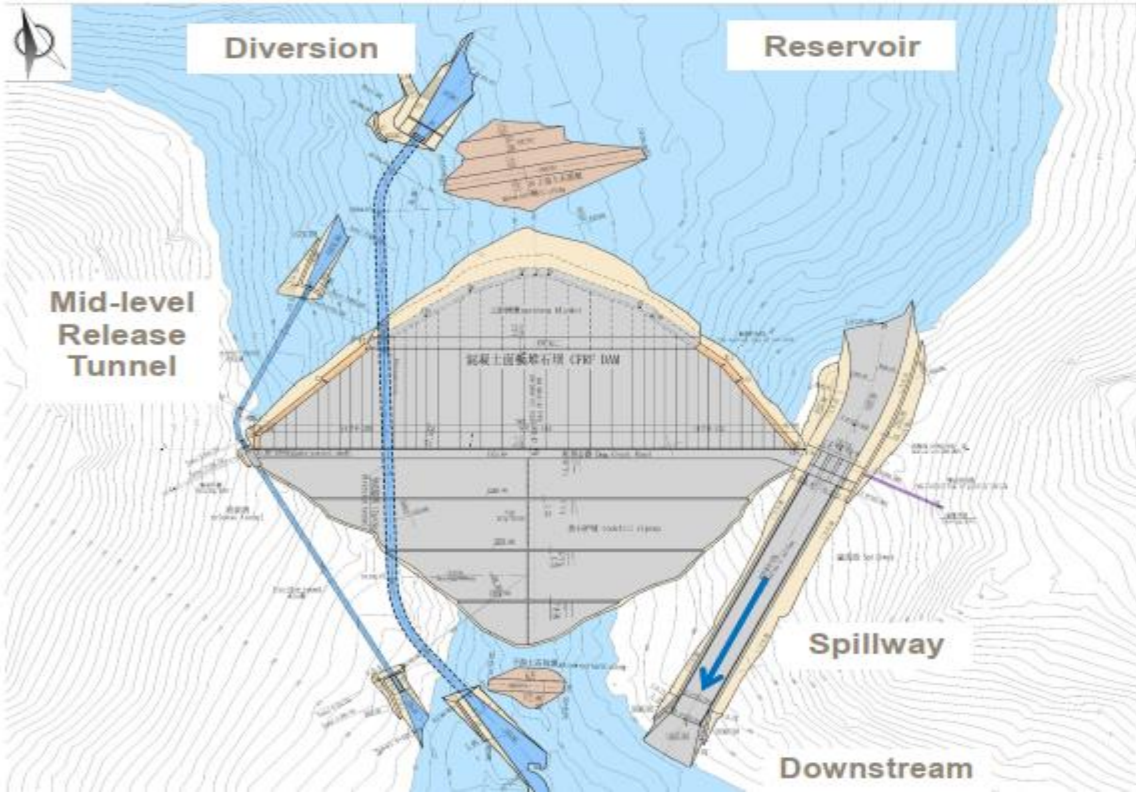

Figure 3-3. Layout Plan of Dam Area Structures [EEP March 2017]

Figure 3-4 Upstream and Down Stream View [EEP March 2017]

3.2.2 Physical Descriptions

The Genale Dawa III Hydropower Project is a concrete-faced rockfill dam located on the Genale River in Ethiopia. It is the third cascade hydropower project of the Genale-Dawa River basin.

Table 3-1 Physical Descriptions of Genale Dawa III Hydropower Concrete Face Rockfill Dam

Dam Type	CFRD
Catchment area	10,445 km^2
Mean inflow	92.6 m^3/s
Normal pool level	1,120 m a.s.l.
Peak maximum Flood Level	1122.8 m a.s.l
Total reservoir storage	2.57 × 10^9 m^3
Available storage	2.31 × 10^9 m^3
Maximum dam height	110 m
Dam crest length	400 m
Top Crest Width	7.5 m
Dam crest elevation	1,125 m a.s.l.
Maximum power generation water head	240 m
Types of Turbines	Francis Turbines
No. of Turbines	3
Generated Power per Turbine	1600 GWh
Total installed capacity	254 MW

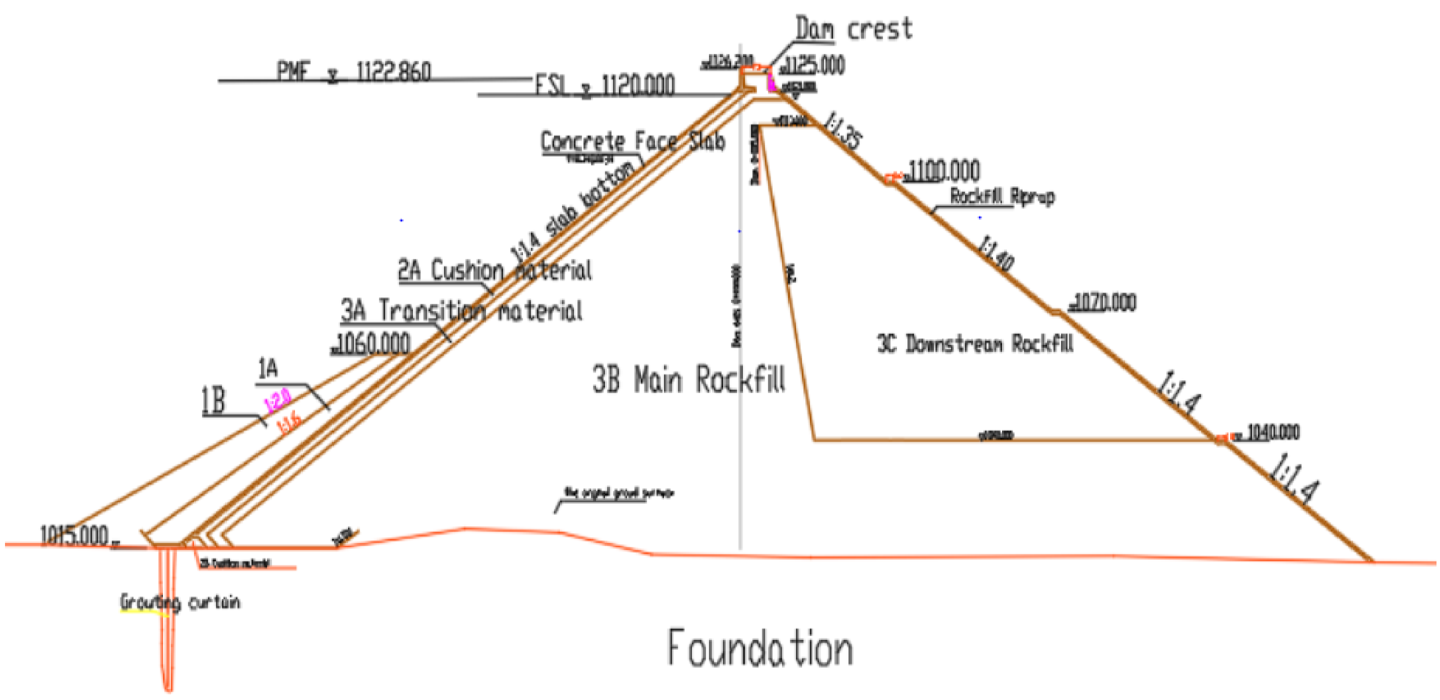

Figure 3-5 Dam section at 0+000

3.2.3 Zoning of Dam Body

The following guidelines should be followed when zoning a dam body: each material zone's permeability should gradually increase from upstream to downstream; each material zone's deformation due to water pressure should be coordinated and minimal; the project's excavated materials should be fully utilized; and the material zone division should be as straightforward as possible to minimize construction challenges.

The dam rockfill zones include the special cushion zone, transition zone, major rockfill zone, secondary rockfill zone, and rubble slope protection. The cushion zone and transition zone have a slope ratio of 1:1.4, and their horizontal widths are 3m and 4m, respectively. A special cushion zone is supplied downstream of the toe slab. The primary rockfill zone is located downstream of the transition zone, with a top height of 1119m. A downstream rockfill zone exists downstream of the main rockfill zone. The downstream rockfill zone has an upstream side slope ratio of 1:0.2, an elevation of 1113m at the top and 1040m at the bottom. Dry rubble facing, with a horizontal width of 1m, is provided over the downstream surface of the dam.

A "clay + rock ballast" cover shall be provided upstream of the face with an elevation of less than 1060m for supplemental anti-seepage function. The top width of the clay cover is 4.00m, and the upstream surface slope ratio is 1:1.6; the top width of the rock ballast-mixing zone is 6.00m, and the upstream surface slope ratio is 1:2.0.

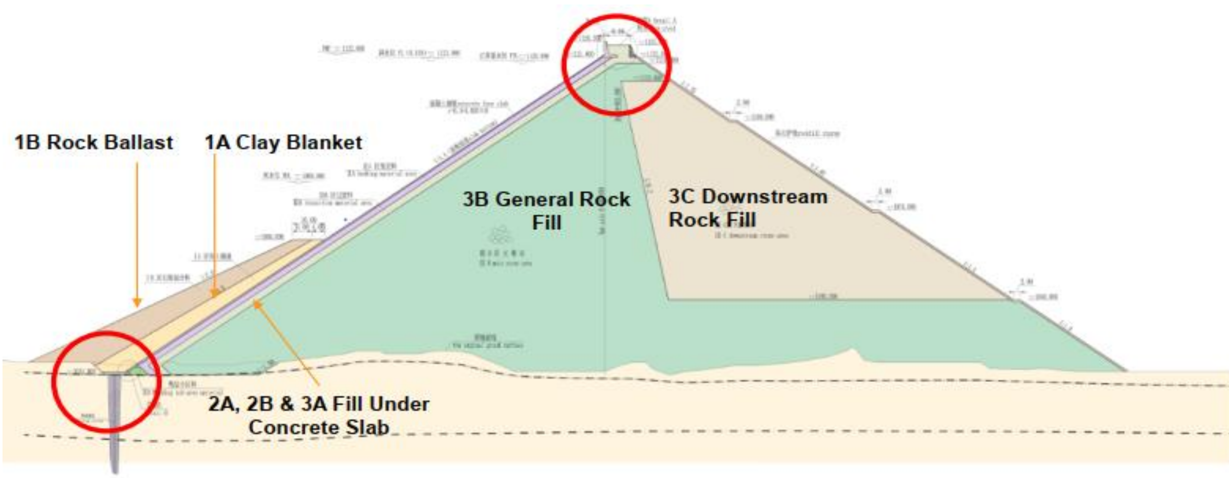

Figure 3-6. Zoning of Dam Body

3.3 Data collection

The required data for the research were collected directly from the project office of the dam owner, namely EEP. The project office provided geometric, material property, and earthquake data inputs, and literature was studied.

3.3.1 Material Properties for the Existing CRFD

Cushion Material

As the primary component of concrete facing a rockfill dam, the cushion material must be semi-permeable, low compressible, strong, and high seepage stable. It must also perform reversed filtering against fly ash. As a result, the horizontal breadth of the cushion material was 3 meters, and processed and slightly weathered rockfill materials with good gradation were used. Main design criteria include a dry density of 21.8 kN/m3, a porosity of 18%, a maximum diameter of 80 mm, a percentage of particles with a diameter of less than 5 mm between 40% and 50%, a maximum percentage of particles with a diameter of less than 0.075 mm of 8%, and a continuous gradation.

Special Cushion Material

The special cushion material was provided in the peripheral joints, and it had a good filtering function for the fly ash placed upstream.

The following are the main design criteria: continuous gradation, maximum diameter of 40 mm, dry density of 22 kN/m3, porosity of 17%, and maximum diameter of 5 mm.

Transition Material

The transition material, which had a horizontal width of 4 meters, was placed between the cushion material and the main rock filling material and had to serve as a filter for the fine materials of the cushion zone. The slightly weathered fresh and processed rocks that can satisfy the graduation requirements were therefore adopted. The transition material must possess low compressibility, strong shear strength, and free drainage after compacting.

Principal design criteria include a dry density of 21.2 kN/m3, a porosity of 20%, a maximum diameter of 300 mm, a content of particles less than 5 mm in diameter of less than 15%, and a continuous gradation.

Main rockfill Material

The main rockfill material served as the body sustaining the majority of the dam, so it had to be sufficiently dense and possess the necessary deformation modulus. The principal rockfill materials will be mined and blasted parent rock from the relatively fresh granite.

The following main design criteria were used: continuous gradation, dry density of 20.5 kN/m3, porosity of 23%, maximum diameter of 600 mm, content of material with a diameter less than 5 mm, and content of particles with a diameter less than 0.075 mm.

Downstream Rockfill Material

The downstream rockfill zone was located beyond the dam. The top of such zone is 1113m above sea level, 5m from the dam axis, and the slope between the top and the main rockfill zone boundary is 1:0.2. Secondary rockfill material requirements can be adequately lowered, and poorly worn granite can be blasted, mined, and utilized.

Main design criteria include a dry density of 20.2 kN/m3, a porosity of 25%, a maximum diameter of 800 mm, a content of particles less than 5 mm in diameter of less than 20%, and a continuous gradation.

Dry Rubble for Downstream Slope Protection

Dry rubble slope protection is adopted over the downstream surface of the dam; such slope protection with a horizontal width of 1m, and the minimum diameter of the rubble shall be no less than 0.5m. See Table 3-3 for the main design index for materials in each zone of the dam.

Table 3-2. Main Design Index for Materials in Each Zone of Dam

Type	Dam filling materials	Dry density γ_d (kN/m^3)	Porosity n %	Maximum diameter (mm)	Permeability coefficient (cm/s)
Special cushion materials	Slightly weathered materials after processing	22.0	17	40	10-3~10-4
Cushion materials	Slightly weathered materials after processing	21.8	18	80	10-3~10-4
Transitional materials	Slightly weathered materials after processing	21.2	20	300	40
Main rockfill materials	moderately-slightly weathered rock fill materials	20.5	23	600	80
Downstream rockfill materials	moderately weathered materials	20.2	25	800	100

In a finite element calculation, it was decided to model the dam's cushion, transition, main rockfill, and secondary rockfill materials, facing and toe slabs of reinforced concrete, and the dam's foundation rock mass, ballast mixture, and blanket using the linear elastic model. The contact surface unit between the two materials was established to precisely represent this discontinuous displacement. The boundary (interface) behavior of similar or dissimilar materials was modeled using the interface behavioral approach. The interface behavioral model, which is based on Coulomb's law of friction (1785), makes the premise that an interface's frictional force is proportional to its coefficient of friction and any confining forces acting on it in directions perpendicular to its normal direction.

The material parameters of analysis were applied using the project report (EEP, 2018), the Sibilo dam design report (2020), and the kinds of literature. The material parameters adopted are shown in Table 3-3 below.

Table 3-3 Parameters of Dam body Material and Concrete face (Elastic Model)

Type	γt (kN/m^3)	γsat (kN/m^3)	E (MPa)	ν
Zone 1A	19	21.5	50	0.2
Zone 1B	17	20.5	50	0.2
Concrete Face and slab	25	25	30,000	0.167
Zone 2B	21.8	22	50	0.21
Zone 3A	21.2	21.8	50	0.32
Zone 3B	20.5	21.5	50	0.33
Zone 3C	20.2	21.5	100	0.34
Bed Rock	26	26	2,000	0.22

3.3.2 Seismic Data

According to Ethiopia's seismic intensity zoning, the project region of the GD3 HPP is in the low seismic intensity zone, with a seismic intensity of VI degrees (see Fig. 3-6). Since November 1988, the Addis Ababa University Seismographic Station has monitored and evaluated the GD3 HPP project site. Table 3-4 indicates the project's regional earthquake peak acceleration as well as the corresponding VI-degree fundamental earthquake intensity. As a result, the basic seismic intensity for the project region was specified as VI degree for protection. The operational basis earthquake peak acceleration was 0.12g, as was the maximum design earthquake peak acceleration. Refer to the Ethiopian Building Code Standard EBCS-8: Design of Structures for Earthquake Resistance Data Base 1875-1974 (Prepared 1995). Bedrock in the GD-3 project area has an acceleration response rate of $\alpha 0 \leq 0.01$ in the Area ○0. See Fig. 3-7 for details.

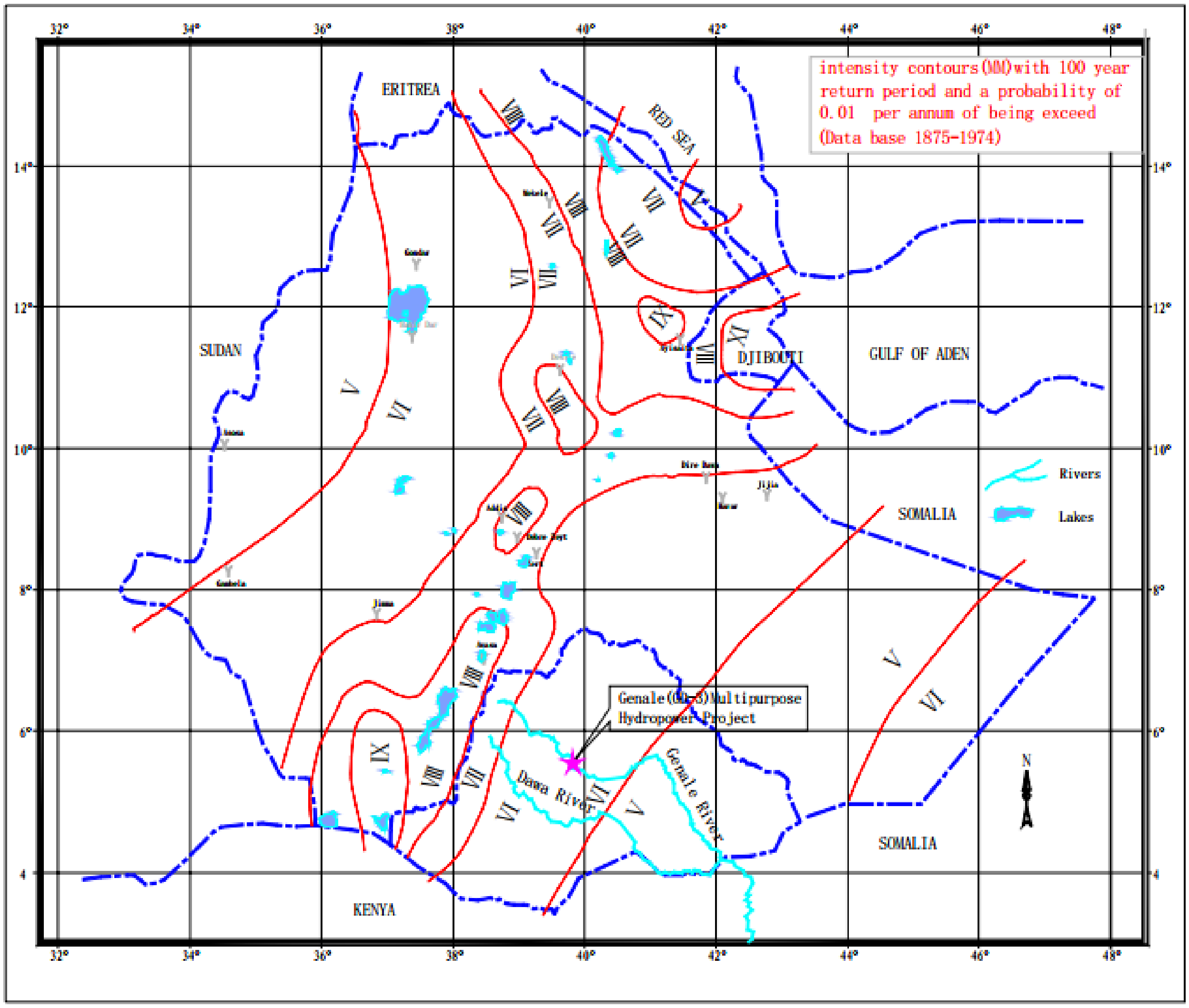

Figure 3-7. Zoning of Seismic Intensities in Ethiopia

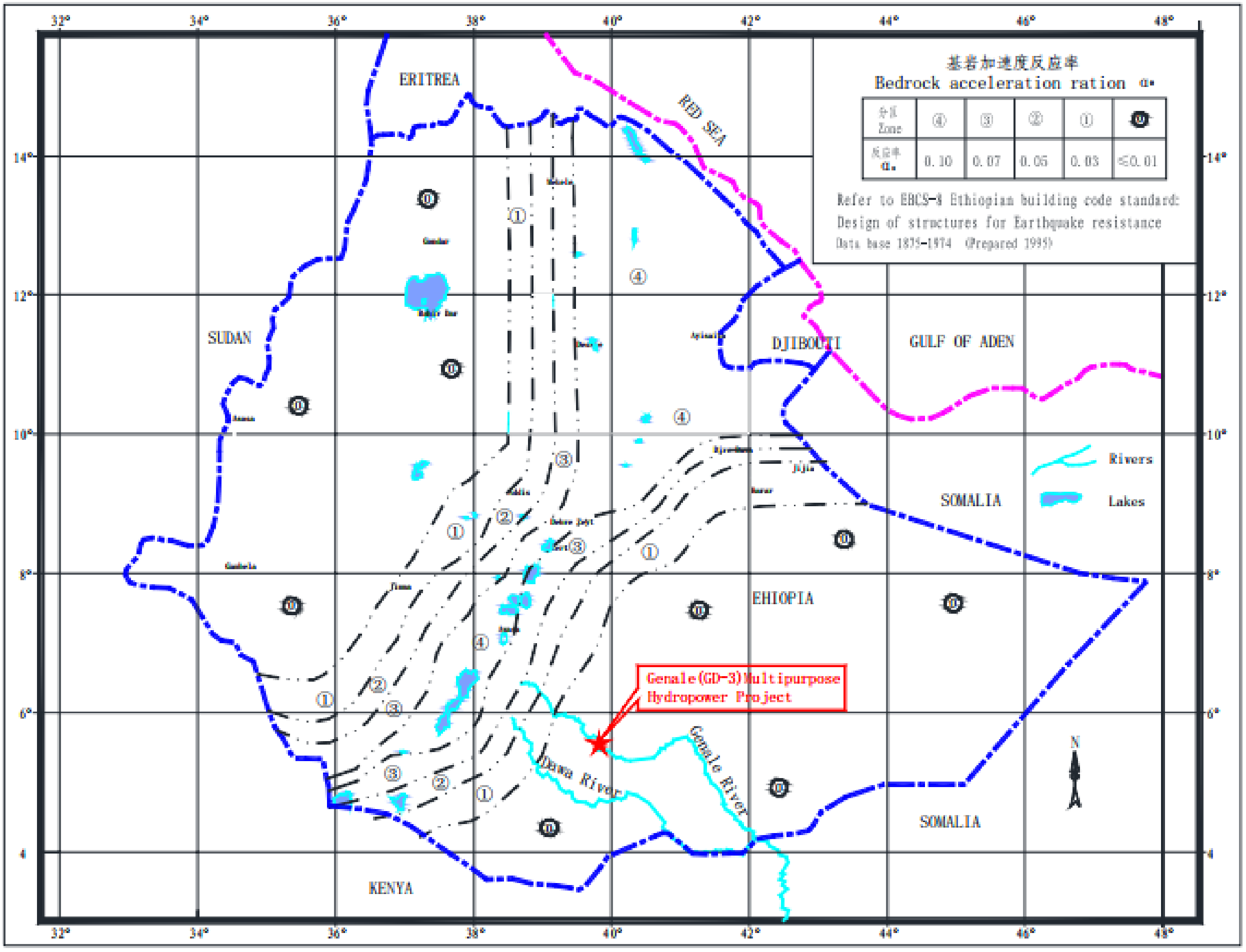

Figure 3-8. Zoning of Seismic Risk in Ethiopia

3.3.2.1 Characterization of Design Earthquakes

The following seismic parameters describe the various design earthquakes (Wieland, M., 2012).

1) The maximum ground acceleration (PGA) of the earthquake's horizontal component.
2) The acceleration response spectra of the horizontal earthquake component, usually for 5% damping, i.e., uniform hazard spectra for OBE and MDE obtained from the probabilistic seismic hazard analysis (mean values) and 84 percentile values of acceleration spectra for MCE obtained from the deterministic analysis using various attenuation models.
3) Acceleration time histories for the horizontal component of the MDE ground motion that are spectrum-compatible and are derived either randomly or by scaling the ground motions of observed earthquakes. The horizontal earthquake components' artificially manufactured acceleration time histories must be stochastically independent. It is advised to prolong the period of intense ground shaking to accommodate for aftershocks.

3.3.2.2 Development of Design Response Spectra

Site-specific procedures or standard procedures for the generation of design response spectra.

Standard Horizontal Response Spectra

Standard response spectra are based on a general characteristic shape that is determined in terms of estimates of certain ground motion parameters. These parameters can be effective peak ground accelerations or spectral accelerations. The following are some factors to take into account and steps to implement while creating a standard response spectrum:

(i) Seismic hazard data

The predicted probability of exceedance should be calculated for a given site location using the PGA calculated using the PSHA technique and the spectral ordinates at intervals of 0.2 seconds (denoted by Ss) and 1.0 seconds (denoted by S1). The shear-wave velocity in the top 30 meters of hard rock locations, which are used to determine ground motion estimates, is approximately 760 m/sec.

(ii) Site Conditions

To reflect site characteristics, one can alter the shape of the conventional response spectra. The effects of the foundation and soil can have a significant impact on how the structure responds. The employment of site coefficients, which scale the spectrum ordinates to the proper values for other local conditions, allows for the accounting of these site impacts in the construction of the standard response spectra.

Table 3-4 Site classification

Site class	Description	Shear wave velocity, Vs (m/s)	Reference
A	Hard rock	1500	USACE (2007), EM 1110-6053
B	Rock	750 - 1500	
A	Rock or other rock-like geological formation	> 800	ES EN 1998-1: 2015

In this study selected, Damping 5%, Ground type: A (rock), Seismic zone: 3, and PGA = 0.12g

Parameters to describe the horizontal elastic response spectrum (ES EN 1998-1: 2015)

Ground type	*S*	*T*B (s)	*T*C (s)	*T*D (s)
A	1,0	0,15	0,4	2,0

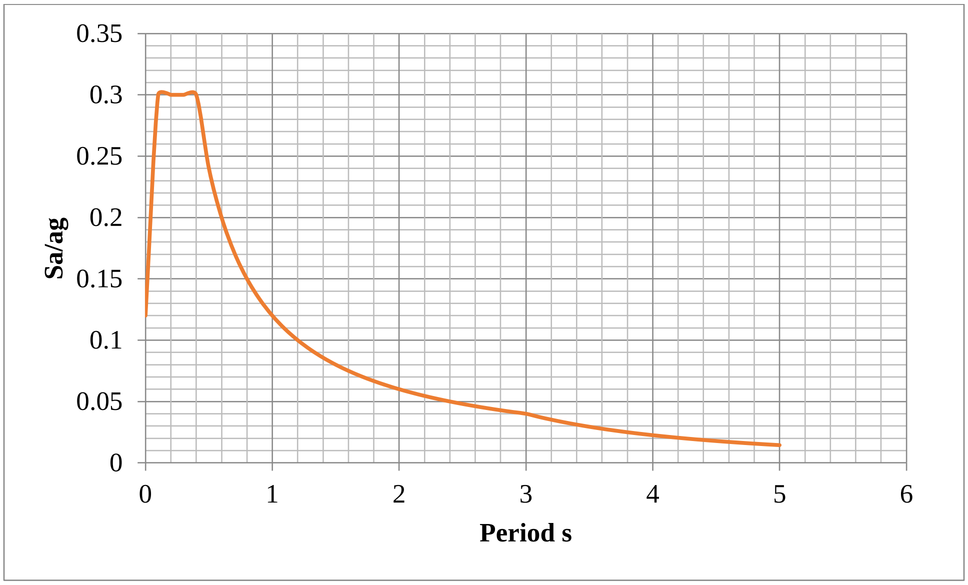

Figure 3-9 Shape of the elastic response spectrum

3.3.2.3 Selection of acceleration-time histories

The initial selection of time histories will be followed once the design earthquakes and design ground motion characteristics are defined for the project. The time histories should, in general, be fairly consistent with the site's tectonic environment, design earthquake parameters (magnitude, source-to-site distance, faulting type), local site conditions, and design ground motion parameters (response spectral content, duration of strong shaking, and special characteristics, such as near-source characteristics). If there are not enough recorded motions, ground motion modeling techniques can be used to create simulated recorded time histories. The smooth design response spectra, which also include the zero-period acceleration that is equal to the peak ground acceleration (PGA), duration of violent shaking, and particular ground motion characteristics (such as near-source pulse movements) are examples of design ground motion characteristics. Following selection of time-history records from the database of observed ground motions that are reasonably consistent with the design parameters and conditions, given these features of the design earthquakes and ground motions, is done (USACE 2003, EM 1110-2-6051).

3.3.2.4 Factors to be considered in selecting time-histories

The acceleration time histories should be chosen whenever possible to be comparable to the design earthquake in the following ways:

- Tectonic environment.
- Earthquake magnitude
- Fault rupture mechanism (type of faulting)
- Earthquake source-to-site distance.
- Site/subsurface conditions: rock, firm soil, etc
- Design response spectrum characteristics: similarity in amplitude
- Duration of strong shaking: < 1.5 times the duration specified for the design earthquake.
- Pulse characteristics and sequencing: fault ruptures process (< 10km).

It is frequently essential to edit already-existing data or create synthetic records that satisfy the majority of these requirements because it is not always possible to discover records that fit all of these characteristics. At least three separate earthquakes must be taken into account for the MDE ground motion during the safety check of a dam (Wieland, 2017).

3.3.2.5 Selected representative peak ground motions for time-history analysis

Among the well-recognized databases for strong ground, motions include the Pacific Earthquake Engineering Research Center (PEER) NGA-West2 database.

A significant amount of ground movements from shallow crustal earthquakes that occurred in active tectonic regimes were collected worldwide and are available in the PEER NGA-West2 database (https://ngawest2.berkeley.edu). One of the most complete sets of metadata, including numerous distance measurements, distinct site characterizations, and earthquake source information, may be found in the database. The PEER Ground Motion database has proven to be quite popular with earthquake-related engineers, who are increasingly using it for record selection and modification while analyzing computer models of different infrastructure projects, including large dams.

Table 3-5 Earthquake time history data obtained from the PEER ground motion database

Earthquake serial no.	Earthquake Name	Year	Station Name	Magnitude	Mechanism	Vs30 (m/sec)
RSN80	"San Fernando"	1971	"Pasadena - Old Seismo Lab"	6.61	Reverse	969.07
RSN455	"Morgan Hill"	1984	"Gilroy Array #1"	6.19	strike-slip	1428.14
RSN788	"Loma Prieta"	1989	"Piedmont Jr High School Grounds"	6.93	Reverse Oblique	895.36

Finally, the fittest design response spectrum was Loma Prieta, and the time history database of this ground motion was adopted by applying a scale factor of 1.3844 for use in the dynamic analysis of the dam.

The following seismic Data will be used in the Finite element analysis model

Peak ground acceleration (PGA) = 0.12g

Duration of Ground motion = 30 seconds

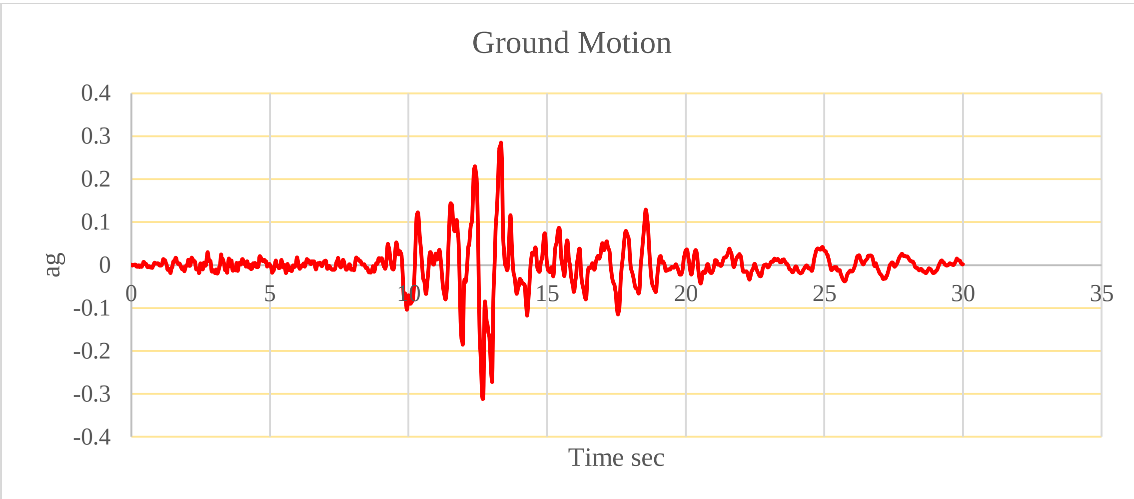

Figure 3-10 Ground acceleration

3.3.3 Loading Conditions and Combinations

Table 3-6 Loading Conditions and Combinations

Case	Condition	Load combinations	Remarks
1	Empty Reservoir	Dead Load	-
2	Normal Pool Level	Dead Load + Hydraulic pressure(F.S.L)	F.S.L(EL. 1120 m)
3	PMF Flood Level	Dead Load + Hydraulic pressure (M.W.L)	M.W.L(EL.1122.86m)
4	Normal Pool Level + Earthquake	Dead Load + Hydraulic Pressure(F.S.L) + Earthquake Load	Earthquake Load (0.12g)

3.4 Analysis method

3.4.1 Finite element Analysis

The Finite Element Method (FEM) is a powerful numerical technique commonly used in various fields of engineering and science for solving complex problems. It has proven to be an effective numerical analysis technique for assessing the behavior and stability of concrete face rockfill dams. According to Zhou et al. (2016), the FEM is a powerful computational tool that allows engineers to model and analyze complex structures, such as CFRDs, by dividing them into smaller elements and solving the governing equations for each element. The FEM offers several advantages for the analysis of CFRDs. Firstly; it can accurately capture the non-linear behavior of concrete and rockfill materials, which is crucial for understanding the response of CFRDs under various loading conditions. Secondly, the FEM allows for the consideration of heterogeneities in material properties and the incorporation of complex boundary conditions, enabling a more realistic representation of the actual dam behavior. Additionally, the FEM provides a flexible framework for investigating different failure mechanisms and evaluating the safety of CFRDs. By simulating the stress distribution, deformation, and stability of CFRDs, the FEM can assist engineers in optimizing dam designs and identifying potential failure modes. Overall, the implementation of the FEM in the numerical analysis of CFRDs offers a powerful and reliable tool for enhancing the understanding and safety assessment of these important infrastructures (Zhou et al. 2016).

3.4.2 Numerical modeling of CFRDs

There are several software packages available for numerical modeling of stress and deformation analysis in Concrete-Faced Rockfill Dams (CFRDs).

Table 3-7 The software packages commonly used for stress and deformation analysis of CFRDs

software packages	Advantages	Disadvantage
FLAC	▪ Specialized for geotechnical analysis, including soil-structure interaction. ▪ Efficient for modeling large-scale problems. ▪ Capable of simulating complex behaviors like nonlinear soil and rock behavior, consolidation, and thermal effects. ▪ User-friendly interface and extensive documentation.	▪ Limited to 2D analysis, although FLAC3D extends to 3D. ▪ Less suitable for complex dam geometries and material interfaces. ▪ May require additional software or scripting for advanced analyses.

Plaxis	▪ Specialized in geotechnical finite element analysis. ▪ Offers both 2D and 3D analysis capabilities. ▪ Provides a wide range of geotechnical material models. ▪ Supports advanced soil-structure interaction modeling. ▪ User-friendly interface with extensive tutorials and examples.	▪ Limited to geotechnical analysis and may lack certain features required for specific applications. ▪ Can be computationally demanding for large-scale models and complex analyses. ▪ Requires expertise to correctly define material parameters and boundary conditions.
MIDAS GTS	▪ Provides comprehensive geotechnical analysis capabilities for a wide range of applications. ▪ Offers both 2D and 3D analysis options. ▪ Includes advanced soil and rock constitutive models. ▪ User-friendly interface with built-in wizards and templates.	▪ Limited documentation and resources compared to other software packages. ▪ May require additional customization for specific applications. ▪ Less commonly used compared to other software options.
ABAQUS	▪ General-purpose finite element analysis software applicable to a wide range of engineering disciplines. ▪ Offers advanced features for complex material behaviors and multi-physics analysis. ▪ Supports both 2D and 3D analysis. ▪ Large user community and extensive documentation available.	▪ Steeper learning curve, especially for geotechnical applications. ▪ Requires expertise to define appropriate constitutive models and material parameters. ▪ Can be computationally demanding, especially for large-scale models and complex analyses.

3.4.3 The capability of Midas GTS NX

Midas GTS NX is a software package that specializes in geotechnical and geo-environmental finite element analysis. It is widely used in the field of civil engineering and geotechnical engineering for analyzing and designing various types of geotechnical structures such as foundations, retaining walls, embankments, tunnels, and slopes. It has the capability to model the complex geometry and material behavior of concrete face rockfill dams.

To investigate soil-structure interaction using a finite element approach, the simulation tool GTS NX was created. Use GTS NX to conduct in-depth analyses of excavation, banking, structural location, loads, and other elements that directly affect design and construction. To replicate actual events, the application supports several conditions (soil properties, water level, etc.).

Nonlinear analysis techniques, including seepage-stress, stress-slope, seepage-slope, and nonlinear dynamic-slope coupled analyses, can simulate settings for a variety of field conditions. These techniques include linear/nonlinear static analysis, linear/nonlinear dynamic analysis, seepage and consolidation analysis, slope safety analysis, and slope analysis.

Midas GTS NX uses the Finite Element Method (FEM) for mathematical analysis. The FEM is a numerical method for solving partial differential equations that arise in various engineering disciplines, including geotechnical engineering. It involves dividing a complex problem into smaller, simpler elements, and approximating the behavior of the solution within each element.

In the context of geotechnical analysis, Midas GTS NX uses the FEM to solve governing equations related to soil mechanics and structural behavior.

In this study, “Midas GTS NX” software is used for modeling and analyses being performed on the CFRD to determine the deformations and stresses made in the dam cross-section and concrete slab. The two-dimensional analyses were carried out using finite element modeling by Midas GTS NX and included both Static analysis and Dynamic analysis.

3.4.4 Governing Equations for FEA

The governing equation used to analyze the stress and deformation of a concrete face rockfill dam is typically based on the principles of solid mechanics, specifically the theory of elasticity. The most common approach is to use the equations derived from the theory of linear elasticity.

The equilibrium equation, which represents the balance of forces, is given by:

$$\nabla \cdot \sigma + \rho g = 0$$

Where $\nabla \cdot \sigma$ is the divergence of the stress tensor σ, ρ is the density of the material, and g is the acceleration due to gravity.

The stress-strain relationship for linear elasticity is expressed as:

$$\sigma = D \cdot \varepsilon$$

where σ is the stress tensor, ε is the strain tensor, and D is the fourth-order elastic stiffness tensor. The elastic stiffness tensor relates the stress and strain components and depends on the material properties of concrete and rockfill.

The strain-displacement relationship relates the strain to the displacement field. For small deformations, this relationship can be approximated as:

$$\varepsilon = ½ (\nabla u + (\nabla u)^T)$$

where ε is the strain tensor and u is the displacement vector.

By combining these equations and applying appropriate boundary conditions, it is possible to solve for the stress and deformation distribution within the concrete face rockfill dam. However, it is important to note that the specific analysis methods and equations used may vary depending on the complexity of the dam and the assumptions made in the analysis.

3.4.4.1 Why used Midas GTS NX

There are several reasons why Midas GTS NX's finite element method (FEM) is preferred over other software in certain cases:

1. **Geotechnical Focus**: Midas GTS NX is specifically designed for geotechnical applications, which means it offers a wide range of specialized features and capabilities tailored to address the challenges and complexities of geotechnical engineering problems. It provides advanced soil constitutive models, soil-structure interaction analysis, and specialized analysis tools for geotechnical simulations.
2. **Comprehensive Analysis Capabilities**: The software offers a comprehensive range of analysis capabilities, including static, dynamic, and nonlinear analysis. It can handle complex soil behavior, consolidation analysis, seepage analysis, and stability analysis, among others. This allows engineers to simulate and study the behavior of geotechnical systems under various loading conditions accurately.
3. **User-Friendly Interface**: Midas GTS NX provides a user-friendly interface that makes it easier for engineers to create and analyze complex geotechnical models. It offers powerful modeling tools, intuitive graphical interfaces, and efficient mesh generation capabilities, allowing users to quickly set up and analyze their models.
4. **Robust Solver**: The software utilizes advanced finite element solvers that are capable of handling large and complex geotechnical models efficiently. It can handle complex soil-structure interaction problems, nonlinear material behavior, and large deformation analysis. The solver's speed and accuracy contribute to the software's reliability and effectiveness.
5. **Integration and Interoperability**: Midas GTS NX allows for seamless integration with other software packages commonly used in civil engineering, such as computer-aided design (CAD) and building information modeling (BIM) software. This facilitates the

exchange of data between different software platforms, streamlining the engineering workflow.

3.4.5 Dam Modeling

The dam will be modeled in Midas GTS NX using Two-dimensional mesh of tetrahedral elements. The mesh will be refined in areas where high Stresses and deformations are expected.

3.4.6 Procedure of Modeling and Analysis

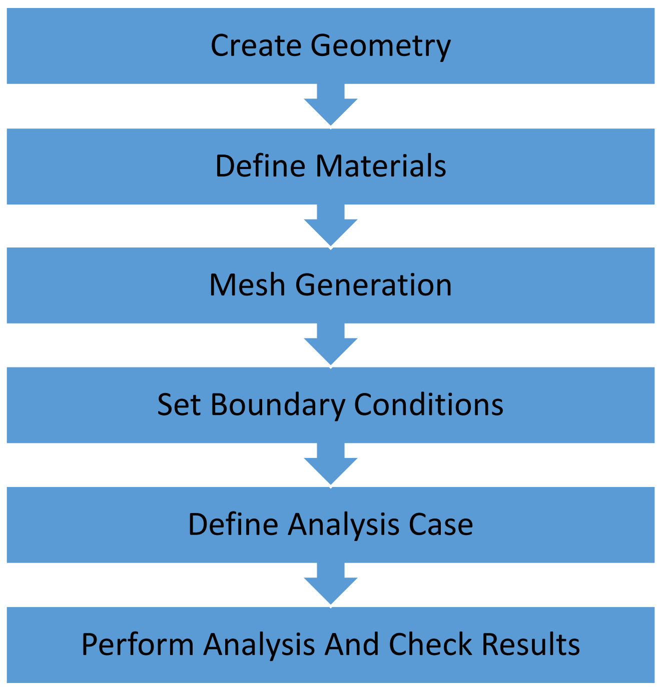

Figure 3-11 Procedure of modeling and analysis

Geometry*:* The geometric model was created as the foundation for developing a finite element analysis model in GTS. Mesh creation and various extra modeling procedures were performed based on the geometry data. The geometric model was created directly in GTS utilizing its modeling features. The Finite Element Analysis of the dam was carried out for the 0+000 Section.

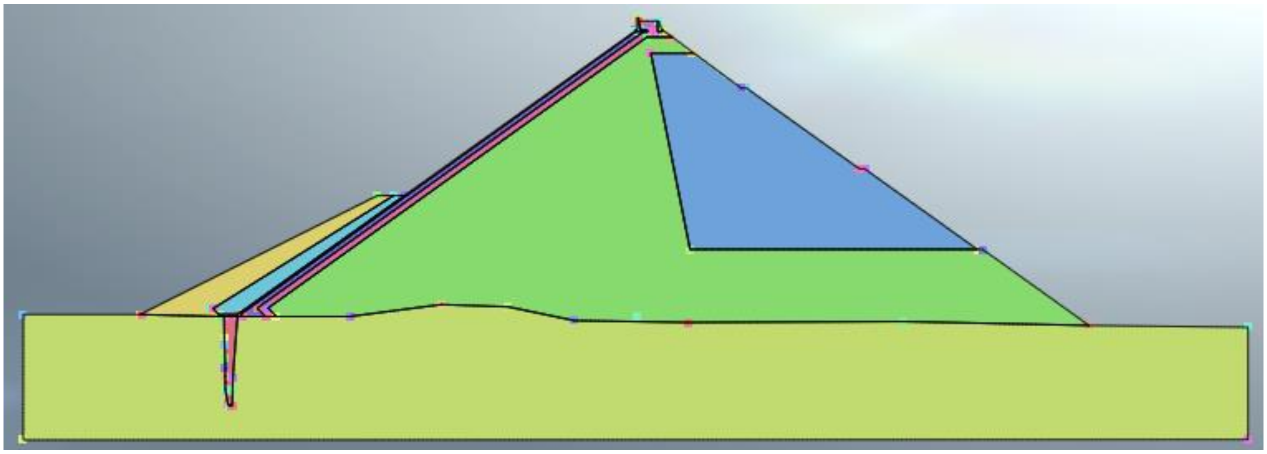

Figure 3-12 Dam Geometry Section (0+000)

Mesh Generation

Mesh was constructed from the geometric model that had previously been created. In general, triangular and quadrilateral pieces are suggested for more accurate analytical results. However, for a complex model, it is preferable to employ quadrilateral or triangular elements generated by GTS's Auto-mesh generation feature. GTS features many mesh control functions as well as three different mesh-generating methods - Auto-mesh, Mapped-mesh, and Protrude-mesh - to maximize its usability. The finite element model used 1m and 2m mesh density, used the various conditions such as normal pool level, peak flood level, and earthquak condition.

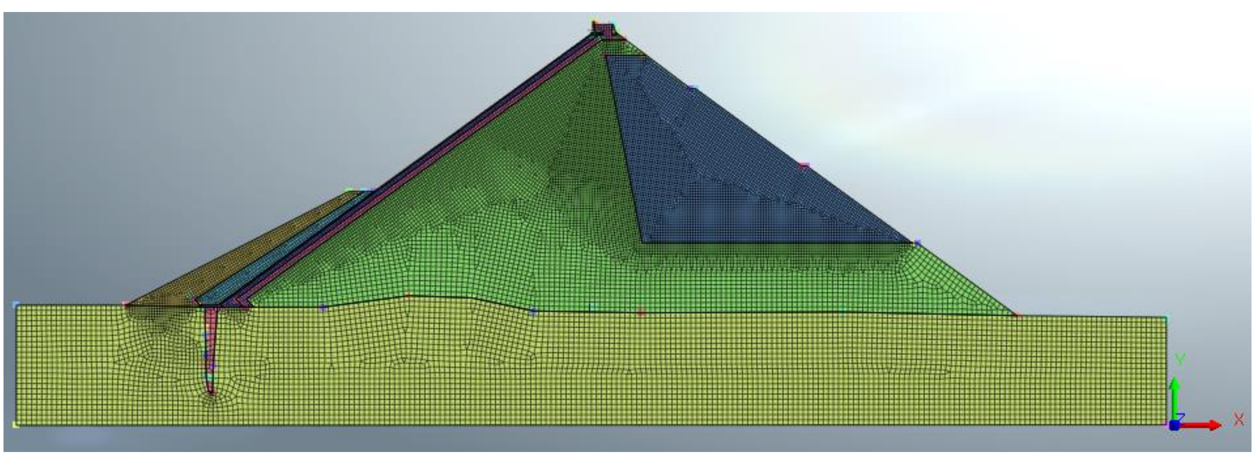

Figure 3-13 Finite Element Analysis Modeling (0+000)

Boundary Condition

GTS offers several different materials, physical attributes, load types, and boundary conditions. The load and boundary conditions can be applied directly to geometric forms rather than nodes and elements, allowing the conditions to be employed efficiently even when the model has a very complicated shape.

Table 3-8 The boundary conditions of the finite element model

case	*Type of analysis*	*Load Type*	*Boundary Condition*
1	Static Analysis	Self-weight	Fixed Ground Condition
2	Static Analysis	Self-weight and water pressure	Fixed Ground Condition, normal pool level
3	Static Analysis	Self-weight and water pressure	Fixed Ground Condition, peak flood level
4	Dynamic Analysis	Self-weight, Earthquake, and water pressure	Fixed Ground Condition, normal pool level, Time Step

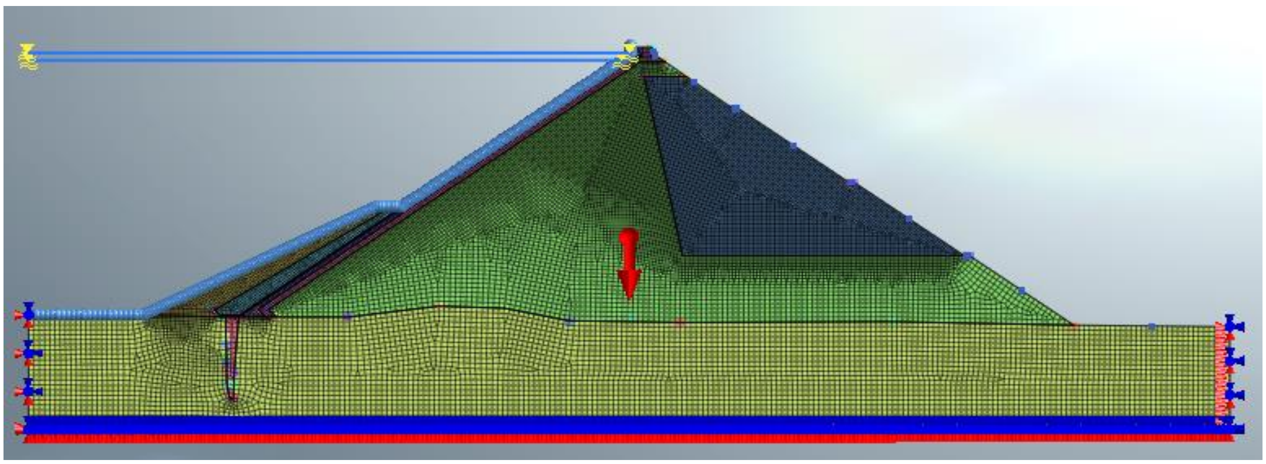

Figure 3-14 Static Boundary Conditions

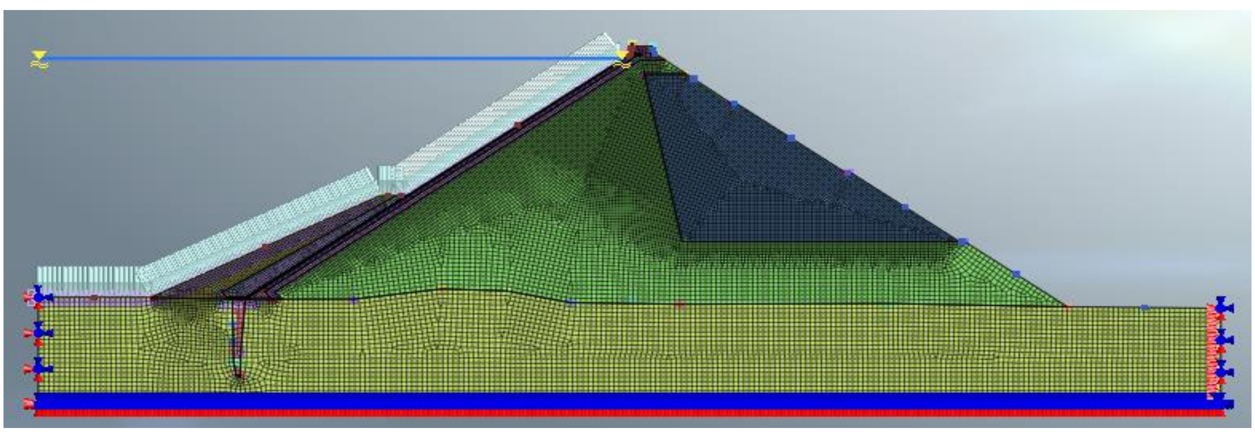

Figure 3-15 Dynamic Boundary Conditions

Two Dimensional Finite Element Analysis

The two-dimensional analyses were performed using Midas GTS NX finite element modeling and analysis program with both static and dynamic analysis included in the computation of stress and deformations at critical locations of the dam body. The analyses provided are significant to section 0+000, which is the most crucial section for the deformation and stress study of the segment of the dam based on the maximum dam height and maximum cross-section of the dam.

For static analysis, linear analysis was considered for self-weight and water pressure considering various boundary conditions such as normal water level, peak flood level, and ground constraint for the empty reservoir condition, and peak flood conditions.

In terms of linear time history dynamic analysis, dynamics, this study considered ground motion acceleration, normal water pressure as surface dynamic load, and self-weight by normal water level and fixed ground constraints boundary conditions.

The time-history analysis was primarily employed for an effective structural damping of 5% of the critical damping. The entire response is derived through discrete time step integration.

Post-Processing and Result Evaluation: GTS organizes and provides the post-data result for the design process once the analysis is performed properly. GTS has several excellent graphical results displays and animation-capturing features that are optimized for displaying large-size models and complex building stage analysis results. GTS's Result Summary function provides a report that comprises a wide range of information, from model data to analysis findings.

Limitations of the FEA model

The FEA model is a simplified representation of the dam. It is important to be aware of the limitations of the model when interpreting the results. Some of the limitations of the FEA model include:

- The model assumes that the materials are linear elastic. In reality, the materials exhibit nonlinear behavior.
- The model does not account for the effects of cracking and delamination in the concrete face slab.
- The model does not account for the effects of construction sequences.

4. RESULTS AND DISCUSSIONS

4.1 General

To observe the status and changing pattern of deformation and stress of facing rockfill dam body and concrete facing under the effect of dead load and water load during different periods, this study selects the greatest section (dam left (right)0+000) to conduct two-dimensional finite element analysis. The Finite element analysis of Genale Dawa dam GD3 (concrete-facing rockfill dam) was carried out using GTS NX by Midas IT.

4.2 Analysis Results

This section briefly presents the results of the deformation and stress analysis of the dam when subjected to the following four loading conditions, which are also presented in section 3.3.3 under "Chapter 3: Materials and Methods."

(i) Loading case-1: Dam at empty reservoir condition
(ii) Loading case-2: Dam at normal pool level condition
(iii) Loading case-3: Dam at PMF level condition
(iv) Loading case-4: Dam at normal pool level, MDE ground motion (PGA 0.12g)

Under loading cases 1 to 3, only static loads were considered for the associated reservoir conditions, whereas under loading case 4 the static loads combined with the dynamic effects of earthquake excitations were considered in both horizontal and vertical directions in the time history analysis of the dam.

The following sections describe the summarized results of the linear-elastic time-history analysis of the dam using the Midas GTS NX finite element modeling and analysis program.

4.2.1 Horizontal and Vertical Displacement

Table 4-1 below presents the summary of the maximum horizontal and vertical displacements of the dam body under loading cases 1 to 4. The deformations in the form of contour bands of maximum horizontal and vertical displacements for the corresponding load combination cases are also illustrated in Figure 4-1 to Figure 4-8 below

Table 4-1 Maximum horizontal and vertical displacements under various load combination cases

Loading Case.	Loading Condition	Horizontal Displacement	Vertical Displacement	Reference
1	Empty reservoir condition	0.594m (d/s slope, mid-elevation)	0.459m (at crest)	Figure 4-1 Figure 4-2
2	Normal pool level condition	0.446m (d/s slope, mid-elevation)	0.357m (u/s slope, 1/5th of dam height below crest)	Figure 4-3 Figure 4-4
3	PMF level condition	0.459m (d/s slope, mid-elevation)	0.386m (u/s slope, 1/5th of dam height below crest)	Figure 4-5 Figure 4-6
4	Normal pool level, MDE ground motion (PGA 0.12g)	0.605m (near crest, on u/s slope)	1.033m (near crest, on d/s slope)	Figure 4-7 Figure 4-8

As can be seen in the above table and also in figures 4-1 to 4-8 below, we note that:

1) Under loading case-1 [Empty reservoir], the maximum horizontal displacement (0.594m) occurs on the downstream slope of the dam, whereas the maximum vertical displacement (0.459m) occurs at the crest level see Figure 4-1Figure 4-2

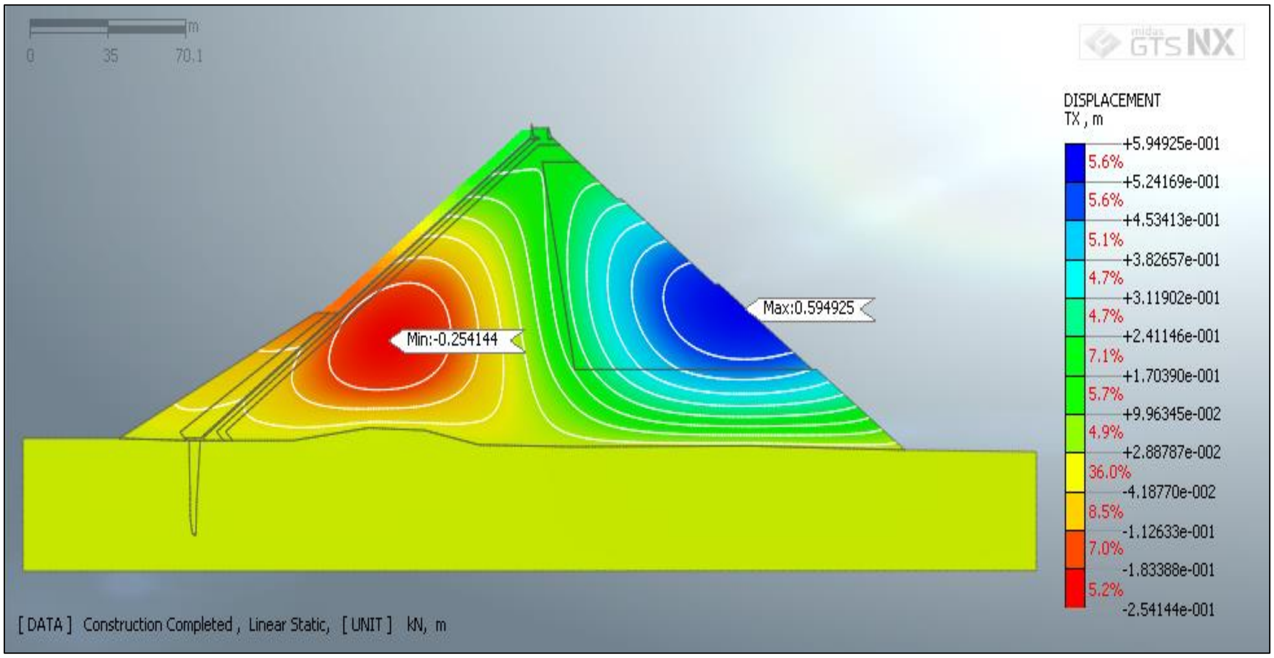

Figure 4-1 Loading Case 1(Empty Reservoir) Horizontal Displacement

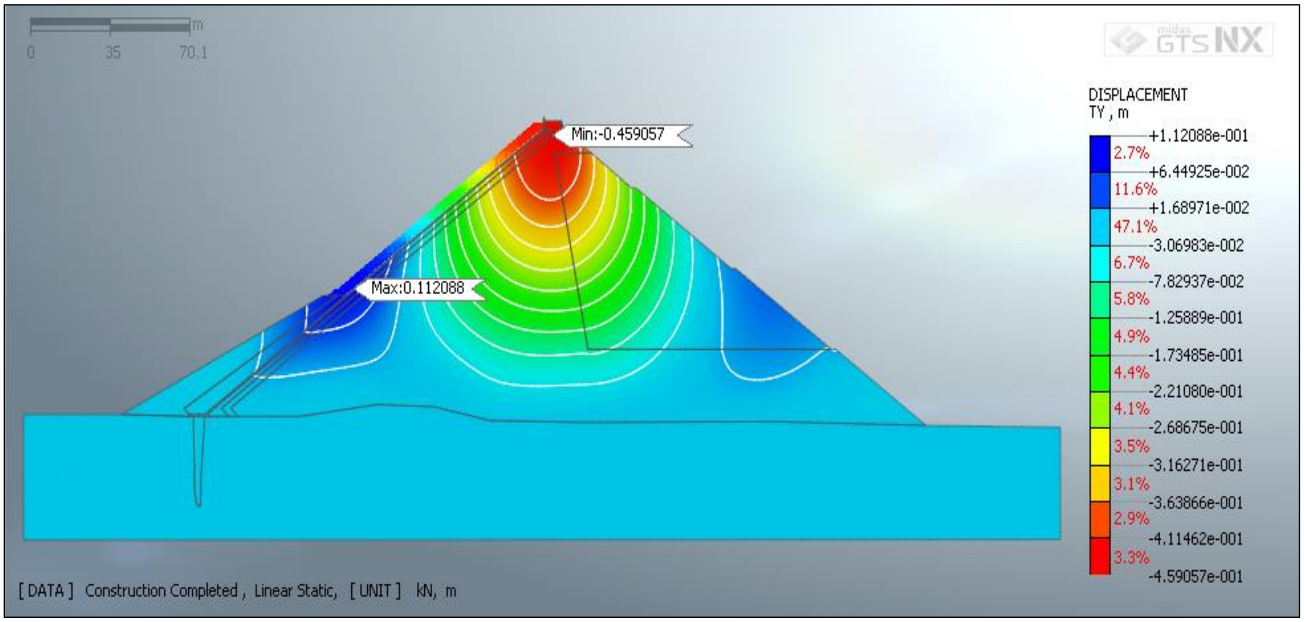

Figure 4-2 Loading Case 1(Empty Reservoir) Vertical Displacement

2) Under loading case-2 [Normal Pool Level Condition], the maximum horizontal displacement (0.446m) occurs on the downstream slope around mid-elevation, and the maximum vertical displacement (0.357m) occurs on the upstream slope around one-fifth of dam height below the crest level (see Figure 4-3and Figure 4-4).

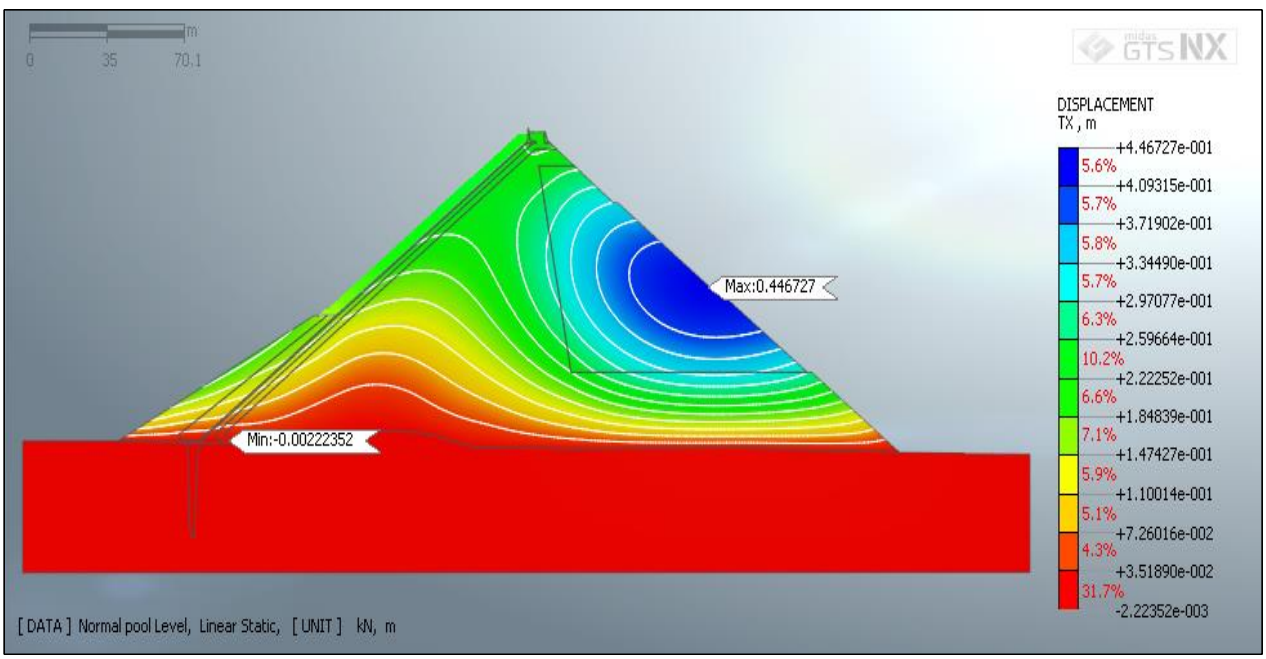

Figure 4-3 Loading Case 2(Normal Pool Level) Horizontal Displacement

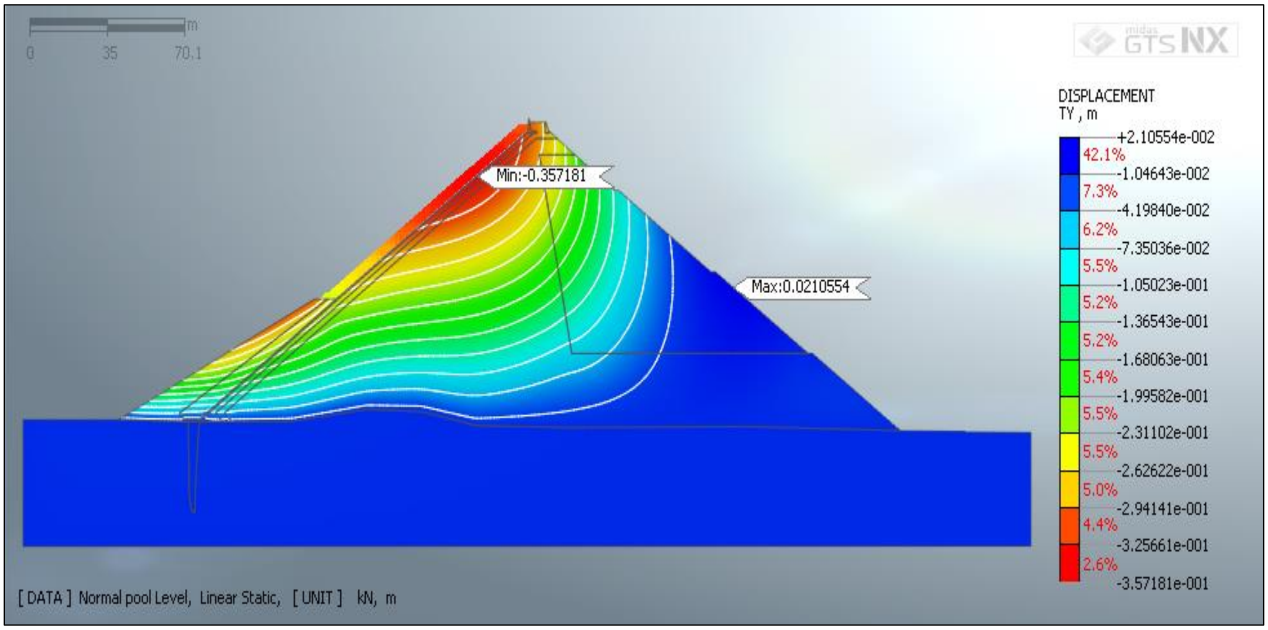

Figure 4-4 Loading Case 2(Normal Pool Level) Vertical Displacement

3) Under loading case-3 [PMF Level Condition], the maximum horizontal displacement (0.459m) occurs in the same location as that for loading case-2. The maximum vertical displacement (0.386m) also occurs at the same location of the dam as that for loading case 2. However, the estimated displacement values under this loading condition are slightly greater than those estimated for loading case-2 at normal pool conditions due to the increased reservoir loads as the headwater has increased Figure 4-5 and Figure 4-6.

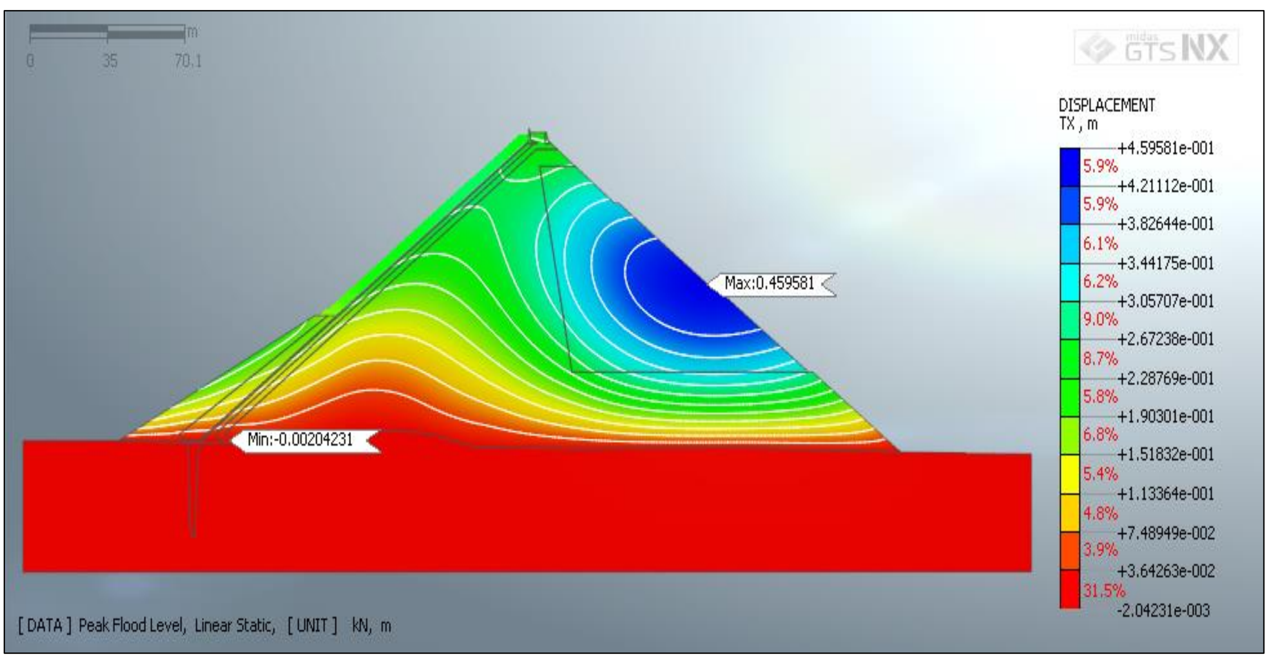

Figure 4-5 Loading Case 3(PMF Flood Level) Horizontal Displacement

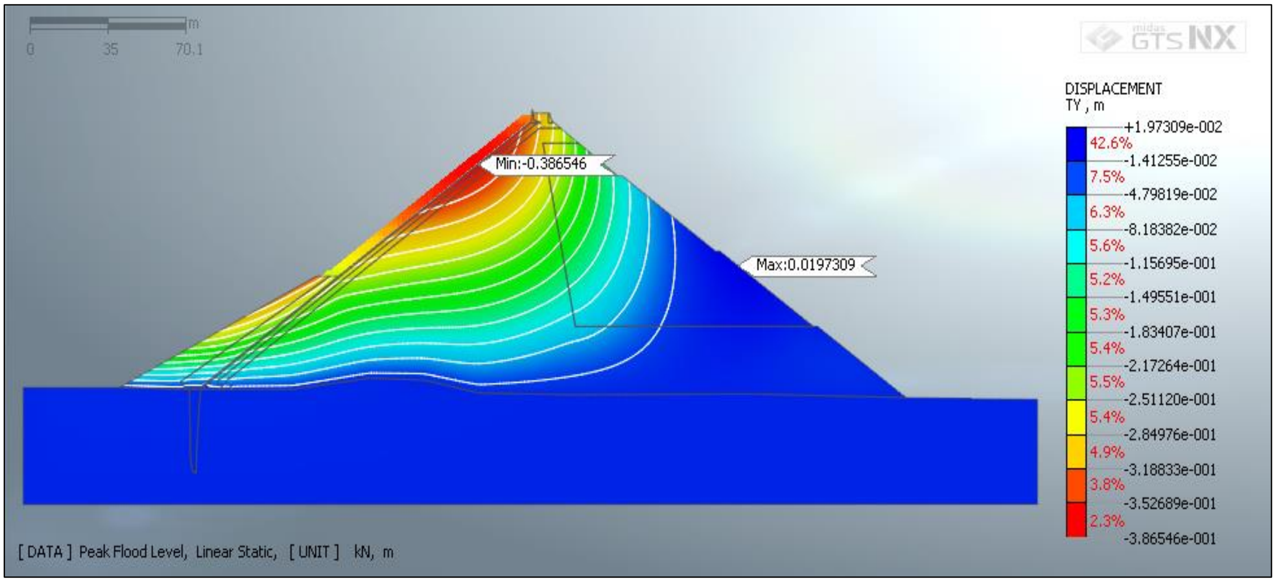

Figure 4-6 Loading Case 3(PMF Flood Level) Vertical Displacement

4) The earthquake loading condition, that is loading case-4 [Normal Pool Level, earthquake loading (MDE 0.12g)], has resulted in the absolute maximum horizontal displacement (0.605m) that occurs on the upstream slope at the crest level, and also the absolute maximum vertical displacement (1.033m) that occurs on the downstream slope around crest level (figures 4-7 and 4-8). More specifically, the vertical displacement (1.033m) accounts for 0.94% of the dam height, and it is understood that the earthquake loading condition has shown a significant increase (2.9 times) in deformation effects when compared to the values estimated for normal operating level under loading case-2Figure 4-7and Figure 4-8).

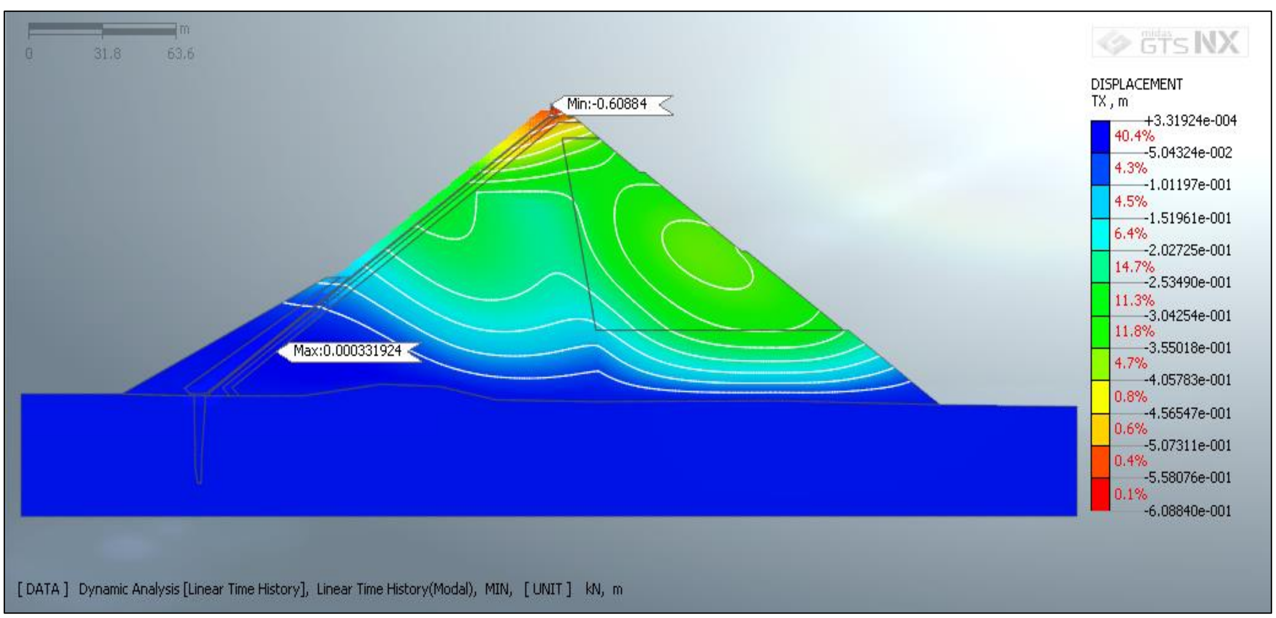

Figure 4-7 Loading Case 4(Normal Pool Level + Earthquake (0.12g)) Horizontal Displacement

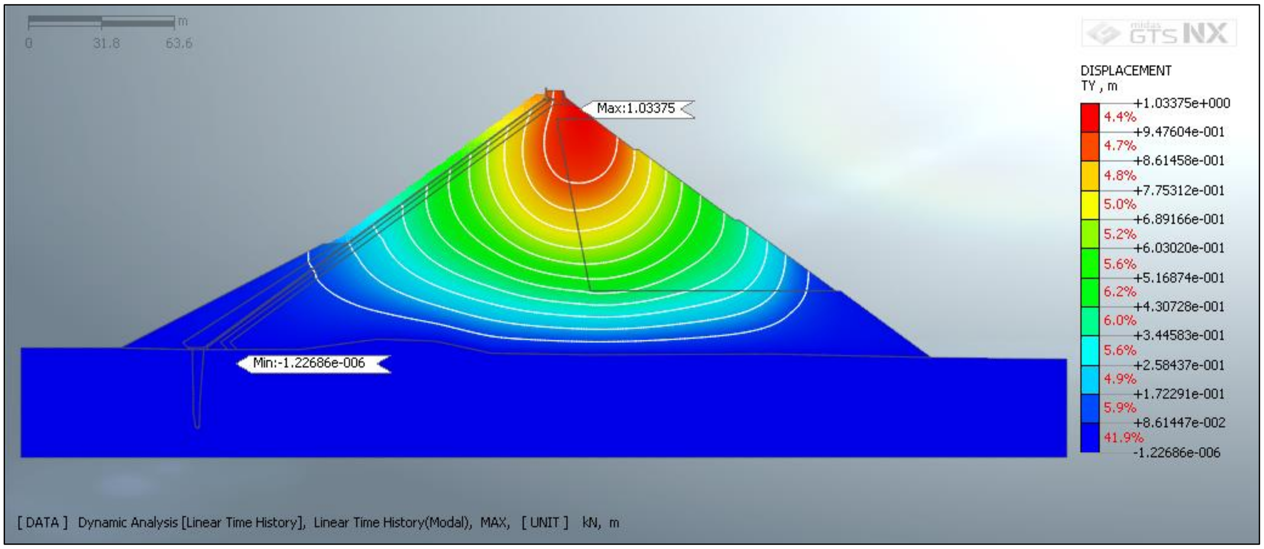

Figure 4-8 Loading Case 4(Normal Pool Level + Earthquake (0.12g)) Vertical Displacement

According to the results, the horizontal displacements of the dam are 0.446 - 0.605 m and the vertical displacements are 0.357 -1.033 m, according to the results displayed in the previous results. The displacements, both horizontally and vertically, are both less than 1% of the dam height. (The maximum vertical displacement is 0.94% and the maximum horizontal displacement is 0.55% of the height of the dam (110m). Therefore, the analysis shows acceptable results.

4.2.2 Major and Minor Principal Stresses on the Rockfill Materials

Table 4-2 below presents the summary of the major and minor principal stresses on the rockfill materials of the CRFD when subjected to loading cases 1 to 4. The principal stresses are depicted in the form of contour bands in Figure 4-9 to Figure 4-16 below

Table 4-2 Major and minor principal stresses under various load combination cases

Loading Case.	Loading Condition	Major principal stress	Minor principal stress	Reference
1	Empty reservoir condition	-1.70MPa (central bottom end of main dam body	-1.98MPa (bottom end of the main dam, slightly shifted to the downstream side)	Figure 4-9 Figure 4-10
2	Normal pool level condition	-0.96 MPa (center of dam-foundation interface)	-1.25 MPa (central bottom end of main dam body)	Figure 4-11 Figure 4-12
3	PMF level condition	-0.97 MPa (center of dam-foundation interface)	-1.26 MPa (dam-foundation interface, slightly shifted to the downstream side)	Figure 4-13 Figure 4-14
4	Normal pool level, MDE ground motion (PGA 0.12g)	+1.16 MPa (at the bottom left end corner of downstream rockfill material)	+0.53 MPa (near the central bottom end of the main dam body at the dam-foundation interface)	Figure 4-15 Figure 4-16
Note: Positive (+) values denote tensile stress and negative (-) values denote compressive stress				

As can be seen in the above table and also in schematic presentations in figures 4-9 to 4-8 below, we note that the observed results reveal that both major and minor principal stresses are showing the maximum values at the empty reservoir condition. Moreover, there are similar trends of increasing response quantities in both major and minor principal stresses developed because of the different loading conditions. Showing increasing stress magnitudes from top of the dam (crest) to the bottom of the dam (dam-foundation interface).

The results are briefly described as follows:

1) At empty reservoir condition, the maximum major principal stress and minor principal stress, respectively, are 1.7 MPa and 1.98 MPa, indicating maximum compressive stresses at or near the central-bottom end of the main dam body (Figure 4-9 and Figure 4-10).

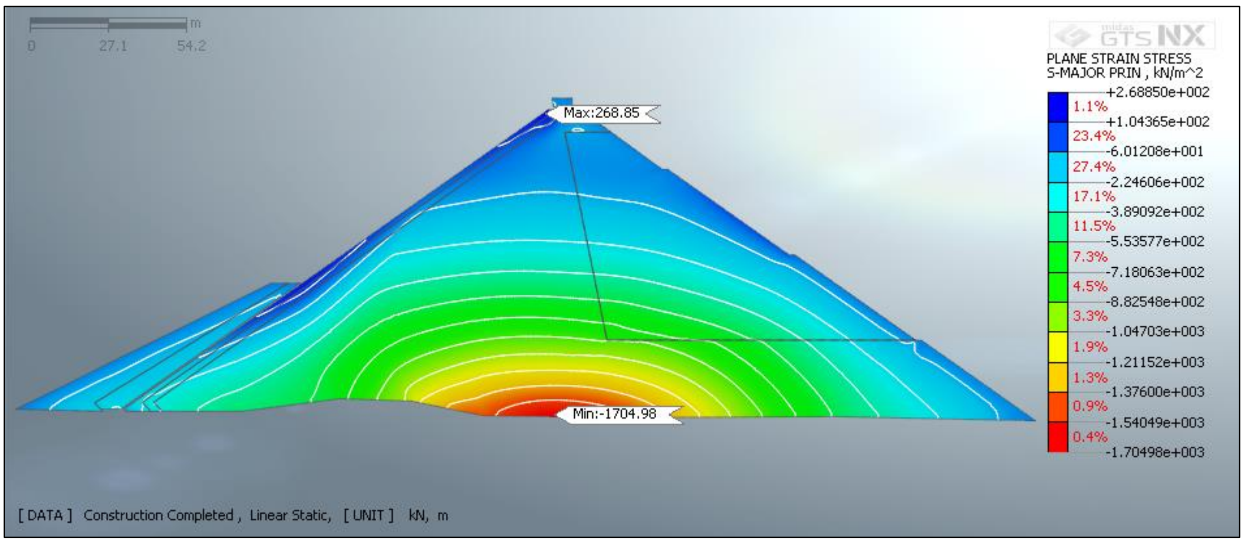

Figure 4-9 Loading Case 1(Empty Reservoir) major principal Stress

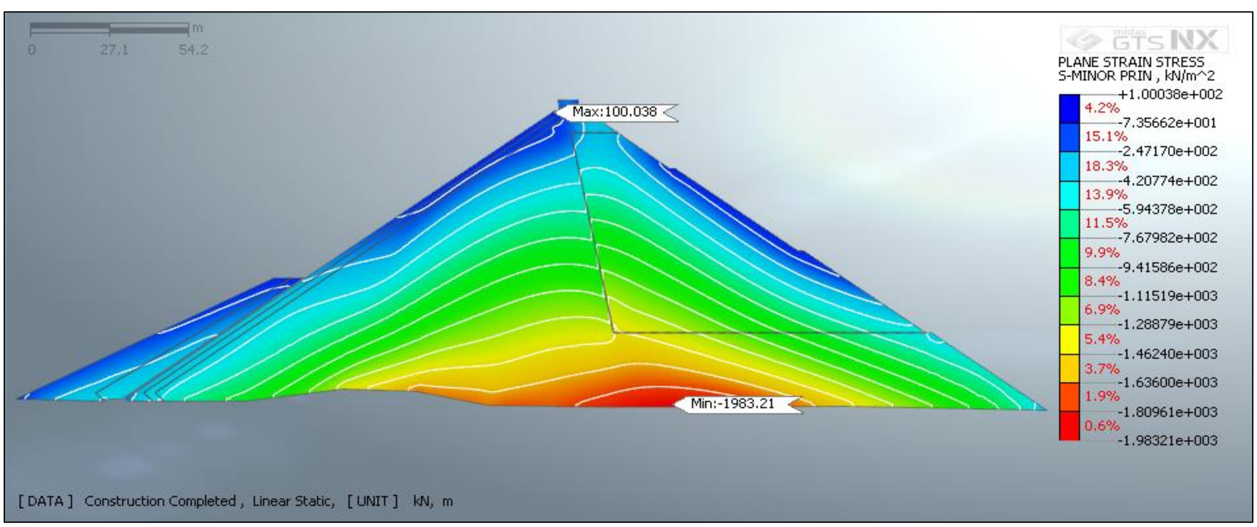

Figure 4-10 Loading Case 1(Empty Reservoir) minor principal Stress

2) At normal water level condition, the maximum major principal stress is 0.96 MPa and the maximum minor principal stress is 1.25MPa both of which are compressive stresses, which occur nearby the central-bottom end of the main dam body (Figure 4-11 and Figure 4-12).

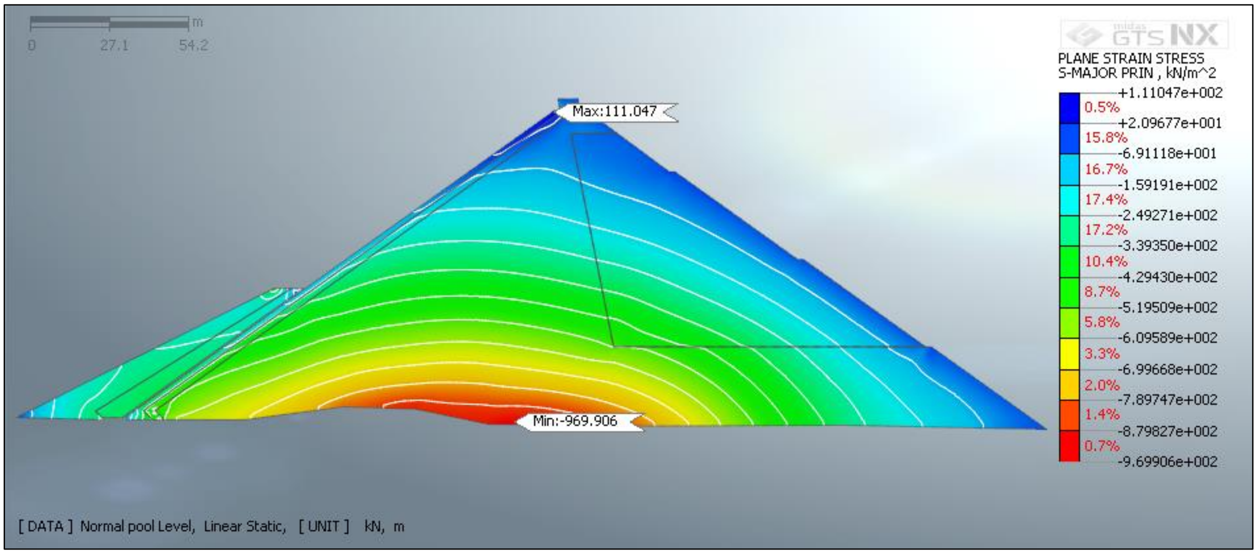

Figure 4-11 Loading Case 2(Normal Pool Level) major principal Stress

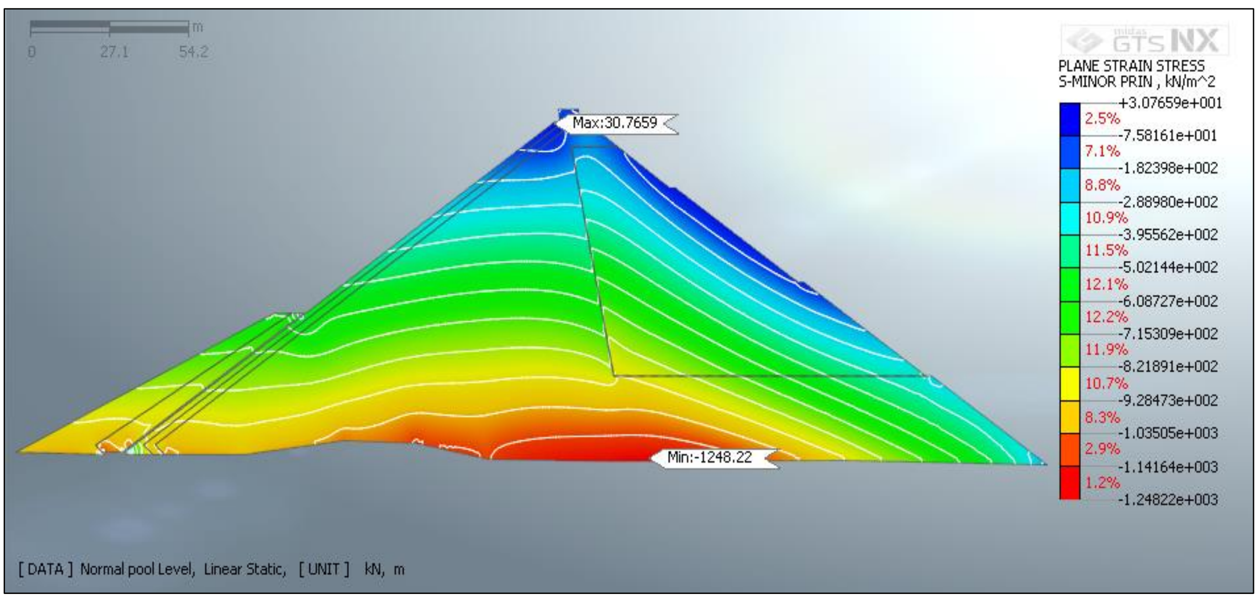

Figure 4-12 Loading Case 2(Normal Pool Level) minor principal Stress

3) At PMF condition, the maximum major principal stress is 0.97 MPa and the maximum minor principal stress is 1.26 MPa both of which are compressive stresses, which occur near the central bottom end of the main dam body (Figure 4-13 and Figure 4-14).

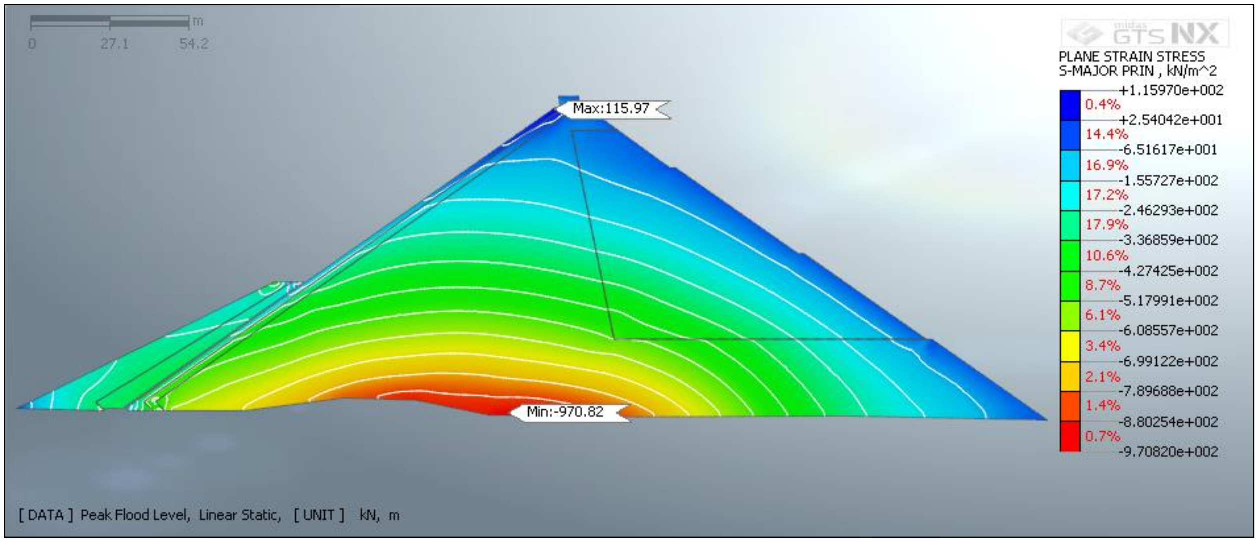

Figure 4-13 Loading Case 3(PMF Flood Level) major principal Stress

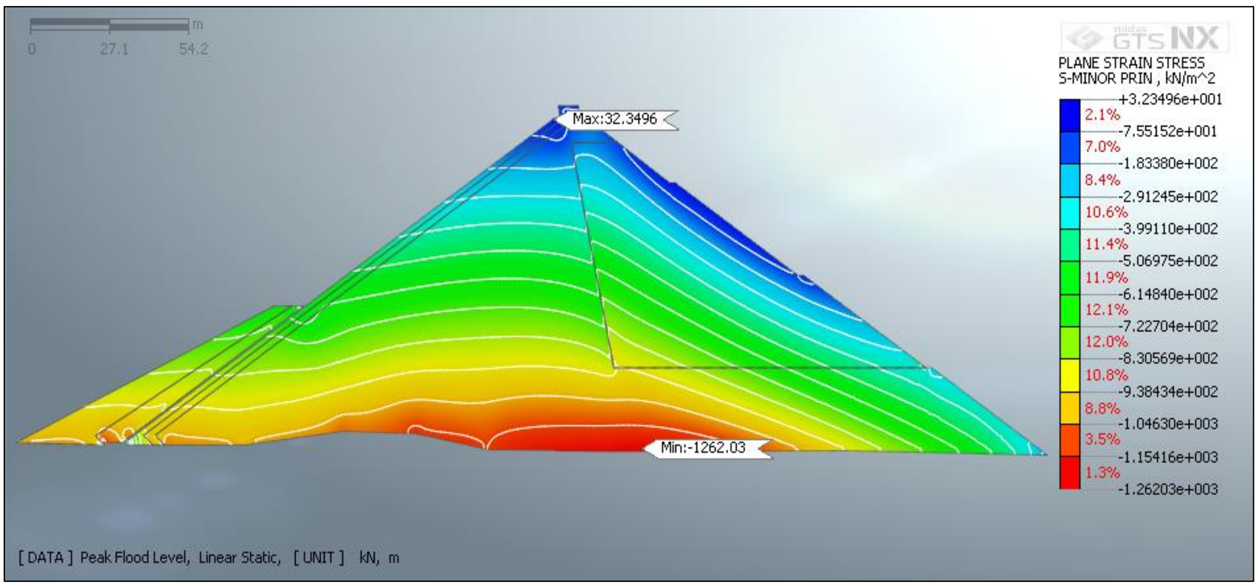

Figure 4-14 Loading Case 3(PMF Flood Level) minor principal Stress

4) At earthquake loading condition, the maximum major principal (tensile) stress is 1.16 MPa which occurs at the bottom left end corner of downstream rockfill material (i.e. at the interface between the main dam rockfill and downstream rockfill material). The maximum minor principal (tensile) stress is 0.53 MPa which occurs near the central bottom end of the main dam body at the dam-foundation interface. From this loading condition, it can be clearly understood that the maximum tensile stresses were developed at the indicated locations within the dam body during earthquake loading conditions only, because of the significant impacts of the horizontal earthquake excitations during ground shaking. Figure and Figure 4-16).

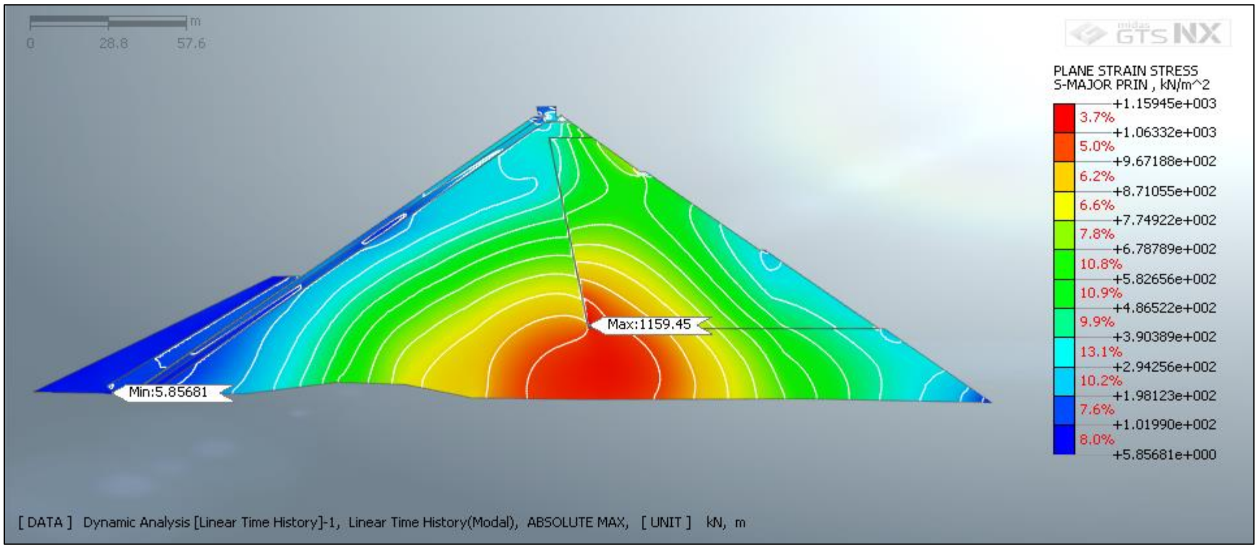

Figure 4-15 Loading Case 4(Normal Pool Level + Earthquake (0.12g)) major principal Stress

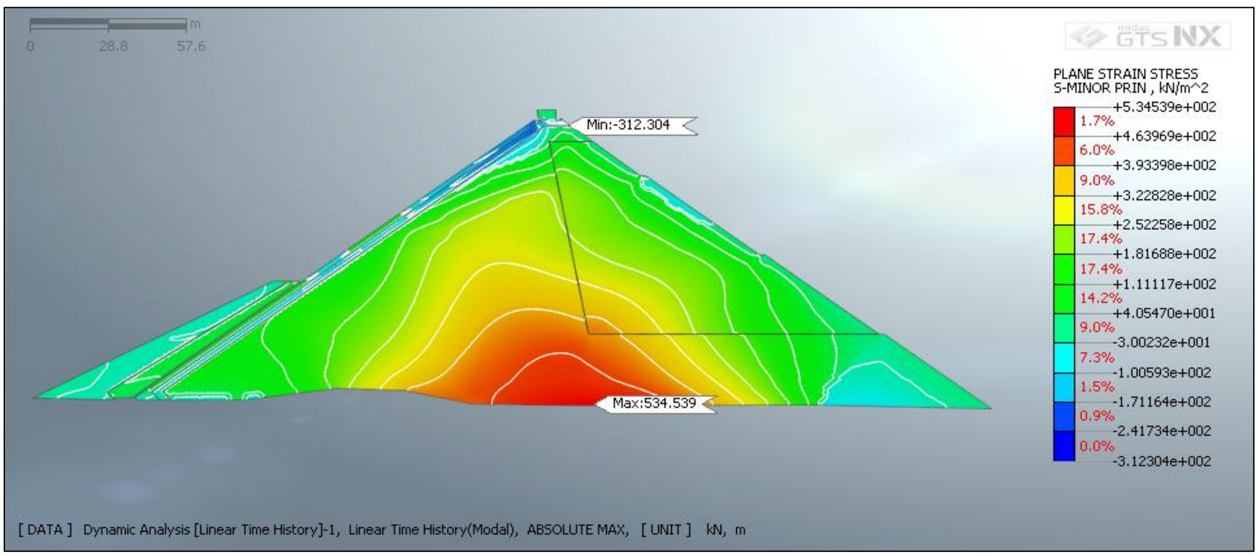

Figure 4-16 Loading Case 4(Normal Pool Level + Earthquake (0.12g)) minor principal Stress

4.2.3 Concrete Facing Displacement

Table 4-13 below presents the summary of the total displacements of the dam body under loading cases 1 to 4. The deformations in the form of contour bands of maximum total displacements for the corresponding load combination cases are also illustrated in Figure 4-17 to Figure 4-20 below.

Table 4-3 Maximum total displacements under various load combination cases

Loading Case.	Loading Condition	Total Displacement	Reference
1	Empty reservoir condition	0.497m (at crest)	Figure 4-17
2	Normal pool level condition	0.434m (u/s slope, 1/5th of dam height below crest)	Figure 4-18
3	PMF level condition	0.473m (u/s slope, 1/5th of dam height below crest)	Figure 4-19
4	Normal pool level, MDE ground motion (PGA 0.12g)	0.552m (near the crest, on u/s slope)	Figure 4-20

As can be seen in the above table and Figure 4-17 to Figure 4-20 below, we note that:

1) Under loading case-1 [Empty reservoir], at the elevation of the top of the dam crest, the slab deflects by 49.7 cm and the perimeteric joint's opening displacement is 0.137 cm, (see Figure 4-17).

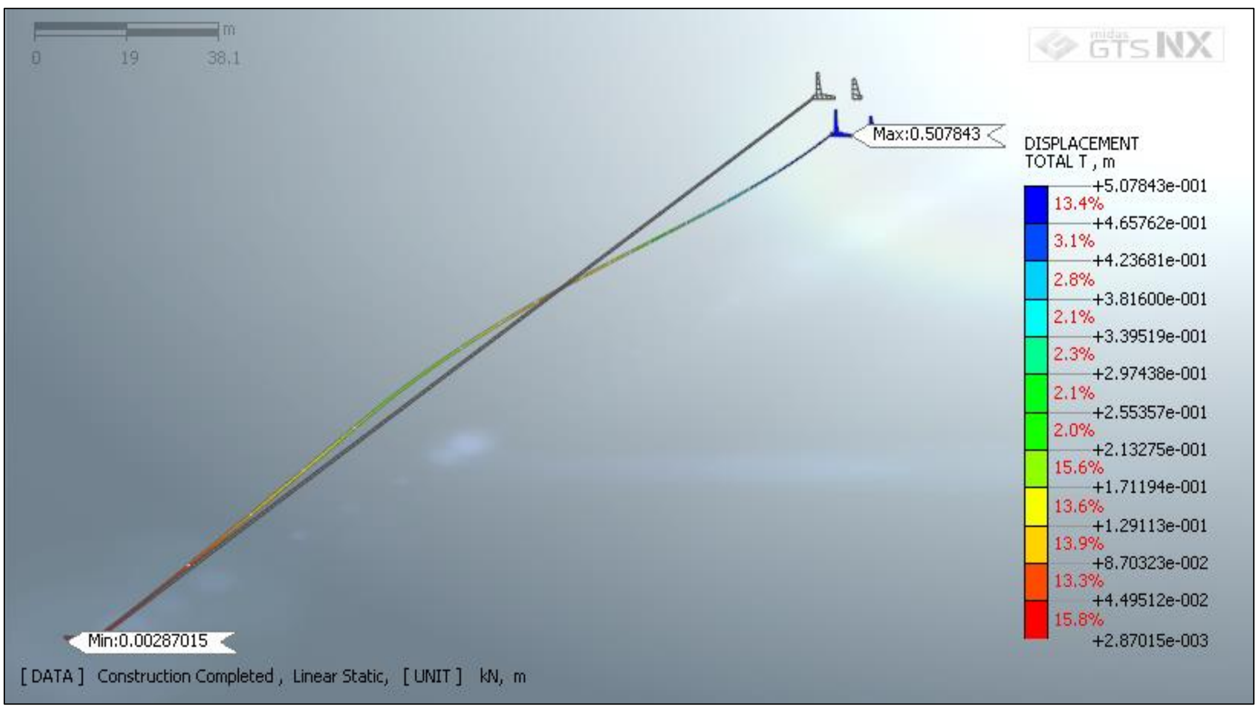

Figure 4-17 Loading Case 1(Empty Reservoir) Concrete Facing Displacement

2) Under loading case-2 [Normal Pool Level Condition], the slab deflects by 43.4 cm, which occurs on the upstream slope around one-fifth of dam height below the crest level and the perimeteric joint's opening displacement is 0.137 cm (see Figure 4-18).

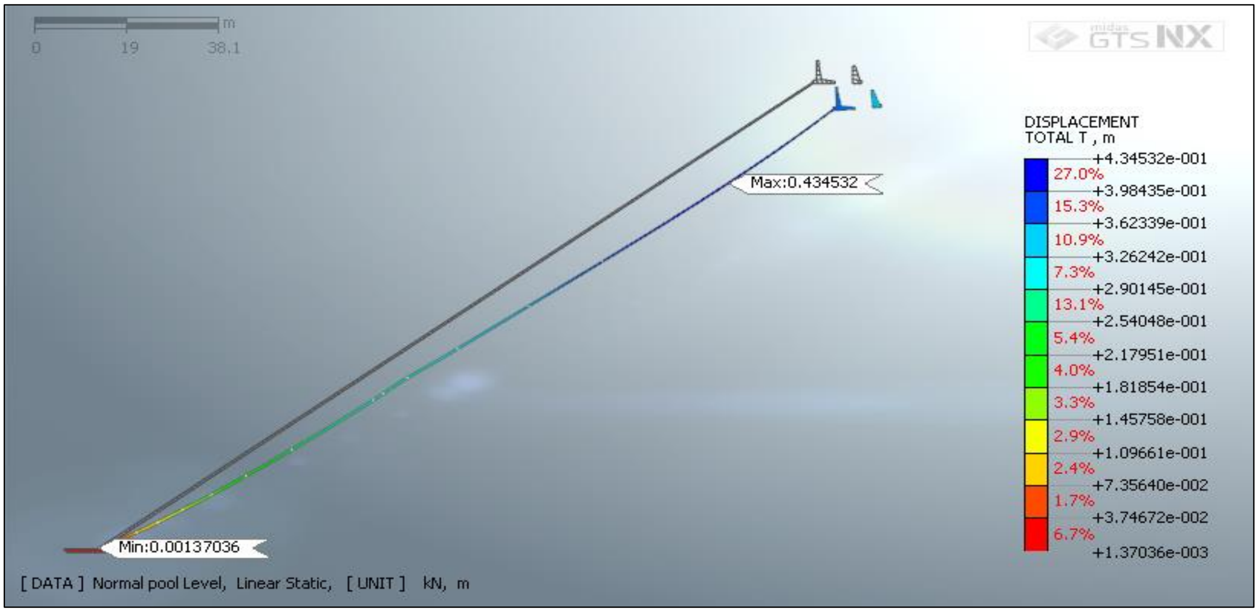

Figure 4-18 Loading Case 2(Normal Pool Level) Concrete Facing Displacement

3) Under loading case-3 [PMF Level Condition], the slab deflects by 47.3 cm, which occurs on the upstream slope around one-fifth of dam height below the crest level and the perimeteric joint's opening displacement is 0.137 cm (see Figure 4-19).

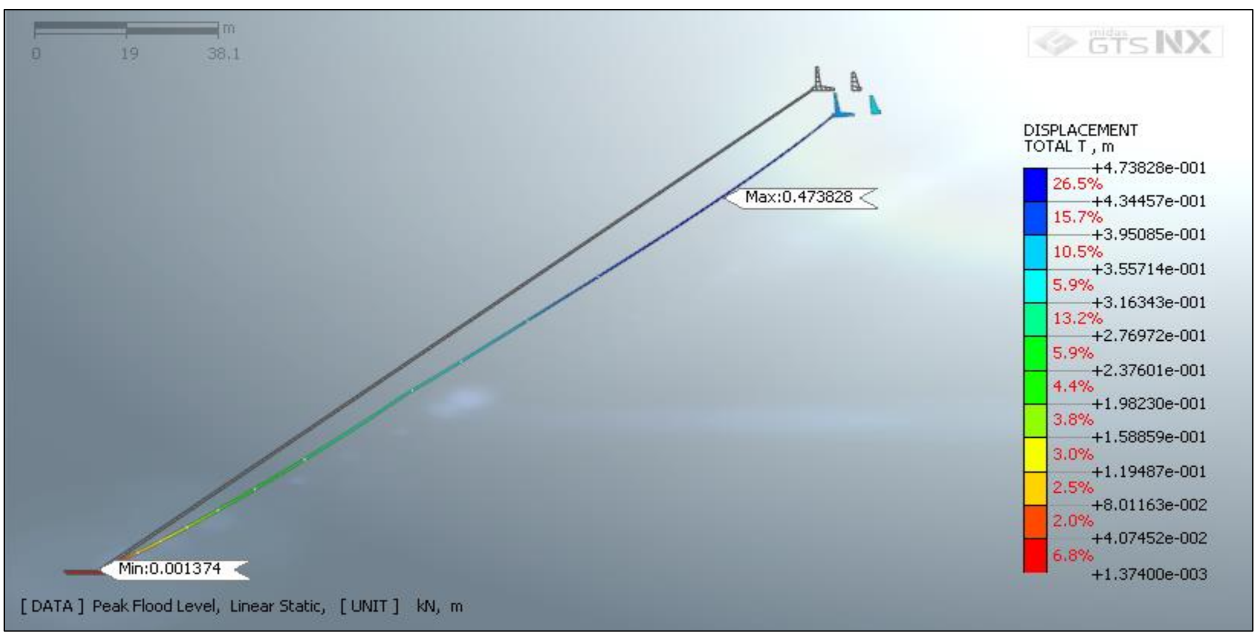

Figure 4-19 Loading Case 3(PMF Flood Level) Concrete Facing Displacement

4) The earthquake loading condition, that is loading case-4 [Normal Pool Level with earthquake loading (MDE 0.12g)], The top of the slab experiences the largest slab deflection under normal water levels with earthquake conditions, with a value of 55.2 cm. The maximum slab deformation is 55.2cm or 0.5% of the dam height, thus are acceptable results. (see Figure 4-20).

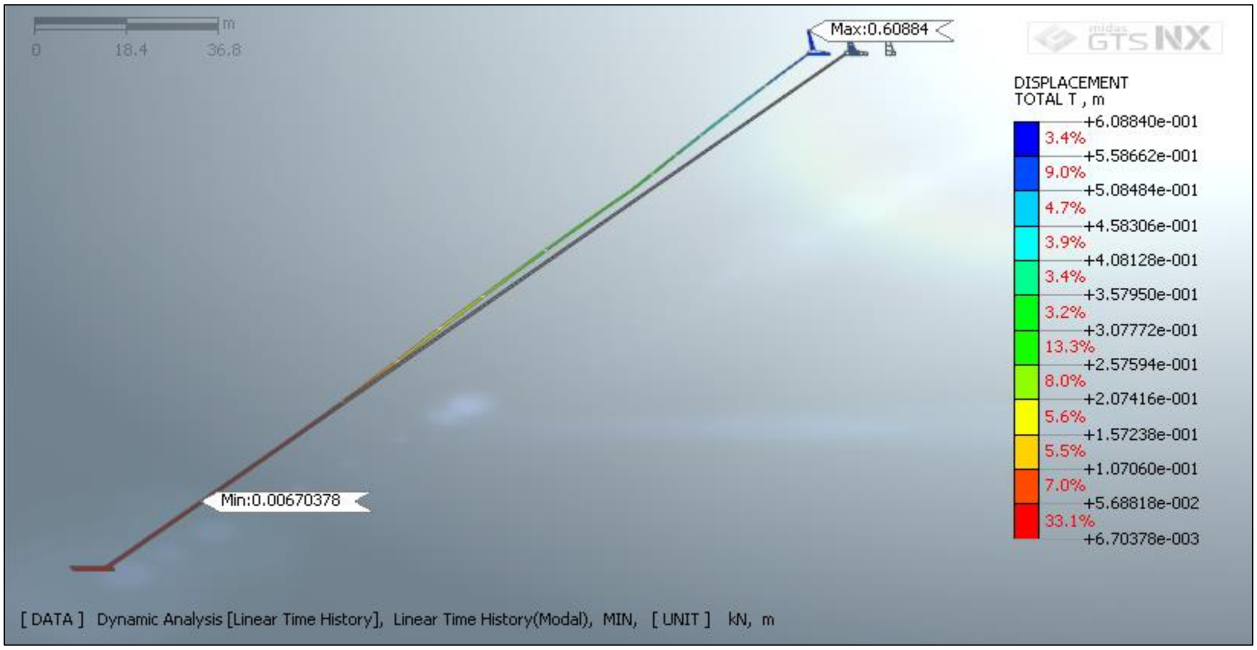

Figure 4-20 Loading Case 4(Normal Pool Level + Earthquake) Concrete Facing Displacement

Among the four loading conditions the dam has experienced maximum deformation at the fourth loading condition 55.2 cm shows that it could have happened due to seismic loading and for further investigation, this loading case is considered to be the worst.

4.2.4 Concrete Facing Stress result

Table 4-24 below presents the summary of the major and minor principal stresses on the rockfill materials of the CRFD when subjected to loading cases 1 to 4. The principal stresses are depicted in the form of contour bands in Figure 4-21 to Figure 4-28 below

Table 4-4 Major and minor principal stresses under various load combination cases

Loading Case.	**Loading Condition**	**Major principal stress**	**Minor principal stress**	**Reference**
1	Empty reservoir condition	-8.45 MPa (near the perimetric joint)	-42.12 MPa (near the perimetric joint)	Figure 4-21 Figure 4-22
2	Normal pool level condition	-4.87 MPa (near the perimetric joint)	-14.51 MPa (near the perimetric joint)	Figure 4-23 *Figure 4-24*
3	PMF level condition	-5.98 MPa (near the perimetric joint)	-13.65 MPa (near the perimetric joint)	Figure 4-25 Figure 4-26
4	Normal pool level, MDE ground motion (PGA 0.12g)	+1.94 MPa (2/3 of the dam height)	-6.24 MPa (1/3 of the dam height)	Figure 4-27 Figure 4-28
Note: Positive (+) values denote tensile stress and negative (-) values denote compressive stress				

The results are briefly described as follows:

1) Due to the empty reservoir condition, the maximum major principal (compressive) stress and minor principal (compressive) stress, respectively, are -0.58 MPa and -42.12 MPa occurred at the position near the perimetric joint. Figures 4-21 & 4-22 show the maximum stress of the concrete face occurs on the lower part of the slab (1/3 of the surface of the face slab) around the perimetric joint.

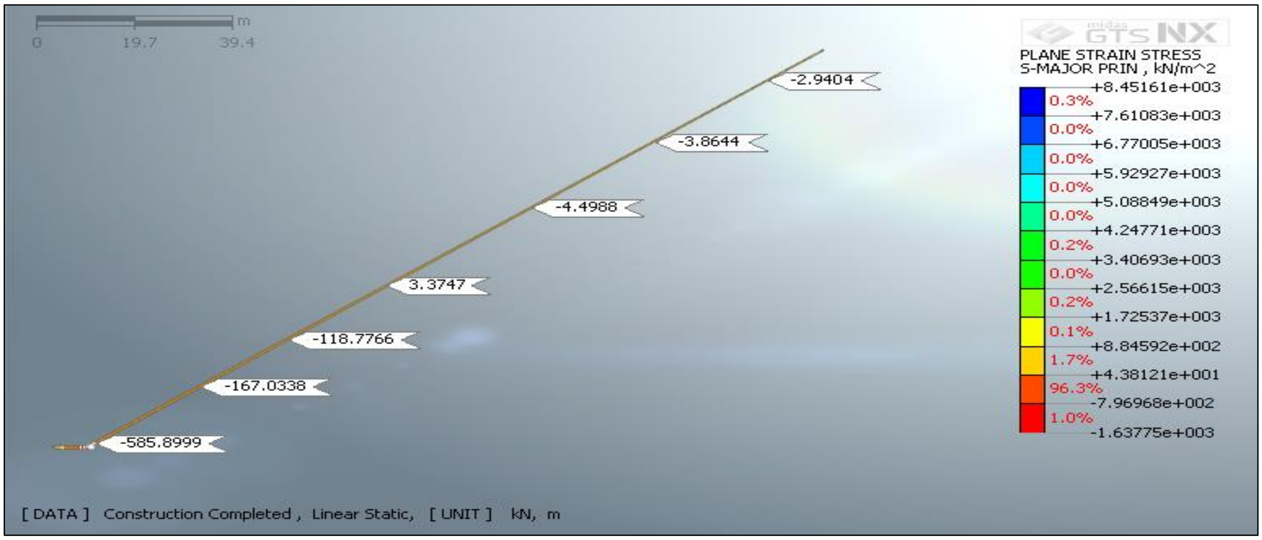

Figure 4-21 Loading Case 1(Empty Reservoir) Concrete Facing major principal Stress

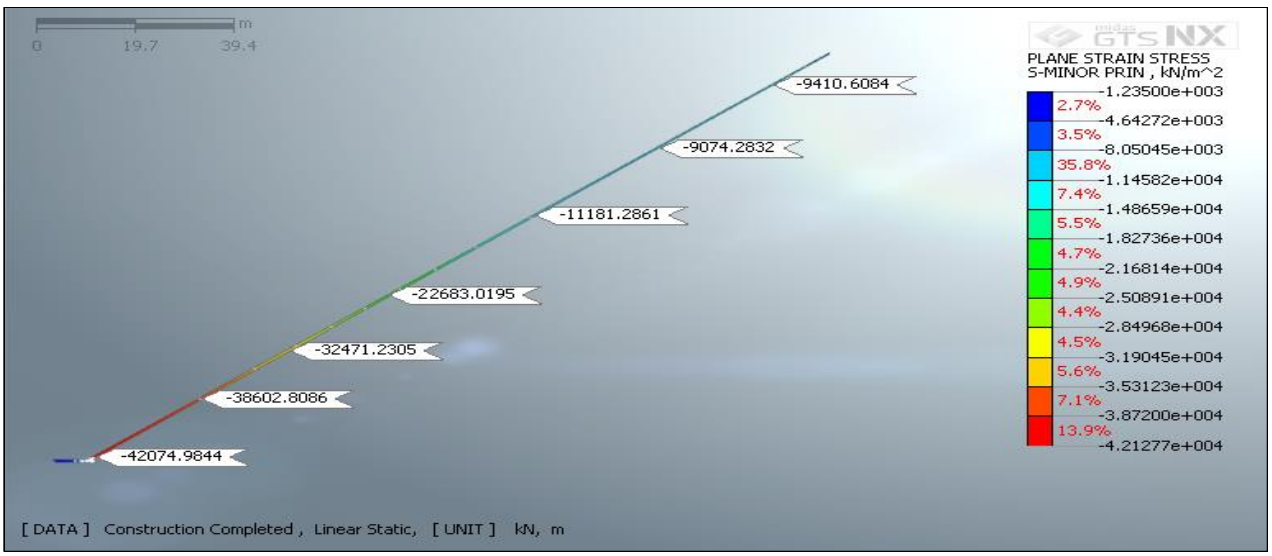

Figure 4-22 Loading Case 1(Empty Reservoir) Concrete Facing minor principal Stress

2) Under normal water level condition, the maximum major principal (compressive) stress and minor principal (compressive) stress, respectively, are -0.94 MPa and -14.5 MPa for the position near the perimetric joint, as illustrated in figures 4-23 and 4-24, and the maximum value occurs on the bottom 1/3 surface of the face slab or around the face slab.

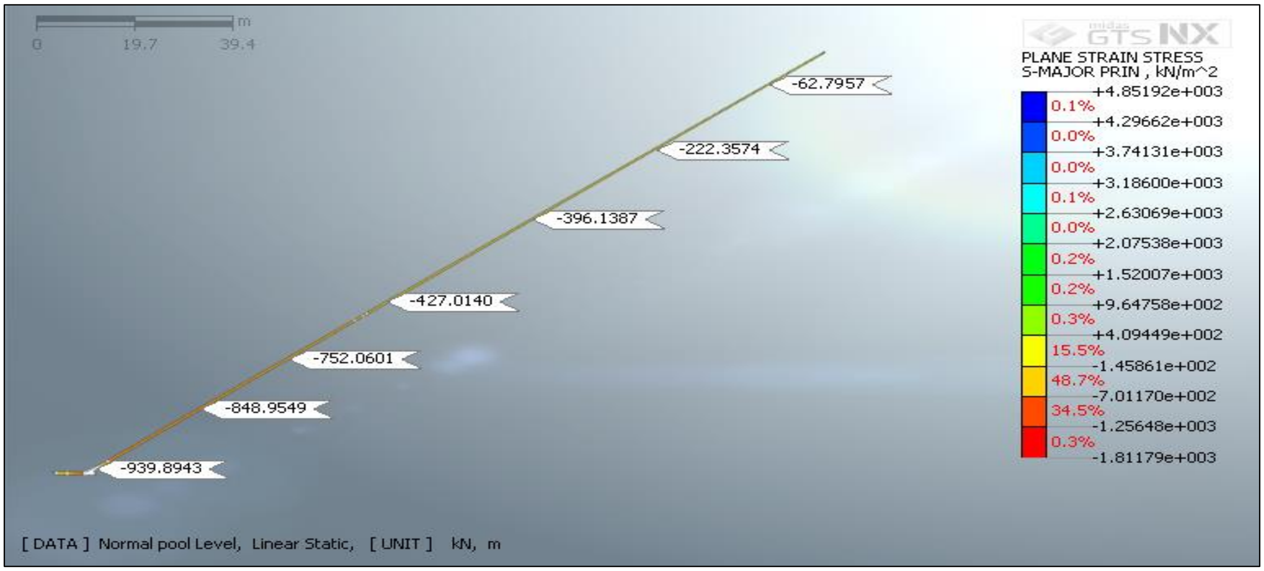

Figure 4-23 Loading Case 2(Normal Pool Level) Concrete Facing major principal Stress

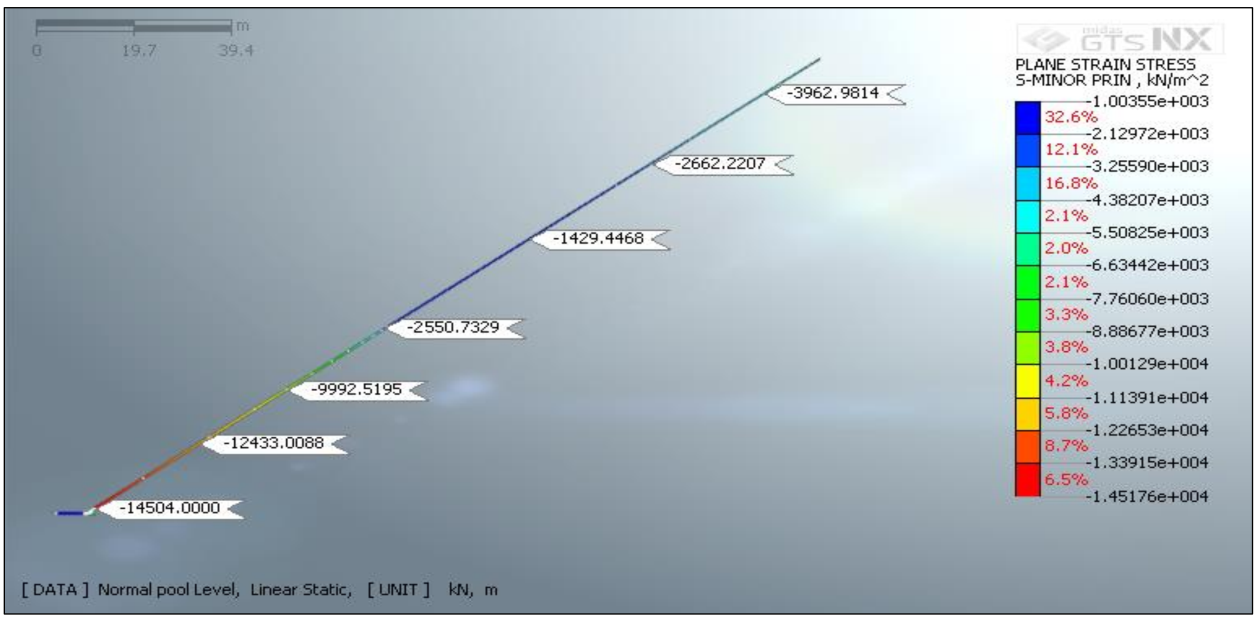

Figure 4-24 Loading Case 2(Normal Pool Level) Concrete Facing minor principal Stress

3) Under the design flood level condition, the maximum major principal (compressive) stress and minor principal (compressive) stress, respectively, are -0.87 MPa and -13.63 MPa, for the position near the perimetric joint, as illustrated in Figures 4-25 and 4-26, and the maximum stress value occurs on the bottom 1/3 surface of the face slab or around the face slab.

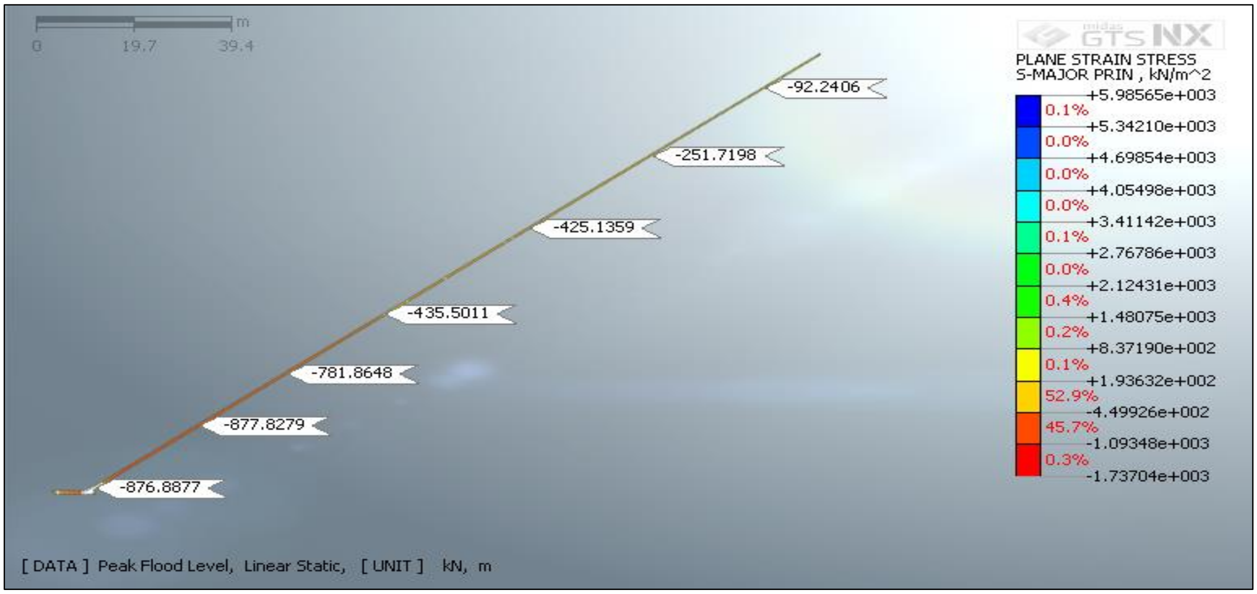

Figure 4-25 Loading Case 3(PMF Flood Level) Concrete Facing major principal Stress

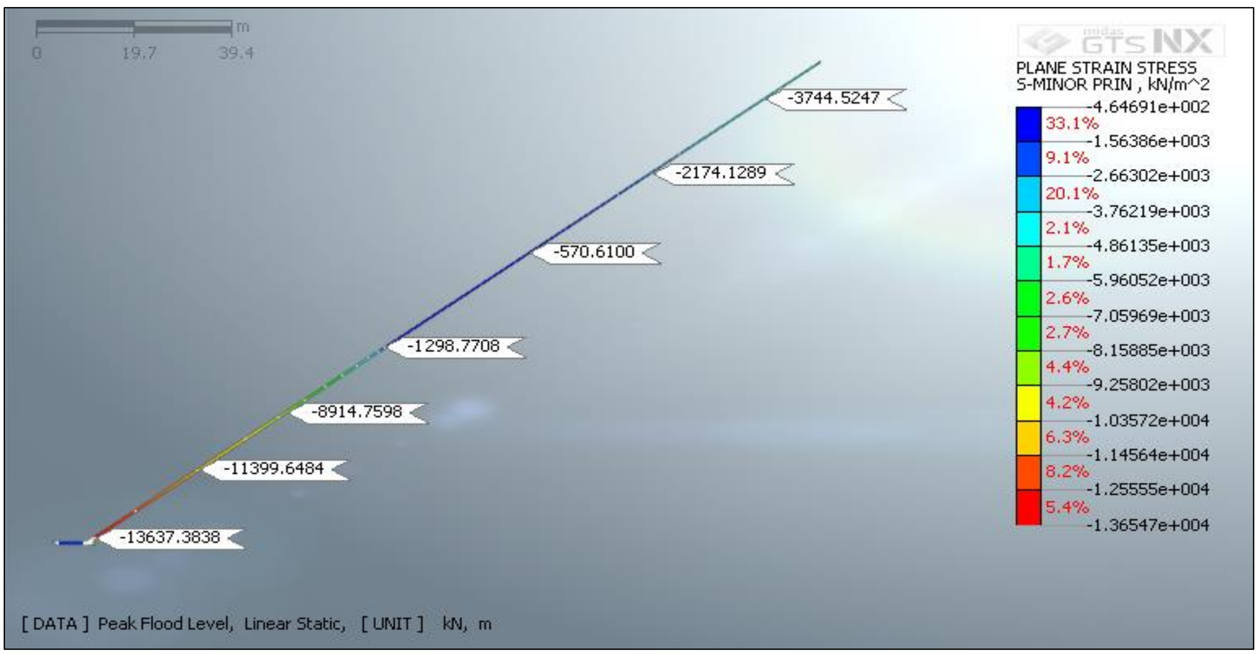

Figure 4-26 Loading Case 3(PMF Flood Level) Concrete Facing minor principal Stress

4) Under a normal pool with earthquake condition, the maximum major principal (tensile) stress is 1.94 MPa which occurs around 2/3 of the dam height. The maximum minor principal (compressive) stress is -6.25 Mpa which occurs around the perimetric joint. Figure 4-27 presents the tensile stresses on the concrete face, whereas Figure 4-28 shows the small values of tensile stress and compression stress around the lower 1/3 surface of the face slab.

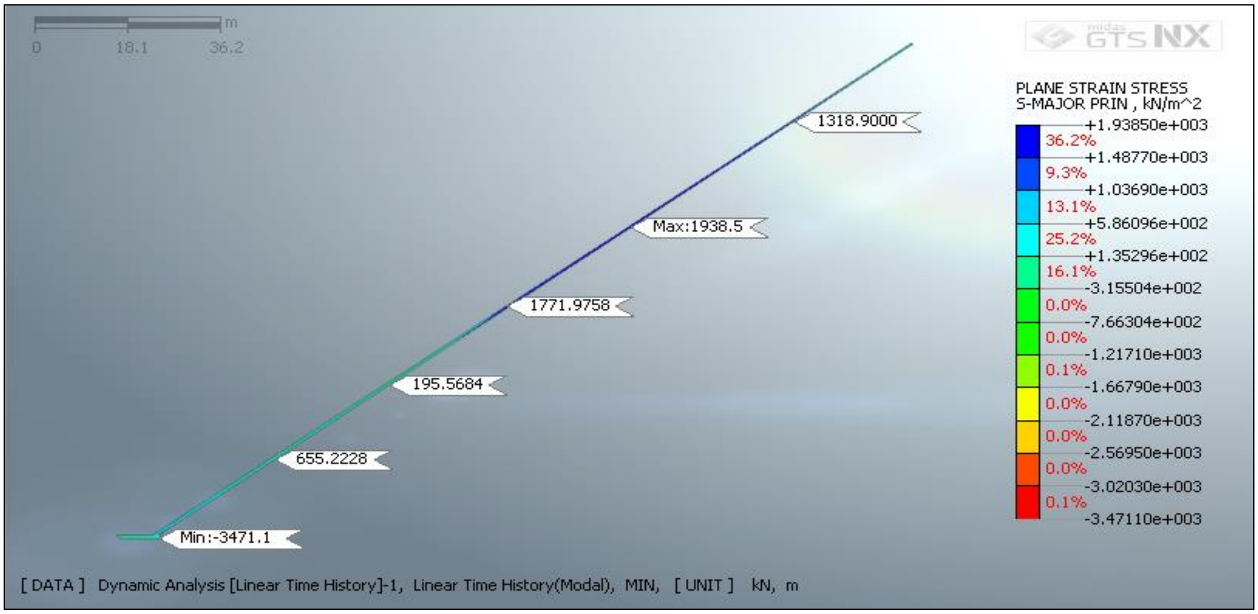

Figure 4-27 Loading Case 4(Normal Pool Level + Earthquake) Concrete Facing major principal Stress

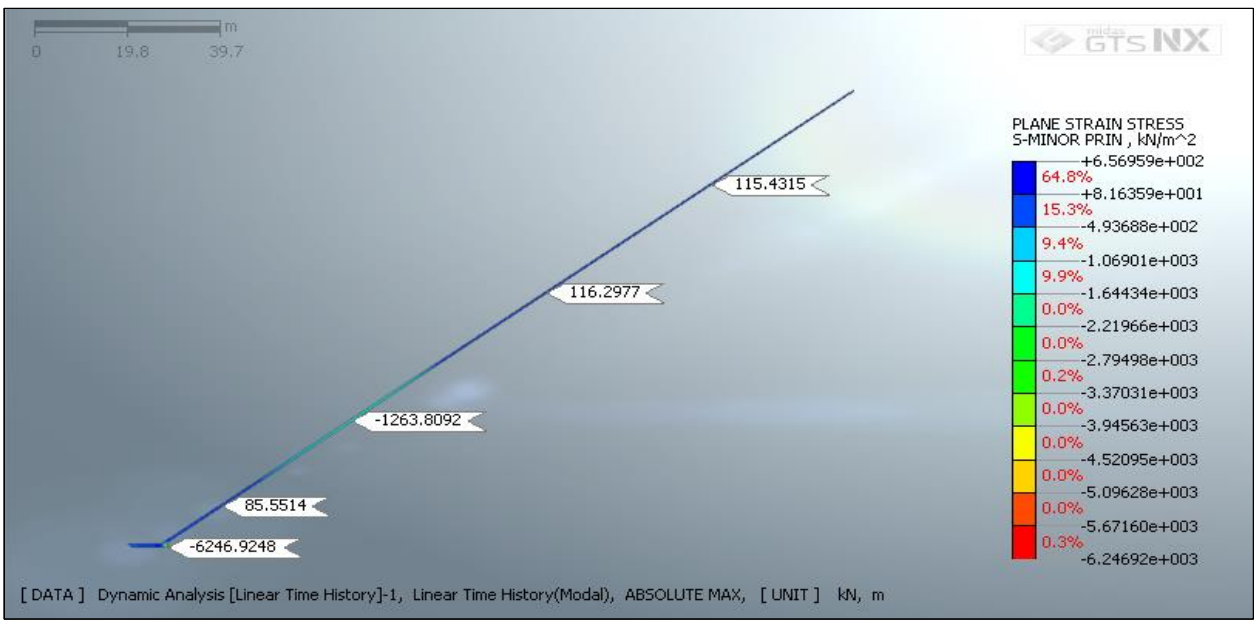

Figure 4-28 Loading Case 4(Normal Pool Level + Earthquake) Concrete Facing minor principal Stress

The study reveals that both major and minor principal stresses exhibit maximum values at the empty reservoir condition, except normal pool level with earthquake condition similar trends of increasing response quantities in both stress types due to different loading conditions. The stress magnitudes increase from the top of the dam to the perimetric joint of the dam.

4.3 Validation of the results

1. Guidelines for Design of Large Earth and Rockfill Dams, IS 8826 (1978), If rockfill was dropped, the deformation caused by embankment compression would be greater. For earth and rockfill dams, a provision of 1 to 2 percent of the embankment height above the specified top level may be made to account for both foundation settlement and embankment compression. The result obtained in Genale Dawa III dam analysis is 0.94% of dam height, which is the very profound result as compared to this design standard.
2. Runying Wang, Keping Yu 2020, Panshitou dam, the basin area of Panshitou Reservoir is 1,915 km^2, with a total storage capacity of 608 million m^3. The dead water level of the reservoir is 208.00 m and the normal water level is 254.00 m. The dam is a concrete face rockfill dam with a crest elevation of 275.70 m and a maximum dam height of 102.20 m. The finite element software COMSOL Multi-physics is used to establish the model. The result shows the maximum horizontal and vertical displacement are 0.67m and 2.01m respectively. The result obtained is similar to the Genale Dawa III dam.

Note: The maximum tensile stress at the concrete facing was 1.94MPa, exceeding the concrete's tensile strength of 1.67MPa. This suggests that the concrete's tensile capacity is insufficient in the linear-elastic range, necessitating non-linear analysis to prevent dam collapse. The maximum compressive stress was 45.12MPa, also exceeding the concrete's compressive strength of 25.3MPa, indicating the need for further non-linear analysis studies.

5. CONCLUSIONS AND RECOMMENDATION

5.1 Conclusions

This study conducted a stress and deformation analysis of Genale Dawa III Dam, the first concrete-faced rockfill dam in Ethiopia, using the Midas GTS NX 2D (2022 version) finite element software. The linear-elastic model was used for the analysis, considering four loading conditions: empty reservoir, normal operating, PMF level, and normal water level with earthquake loading. Based on the results obtained from the linear-elastic analysis using finite element modeling, the following conclusions were drawn:

1) The dam's horizontal and vertical displacements are less than 1% of its height, with a maximum settlement of 1.033cm under normal pool level with earthquake loading. The maximum horizontal displacement is 0.55% of the dam's height (110m). The deformation properties of the concrete facing and dam body are within acceptable limits, with the maximum settlement occurring at 0.94% of the dam's height and the maximum horizontal displacement at 0.55%.
2) The dam body's major and minor principal stresses are highest at empty reservoir conditions, with increasing response quantities due to different loading conditions. Stress magnitudes increase from the dam's top (crest) to the bottom (dam-foundation interface), indicating a consistent trend in stress responses.
3) The dam has experienced maximum deformation of concrete face at the normal water level with earthquake loading condition is 55.2 cm shows that it could have happened due to seismic loading and for further investigation, this loading case is considered to be the worst.
4) The concrete face's major and minor principal stresses peak at empty reservoir conditions, with similar trends of increasing response quantities in both stress types, except under earthquake conditions, due to different loading conditions magnitudes from the dam's top to the perimetric joint.
5) Under normal pool conditions with earthquake conditions, concrete faces develop tensile stress, with maximum stress reaching 1.94 MPa, occurring around 2/3 of dam height.

5.2 Recommendations

1) **The need for nonlinear analysis**

   As suggested by USBR (2006), it is recommended that nonlinear analysis be considered whenever a linear elastic analysis results in tension or shear stress values that significantly exceed the measured or anticipated tension strengths (over wide areas) because damage and stress redistribution are not taken into consideration in linear elastic analysis. In this particular study both major and minor principal stresses of concrete face have exceeded the tensile and compressive strength of concrete respectively. Accordingly, the need for nonlinear analysis considering realistic behavior of the material is recommended. The nonlinear analysis should be conducted using a suitable nonlinear material model and should consider all relevant loading conditions, such as empty reservoir condition and normal water level with earthquake loading condition.

2) **Development of national guidelines for static and dynamic analysis of embankment dams**

   Many site-specific studies for many civil engineering structures in Ethiopia are being carried out on a subjective approach that primarily depend upon the knowledge and experiences of the professionals involved in the data collection, processing, and formulation of approaches and methods of analysis. For large-scale development projects such as dams, there are no standards and guidelines to serve the professionals in the static and dynamic analysis evaluation of new and existing dams. The guidelines should be developed in consultation with key stakeholders, such as dam engineers, government officials, and academics. The guidelines should address all relevant issues, such as the selection of input parameters, the modeling of dam behavior, and the interpretation of results. To provide good ground for the upcoming projects, the development of standard guidelines for large civil engineering projects and infrastructure projects should be considered in the near future.

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