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Gravity Dams on Karst foundation

Habibullah Khan Habib

Hydropower Development
Submission date: October 2018
Supervisor: Fjola Gudrun Sigtryggsdottir, IBM

Norwegian University of Science and Technology
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Submission Date: 10-Oct-2018

Supervisor: Dr. Fjola Gudrun Sigtryggsdottir

NTNU- Department of Hydraulic and Environmental Engineering

M.Sc. THESIS ASSIGNMENT

Candidate: Mr. Habib, Habibullah Khan

Title: Dams on karst foundation

1. Background

Geological conditions at a dam site are influential for selection of suitable dam type. Adverse geological condition in a dam/reservoir foundation may lead to foundation instability, subsurface hydraulics, and/or foundation subsidence. Materials suspended, from the foundation, by subsurface hydraulics such as erosion, piping, and/or dissolution of rock (karstification), are one of a few primary causes of dam failures. These processes pose threats of both increased leakage and subsidence, ultimately progressing into failure of the dam and/or its foundation.

Concrete gravity dams rely their own weight for stability, however for this to be possible a good foundation support is essential. The main failure mechanism to consider for this dam type are overturning and sliding. Concrete dams are sensitive to foundation subsidence, and particularly differential deformations. Building a dam on foundation with karst phenomenon (cavities developed through dissolution of limestone) possess the risk of (a) large settlements (b) high seepages/leakages, and (c) initiation of failure mechanism leading to catastrophic failure if the dam is not properly designed and monitored.

A 67 m high gravity dam has been proposed at Almar site (located in north of Kabul) in Afghanistan. The dam design is challenging due to karsting problems in the foundation. At the dam site cavities, developed through dissolution of limestone, are visible at the surface. The project is currently in paused stage, due to internal conflicts in the area but its construction might soon be resumed. This dam and the site conditions is the subject of the thesis.

2. Work description

The thesis shall cover, though not necessarily be limited to the main tasks listed below. The candidate must collect available documents such as reports, relevant studies and maps regarding the Almar site. Based on the available documentation the following shall be carried out:

1. Literature review on karst foundations and concrete gravity dams as well as other potential concrete dam types.
2. Field trip to the proposed site to see the challenging conditions.
3. Identify failure mechanism due to the foundation conditions
4. Investigate through calculations, and Finite Element Analysis the stability of the dam under the conditions identified in 3.

5 Carry out a sensitivity analysis of the parameters influencing the stability and thus safety of the dam.
6 Draw conclusions from the work and provide some recommendations.

3 Supervision

Associate Professor Fjóla G. Sigtryggsdóttir will be the supervisor and provide guidance on the process of the study.

Discussion with, and input from colleagues and other researchers or engineering staff at NTNU, SINTEF, power companies or consultants is highly recommended. Significant inputs from others shall, however, be referenced to in a convenient manner.

The research and engineering work carried out by the candidate in connection with this thesis shall remain within an educational context. The candidate and the supervisors are free to introduce assumptions and limitations which may be considered unrealistic or inappropriate in a contract research or a professional/commercial context.

4 Report format and submission

The report should be written with a text editing software, and figures, tables, photos etc. should be of good quality. The report should contain an executive summary, a table of content, a list of references and information about other relevant sources.

The report should be submitted electronically as pdf-file in DAIM, and three paper copies should be handed in to the institute.

The executive summary should not exceed 450 words, and should be suitable for electronic reporting.

The Master's thesis should be submitted within 11th of October 2018.

Trondheim,

____

Fjóla Guðrún Sigtryggsdóttir
Associate Professor
Department of Civil and Environmental Engineering
NTNU

Forwards

This thesis report entitled "*Concrete Gravity Dams on Karsting Foundation*" is written in a partial fulfilment of the requirements of the Master of Science in Hydropower Development.

This work is carried out in 2018 in Afghanistan, and is completed and submitted to the Department of Hydraulic and Environmental Engineering at the Norwegian University of Science and Technology – Trondheim under the supervision of Dr. Fjola Gudrun Sigtryggsdottir.

The work presented in here belong to the author, and proper references are given at the time when other sources are used.

Thanks

Habibullah Khan

October, 2018

Kabul, Afghanistan

Acknowledgement

My study at the NTNU, and subsequently development of this report took an enormous energy at time when I was full time employed. I used to visit my family twice each semester going back from Norway to Afghanistan and then working on thesis only at the weekends and during the nights when I have already spent most of my body support in my official duty at the Ministry of Rural Rehabilitation and Development (MRRD) was a bitter experience.

So, my heartfelt thanks goes to Allah for the strength that he gives me and for the support at a time when I was at the brink of failure.

Why, so much efforts at time when I have already a master, and a job; the answer is simple, I love NTNU the most, and its standard of teaching a single subject with several world renown professors and industry highly experienced staff. The courses of Hydropower Development are nicely organized, well thought-out, perfectly taught kept me in the race of completing my studies for which I never thought at the time of joining the program.

Professors were kind, with always available support especially from my advisor Dr. Fjola is unbelievable. Her experience from the field and academia gave me the starting point of my thesis work, and always guide me towards completion of this goal. Her instruction through emails, and phone calls are all unforgettable for me.

I would also like to appreciate Faculty of Engineering for being extremely supportive during my course work and thesis completion. I always found them standing beside me, so thank you again for all the encouragement and wonderful support.

I would also like to thank my young family for leaving them in a war zone while I was gone not for war, but for bringing knowledge (a source of prosperity) to this country.

Abstract:

Foundations with dissolvable limestone possess difficult situation for the dams. Several dams around the world are either abandoned and never used such as Anchor Dam (USA) or requires continuous maintenance e.g Mosal dam (Iraq) due to the presence of karsts/cavities/caverns in their foundation. This thesis considers one such difficult dam site, the proposed Almar dam site in the north of Afghanistan and investigates how the foundation conditions may affect the stability of the 67 m high gravity dam proposed for this location.

The Almar dam site has visual signs of karst cavities at its abutments, and the foundation consists of weak limestone/dolomite, gypsum and chalk and are considered as stiff soil during the stability analyses conducted as a part of this work. Gypsum ($CaSO_4{:}2H_2O$) which is precipitated limestone have low strength and are prone to large settlement when soaked. Fifteen boreholes of various depths were drilled at the dam site, and borehole no A10 near the toe of the dam show existence of possible karst cavity. Due to the remoteness of the dam site, and security risks incomplete and insufficient subsurface investigations were carried out, and many parameters values in this analysis were taken from reference studies.

The dissolution rate of limestone of 141 mg/litre was found using Roque's curves, which can result in reduced contact surface between the dam and its foundation. To initiate failure, the contact width of dam body with its foundation must reduce from its designed based-width of 53.3 m to 37 m.

Induced stresses at different depths within the foundation were estimated to check if they would cause collapse of hidden karst cavities. For example, At a depth of 16 meters below the base of the dam, the induced stresses from this 67 m high gravity dam would be 705 KPa (750 kPa as measured by Plaxis-2D).

The foundation was further evaluated for (a) Bearing capacity failure, (b) Settlement. Bearing capacity was found to be 6,581 KPa as per Terzaghi method, and 6,106 KPa (Vesic method) assuming no cavities. FEM analysis also shows no bearing capacity failure for the empty dam except initial settlement of 2 cm. Classic method were then used to estimate total settlement of 573 mm by dividing the foundation strata into 5 layers. FEM shows foundation settlement of 600-700 mm and the crest of the dam moved downstream by 1.25 meters after the partial failure of the cavity at the toe.

Sensitivity analysis of design parameters e.g Cohesion, Friction angle etc, were carried. $F.S_{sliding}$ significantly increased when cohesion of the foundation material is increased from 600 KPa to 1 MPa, but has no effects over $F.S_{overturning}$. Similarly, tensile stress were developed at the toe of the dam when the dam is overtopped.

As a Conclusion, structural integrity of this 67 m high concrete gravity dam will be at risk as were found by this study if a settlement of 573 mm happens or if the cavities in the foundations fails suddenly, and thus requires active surveillance program.

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1. Introduction

1.1 Background

Several factors such as dam site, foundation and abutments rock (geology), construction material availability (in a distance of 2-4 km), required safety level, cost, and site accessibility decide on the selection of a particular dam type.

Concrete dams typically requires strong geological formations in its foundation, i.e good quality rock with adequate strength (compressive, shear strength, and bearing capacity mainly). Hence, Geology is the key as it controls engineering and hydrologic (seepage, leakages) conditions at the dam site, and if geology is properly understood then all subsequent work can be performed with more confidence including design, construction, remediation, modelling and long term operation of the dams *(Richard C. Benson. et. al).*

Foundation stone with karst phenomenon poses a unique challenge to the stability through (a) settlement and (b) excessive leakage through interconnected cavities/channels and crakes. Several dams around the world constructed on such foundations have either been abandoned or never used such as Anchor Dam (USA) or requires continuous maintenance e.g Mosal dam in Iraq. This thesis considers one such difficult dam site, the proposed Almar dam site in the north of Afghanistan and investigates how the foundation conditions may affect the stability of the 67 m high gravity dam proposed for this location.

Almar dam site surface geology (Figure-1) has visible cavities. Foundation rock consists of limestone, marlstone, dolomite, and gypsum. An underground exploration (borehole-A10) shows 50 meter deep cavity filled with riverbed material. Similarly, rock fall is common at the dam site showing weak geological formations.

Several uncertainties are involved in concrete dams built on weak rock with karsts. It is difficult to predict rate of settlement, rate of dissolution and thus seepages through cavities adding to the challenges such as selection of dam type, design, construction, and foundation treatment as were seen in Mosal Dam of Iraq.

Predicting dam and foundation behaviour at the project design time is extremely important to prevent excessive design modifications once it is approved. In Afghanistan, up to 25% changes are allowed in a design after its approval, and if needed changes are more than it then project needs to be re-advertised. If detail sub-surface investigations are carried out, and engineering properties are known, then there exists methods/relationships, and computer softwares to predict settlement in the foundation which helps in preparation of accurate designs.

Figure-1. Almar Dam site. Limestone dissolution cavities at the surface

In this study several well-known theoretical methods, and a computer program known as PLAXIS-2D are used to predict foundation response with respect to loading, and stability by estimating settlement, and seepages through subsurface strata.

1.2 Objective of the study

Construction of dam, and its associated lake can saturate the underlying limestone at Almar site, and under the increased hydraulic head it can cause large settlement/collapse associated with caverns made through slow dissolution of limestone and marlstone into gypsum *(Tod Jarvis, 2003).* Hence, the intension of this study is to determine if the concrete gravity dam would be a good option to absorb the estimated settlement as a result of full or partial collapse of karst cavities; if not then which other type of dam would prove to be a better option.

1.3 Scope of study.

In this study the following activities will be carried, and the results will be compiled to see if conventional concrete gravity dam would survive the caverns associate hazards.

i. To review relevant literature on different types of soluble rocks and their associated risks
ii. Detail review of shallow, and deep foundations literatures to estimate extent of stress zone, settlement, shear strength, and bearing capacities of limestone (dolomite), and marlstone under 67 meter high Almar Gravity Dam
iii. Stability analysis of conventionally vibrated concrete (CVC) concrete gravity dam constructed on soluble rocks
iv. Sensitivity analysis of design parameters
v. Use of Finite Element software of Plaxis-2D to compare and augment step (ii) handmade calculations

1.4 Structure of Thesis

Each of the activity mentioned in the scope has a separate chapter beside some additional information to cover the topic in detail.

1. Chapter 1 is about the general information of the dam's foundation, and scope Of this study
2. Chapter 2 is about the literature review mainly about the weak foundation Such a weak limestone, dolomite, gypsum, and their dissolution rate
3. Chapter 3 is on concrete dams in general
4. Chapter 4 is about the Almar dam project site
5. Chapter 5 is about the stability analysis
6. Chapter 6 is about the use of FEM
7. Chapter 7 is about the conclusion and recommendations

1.5 Limitations

Due to security risks, and ongoing war in Afghanistan, not too many good engineering companies participate in the bidding process of large hydropower and irrigation projects. This site is currently under the high security risk, and project work is paused for some time. Therefore, most of the required sub-surface investigations are poorly, and insufficiently carried out, and there exist gapes in the data availability, and its integrity is always under the question mark for example bringing of undisturbed samples for geotechnical tests from this site is a challenge. So, this report is based on limited data, and there exists a potential of further improvements in this work which can be used as tool in making decision in the selection of type of dam, its successful completion and in extending the useful life of the project.

2. Soluble Rocks

In this chapter soluble rocks will be introduced along with limestone dissolution and geographic features supporting dissolution process. Furthermore, conditions at the Almar dam site are brought into the discussion, and the potential dissolution at the site is evaluated.

2.1. Soluble rocks-general

Soluble rocks are also known as carbonate rocks, and are composed of minerals characterized by the presence of CO_3^{2-} ion. The most common types of these mineral are calcium carbonate, Ca^{2+} CO_3^{2-} known as Calcite, and Aragonite, and Calcium and Magnesium Carbonate $(Ca, Mg)^{2+}$ CO_3^{2-} also known as Dolomite.

Carbonate rocks comprise a vast family including limestones, marls, and dolomites and makes up to 20% sedimentary rocks.

Table-1. *Carbonate rocks (adopted from karstology by Eric Gilli et. Al)*

Rock type	Pure Limestone	Marly limestone	Marl	Calcareous clay	Calcareous sandstone	Conglomerate
$\%CaCO_3$	>95%	95-65%	<65%	<50%	Variable	Variable

2.1.1. Limestone

Limestone can contain up to 65% $CaCo_3$, and are immensely variable in its chemical nature, and has several types such as crystalline limestone, mudstone, packstone (grains-supported, with pore spaces filled by micrite), grainstone (pore space filled with sparite), and boundstone. Both, micrite, and sparite have cementitious characteristics.

2.1.2. Chalk.

Chalk is a soft porous rock consisting of more than 90% of $CaCo_3$ and is always exists in shallow depositions.

2.1.3. Dolomitic Rocks

This is a family of carbonate rocks containing a significant proportion of dolomite (Magnesium carbonate) beside others carbonates from the family of soluble rocks. In outcrops, these rocks often look like elephant skin. When changed by weathering, dolomites result in a characteristic ruined-castle-like topography. Dolomite are less soluble than calcite. The dissolution of $CaCO_3$ in a dolomitic rock results in voids that may give the rock a very high porosity.

Table 2. *Dolomitic rocks.*

Rock type	**Limestone**	**Dolomitic limestone**	**Calcareous clay**	**Calcareous sandstone**	**Conglomerate**
$\%(Ca,Mg)CO_3$	>95%	95-65%	<50%	Variable	Variable

2.2. Limestone Dissolution:

Three primary agents of karstifcation are (a) Rock (b) Water, and (c) Carbon Dioxide (CO_2). The latter plays an important role in the overall dissolution reactions for limestone. Cavities are formed by repeated scouring as result of limestone dissolution and draining. In hot weather, and basic-environment (with pH>7) the precipitation settle down and makes another (weak) rock type called evaporates such as Gypsum. Large cavities are unusual in Gypsum as it collapse in on themselves, but its dissolution rate is ten time faster than limestone. The top few meters below the river overburden in Almar is excavated with backhoe excavator, and the strata behaved like stiff soils.

The two crystalline forms of calcium carbonate, calcite and aragonite, are water-soluble only in small amounts: 14 mg.L–1 at 25°C, however, other chemical mechanisms involving CO_2 allow a considerable increase in the amount of $CaCO_3$ dissolution.

With introduction of CO_2 from the environment (0.03 – 0.05%), calcium bicarbonate forms and is ten time more soluble in water than calcium carbonate, and at room temperature up to 300 mg of $CaCO_3$ can be dissolved per litre of water. The overall dissolution reaction is shown below.

$$CaCO_3 \longleftrightarrow Ca^{2+} + CO_3^{\ 2-}$$

$$CO_2 + H_2O \longleftrightarrow HCO_3^{\ -} + H^+$$

$$HCO_3^{\ -} + H^+ + Ca^{+2} + CO_3^{\ 2-} \longleftrightarrow (HCO_3)_2\ Ca^{2+}$$

At Almar, the pH of the water is 7.8, and at that level, Rogues curves shows a dissolution of 140 mg L^{-1}.

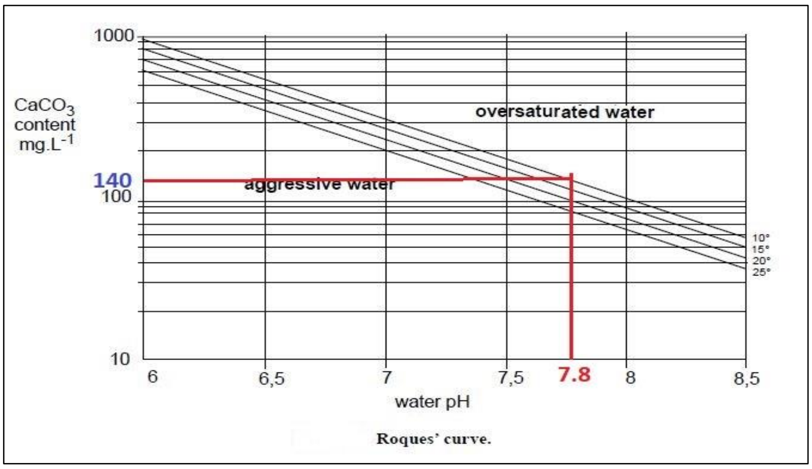


Figure 2. Roque's curve relating $CaCO_2$ dissolution with pH of dissolving water

Factors that favours or inhibit limestone dissolution as function of temperature and partial pressure of CO_2, as show below in the table.

Table-3. *Factors favouring dissolution and precipitation*

Factors favouring dissolution of limestone	Factors favouring precipitation of dissolved limestone
Cold	Hot
Acidic Environment	Basic Environment
High CO_2	Ventilation
Abundance of Water	Aridity
Long Contact time	Short contact time

2.3. Favourable Geographic features supporting dissolution

Favourable geographic features supporting dissolution include:

- Abundant vegetation favours higher concentration/ production CO_2, and humic acid
- High rainfall increase contact time
- Low temperature at mountain favours CO_2 dissolution in the water
- Flowing water will promote dissolution rate as it will never reach saturation

- Presence of discontinuities, fissures, and joints will further promote development of cavities
- Presence of pyrite FeS_2, and H_2S makes H_2SO_4 which increase limestone dissolution and converting it to Gypsum ($CaSO_4$ $2H_2O$).

Almar dam site and surrounding watershed grows abundant grasses as the topsoil has high organic contents, and is source of producing high CO_2 concentrations.

3.0 Concrete dams

In this chapter different types of concrete dams are introduced along with their foundation requirements. These are simultaneously compared to the Almar dam site. Subsequently, analysis of concrete gravity dams is reviewed along with some different requirements on the factor of safety depending on failure mechanism and guidelines.

3.1 Concrete Dams and their foundation requirements.

Concrete dams are classified as (i) Gravity Dam (b) Arch Dam (c) Buttress dams. Each of these dam have different foundation rock strength requirements.

3.1.1 Gravity Dams

Gravity dam are made of (a) Conventional vibrated concrete (CVC) (b) Roller Compacted concrete (RCC) and (c) Stone masonry. 25% of the gravity dam built around the globe is made of stone masonry and are not strong as concrete (i.e lower tensile strength, higher perviousness, doubtful quality control (*ICOLD bulletin B117*). Curved gravity dam are more stable and is proved by the old dams still standing.

The required rock uniaxial compressive strength for concrete dam are 5 MPa. In Almar Dam some of the foundation rocks has compressive strength of up to 3 MPa (used in Plaxis-2d analysis), but the danger is the existence of cavities, and interconnected channels, low strength, and deformation characteristics of the rock at the foundation and abutments. Rock fragments collected from some points in the foundation can be squeezed even in figures and it shows existence of gypsum formed through deposition of limestone precipitation.

3.1.2 Buttress Dam

Buttress dam are also made of concrete and has two main parts (a) the inclined slab (b) supported by buttress which transfer the load to the foundation rock. The suitability of this type dam for the Almar site is never considered as buttress dam are normally constructed up to a height of 30-35 meters, while Almar dam proposed dam height is 67 meters. Foundation rock compressive strength requirement for buttress dam is in the range of 5-8 MPa, which is not available at this site.

3.1.3 Arch Dams.

Arch dams constructed in narrow gorges where B/H<4 *(B=base width, H=height of the dam),* and have foundation rock strength of 10 MPa or more. Arch dams are curved upstream, and resists hydrostatic pressure through its arch nature transferring loads to its foundation and abutments. Main loads acting on arch dams are dead load, water pressure, temperature and earthquake. Other loads are ice, silt, and uplift pressure. Temperature is the biggest threat if arch dams are empty as its thin curved concrete slab expands/contracts and causes cracks in it.

3.2 Analysing Stability of Concrete Gravity Dams

Concrete gravity dams also known as dams of the future are designed to resist mainly the hydrostatic pressure of water, uplift pressure, earth quack load, sediment load, ice load in cold regions etc by using weight of the material. Concrete dam stability can be assessed through

i. Sliding failure criterion
ii. Overturning failure criterion, and
iii. Stress level in the dam body, as well as in the foundation, and abutments rock.
iv. Stability of the foundation.

Gravity dams consists of sections, and concrete quality differs for each sections. For example, it develops appreciable shear stresses at their contact with foundation and thus cannot tolerate large deformations so it requires superior quality concrete at foundation, as well as at its upstream and downstream faces. Similarly, mass concrete is impervious and uplift-pressure release is obtained by having drainage galleries, and curtain grouting which stays as integral part of the dam works.

The list of various forces acting on the gravity dam is given below.

- Dead weight of the dam
- Upstream hydrostatic water
- Accumulated sediment load and upstream face
- Earthquake forces
- Heat (Temperature) causing expansion and extraction
- Wave pressure
- Tail water pressure (if existed)

3.2.1 Sliding Stability

Many different criteria exists for estimating sliding stability for the gravity dams in different parts of the world. For example, countries like China, US, Sweden, Germany, India and Norway has developed their own criterion, and regulations *(ICOLD: Sliding stability of existing gravity dams*). Similarly, their Factory of Safety values also changes with different region.

In **Switzerland** they use this below equation to estimate sliding factory of safety.

$$FS = \frac{(tan\theta.\sum V) + C.A}{\sum H}$$

Or $$\sum H = \frac{tan\theta.\sum V}{FS1} + \frac{c.A}{FS2}$$

Where, FS = Factor of safety

FS1= FS if cohesion is assumed =0

FS2 = 5, 4, 3 for load (normal, exceptional, and extreme)

∑ V: Sum of all vertical forces acting on the foundation

∑ V: Sum of all horizontal forces acting above foundation

A: area of the dam-foundation contact surface

ϴ: Internal frictional angle

C: Cohesion

Table 4. *FS varies in response to different loading conditions as shown*

Sliding FS based on Type of Load in Switzerland		
Normal	**Exceptional**	**Extreme**
1.5	1.3	1.1

In **Norway,** slightly different equation is used to estimate sliding factor of safety, and looks more reasonable to common sense.

Factor of safety against sliding is given by **S = F/$\sum H$**

Where, F = maximum shear resistance that can be mobilized

$$\boldsymbol{F} = \frac{\boldsymbol{c.A}}{\boldsymbol{cos\alpha}(\mathbf{1} - \boldsymbol{tan\theta.tan\alpha})} + \sum \boldsymbol{V}.\,\mathbf{tan}(\theta + \boldsymbol{\alpha})$$

α = inclination of the sliding plane in relation to the horizontal plan as shown below in the schematics

if α=0, then

$$F = C.A + \sum V.tan\theta$$

$$S = \frac{C.A + \sum V.tan\theta}{\sum H}$$

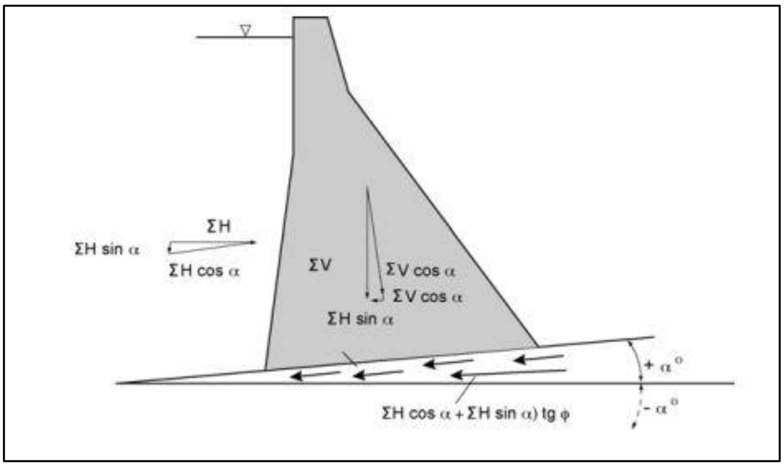


Figure 3: Showing inclination angle of α

Rules of thumbs:

a. If cohesion is considered, then FS = 3 for design loads, but = 2.5 if cohesion values are verified through tests
b. FS= 2 for unusual and extreme loads, and = 1.5 if cohesion values are verified through tests).
c. If friction angles are not measured through tests, then the below maximum values may be used:

θ = 50° for hard rock, rough surface

θ = 45° for hard rock, and low roughness

θ = 40° for loose rock, and low roughness

3.2.2 Overturning stability

All gravity dams are constructed without structural reinforcement, however some reinforcement is generally provided near the surface to distribute small cracks that may form. Regardless of considering failure of a Gravity Dam from overturning or sliding, most of the gravity dams starts failing by overturning but then ends up in sliding because all forces transfer to the toe of the dam. Factor of Safety against overturning can be estimated through following equation.

$$FS = \frac{Overturning\ Movment\ (Mo)}{Stabilizing\ Movement\ (Mst)}$$

Another criterion for checking if a gravity dam is stable against overturning; is if the resultant of all forces a_R (as shown in red colour in the below Figure-4) acting on the dam is greater than $B_B/3$ from the downstream face of the foundation, so that the dam always stay in compression.

FS_{ovt} all around the world is in range of 1.30 to 1.5, with FS=1.1 for accidental loads.

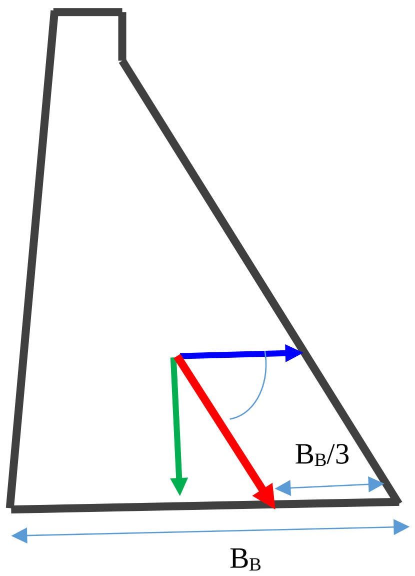


Figure 4: Showing requirement for the ultimate resultant force (in red colour) location (towards the downstream) for stability criteria against overturning

4.0 Almar Dam project site

Overall, Afghanistan climate is arid to semi-arid, with cold winters and dry summers. Southern and Western areas in Afghanistan has a wind blowing known as "120 days wind" from June to September having speed of 97 -177 Km/hr bringing clouds of dust. By the end of wind period, a guest visitor can easily observe that all trees are inclined in one direction showing existence of strong winds in the area.

Almar Dam & Irrigation Network Project of Afghanistan is USD 52 million project and is located about 33 km away from Maymona, centre of Faryab Province in North West of Afghanistan and is in the south city of Almar as shown in below figures. This project is planned to supply irrigation water to more than 2,700 ha agricultural land as well as generate some hydropower, and supply drinking water to the nearby town.

Figure 5: Showing the location of the Almar dam site with respect country capital

The dam site is located at E 64 32 17, and N 25 46 14. The valley at the dam axis location is V shape and is relatively symmetric. Almar catchment characteristics is mentioned in the below table 4. The river bed width for the time being is about 65 m to 70 m around the dam axis. The river bottom level is about 997 masl at the dam axis location. The topographic slope in the dam abutments is in the range of 30 to 35 degrees. Various elevation levels are shown in Figure-7.

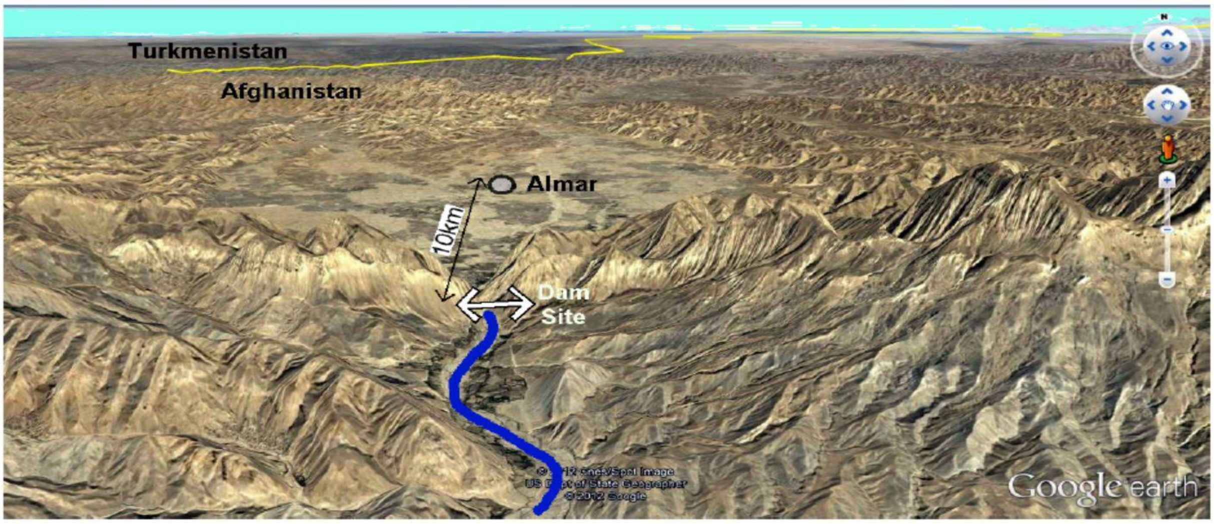


Figure 6: Almar town and dam site (from google earth)

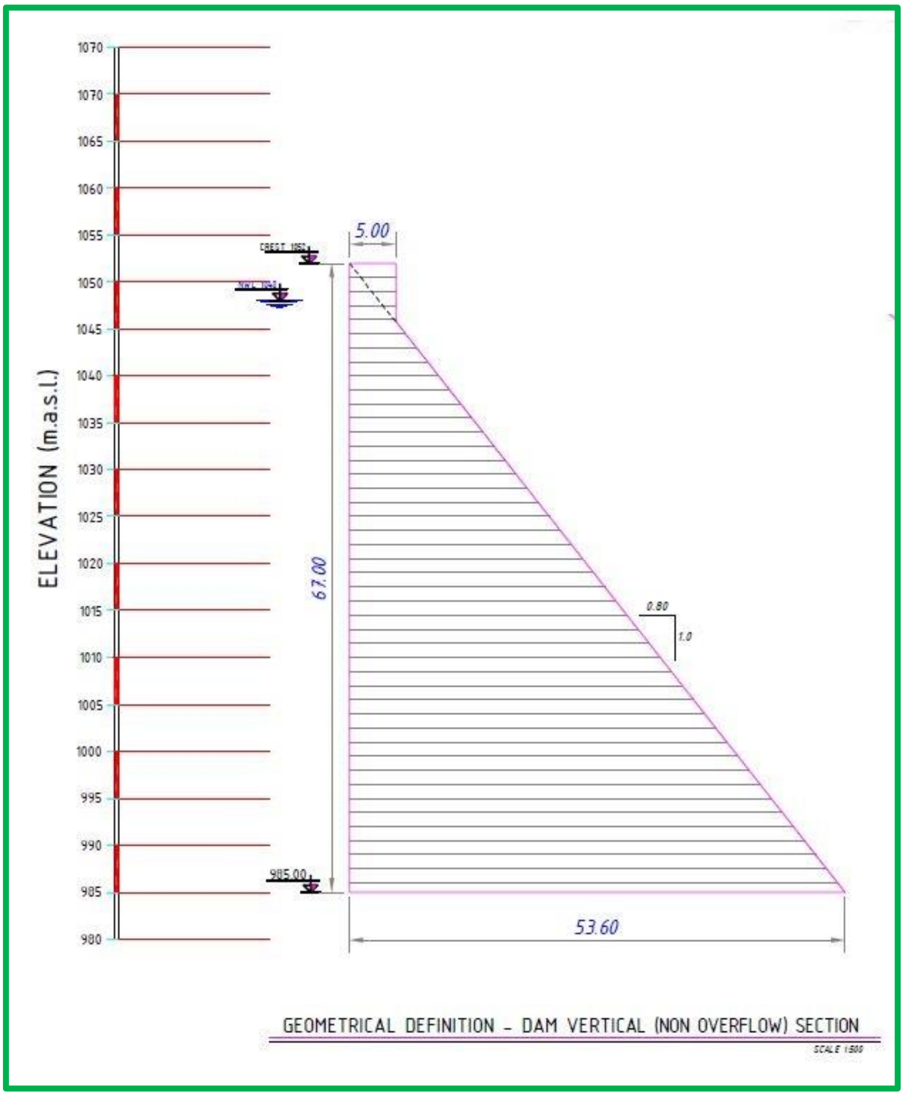


Figure 7: Proposed Almar Gravity Dam and its various elevations

4.1 Hydrology:

Rainfall varies with elevation changes in a catchment, and this phenomenon is known as hypsographic effect. This variation in precipitation in the Almar catchment is given by the formula **$P=0.11H+170$ mm, $R^2= 0.86$** Where; P= Precipitation, H= Elevation in meters. Average annual precipitation is 430 mm, with the 24 hrs maximum precipitation recorded is 69 mm. Average temperature is 15.2 C^o with average 83 freezing days in a year, so in the dam stability analysis ice load of 50 KN/m acting on the upstream of the dam is also included. Annual evaporation for the dam site is measured to be 1,549 mm per year. Other relevant parameters are given below.

Table 5: *Almar catchment characteristics*

Alamar Gravity Dam catchment area (km^2)	679
Catchment Perimeter (km)	125.5
Elevation (m)	1010 - 3280
River Length (km), with slope %	50, 9.2%
Catchment concertation time (hrs)	6-10

According to the hydrological analysis, the mean, minimum and maximum annual flow at the Almar dam site are 37.9, 17.2, 59.5 million m^3 with maximum average flow of 4.65 m^3 /s occurs in the month of April.

4.2 Geology

As shown in the below Figure-8, 15 boreholes with more than 640 m depth were drilled in dam axis, reservoir, and abutments. Each borehole depth, and core detail is given

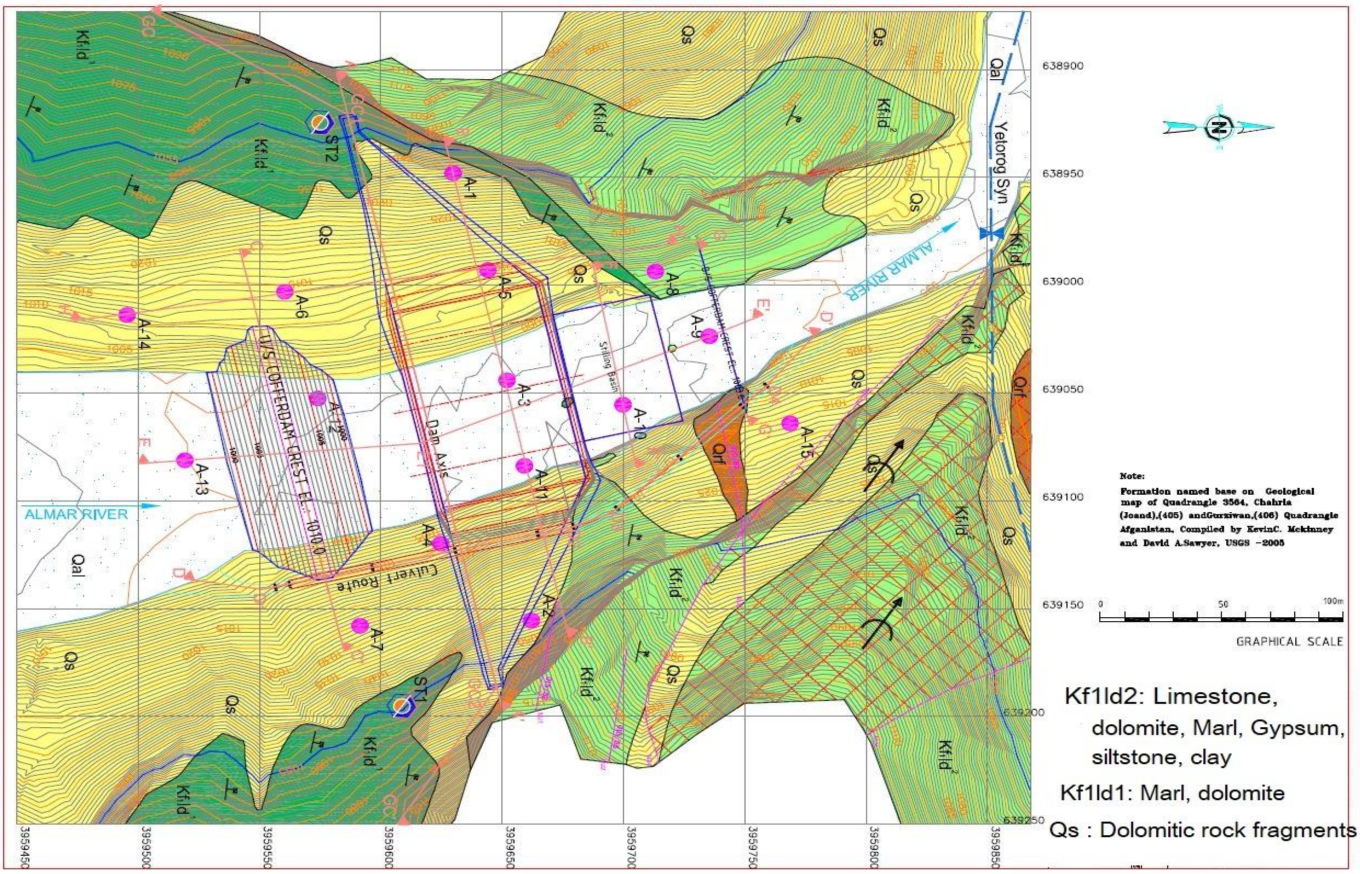


Figure 8: Geological map showing main dam, copper dam, and 15 boreholes locations

in table 6 below. As shown above in the Figure 8, dam axis sets on dolomitic rock with precipitated deposits of Gypsum, and intrusion from limestone and marlstone. The compressive strength of intact rock varies (through Triaxial testing) from 3 MPa – 32 MPa, which is enough for carrying empty dam load i.e load of concrete of the dam of 1.68 MPa (168 T/m^2), but is not enough for extreme conditions when the water level in the reservoir is at its highest level. The 3 MPa compressive strength is recorded for the white chalk which makes major part of the excavated foundation and is visible in the below Figure-9, while 32 MPa compressive strength was recorded for subsurface strong limestone at various depths. Gypsum/chalky part of the foundation rock is excavated with excavator showing its softness, and a sample small piece of the rock can be compressed within the fingers.

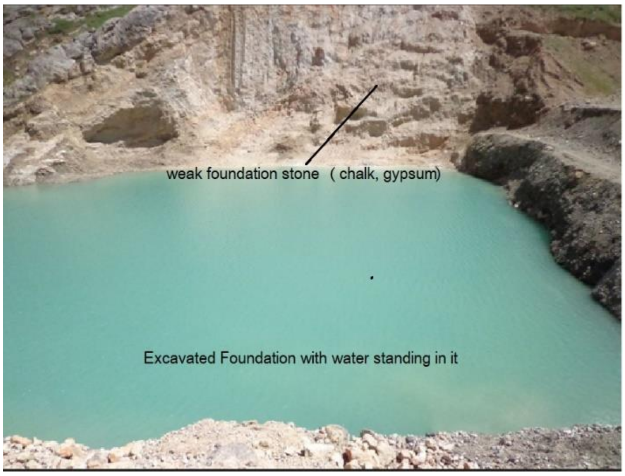


Figure 9: Piece of foundation soil/rock held in hand, and excavated foundation of Almar dam excavated with backhoe excavator

Table 6: *Boreholes details*

BH	Depth (m)	Number of Box
A1	70	12
A2	70	14
A3	80	17
A4	45	8
A5	35	7
A6	45	8
A7	45	9
A8	30	5
A9	26	5
A10	50	10
A11	35	7
A12	25	5
A13	25	5
A14	35	6
A15	30	6
Total	**646**	**124**

The thickness of overburden at Almar Dam Site varies depending on the topographic slopes. The maximum thickness of overburden is reportedly 10 m. The overburden around the dam site includes slope washes and colluvial materials, rock falls and the river alluvial materials. The size of the segments constituting these rock falls is minimum 1 m and maximum 5 m. There are 5 joint sets (faults) found consisting of total of 698 joints.

5.0 Stability analysis of the Almar Dam

Each dam type has different normal stress transmitted to the foundation. For example, Arch Dams typically transmits 7.5 to 10 MPa/m^2, while Embankment dam transmits 1.8 MPa/m^2, and gravity dam transmits up to 5 MPa to its foundation.

On compressible rock such as at Almar site, Earth fill dam is more appropriate than concrete dams because low bearing loads are transferred to the foundation strata. Construction material is also in the vicinity such as good clay, and sand-gravel or random fill, and the only concern is the spillway's high cost and potential erosion of dam material into cavities at its foundation.

Large lateral pressure caused by resultant force coming from the reservoir full dam can cause the failure of karsting cavity existing at the toe. This can only be prevented if (a) the dam site (foundation) is moved away from this cavity/ies or (b) a plate-form is created supported by Batter Piles (driven with 1H: 4V ratio) which makes a stiff foundation system against the lateral loads. However, operation like this requires additional cost and sometime is not economical *(Coduto -2011).* Furthermore, the extent of this cavity is not exactly known unless confirmed with further excavations. In that case one measure could be to fill the cavity with concrete, or some other locally available suitable material.

The stability of Almar dam is to investigate it for normal foundation conditions i.e good foundation and no karst cavities, and then with existence of a cavity near the toe.

1. Case-1: Normal conditions i.e with good foundation and existence of no cavities

2. Case 2: If a large cavity/ cavities collapse suddenly under the high hydraulic pressure when the reservoir is full, what will happen to the Factor of safety, i.e when will this be F.S=1

3. Case 3: Bearing Capacity Failure i.e to evaluate if there is a danger that dam will sink due to soft rock foundation

4. Case-4 conduct settlement calculations

And finally, conduct sensitivity analysis beside using Plaxis-2D.

5. Sensitivity analysis of the design parameters such as Cohesion, Frictional angle of underlying strata (as these parameters cannot be precisely measured), overflow of the dam during floods, and increasing volume of the dam

5.1. Case-1. Normal conditions

Assuming that the foundation is stable, and that cavities does not have any effect over the stability of dam, the various forces acting on the dam body (Figure-10) is calculated as shown in the below Table-7, and the factor of safety for overturning and sliding were found to be

$FS_{overt} = 1.5$

$FS_{Sliding} = 1.89$

Table 7: *Normal conditions with good foundation. Showing various forces acting over the dam*

No	Type of load	Load (KN/unit length)	Level Arm (m)	Moment (KN-m)	Comment
1	load due to water, $h_v= \Upsilon_w.\ H^2/2$	**21125**	21.667	457708.3	Overtrun
2	load due to vertical column of water at sloped surface, $V_v= \Upsilon_w.n1.H^2/2$	**0.00**	53.300	0.0	stabilizinng
3	Load due to pore pressure $U= \Upsilon_w.B_B\ .H/2$	**17322.5**	35.533	615526.2	Overtrun
4	Self weight, G1= ϒc.h.B1/2	**38833.2**	32.200	1250429.0	stabilizinng
5	Self weight, G2= ϒc.h.B2	**8040**	50.800	408432.0	stabilizinng
6	Self weight, G3= ϒc.h.B3/2	**0**	55.533	0.0	stabilizinng
7	Ice Load	**50**	63.75	3187.5	Overtrun
8	River Sediment load Ps=0.5. $\Upsilon sub.\ H^2s.\ (1-\sin\varphi/1+\sin\varphi)$	**3098.636**	11	34085.0	Overtrun

The strength parameters measured e.g frictional angle of the foundation material is φ=25 while cohesion, c=600 kN/m². River sediment deposit is 33 meters high against the wall of the dam with φ=35° with submerged unit weight of 21 kN/m³

The dam is also checked for location of the resultant force, and thus tensile stress, and it was found that $a_R > B_B/3$. (aR=18.6 m, while $B_B/3$=17.77m). The resultant of all forces, R = 38,242 kN/m and is acting in direction of θ=51° (with x-axis). Thus the resultant lies within the middle third of the dam as required by the Norwegian regulations. However, the location of the resultant is close to the downstream boundary or 18.6 -17.77 = 0.83 m.

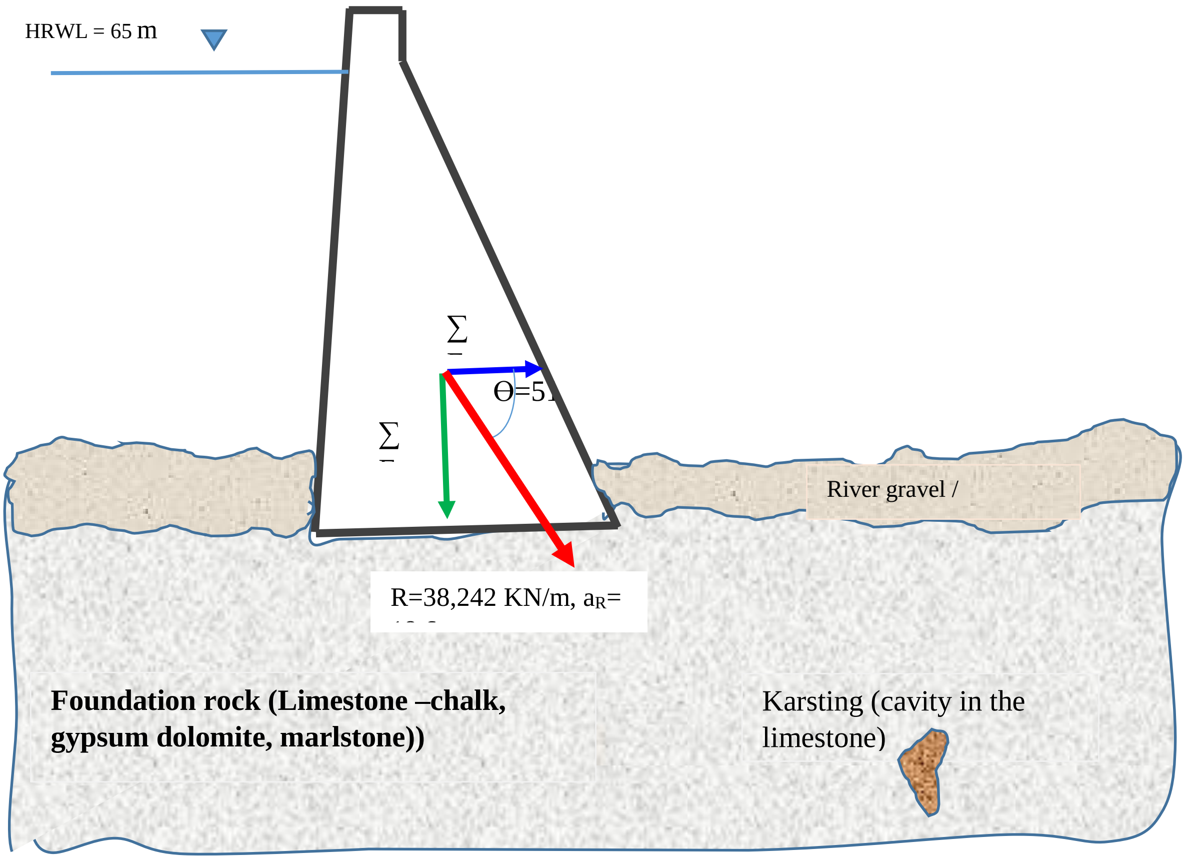


Figure 10: Schematic showing normal loading conditions and fine foundation

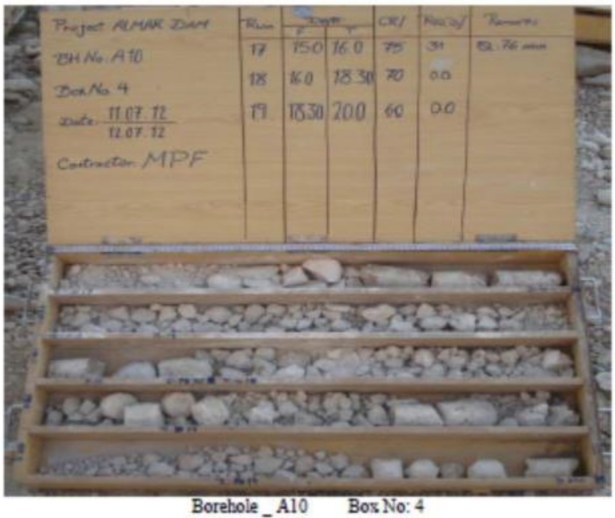
Discussion:

FS $_{overt}$ = 1.5 is very safe even if all forces increases by 50%. FS $_{Sliding}$ =2 is considered stable if c is taken from reference material, while FS $_{Sliding}$ =1.5 is considered stable if value of c is based on actual measurements in the field. So, in our case the dam is very stable under normal conditions.

5.2. Case -2: When Karsting cavities/ large cavity at the toe fails suddenly.

The Bore Hole No -A10 drilled up to a depth of 50 meter in the stilling basin areas i.e a few meters away from the toe of the dam; shows gravel, sand, precipitated limestone (gypsum) as shown in the below Figure-11. The extent of this cavity is precisely not known as it needs additional investigations such as digging more boreholes or conducting of geophysical methods to find the boundaries of the cavity/cavities. The author personally visited the site, and stay for couple days to collect as much information as possible. Still, some assumptions are required. So, it os assumed that if this cavity/cavities fail suddenly when the dam reservoir is full, and investigated what will happen to the safety of the Dam? An excel sheet has been develop to calculate various forces acting on the dam in a When cavities/cavity fails suddenly as shown in Figure 13, with water level at 65 m from the foundation of the dam.

Borehole _ A10 Box No: 3



Borehole _ A10 Box No: 4

Figure 11: Borehole-A10, core samples at depth of 11-20 meters below the dam contact with foundation

The various forces acting on the dam body were calculated as shown in the below table 8, and the factor of safety were found to be primarily dependent on the contact length of dam base and foundation surface denoted by $B_{B.}$ So, by keeping all other parameters in place the factor of safety for both Overturning and Sliding will reduce as shown below. The tilted dam will be under tensile stress at its heal, and compression cracks will develop at the toe as shown in the schematic Figure-13. If, B_B reduces from 53.6 m to 37 m, the F.S $_{\text{Overturning}}$ =1.00, F.S $_{\text{Sliding}}$ =1.48. Any further reduction in contact length cause failure of the dam.

Table 8: *Forces acting on Dam when cavity at the toe fails suddenly, and the contact length of Dam and foundation reduces from 53.6m to 37m*

No	Type of load	Load (KN/unit length)	Levell Arm (m)	Moment (KN-m)	$B_3/3$	a_R	
1	load due to water, $h_v = \gamma_w \cdot H^2/2$	**21125**	21.7	457708.3	**12.33**	**-0.2**	High Risk of Tesile stress at the toe, and dam will fail by overtuning
2	load due to vertical column of water at sloped surface, $V_v = \gamma_w . n1. H^2/2$	**0.0**	37.0	0.0			
3	Self weight, G1= Yc.h.B1/2	**38833.2**	21.3	828441.6			
4	Self weight, G2= Yc.h.B2	**8040**	34.5	277380.0			
5	Self weight, G3= Yc.h.B3/2	**0**	37.0	0.0			
6	Uplift	**17322.5**	35.53	615526.2			
7	Ice Load	**50**	63.75	3187.5			
8	River Sediment load Ps=0.5. Ysub. H^2s. (1-sinφ/1+sinφ)	**3098.636**	11.00	34085.0			

Discussion:

The failure starts by tilting of the dam, but then ends in sliding. The resultant force R= 38,242 kN/m, is making ϴ= 51° with x-axis. a_R= - 0.2, and the resultant force acts out of base of the dam as shown in the schematic Figure-12 below.

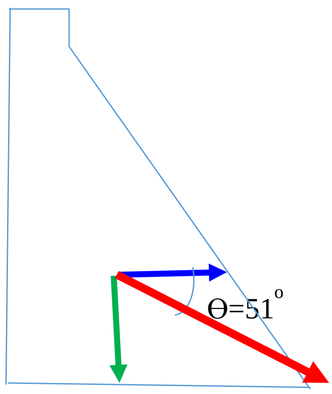


Figure 12: showing schematic of resultant force acting outside of the base at the time when karst cavity at the toe fails suddenly.

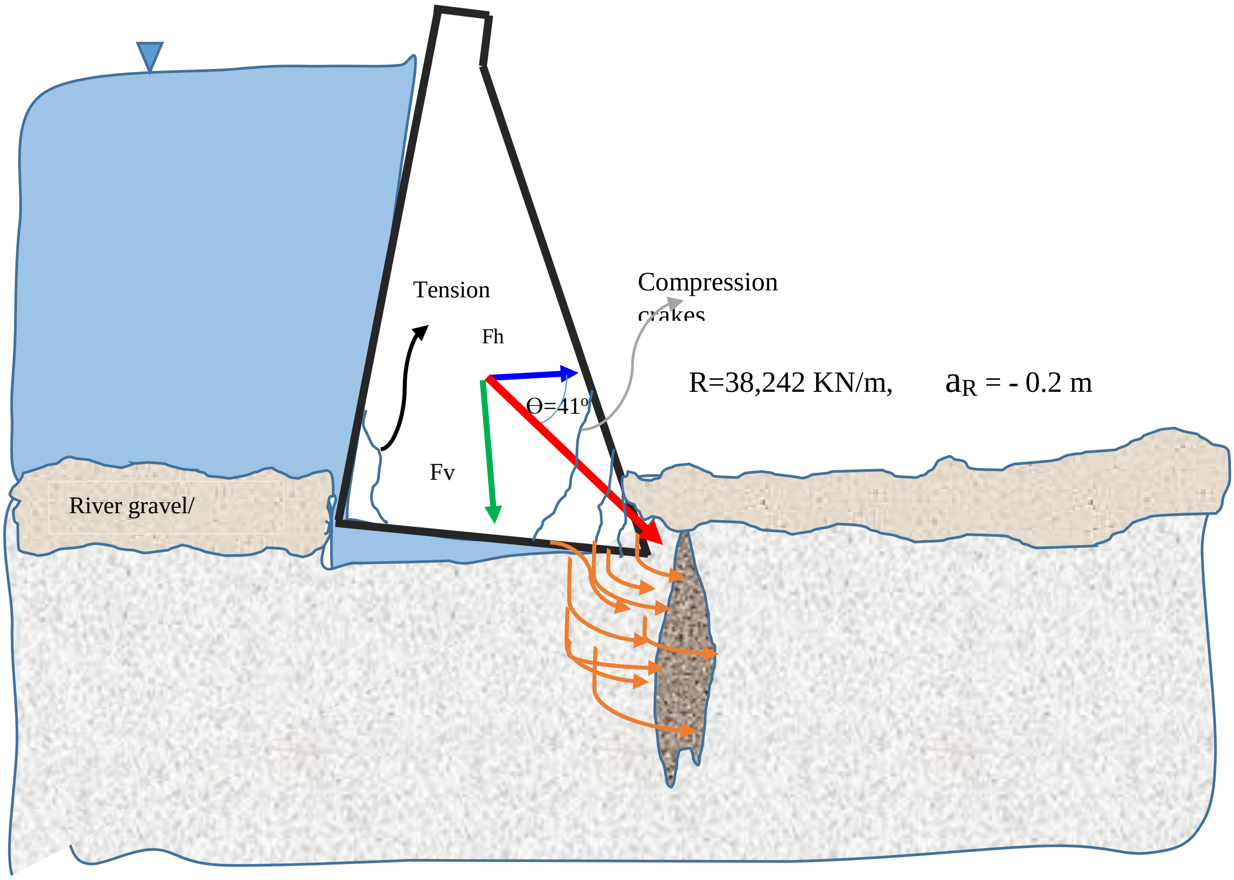


Figure 13: Schematics showing dam failure starts after collapse of the karsting cavity at the toe, followed by tilting of the dam, and the resultant of all forces are acting outside of the B_B causing cracks in the toe of the dam

5.3. Case-3. Bearing capacity of the weak foundation to carry the load of the dam

Dam foundations must satisfy various performance requirements e.g bearing capacity, and settlements. Dam weight, and other forces acting over the dam body transfer both compressive and shear stresses to the foundation rock/soils. In some cases the shear stress may surpass the shear strength of underlying rock resulting in bearing capacity failure. As were mentioned above, in Almar Dam site, the underlying rock type consists of chalk, gypsum, dolomite and limestone, and is highly susceptible to shear strength failure.

In the reference studies, three different types of bearing capacity failures are reported namely (a) General Shear Failure (b) Punching Shear Failure and (c) Local Shear Failure

General Shear Failure: is common in rock, incompressible soils, and consolidated clays. This type failure happens suddenly under the quick loading conditions. The failure surface is clearly defined by appearance of bulge. See Figure-14. For nearly all shallow foundations (D/B<=1), it is necessary to check the general shear case, and then conduct settlement analysis to verify that the foundation will not settle excessively (*Donald P. Coduto).*

Punching Shear Failure: happens in in very loos sand, weak clays under slow loading conditions.

Local Shear Failure: is different from general shear failure in way that it did not forms a heave over the surface, but it has a clear shear failure surface.

For all concrete gravity dams foundations, general shear failure should be checked, while the remaining two types of bearing capacity failures are covered by settlements analysis.

In order to actually measure bearing capacity of Almar Dam foundation rock, a full scale load test is needed, which is consist of constructing of real spread footing and loading it until failure happens. This method is the most accurate method but is expensive and is not applied. The alternate method is to use (a) Limit Equilibrium Analysis which includes empirical factors developed from model tests, and (b) using of Numerical methods e.g finite element method (FEM). Here, both methods i.e Limit Equilibrium Analysis to measure the bearing capacity, supported by FEM are considered. Furthermore, the foundation rock is not homogeneous and thus γ, φ, and c varies with layers. For the sake of accuracy, lowest values of γ, φ, and c in the zone between bottom of the dam foundation and a depth B (where B=53.6 meters) were considered. The alternate option would be to consider weighted average of γ, φ, and c of all subsurface layers. Similarly, in the next section settlement analysis is also carried out. As rule of them bearing capacity failure occur only in a zone up to depth of B_B.

Allowable bearing capacity:

Allowable bearing capacity (q_a) is given by the relation of ultimate bearing capacity (q_{ult}) and factor of safety (FS).

$$\mathbf{qa} = \frac{qult}{FS}$$

q_a =Allowable bearing capacity

q_{ult}= Ultimate bearing capacity at failure

FS = required factor of safety (2.5 or 3.5)

q= design load < q_a

Factor of safety value given above is based on the professional judgement of engineers. In this analysis a FS=3 based on reference studies. The lesser the subsurface and laboratory exploration results in more uncertainty and thus requires higher FS. Similarly, importance of structure and consequence of the failure play important role in selecting a value for FS. Structures with higher H/B ratio such as dams, chimney which face catastrophic failure should have higher FS.

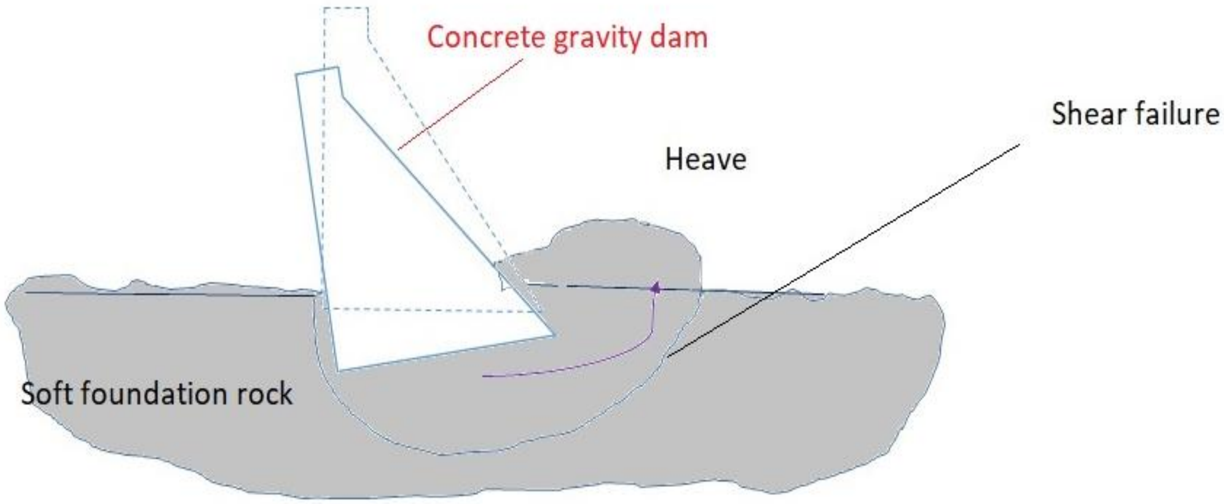


Figure 14: Schematic showing general bearing capacity failure mechanism

Bearing capacity analysis

Various Limit Equilibrium Methods are available in the literature, but the most widely used methods are (a) Terzaghi and (b) Vesic's bearing capacity equations *(Coduto-2011)*

Terzaghi computation of bearing capacity is based on the following assumptions

i. The depth of foundation is less than or equal to its width (D<=B), which in our case is satisfied i.e D=10 m, and B=53.3 m
ii. The shear strength of the soil described by the formulas $s = c' + \sigma' \tan\varphi'$
iii. The foundation (base of the structure) is very rigid as compare to the underlying soil/rock (this is also satisfied as the underlying soft rock consists of limestone, precipitated limestone (gypsum) and chalk)

iv. The applied load is compressive and is applied vertically, and no applied moment loads are present i.e the dam is empty (for empty dam, can the underlying soft rock will able to provide required bearing strength)

Terzaghi's theory is based on continuous foundation i.e with large L/B ratios, and is given by below equation

$$qult = 1.3\, c' N\varsigma + \sigma' N\mathrm{q} + 0.4\gamma' B N_{\gamma}$$ -------------------- Terzaghi eq.

Where,

q_{ult} = Ultimate bearing capacity of the foundation rock/soil at failure

c' = effective cohesion

σ' = effective stress at depth D below the ground surface

γ' = effective unit weight of the soil/rock (γ' = γ if ground water level very deep)

D = depth of foundation below ground surface

B= width of the foundation

$N\varsigma, N\mathrm{q}, and\ N_{\gamma}$ are Terzaghi's bearing capacity factors and function of φ'. They are also computed through the following equations.

Terzaghi's formulas is based on effective stress, but in case of saturated undrained conditions when φ=0, then Nc = 5.7, Nq=1, and Nc=0.0.

Terzaghi bearing capacity factors are given through following equations

$$Nq = \frac{a_{\theta}^{2}}{2\, cos^{2}\ (25 + \frac{\varphi'}{2})}$$

$$a_{\theta} = e^{\pi(0.75 - \phi'/360) \tan \phi'}$$

$$Nc = 5.7 \qquad \text{for } \phi' = 0$$

$$Nc = \frac{Nq-1}{tan\varphi\prime} \qquad \text{for } \phi' > 0$$

$$Nr = \frac{tan\varphi\prime}{2} \left(\frac{Kpr}{cos^{2}\ \varphi\prime} - 1\right)$$

Terzaghi method is still often used because it is simple and familiar. Vesic's produce an alternate equations to Terzaghi's method, and it produce more accurate bearing values and can be applied to broader range of loading and geometry conditions.

$q_{ult} = c' N_c s_c d_c i_c b_c g_c + \sigma'_{zD} N_q s_q d_q i_q b_q g_q + 0.5\gamma' B s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma$ ---- Vesic's equation

Where,

s_c, s_q, s_γ are shape factors given by different equations. Mentioning all equations is beyond the scope of this report.

d_c, d_q, d_γ are depth factors given by different equation used in this analysis

i_c, i_q, i_γ are load inclination factors

b_c, b_q, b_γ are base inclinations

g_c, g_q, g_γ are ground inclinations

$Nc = \frac{N_q - 1}{tan\varphi'}$ for $\varphi'>0$,

$N_c = 5.14$ for $\varphi'= 0$,

Considering a FS=3 for foundation material uncertainties, while using both Terzaghi's and Vesic's equations give us bearing capacities values given in table 9.

Table-9. *Showing bearing capacity of the dam foundation*

Stress at the foundation due to vertical load of concrete	Bearing capacity of the foundation rock using **Terzaghi's Equation**	**Vesic's Equation**
1,608 KPa	6,581 KPa	6,106 KPa

So, no bearing capacity failure will take place considering no karsting cavities at heal of the dam. For detail calculation, please refer to *Annex- 3*

Discussion:

Proper selection of soil strength parameters like $\acute{c}$, and φ' is the most difficult part of the performing bearing capacity analysis. Field or laboratory test data is often contain errors or is incomplete. For example, bearing capacity would be 50 to 60% less than the actual bearing capacity if $\varphi'=35^o$ instead of actual 40^o. Thus it is extremely necessary not to overestimate the soil strength parameters. This is why most engineers use conservative values for soil strength parameters. In Almar Dam case, I used $\varphi'=25^o$, and $\acute{c}$=600 KPa, and estimated bearing capacity of the foundation rock and is equal to 6.5MPa, while the stress of the concrete dam is 1.68 MPa. Though, foundation rock is very weak and soft but is still enough to resist the load of 67 m high concrete gravity dam.

5.4. Case-3. Settlement analysis

Even if the bearing capacity of the foundation is enough, still independent settlement analysis is required (*Hough-1959*). Generally the foundation must meet two settlement criteria: total settlement, and differential settlement. Soft rocks such as one encountered in the Almar dam site consisting of soft dolomite, gypsum and chalk, and elsewhere reported soft rock such as siltstone, claystone, and mudstone are very similar to hard soil and often can be sampled, tested and evaluated using methods develop for soils *(Coduto -2011*). Here, my emphases is only to estimate settlement caused by the structural load on foundation, but settlement can also be caused by:

- A falling groundwater table (such as in Mexico city)
- Settlements caused by collapse of karsting cavities
- Settlements caused by the weight of a recently placed fill.
- Settlements caused by lateral movements from nearby excavations

The magnitude of load on foundation from the dam decreases with depth and becomes very small at a depth of about 2B to 6B, where B_B is the dam width. The distribution of load in the foundation strata below the dam can be calculated through Boussinesq's method, Vestergaard's or through simplified method.

When, the weight of the dam is placed on the foundation, then the increase in vertical compressive stresses also known as induced stress in the underlying soil layers is given by the following equation.

$$\Delta\sigma_z = I_\sigma (q - \sigma'_{zD})$$

Where,

$\Delta\sigma_z$ = Induced vertical stress due to load from dam (decreases with depth as the load propagate over a larger area)

I_σ= Stress influence factor (decreases with depth, and is 1 just below the dam as shown in the below figure)

q = Bearing pressure along the bottom of foundation caused by structural load of the dam

σ'_{zD} = Vertical effective stress at depth D below the ground surface

Boussinesq's Method:

Boussinesq developed a solution to find induced stresses in an elastic material due to an applied point load. Newmark then used Boussinesq equation to find I_Q (stress influence factor) at a depth Z_f. The reason for using Newmark solution is to find stress zones say at a depth of 15 meter below the dam foundation although it can also be used to measure induced stresses at other locations i.e beneath and beyond the base of the dam. So, if karsting cavity exist at greater depth below the dam it will have diminishing effects over the settlement.

Newmark equation says if:

$\mathbf{B^2 + L^2 + Z_f^2 > B^2 L^2 / Z_f^2}$ then

$$I_Q = \frac{1}{4\pi}[\frac{2BLZ_f\sqrt{B^2+L^2+Zf^2}}{Zf^2(B^2+L^2+Zf^2)+B^2L^2}\left(\frac{(B^2+L^2+2Zf^2)}{(B^2+L^2+Zf^2)}\right) + Sin^{-1}\frac{2BLZf\sqrt{(B^2+L^2+Zf^2)}}{Zf^2(B^2+L^2+Zf^2)+B^2L^2}$$

Or

$$I_Q = \frac{1}{4\pi}[\frac{2BLZ_f\sqrt{B^2+L^2+Zf^2}}{Zf^2(B^2+L^2+Zf^2)+B^2L^2}\left(\frac{(B^2+L^2+2Zf^2)}{(B^2+L^2+Zf^2)}\right) + \pi - Sin^{-1}\frac{2BLZf\sqrt{(B^2+L^2+Zf^2)}}{Zf^2(B^2+L^2+Zf^2)+B^2L^2}$$

Both of above Newmark's equations were used, and was giving underestimated results. The same issue with Newmark's equations were also mentioned in some other cases in the reference material. Then, simplified method to estimate induced stress at different depths below the dam foundation to see if they are enough to cause collapse of Karsting cavity at various depths. The equation is given below and the results are in Table-10. Figure 15 shows stress bulb beneath the dam based on Newmark solutions

Simplified Method for estimating induced stress under the 68 meters gravity dam is given below.

$$\Delta\sigma_z = \{1 - (\frac{1}{1+(\frac{B}{2Zf})^2}))^{2.6}\}\ (q - \sigma'_{zD}) \quad \text{------- Simplified Method}$$

Where,

$\Delta\sigma_z$ = induced vertical stress due to load from dam (decreases with depth as the load propagate over a larger area)

B= Dam width

Z_f = Depth to point of interest within the foundation beneath the dam

q= Bearing pressure due to structural loads of the dam

Table 10: *estimated induced stresses at various depths beneath the dam*

Sno	Depth (m) beneath the Dam	Induced stress (KPa)
1	5	728
2	10	725
3	12	721
4	16	705
5	20	673
6	30	564

Same stresses were also estimated by using Finite Element Method.

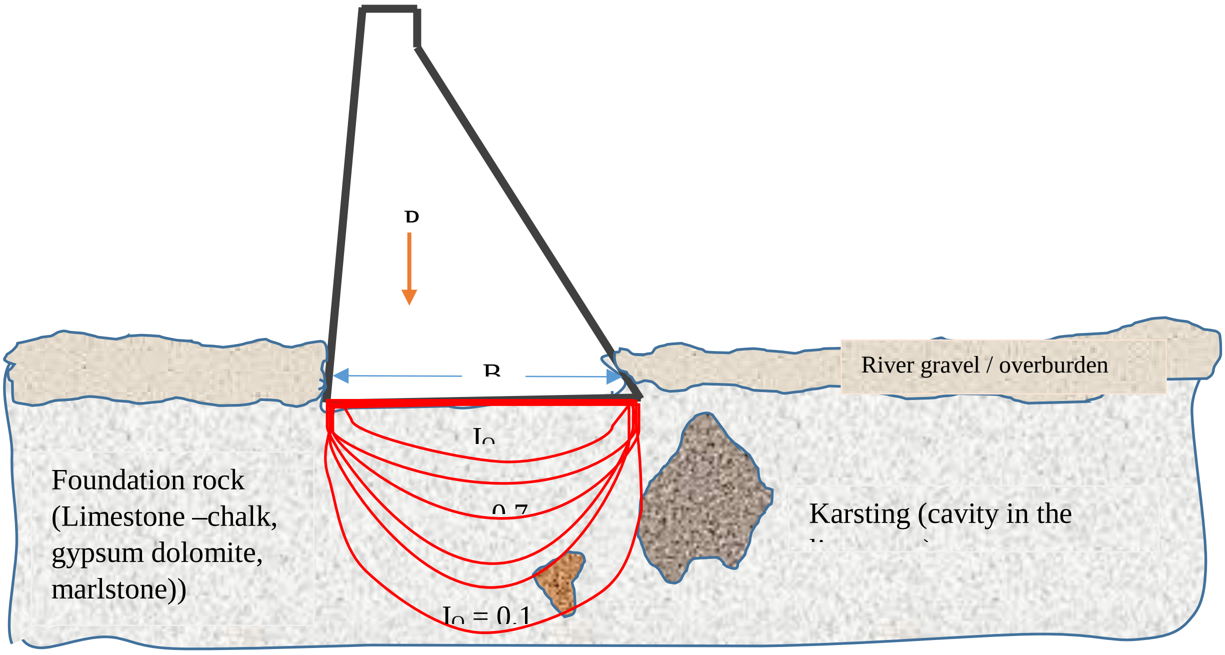


Figure 15: Schematic showing stress bulbs beneath the dam based on Newmark solutions

The most common source or the significant source of settlement in clays, silt and stiff soils are consolidation which is caused by shifting of the solid particles in response to an increase in the vertical effective stress also known as primary settlement. Another type of settlement called Secondary settlement is caused by decomposition of organic matter and is not considered in sand and over consolidated stiff soils. There are several methods described in the reference studies for estimating total settlements e.g (a) *Classical method* (b) *Skempton and Bjerrum* method used for taking three dimensional settlement instead of considering only vertical settlement, and (c) *Janabu's method*. Classical method is used here, and it is based on Terzaghi's theory of consolidation and considers only vertical strains. It divides the soil beneath the footing into layers. So, the foundation strata of 30 meters was divided into five layers as are shown below in Figure-16, and the settlement of each layer were estimated separately, and the total settlement was the sum of all layers settlement and was equal 573 mm.

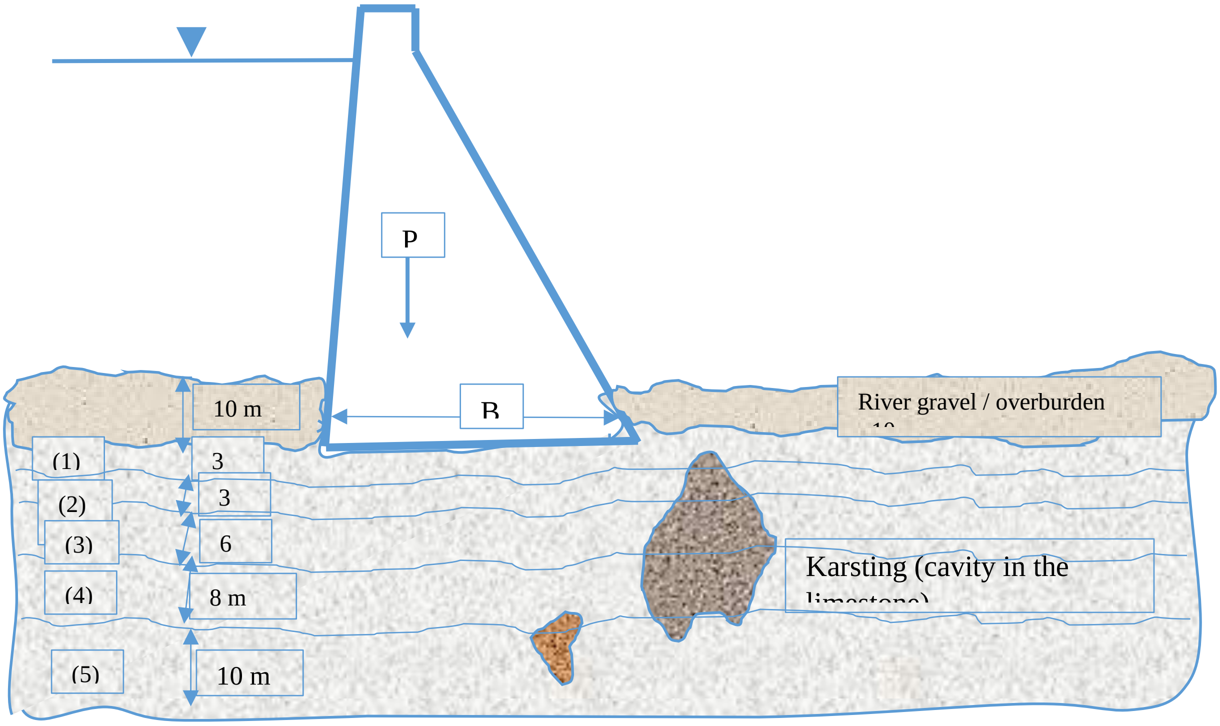


Figure 16: schematic showing different layers of foundation strata (30 m in total) with different thickness used in settlement estimations.

At the Almar dam site, dolomite/gypsum rock is considered as stiff soil and by looking to soil profiles, the unit weights, reference material, and field experience decision about various layers were made to consider them normally consolidated or over consolidated.

The total statement is then given by the following equations. Detail calculation about the settlement analysis is given in Annex-5.

(a) For normally consolidate soils $(\sigma'_{z0} \approx \sigma'_c)$

$$\delta_c = r\sum \frac{C_c}{1+e_0}\ H\ Log\ (\frac{\sigma'_{zf}}{\sigma'_{z0}})$$

(b) For over consolidate soils – Case I $(\sigma'_{zf} < \sigma'_c)$

$$\delta_c = r\sum \frac{C_r}{1+e_0}\ H\ Log\ (\frac{\sigma'_{zf}}{\sigma'_{z0}})$$

(c) For over consolidate soils – Case II $(\sigma'_{z0} < \sigma'_c < \sigma'_{zf})$

$$\delta_c = r\sum[\frac{C_r}{1+e_0}\ H\ Log\ \left(\frac{\sigma'_c}{\sigma'_{z0}}\right) + \frac{C_r}{1+e_0}\ H\ Log\ \left(\frac{\sigma'_{zf}}{\sigma'_c}\right)]$$

Where:

δ_c= ultimate consolidation settlement

σ'_{z0} = initial effective stress at the middle of each layer

σ'_{zf} = Final effective stress at the middle of each layer

r= rigidity factor (0.85)

C_e= compression index

e_{e0}= initial void ratio

H= thickness of soil layer

σ'_{z0} = initial vertical effective stress at midpoint of soil layer

5.4 Sensitivity analysis of the Design Parameters.

Sensitivity analysis of the design parameters were conducted as reported below for i) cohesion and friction angle, ii) reservoir water level and iii) changing the dam width BB or applying upstream slope.

5.4.1 Cohesion (C), and Friction angle (φ).

There are many different checks to see if the design of a gravity dam is safe e.g (a) through finding *Factor of Safety again sliding ($FS_{Sliding}$)*, (b) *Factor of* Safety again overturning, and (c) finding if the *resultant of all forces stay within the dam body* so that the dam is in compression i.e $a_R > B_B/3$ where a_R is the distance of resultant of all forces from the toe, and B_B is the base width of the dam.

Shear strength parameters (c, and φ) of the foundation material play a vital role in determination of the sliding stability. For example, in case of Almar dam cohesion of the foundation material is C=600KPa, and friction angle is, $\varphi = 25^o$ giving us $F.S_{sliding}$=1.9, and if we change cohesion, C=1MPa, while keeping frictional angle the same, then $F.S_{sliding} = 2.78$ which is a significant increase in the safety again sliding of the dam. Both parameters of C, φ has no impacts over the overturning stability of the Dam as shown below in the table 11.

Similarly, if we increase φ from 25^o to 35^o again the $F.S_{sliding}$ is increased and is = 2.18, keeping Cohesion the same i.e C=600 KPa,

Table 11: *showing sensitivity analysis of shear strength parameters.*

	Sensitivity Analysis of Design Parameters: Cohesion C, and Frictional angle, φ				
1	for				
	C (Kpa)=	600			
	Friction angle (φ, degree)=	25			
	F.S$_{overturning}$	**F.S$_{sliding}$**	**$B_3/3$**	**a_R**	
	1.5	1.9	17.87	18.8	
2	for				
	C (Kpa)=	1000			By changing Cohesion, and Frictional anagle Stability against sliding changes. i.e by increasing C values from 600 Kpa, to 1000 Kpa and keeping Friction aganle the same, sliding safety is increased drastically. Similarly, by increaseing Frictional angle, and keeping C the same, again F.S agains sliding increased.
	Friction angle (φ, degree)=	25			
	F.S$_{overturning}$	**F.S$_{sliding}$**	**$B_3/3$**	**a_R**	
	1.5	2.78	17.87	18.8	
3	for				
	C (Kpa)=	600			
	Friction angle (φ, degree)=	30			
	F.S$_{overturning}$	**F.S$_{sliding}$**	**$B_3/3$**	**a_R**	
	1.5	2.03	17.87	18.8	
4	for				
	C (Kpa)=	600			
	Friction angle (φ, degree)=	35			
	F.S$_{overturning}$	**F.S$_{sliding}$**	**$B_3/3$**	**a_R**	
	1.5	2.18	17.87	18.8	

5.4.2 Changing design flood level from 65 m to 67 m

Here it is assumed that the dam will be overtopped in a climate change scenario, and the effect on the factor of safety investigated.

If we assume the dam is overflowed i.e Highest Water Level (HWL) changes from 65m to 67m, then what will happen to the dam safety? Analysis results are given in table 12 and it shows that both factor of safety again overturning and sliding will reduce but stays in acceptable range i.e F.S $_{overturning}$ reduces from 1.5 to 1.42, and F.S $_{overturning}$ reduces from 1.9 to 1.79. The only issues that dam goes under tension as $a_R<B_B/3$. So, if the Concrete Mix design of the concrete is not properly tested and sufficient

surface reinforcement not included then tension crakes will develop in at the upstream heal of the dam which is not ideal condition and is against of the Norwegian standards.

Table 12: *showing factor of safety when dam is overtopped by flood.*

5	**Sensitivity Analysis of Design Parameters**: changing design flood level from 65 to 67 m				
	$F.S_{overturning}$	$F.S_{sliding}$	$B_3/3$	a_R	Dam toe is under the tension which is against the Norwegian standards
	1.42	1.79	17.87	17	

5.4.3 Changing the volume of the dam body by changing base width (B_B)

Concrete density is in a range of 24 KN/m^3 and cannot be increased much, but volume of the dam could be increased to improve stability of the dam against overturning and sliding failures. The volume of concrete could be increased in two ways (a) increase base width of the dam or (b) apply upstream face slop of 1/10. For example, if we increase B_B from 53.3 meters to 60 meters (which is not economical) as shown in table 13 then factor of F.S $_{overturning}$ will increase from 1.5 to 1.63, and F.S $_{sliding}$ will increase from 1.9 to 2.11, and the dam will stay in compression.

Table 13: *showing factor of safety when dam concrete volume is increased*

6	**Sensitivity Analysis of Design Parameters**: changing volume of the dam by changing B_B from 53.3 to 60 meters				
	$F.S_{overturning}$	$F.S_{sliding}$	$B_3/3$	a_R	Dam is very safe, but is not economical
	1.63	2.11	20	24.7	

Similarly, if we apply slope of 1/10 to the upstream face of dam, then factor of safety is drastically increased as shown in Table-14, which is also economical as we use little more concrete than the actual amount. The factor of F.S $_{overturning}$ will increase from 1.5 to 1.59, and F.S $_{sliding}$ will increase from 1.9 to 1.93, and the dam will stay in compression as shown in the below table.

Table 14: *showing factor of safety when upstream slope of 1:10 (H: V) is applied*

7	**Sensitivity Analysis of Design Parameters**: Apply upstream slope of 1:10 (H:V)				
	$F.S_{overturning}$	$F.S_{sliding}$	$B_3/3$	a_R	Dam is very safe, and is economical as we use less concrete in making upstream face slope
	1.59	1.93	17.77	20.7	

6.0 Use of Finite Element Model (Plaxis-2D)

Plaxis-2D is a two dimensional finite model software used commonly for the analysis of deformation, stability, and flow in geotechnical problems. In this study, it is used for modelling stresses, displacement, deformations, and seepages in the foundation strata and at the karsting cavity present at the toe of Almar concrete gravity dam as shown in the Figure-19.

Foundation strata consists of two layers i.e the upper 10 m layer of river bed material (sand, shingle), followed by a 30-40 m weak limestone layer. The dam body itself is composed of concrete with unit weight of 24 kN/m^3, and 25 MPa compressive strength. It is obvious that different zones of the dam consist of concrete with different strength but for simplification, concrete with 25 MPa were used in this analysis. The construction of dam is simulated by a stage construction method to catch actual site (construction) activities.

The upper 10 m river bed material has been excavated, and the dam itself is directly placed on the top of second layer. Slab for the downstream stilling basing has not been included in the analysis in order to simulate the direct impacts of dam dead weight over the cavities.

Three different shapes of cavities were tested in the simulation to get results close to the reality as were reported in Borehole-A10.

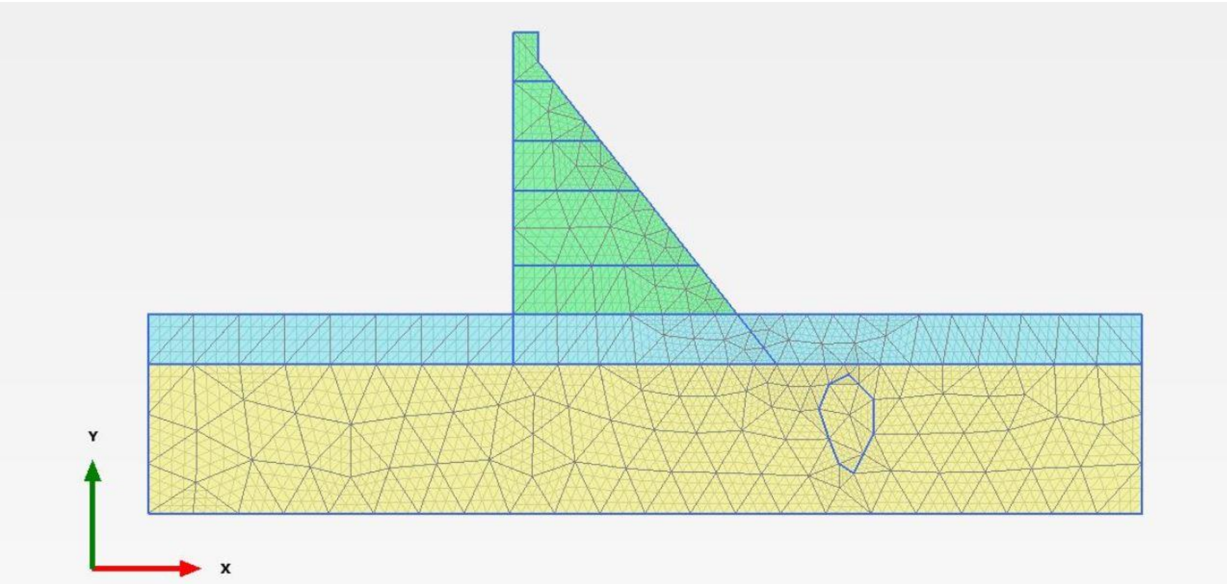


Figure 17 (i): Irregular shape Karsting cavity (9m wide, and 17 m deep)

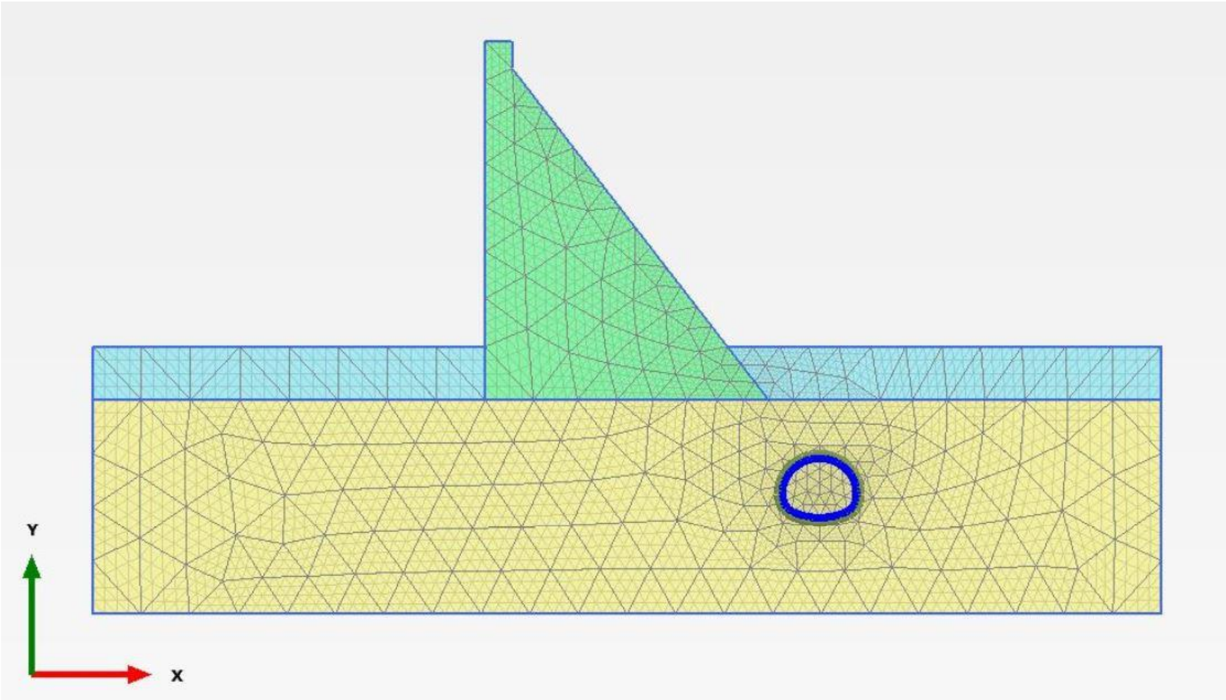


Figure-17 (ii): Simi-circular (tunnel_ shape cavity with 6 m radius), and a temporary lining shown in blue

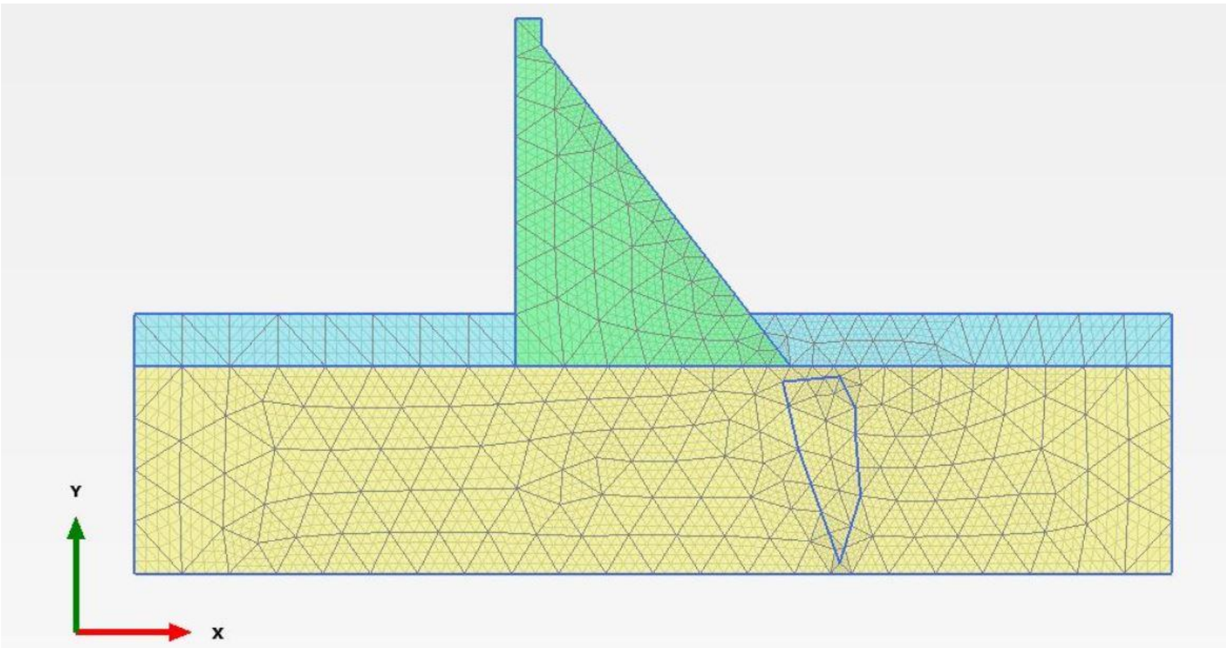


Figure-17 (iii): Elongated cavity with a width of 11m in its top then narrowing down and reaching to a depth of 39 m

As cavities in limestone is prone to further widening/deepening due to various factors, and as supported by borehole A10, Figure-17 (iii) is considered for further detail analysis.

6.1 Soil layers material properties

Plaxis-2D required numerous soil parameters mentioned in the below Table-15, and all of those parameters were not available from the geotechnical report, and feasibility study of Almar dam. Hence, many parameters values were taken either from similar soil models available within the FE model, or taken from other relevant literature.

Table 15: *Soil layers, and concrete parameters and their values used in the Plaxis-2D model.*

Parameter	Name	Riverbed Material (sand)	Weak foundation Limestone	Dam body Concrete	Unit & remarks
General					
Material Model	Model	Mohr-Coulomb	Hoek-Brown	Linear-elastic	
Type of material Behaviour	Type	Drained	Drained	Non-porous	
Unsaturated unit weight	γ	17	21	24	kN/m^3 from geotechnical report
Saturated unit weight	γ_s	21	23.7	24	kN/m^3 from geotechnical report
Initial Void ratio	e_{in}	0.6	0.5	-	From reference soil models
Parameters					
Secant stiffness in standard drained Triaxial test	$E^{ref}{}_{50}$				
Tangent stiffness for primary Odometer loading	$E^{ref}{}_{oed}$				
Unloading and reloading stiffness	$E^{ref}{}_{ur}$				
Power for stress-level dependency of stiffness	m				
Young's modulus	E'	1.3E4	2.5E6	25E6	kN/m^2
Poisson ratio	V'_{ur}	0.3	0.25	0.15	

Uniaxial compressive strength	σ_{ci}		3000		kN/m^2 from geotechnical report
Material constant for intact rock	m_i		8.5	-	
Geological Strength Index	GSI		42	-	
Disturbance factor	D				
Cohesion	C'_{ref}	1	600		From Geotechnical report
Friction angle	Φ’	35	25		From Geotechnical report
Dilatancy parameter	ψ	1			From reference soil models
Ground Water					
Horizontal Permeability	Kx	7	1.8		m/day – from geotechnical report
Vertical permeability	Ky	7	1.8		m/day – from geotechnical report

6.2. Analysis cases

Case-1. Normal condition with no impounding of water, and no karsting cavity/cavities

Only concrete gravity dam is placed over the weak limestone after removing the top 10m layer of riverbed material to see if the foundation strata would be able to resist the deadweight of the dam without significant settlement or bearing capacity failure. The water level is at 4.5 m depth. As were mentioned above, construction of downstream stilling basin is ignored to purely model the stresses and deformation caused only by the dam weight.

Case-2. Full Reservoir, and existence of elongated karsting cavity at the toe.

Reservoir is filled up to 65 m height (HRWL), with a karsting cavity at the toe. As were mentioned above the width of the cavity at the top is 11 m and then narrowing down up to a depth of 39th meter.

Mesh:

Plain strain model, and 15- node element were used to generate fine mesh for the Almar concrete gravity dam and its two soil strata as shown below in the Figure-18, and 19. Mesh quality were carefully examined for example it was sufficiently fine to get accurate numerical results and without delaying processing time.

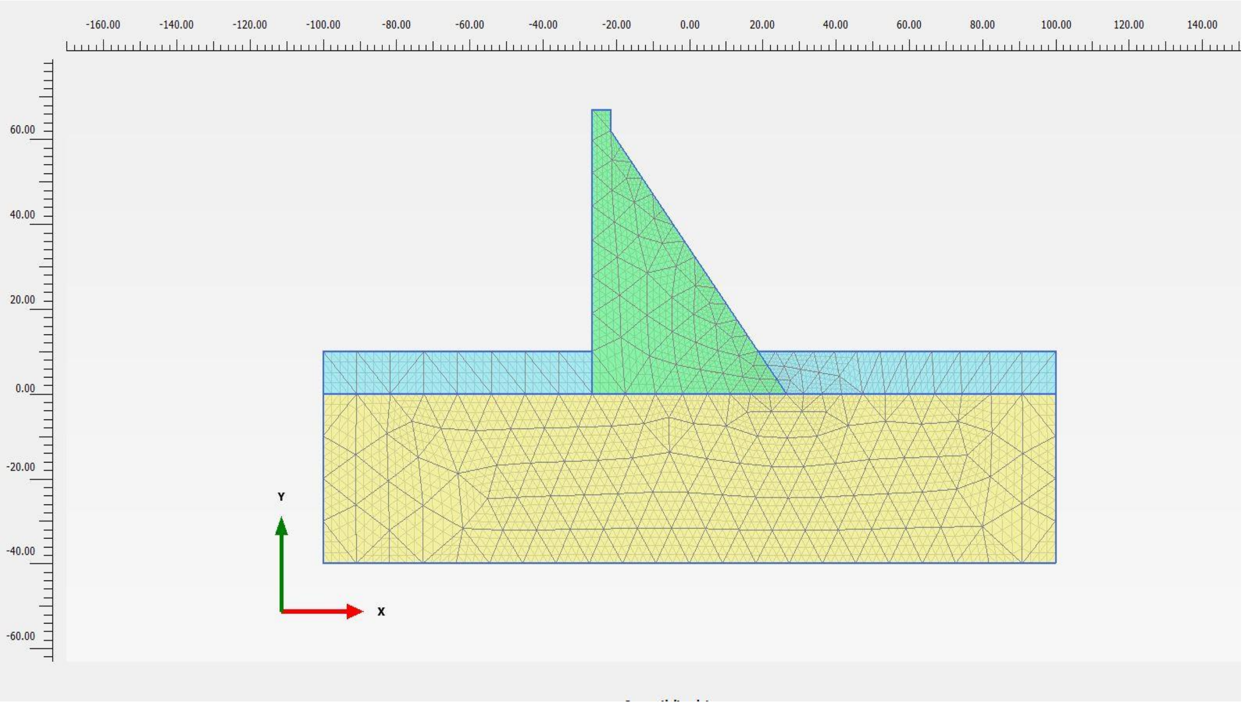


Figure 18: (Empty reservoir, with no cavity) generated mesh (361 elements, and 3099 nodes)

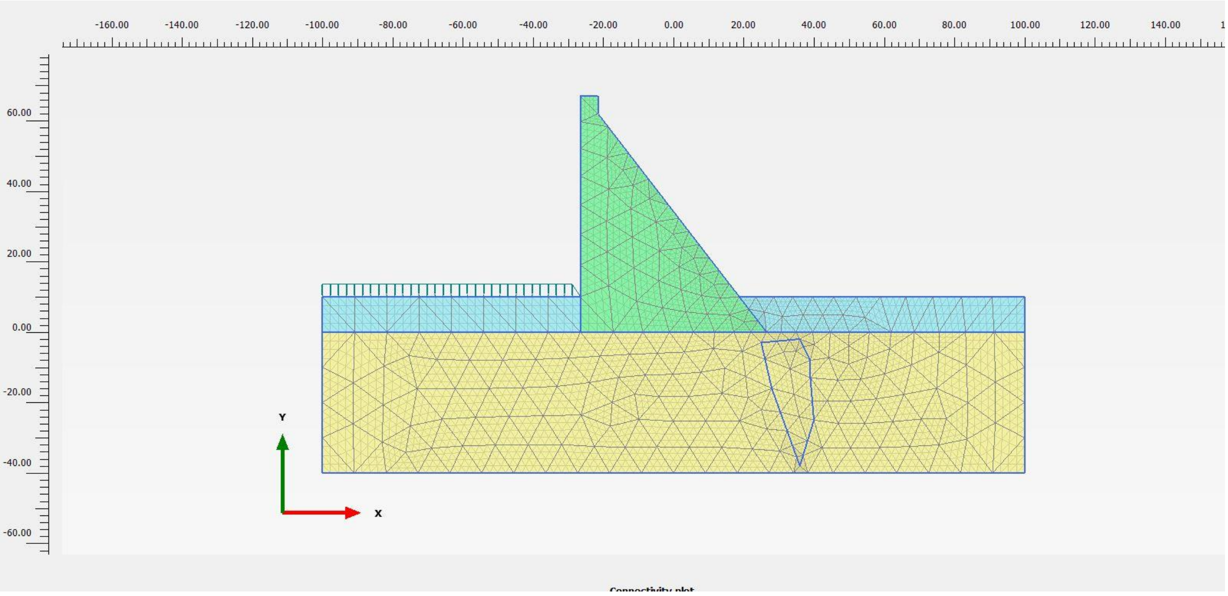


Figure 19: (Full reservoir, with karsting cavity) generated mesh (432 elements, and 3609 nodes)

Total displacement:

With empty reservoir, and no cavity, the high load of the concrete on upstream side of the dam, and weak foundation causes the displacement towards upstream, and the crest of the dam moved to upstream by 2.41 cm (Figures 20, and 21), while the average displacement in the foundation strata is 1.2 cm (Figure-20) confirming that there will be no bearing capacity failure at the foundation or in other words the concrete gravity dam will not sink in the foundation strata due to its high load. The figures are scaled up by the FE model to enhance visual clarity.

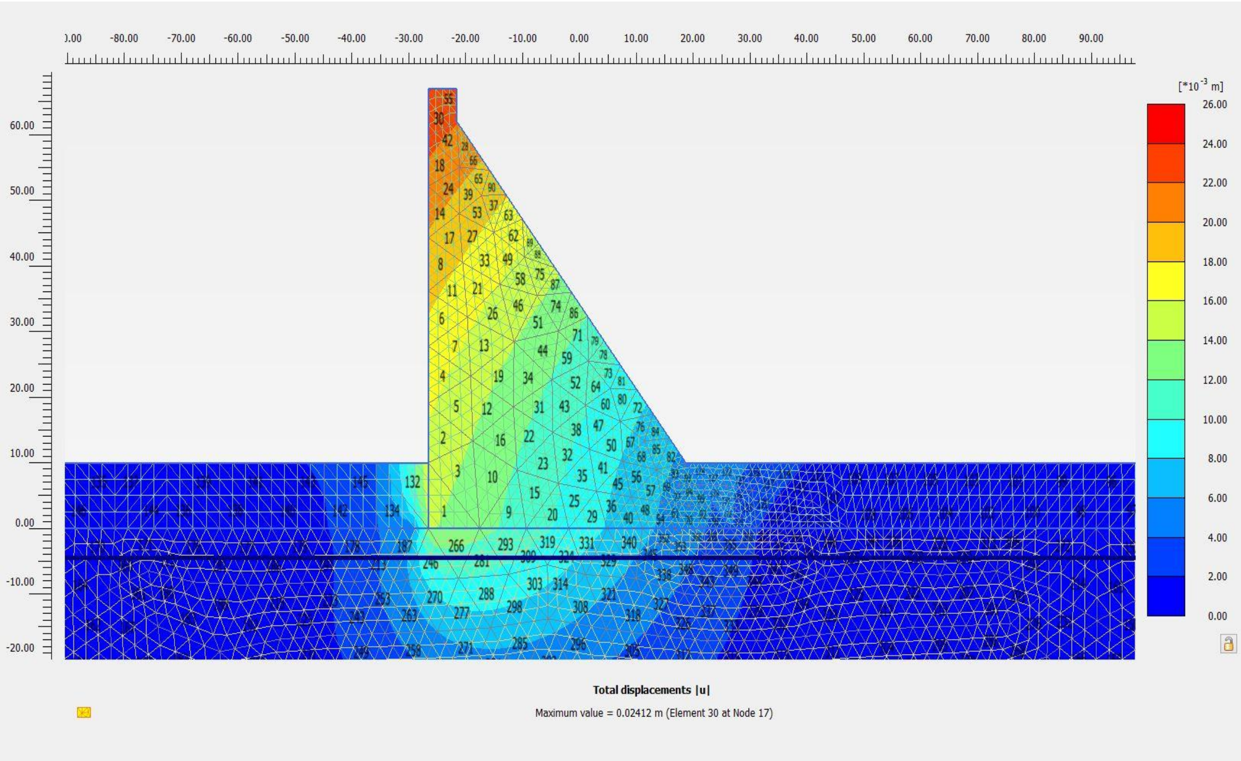


Figure 20: (empty reservoir, and with no cavity) Shows displacement contours, and a total displacement of 2.41 cm at the crest level, and up to 4 mm in the foundation stratum.

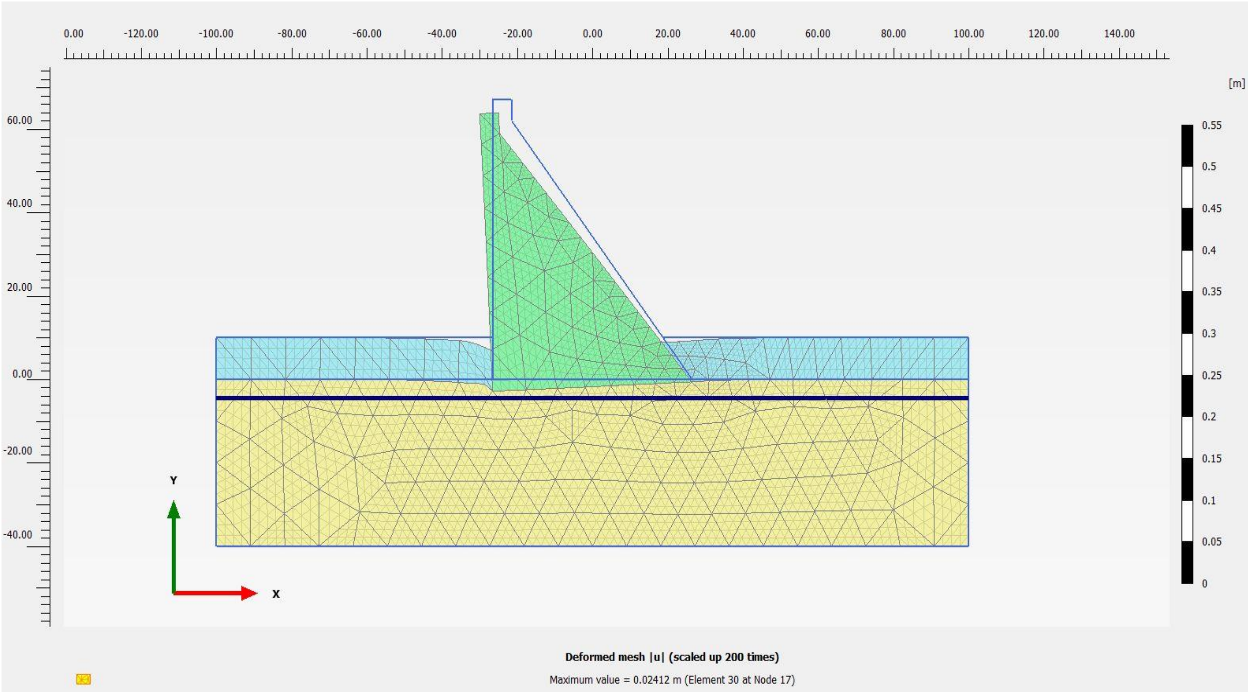


Figure 21: (empty reservoir, & no cavities) Deformed mesh geometry showing total displacement of 2.41 cm scaled up 200 times to enhance visual clarity

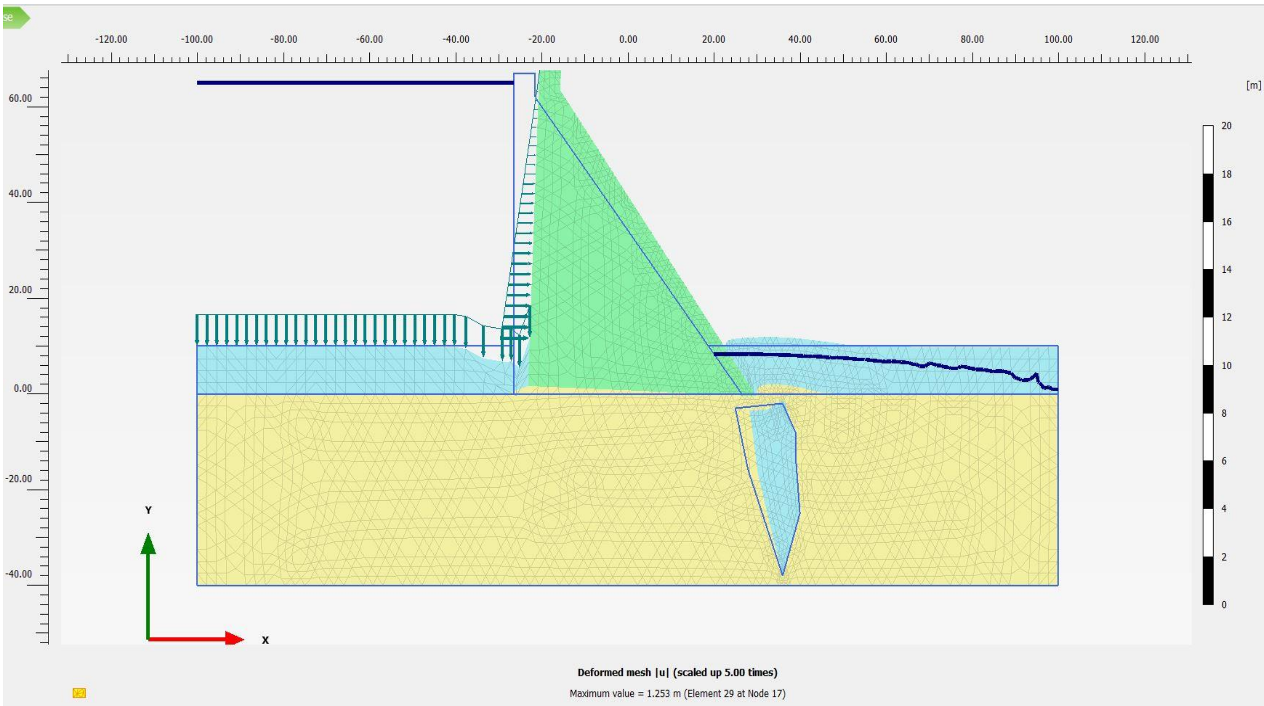


Figure 22: (Full reservoir with karsting cavity at the toe). Deformed mesh geometry showing a total downstream displacement of 1.25 m at the dam crest level due partial collapse of the cavity, and a heave of 39 cm at the toe is formed .

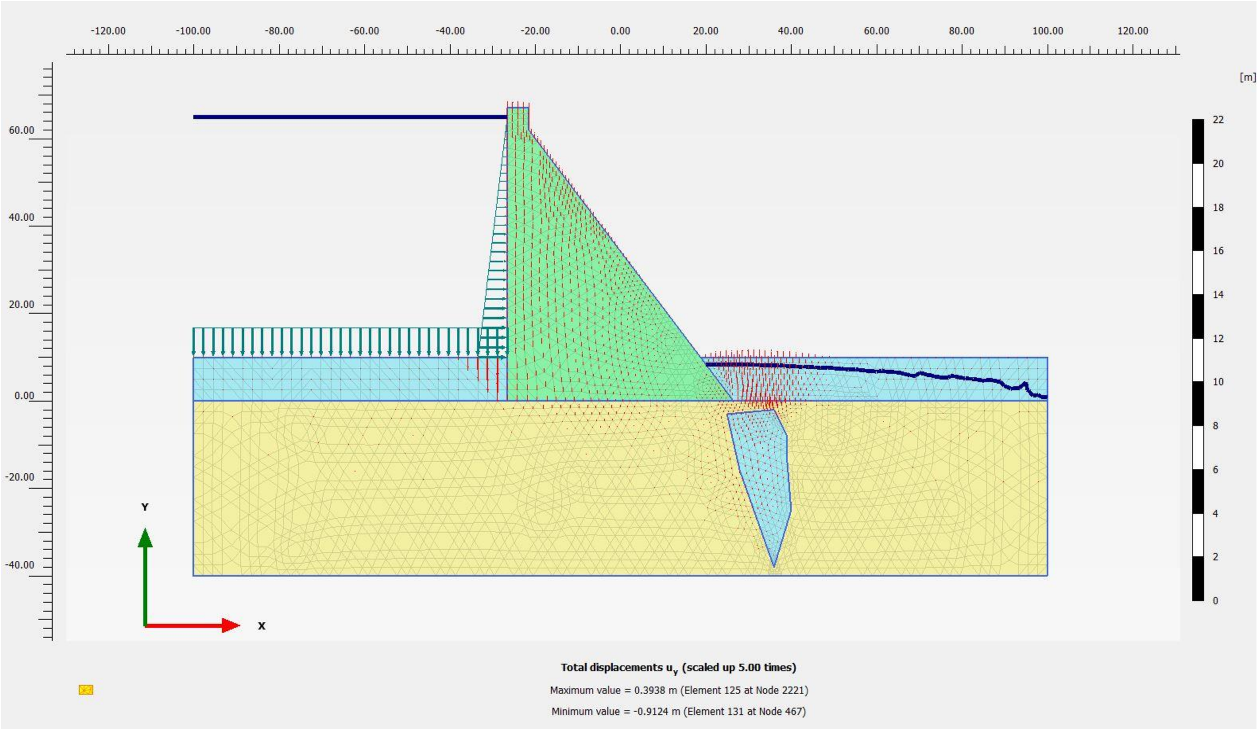


Figure 23. Total vertical displacement (Uy) showing formation of 39 cm heave above the karsting cavity

Discussion:

Existence of a karsting cavity at the toe of the dam, and with filled reservoir shows an enormous displacement of 1.25 m at the crest level towards downstream, and a maximum displacement of 600 mm in the foundation strata as shown in the graph of Figure-24, 25 which matches total settlement of 573mm estimated through handmade calculations. Karsting cavity is filled with riverbed material. When, cavity is filled with precipitated limestone (gypsum) then cavity fails completely preventing further process of the model causing failure of the dam.

The failure points shown in the Figure-26, and Figure-27 below outline the highest need of consolidation grouting for limestone foundations which adds additional cost to the operation and maintenance cost for the dams constructed on karsting limestone. Similarly, highest shear stress of 742 kN/m^2 developed near the cavity as shown in Figure-28 would further play a major role in the complete collapse of the cavity. High pore pressure as shown in the Figure-28 would further enlarge the cavity by enhancing the dissolution of limestone into gypsum. So, before even construction the dam, the cavity should be exposed and filled with concrete to prevent its further enlargement.

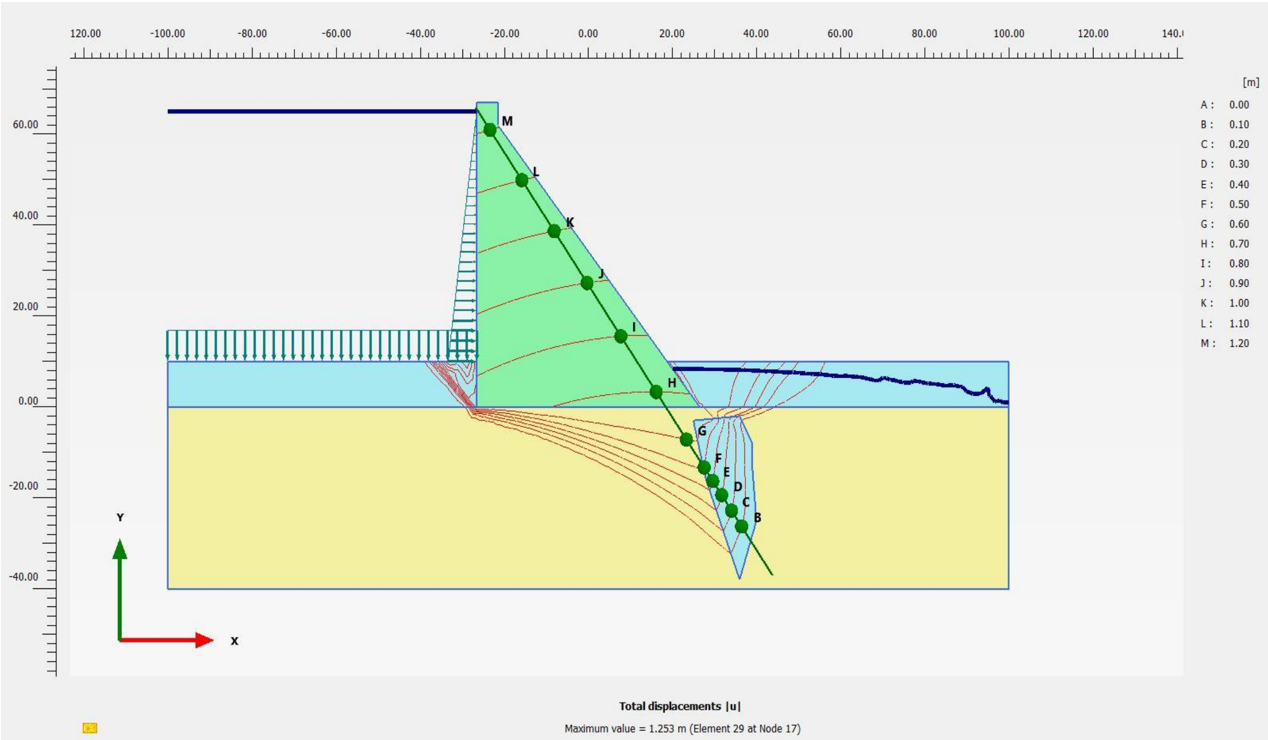


Figure 24: Contours showing various displacement levels after the partial collapse of the karsting cavity at the toe.

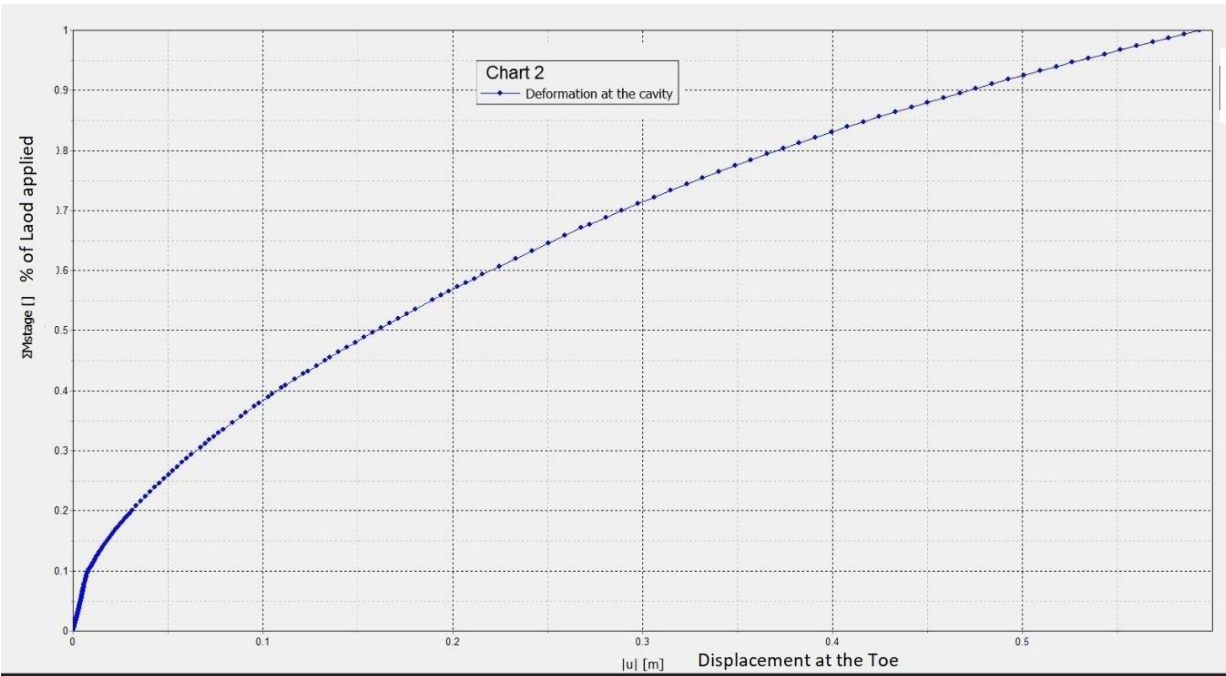


Figure 25: Load vs Displacement curve right below the toe (slightly above the cavity) of dam showing a total displacement of 0.6 m after applying full load (with full reservoir)

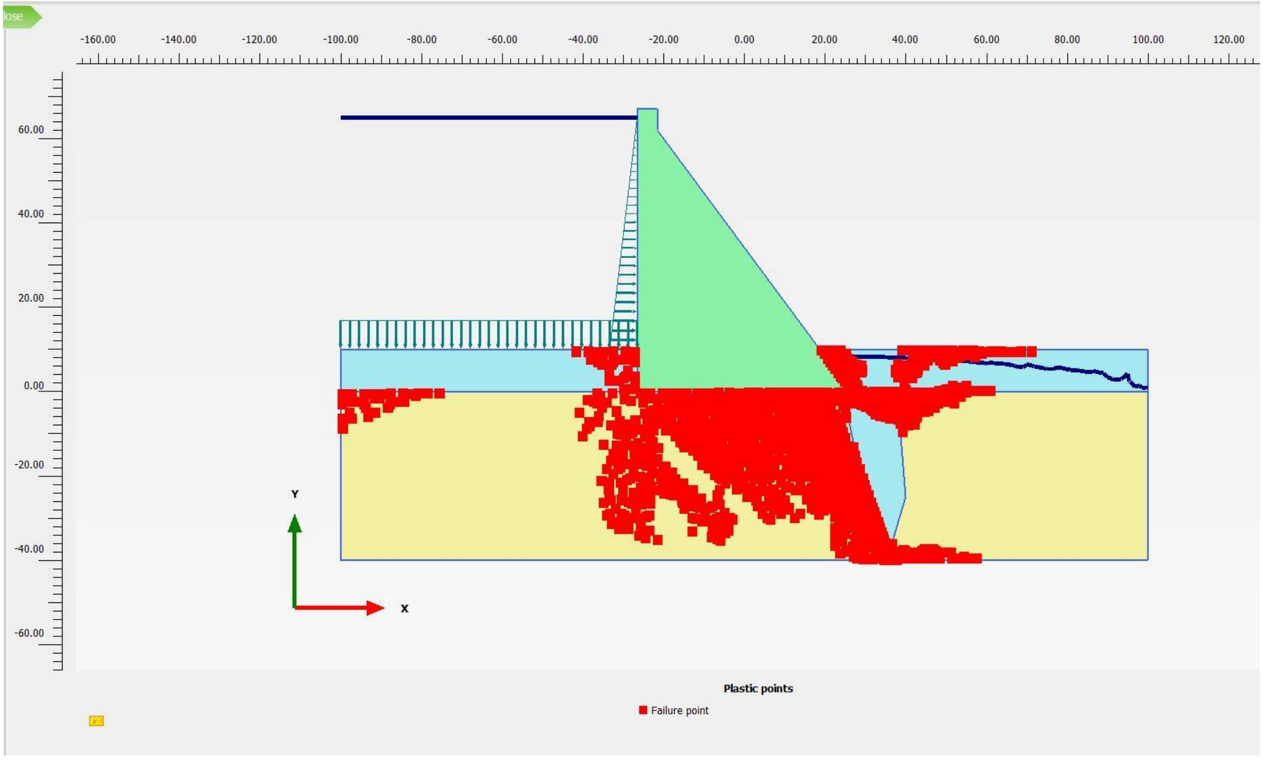


Figure 26: Failure points at the foundation going deep up to 40m in (the foundation strata) outlining the need of consolidation grouting for weak limestone foundation.

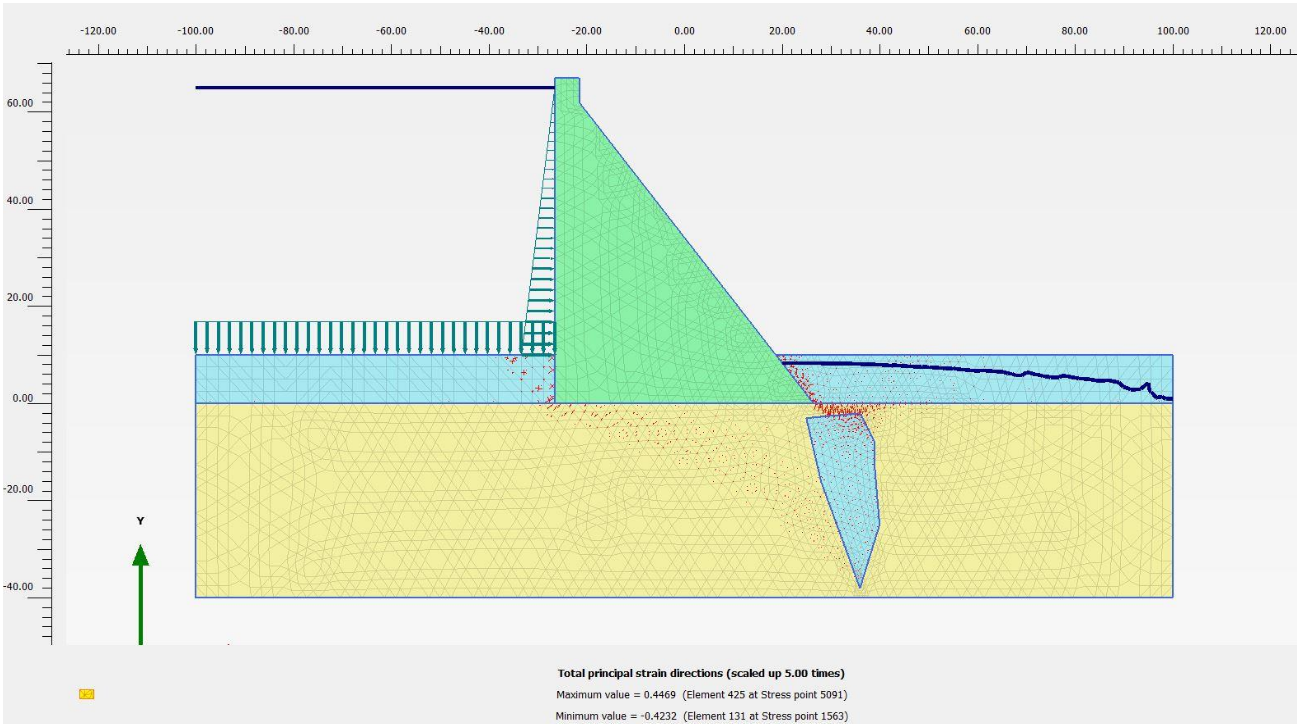


Figure 27. Principal Strain direction showing the areas of highest damage (shear failure)

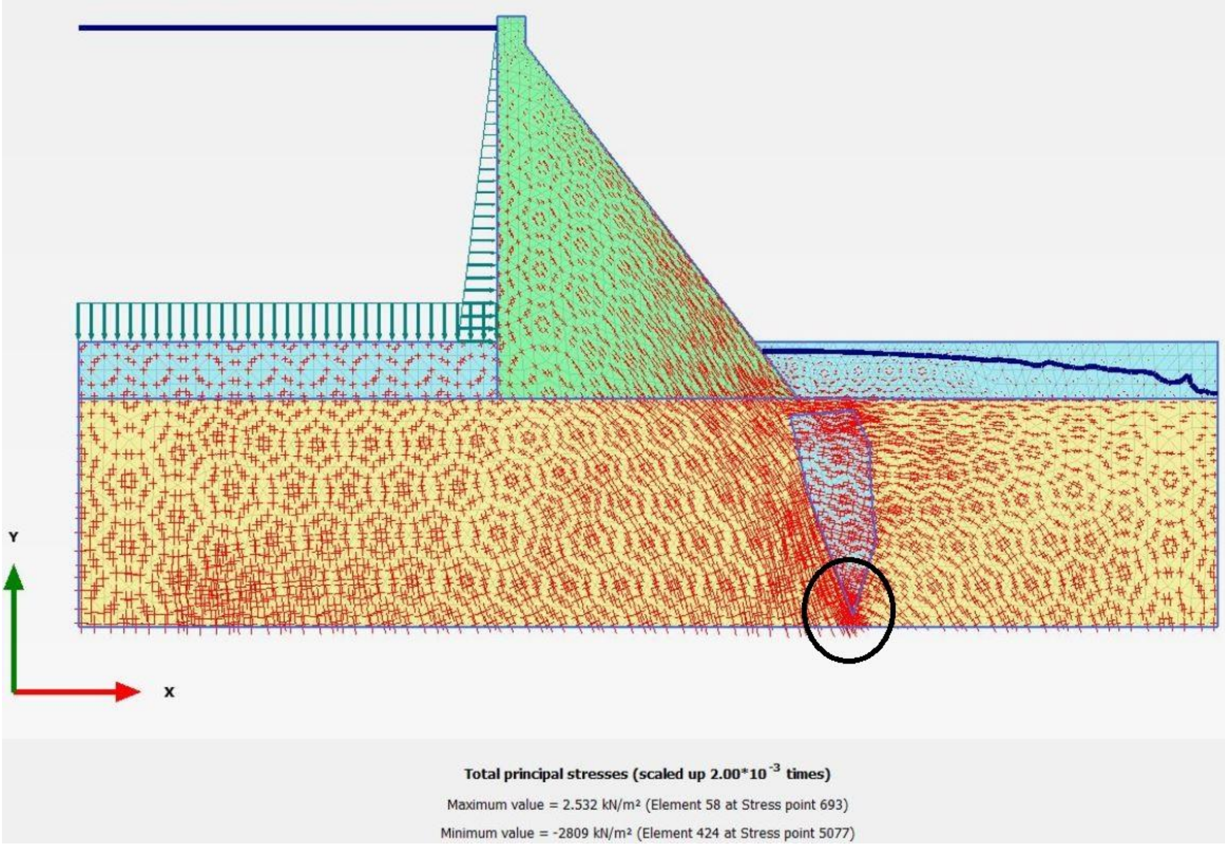


Figure 28. Plot of Total Principal Stresses accumulation at the cavity with highest concentration value of -2809 kN/m^2 at the circle shows that cavities present in the dam foundation can easily be collapsed.

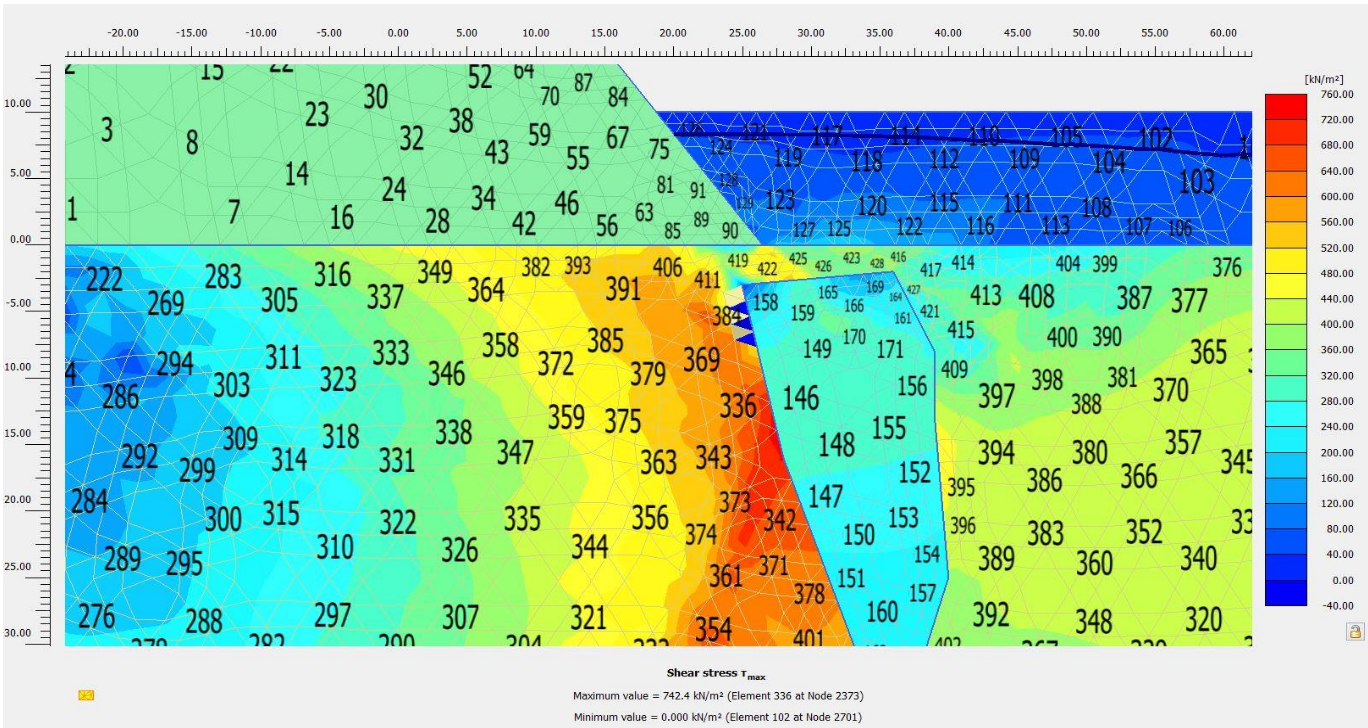


Figure 29. Highest shear stress of 742 kN/m^2 is developed near the cavity

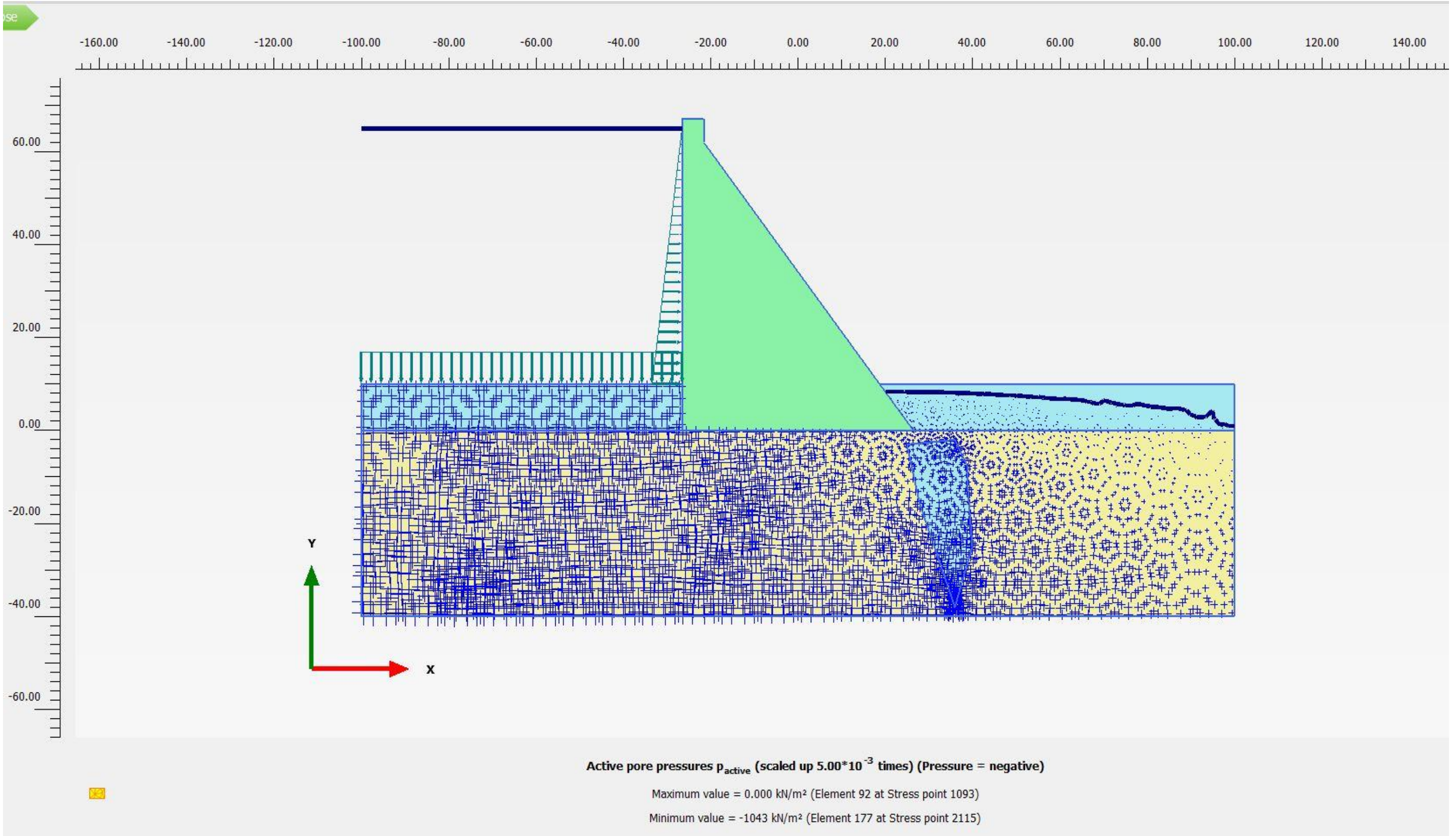


Figure 30. Active pore pressure built up at the upstream, and cavity will further improve the dissolution process resulting in the widening of the cavity.

Differential settlement

Differential settlements plots for both cases (i) empty reservoir, with no cavity, and (ii) filled reservoir with karsting cavity are shown in the below Figures of 31, and 32. Differential settlement induces stresses in the dam body for both cases should not exceed allowable stress in concrete. This can be seen from deviatoric stresses plots given in Figures 32, and 34.

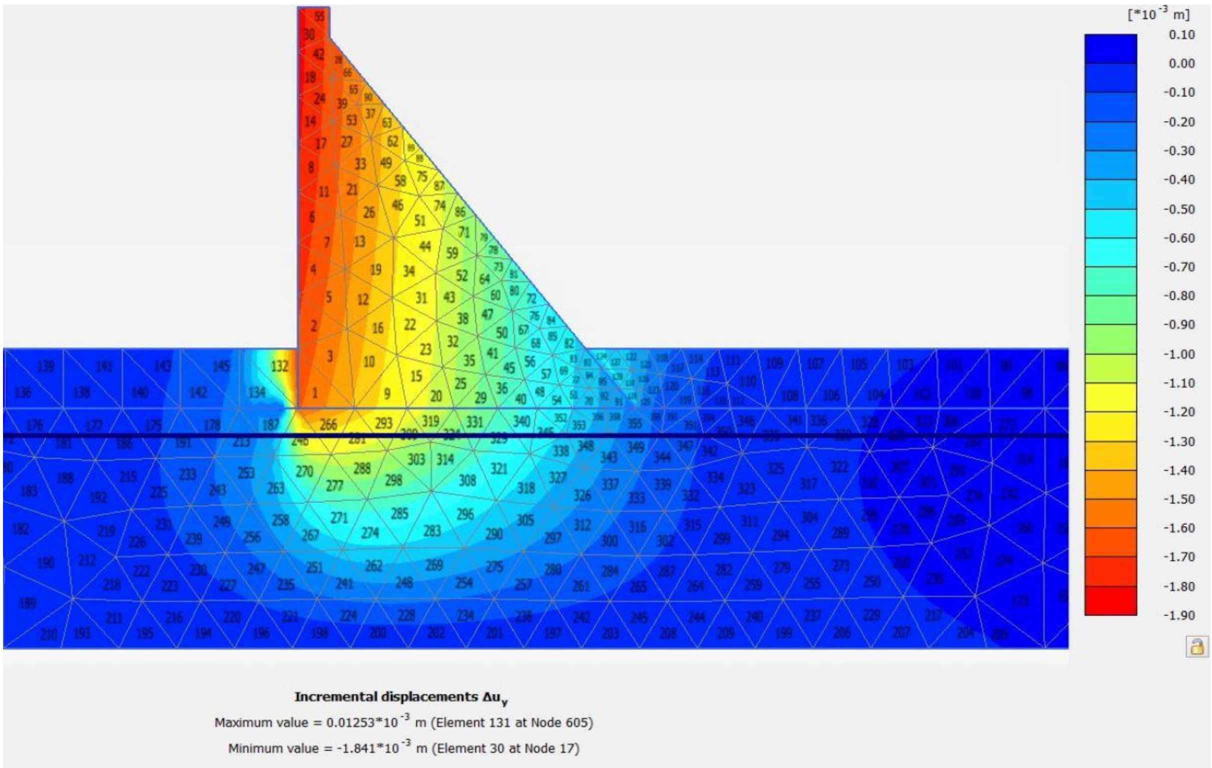


Figure 31: Differential settlement (empty dam with no cavity)

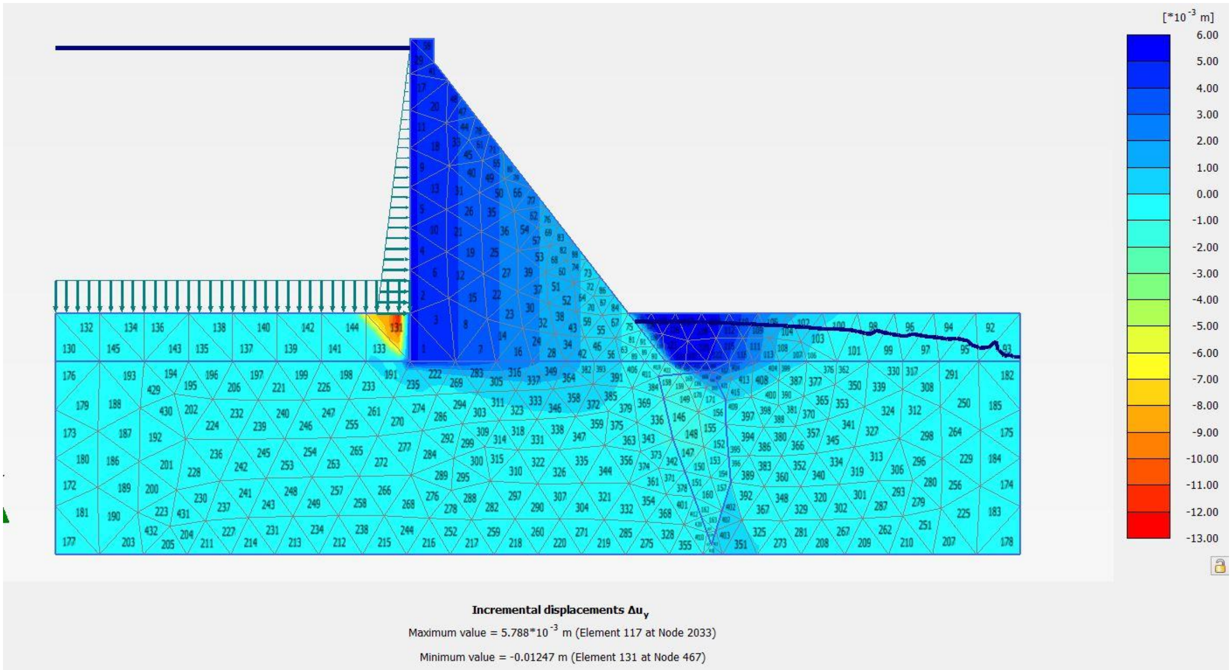


Figure 32: Differential settlement (Full reservoir with cavity)

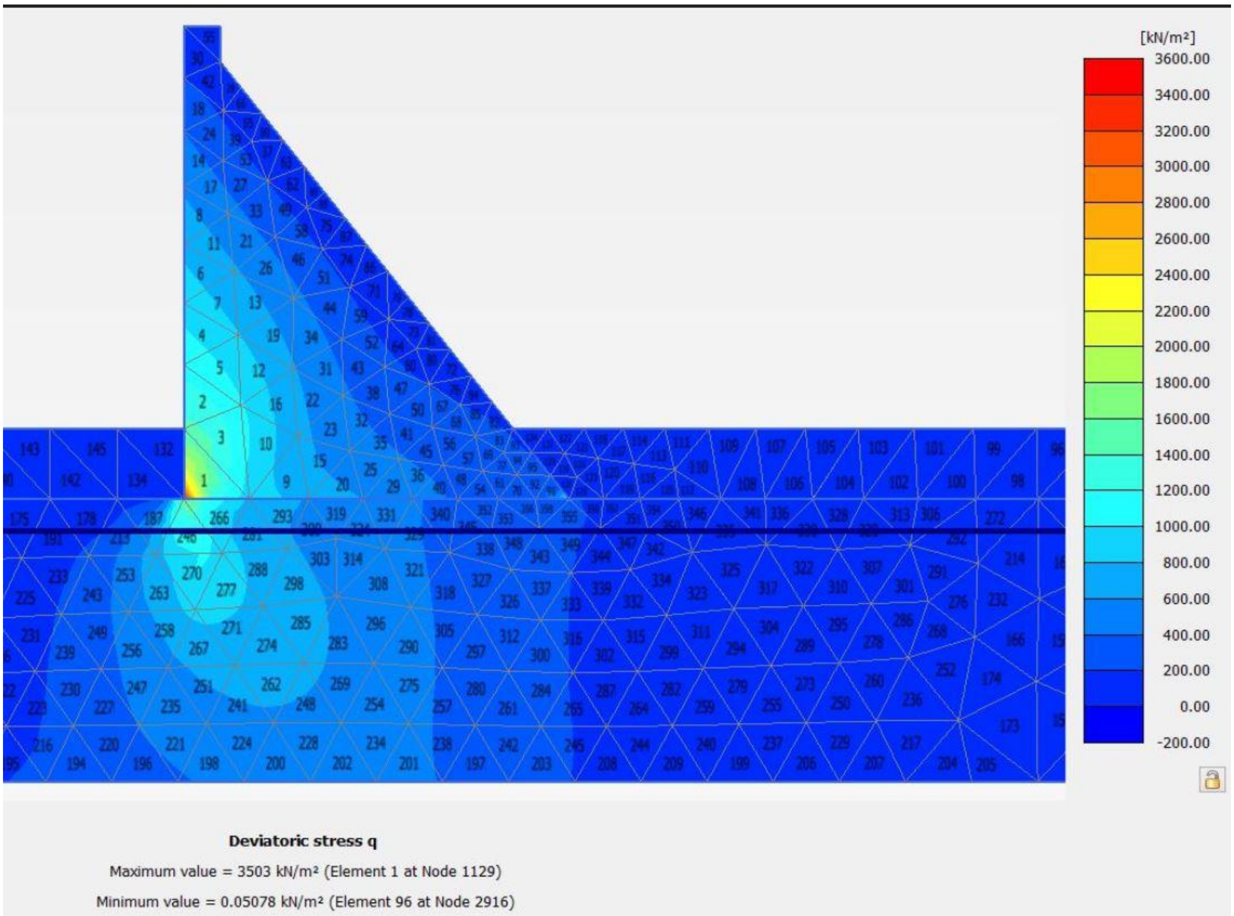


Figure-33: Deviatoric stress (empty dam with no cavity)

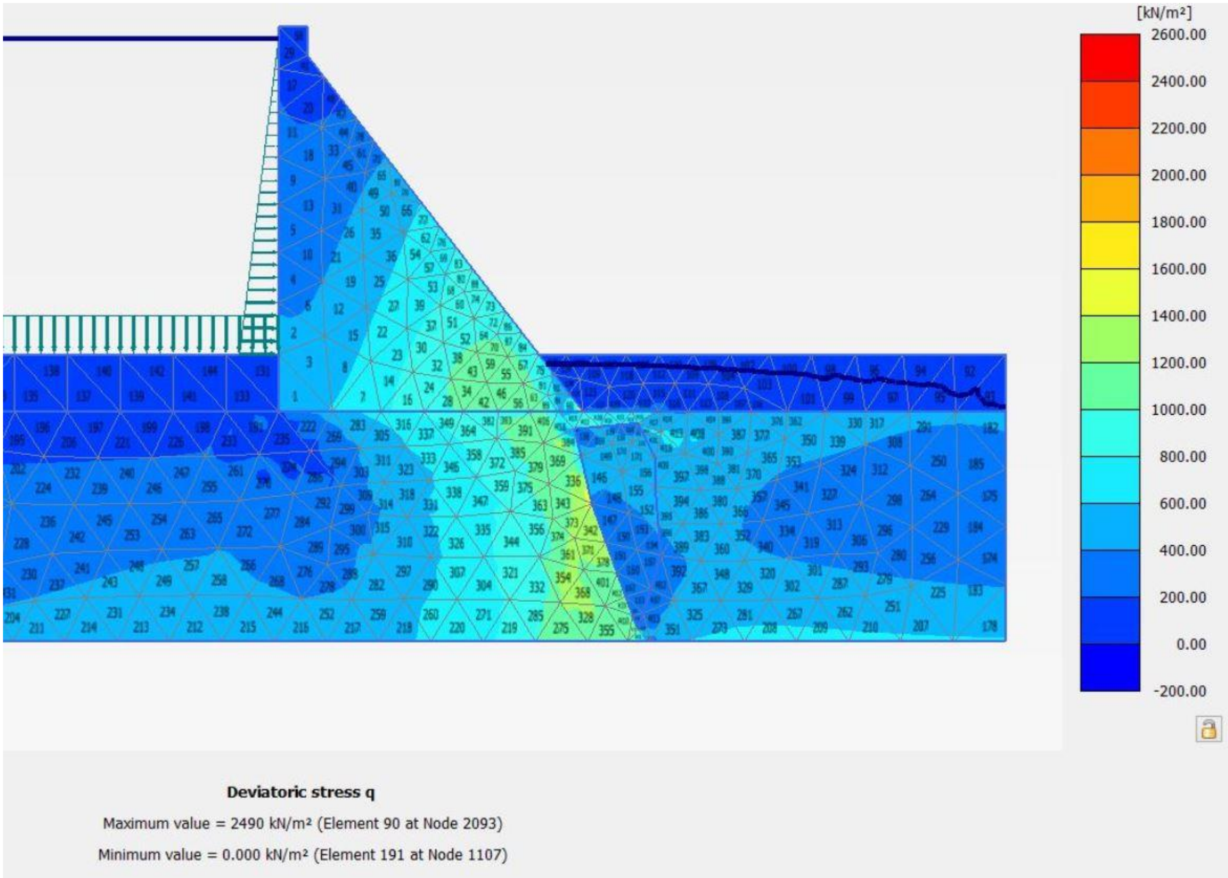


Figure-34: Deviatoric stress (Full reservoir with karsting cavity)

Discussion:

When, the stresses induced in the dam body due to differential settlement is less than the allowable concrete stresses then that limit of differential settlement is acceptable. The two plots of deviatoric stresses shown in the Figures-31, and 32 shows developed compressive stresses (2490 kN/m^2, and 3503 kN/m^2) are less than 4 MPa which is far less than the uniaxial compressive stress of concrete of 25 MPa. The value of highest deviatoric stress developed in toe of the full reservoir is less than the deviatoric stress developed at the heal of empty reservoir is probably because the deviatoric stress is distributed over a large section of the dam as it tilted, and cavity right below the toe is partially collapsed. Similarly, tensile stress develop is in the range of 0.95 MPa, while the allowable tensile stress as per the Eurocode is 1.8 MPa for $f_{c'}$ = 25MPa.

7.0 Conclusions and recommendations.

7.1 Conclusion:

Structural integrity of this 67m high concrete gravity dam will be under the serious risk as were found by this study if a total settlement of 573mm happens, and if the construction of the dam has strategic importance then embankment dam would be more appropriate to accommodate high settlements unless the cost of spillway prevent to do so. Careful, benefit cost analysis is required to compare different dam types.

Similarly, if the cavities in the foundations fails (as is predicted by the FE analysis a 1.25 m downstream displacement of the crest will occur due to the partial failure of karsting cavity) suddenly under extreme conditions such as flood event; failure will be catastrophic and requires an active dam surveillance program. Furthermore, high leakages through cavities are expected, and it might be very difficult to keep reservoir level at Highest Regulating Level reducing the effectiveness of the investment.

7.2 Recommendations:

1. There is definite need of conducting complete field and laboratory investigations. New boreholes needs to be drilled to confirm and augment the existing investigations, and acquire undisturbed samples for complete laboratory investigations.
2. New set of geophysical investigations are required to identify all existing cavities in the foundation rock. If cavities were found then they should be completely exposed and filled with concrete or other locally available material to prevent their sudden collapse, and thus possible failure of the dam
3. Detail hydrologic studies need to be conducted to see if the sub basin have surplus flow to be stored through this project otherwise the project does not worth spending
4. Structural integrity of the dam should be evaluated through various means including computer programs in case excessive settlement of 573mm happens and complete failures of cavity happens
5. Detail cost benefit studies are required to compare different dam options

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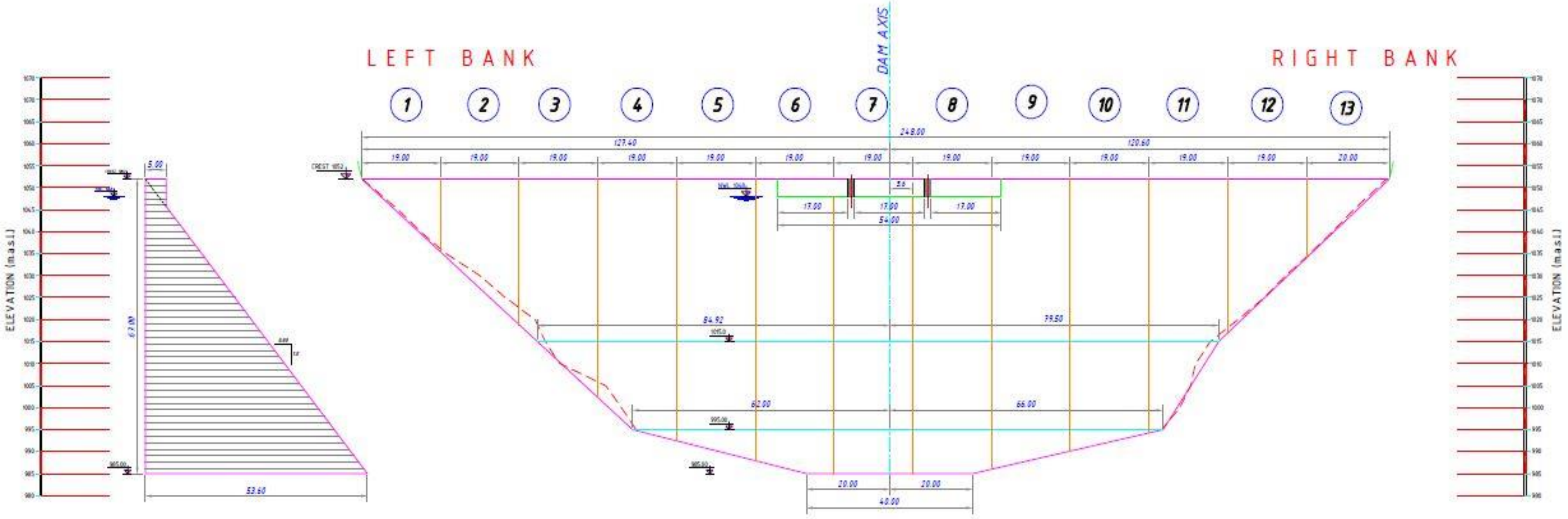
Annex-1. Drawings

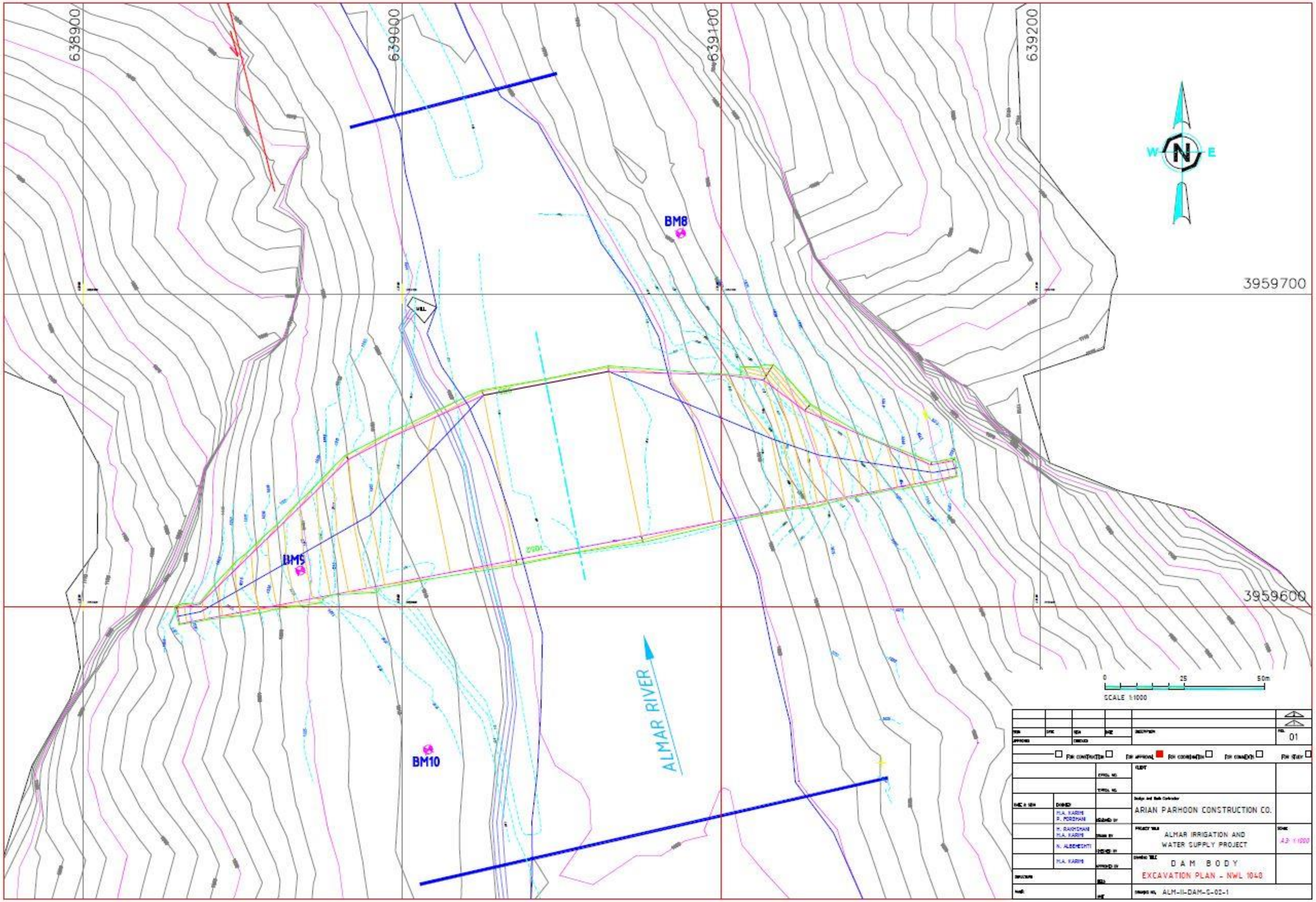


GEOMETRICAL DEFINITION - DAM VERTICAL (NON OVERFLOW) SECTION

0 25 50m

SCALE 1:1000

REV.	01
Status	FOR CONSTRUCTION ☐ FOR APPROVAL ■ FOR COORDINATION ☐ FOR COMMENTS ☐ FOR STUDY ☐
Designed by	M.A. KARIMI, P. PORDHANI
Drawn by	M. RAKHSHANI, M.A. KARIMI
Checked by	N. ALBEHESHTI
Approved by	M.A. KARIMI
Contractor	ARIAN PARHOON CONSTRUCTION CO.
Project title	ALMAR IRRIGATION AND WATER SUPPLY PROJECT
Scale	A3: 1:1000
Drawing title	DAM BODY DAM SECTIONS - NWL 1048
Drawing no.	ALM-II-DAM-S-01-1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639000
639100
639200
3959700
3959600
BM8
BM5
BM10
ALMAR RIVER
W
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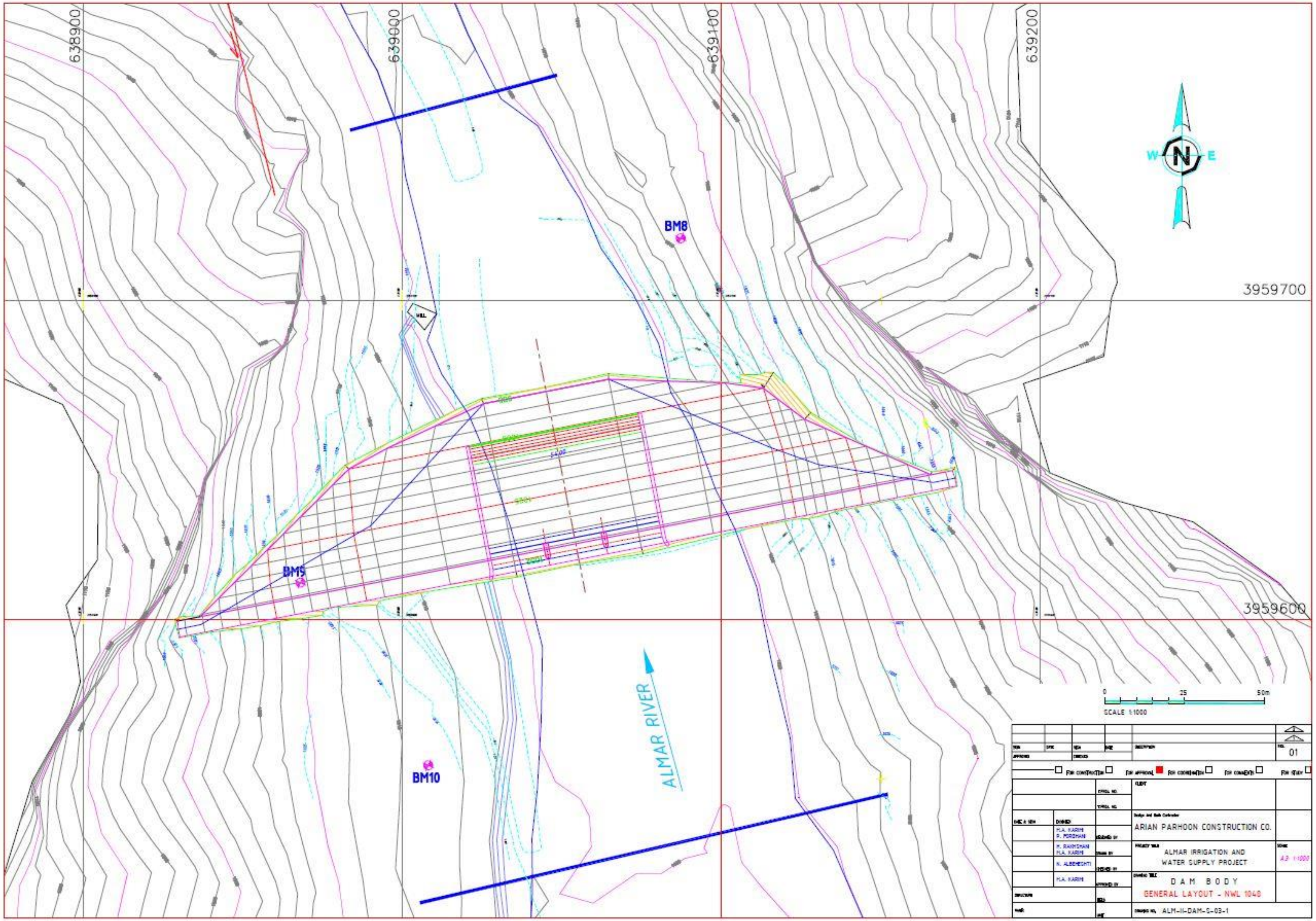
25
50m
SCALE 1:1000
01
FOR APPROVAL
ARIAN PARHOON CONSTRUCTION CO.
M.A. KARIMI
P. PORDHANI
H. RAKHSHANI
M.A. KARIMI
N. ALBEHESHTI
M.A. KARIMI
ALMAR IRRIGATION AND
WATER SUPPLY PROJECT
A3: 1:1000
D A M B O D Y
EXCAVATION PLAN - NWL 1040
ALM-II-DAM-S-02-1



638900
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639100
639200
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E
BM8
BM5
BM10
WELL
ALMAR RIVER
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50m
SCALE 1:1000
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FOR APPROVAL
ARIAN PARHOON CONSTRUCTION CO.
M.A. KARIMI
P. PORSHANI
M. RAKHSHANI
M.A. KARIMI
K. ALBEHESHTI
M.A. KARIMI
ALMAR IRRIGATION AND
WATER SUPPLY PROJECT
A3 1:1000
D A M B O D Y
GENERAL LAYOUT - NWL 1040
ALM-II-DAM-S-03-1

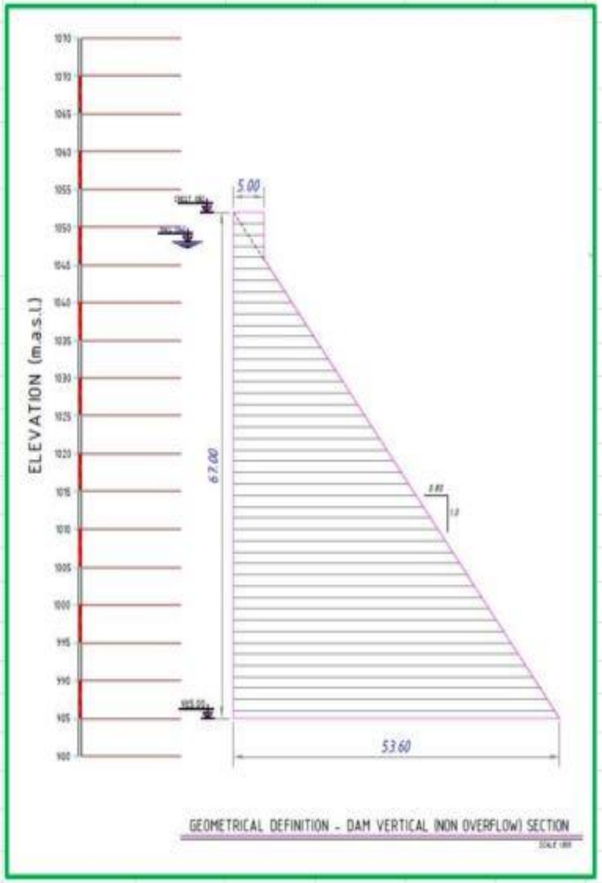
Annex-2. Dam Stability Calculations

Normal conditions with good foundation

Almar Gravity Dam (Afghasitan) **Given Data:**

Normal conditions with good foundation

Dam Height (h, m)= 67	Sediment Hs (m)= 33
Heighest water level, HRWL (m)= 64	Friction angle (φ, degree) of river sand and gravel= 35
Design flood level (H, m)= 65	Submerged weight of sediment (Ysub, KN/m3)= 21
U/s face slope (V:H, m)= 0:00	
n1= 0	
D/s face slope (V:H, m)= 1 : 0.8	
n2= 0.8	
Crest width (B, m)= 5	
Tail water depth (H2, m)= 0	
Cohesion between dam foundation and limestone/dolomite (C, KN/m^2) = 600	
Friction angle (φ, degree)= 25	
angle of inclination/aspierities (i, degrees)= 0	
Unit wt of water (Υ_w KN/m^3) = 10	
Dam base width (B_B, m)= 53.3	
Ice load (P, KN/m)= 50	
B_1(m)= 48.3	
B_2(m)= 5	
B_3(m)= 0	6.7
Unit wt of concrete (Υ_c KN/m^3) = 24	



Find all the forces acting on the dam

Step-A Find all the forces acting on the dam

No	Type of load	Load (KN/unit length)	Level Arm (m)	Moment (KN-m)	Comment
1	load due to water, $h_v = \Upsilon_w . H^2/2$	**21125**	21.667	457708.3	Overtrun
2	load due to vertical column of water at sloped surface, $V_v = \Upsilon_w . n1.H^2/2$	**0.00**	53.300	0.0	stabilizinng
3	Load due to pore pressure $U = \Upsilon_w . B_B . H/2$	**17322.5**	35.533	615526.2	Overtrun
4	Self weight, G1= ϒc.h.B1/2	**38833.2**	32.200	1250429.0	stabilizinng
5	Self weight, G2= ϒc.h.B2	**8040**	50.800	408432.0	stabilizinng
6	Self weight, G3= ϒc.h.B3/2	**0**	55.533	0.0	stabilizinng
7	Ice Load	**50**	63.75	3187.5	Overtrun
8	River Sediment load Ps=0.5. $\Upsilon sub. H^2 s. (1-\sin\varphi/1+\sin\varphi)$	**3098.636**	11	34085.0	Overtrun

Step-B. Find factor of safety bor both overturning, FS_{OT}, and Sliding FS_s

$$FSot = \frac{\Sigma M\ stablizing}{\Sigma M\ over\ turning} = \frac{Mvv + Mg1 + Mg2 + Mg3}{Mvh + Mu + Mice + Msedi}$$

where, Pice, and Mice = 0

FS_{OT}= 1.49

FS_o > 1.25 is accetable, while F FSo > 1.5 desireable (Kennard, Ownen, 1996), and is very stable

$$FS\ sliding = \frac{C.A + \Sigma Fv.Tan\phi}{\Sigma Fh} = \frac{(Vv + G1 + G2 + G3 - U)\,Tan\phi}{(hv + Pice + Ps)}$$

$FS_{Sliding}$= 1.89

If cohesion (c) is considred then, FS against sliding should be at minimum 2, So dam is not safe against extreme conditions

If cohesion is tested then, FS against sliding should be at minimum 1.5, So dam is safe against extreme conditions

In this case, C values is based on the measured value

Step- C. To check if the Resultant Force stays within $B_B/3$

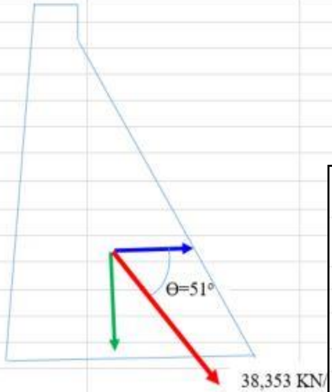
$$no\ tensile\ stress\ if\ a_R > \frac{B_B}{3} \qquad a_R = \frac{\sum M_R}{\sum F_{vertical}}$$

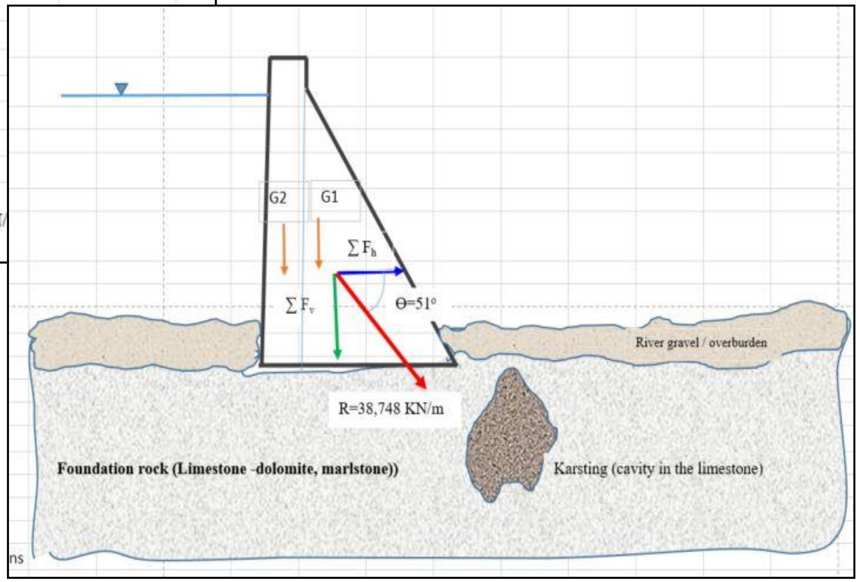
Lets check safety of the dam by checking resultant

No	Type of load	Load (KN/unit length)	Levell Arm (m)	Moment (KN-m)	$B_3/3$	a_R	
1	load due to water, $h_v= Ɣ_w. H^2/2$	**21125**	21.7	457708.3	**17.77**	**18.6**	NO risk of Tesile stress at the toe, and the dam is safe against overtuning
2	load due to vertical column of water at sloped surface, $V_v= Ɣ_w.n1.H^2/2$	**0.0**	51.1	0.0			
3	Self weight, G1= Ɣc.h.B1/2	**38833.2**	32.2	1250429.0			
4	Self weight, G2= Ɣc.h.B2	**8040**	50.8	408432.0			
5	Self weight, G3= Ɣc.h.B3/2	**0**	55.5	0.0			
6	Uplift	**17322.5**	35.53	615526.2			
7	Ice Load	**50**	63.75	3187.5			
8	River Sediment load Ps=0.5. Ɣsub. H^2s. (1-sinφ/1+sinφ)	**3098.636**	11.00	34085.0			

Step-D. **To find Reultant (R) of all forces**

∑ H (Horizental forces, KN)=	24273.6
∑ V (Verrtical forces, KN)=	29550.7
Resultant Force, R (KN)=	38242.04
Direction for R (Θ, **degrees)=**	51





When Cavity fails:

When Cavities at the toe of the Dam collapse suddenly under High Pressure			
Given Data:			
Dam Height (h, m)=	67	Sediment Hs (m)=	33
Heighest water level, HRWL (m)=	64	Friction angle (φ, degree) of river sand and gravel=	35
Design flood level (H, m)=	65	Submerged weight of sediment (ϒsub, KN/m3)=	21
U/s face slope (V:H, m)=	0:00		
n1=	0		
D/s face slope (V:H, m)=	1 : 0.8		
n2=	0.8		
Crest width (B, m)=	5		
Tail water depth (H2, m)=	0		
Cohesion between dam foundation and limestone/dolomite (C, KN/m^2) =	600		
Friction angle (φ, degree)=	25		
angle of aspierities (i, degrees)=	0		
Unit wt of water ($\Upsilon_{w,}$ KN/m^3) =	10		
Dam base width (B_B, m)=	53.3	37	
Ice load (P, KN/m)=	50		
B_1(m)=	48.3	32	
B_2(m)=	5	5	
B_3(m)=	0	0	
Unit wt of concrete ($\Upsilon_{c,}$ KN/m^3) =	24		

Step-1 Find all the forces acting on the dam

No	Type of load	Load (KN/unit length)	Level Arm (m)	Moment (KN-m)	Comment
1	load due to water, $h_v = \gamma_w . H^2/2$	**21125**	21.667	457708.3	Overtrun
2	load due to vertical column of water at sloped surface, $V_v = \gamma_w . n1 . H^2/2$	**0.00**	37.000	0.0	stabilizinng
3	Load due to pore pressure $U = \gamma_w . B_B . H/2$	**17322.5**	35.533	615526.2	Overtrun
4	Self weight, $G1 = \gamma c.h.B1/2$	**38833.2**	21.333	828441.6	stabilizinng
5	Self weight, $G2 = \gamma c.h.B2$	**8040**	34.500	277380.0	stabilizinng
6	Self weight, $G3 = \gamma c.h.B3/2$	**0**	37.000	0.0	stabilizinng
7	Ice Load	**50**	63.75	3187.5	Overtrun
8	River Sediment load Ps=0.5. $\gamma sub. H^2 s. (1-\sin\varphi/1+\sin\varphi)$	**3098.636**	11	34085.0	Overtrun

Step-B. Find factor of safety bor both overturning, FS_{OT}, and Sliding FS_s

$$FSot = \frac{\Sigma M\ stablizing}{\Sigma M\ over\ turning} = \frac{Mvv + Mg1 + Mg2 + Mg3}{Mvh + Mu + Mice + Msedi}$$

where, Pice, and Mice = 0

FS_{OT}= 1.00

FS_o<1.1, and it could fail under any condition

$$FS\ sliding = \frac{C.A + \Sigma Fv.Tan\phi}{\Sigma Fh} = \frac{(C.A) + (Vv + G1 + G2 + G3 - U)\,Tan\phi}{(hv + Pice + Ps)}$$

$FS_{Sliding}$= 1.48

If cohesion is considred then, FS against sliding should be at minimum 2, So dam is not safe against extreme conditions

Step- C. To check if the Resultant Force stays with $B_B/3$

$no\ tensile\ stress\ if\ a_R > \frac{B_B}{3}$ $\qquad a_R = \frac{\sum M_R}{\sum F_{vertical}}$

Lets check safety of the dam by checking resultant

No	Type of load	Load (KN/unit length)	Levell Arm (m)	Moment (KN-m)	$B_3/3$	a_R	
1	load due to water, $h_v = \Upsilon_w . H^2/2$	**21125**	21.7	457708.3	**12.33**	**-0.2**	High Risk of Tesile stress at the toe, and dam will fail by overtuning
2	load due to vertical column of water at sloped surface, $V_v = \Upsilon_w . n1.H^2/2$	**0.0**	37.0	0.0			
3	Self weight, G1= Ƴc.h.B1/2	**38833.2**	21.3	828441.6			
4	Self weight, G2= Ƴc.h.B2	**8040**	34.5	277380.0			
5	Self weight, G3= Ƴc.h.B3/2	**0**	37.0	0.0			
6	Uplift	**17322.5**	35.53	615526.2			
7	Ice Load	**50**	63.75	3187.5			
8	River Sediment load Ps=0.5. Ƴsub. H^2s. (1-sinφ/1+sinφ)	**3098.636**	11.00	34085.0			

Step-D. **To find Reultant (R) of all forces**

∑ H (Horizental forces, KN)=	24273.6
∑ V (Verrtical forces, KN)=	29550.7
Resultant Force, R (KN)=	38242.04
Direction for R (ϴ, **degrees)=**	41

Step E:

Annex-3. Bearing capacity calculations

Bearing Capacity of Almar Dam foundation

Using Terzaghi and Vesic Equations

Project :	Almar Dam
Date:	9-Oct-18

Input Parameters

Unit of Measurement

	SI	SI or E

Foundation Information

Shape =	CO	SQ, CI, CO, or RE
width of foundation, B=	53.3	m
Length of foundation, L=	1	m
Depth of foundation, D=	10	m

Soil/rock Information

c=	600	kPa
ϕ=	25	deg
γ=	21	KN/m^3
Dw=	4	m

Factor of Safety

FS=	3

Reference sheet by : Donald Cudoto.

Results

Bearing Capacity	**Terzaghi Method**	**Vesic Method**
q ult =	19,744 kPa	18,318 kPa
q a =	6,581 kPa	6,106 kPa

Stress due to concrete load at the Heal of the dam= 1608 Kpa

So, there is no risk of bearing capacity fialure if no karsting cavities exist at the heal of the dam

Unit conversion	1	
Gamma w =	9.8	KN/m^3
phi (radians)	0.436	

Terzaghi Computations

a theta =	2.71
Nc =	25.13
Nq =	12.72
N gamma =	9.18
gamma' =	11.2
coefficient #1 =	1
coefficient #3 =	0.5
sigma zD' =	151.2

Vesic Computation

Nc =	20.72
sc =	1.00
dc =	1.08
Nq =	10.66
sq =	1.00
dq =	1.06
N gamma =	10.88
s gamma =	1.00
d gamma =	1.00
B/L =	0
k =	0.187617
W sub f	0

Annex -4. Estimation of Induced Stresses

1. Newmark solution of Boussinesq's Equation to find induced vertical stress at depth of 15 meter

Given data

depth of water d(m)=	4.5
Depth of foundation, D(m)=	10
Width of Dam fondation, B(m)=	53.3
L(m)=	175
Vertical distace from the bottom of the dam foundation to the point of interest Z_f(m)=	30
$\gamma_{overburdon}$ (KN/m^3)=	21
γ_{water}(KN/m^3)=	10
I_Q	?
$B^2+L^2+Z_f^2$	34365.89
B^2L^2/Z_f^2	96669.17

$$\Delta\sigma_z = I_Q(q - \sigma_{zD}')$$

where,

$\Delta\sigma$ = induced stress in the foundation due to load P

I_Q= Stress influence factor

q=bearing pressure along bottom of the foundation

σ_{zD}'= verticval effective stressat a depth below the ground surface

$$I_Q = \frac{1}{4\pi}\left[\frac{2BLZ_f\sqrt{B^2+L^2+Zf^2}}{Zf^2(B^2+L^2+Zf^2)+B^2L^2}\left(\frac{(B^2+L^2+2Zf^2)}{(B^2+L^2+Zf^2)}\right)+Sin^{-1}\frac{2BLZf\sqrt{(B^2+L^2+Zf^2)}}{Zf^2(B^2+L^2+Zf^2)+B^2L^2}\right.$$

So, B2+L2+Zf2 > B2L2 /Zf2, and I have to use below equation

I_Q=	**0.236144**
q(KN)=	$\frac{P/b - W/b}{B}$
b (m)	1
weight of the footing, W(kN/m)=	0.0
P (load, KN/m)=	46873.2
q(KN)=	879.4
σzD '	155
Δσ (KN/m)=	171.1

The induced stress estimated by Newmarks equation doe not seem right and has under estimated the stress developed after constructing 67 meters hight dam

2. Simplified Method

$$\Delta\sigma_z = \{1-(\frac{1}{1+(\frac{B}{2Zf})^2}))^{2.6}\}\,(q-\sigma'_{zD})$$

Δσ (KN/m)=	**564.8**

Ans. So, up to the depth of 16 meter the induced stress are enough to cause karsting cavity collapse.

Annex-5. Almar Dam Settlements

Almar Dam settlment Analysis (Classical Method)

Input

Units	SI	**E or SI**
Shape	CO	**SQ, CI, CO, or RE**
B=	53.3	
L=	1	
D=	10	m
P=	31663	m
D_w=	4	
r=	0.85	

γ soil (KN/m^3)=	22.5	Layer 5
γ soil (KN/m^3)=	22	Layer 3, 4
γ soil (KN/m^3)=	21.5	layer 1, 2
γ soil (KN/m^3)=	19.5	Over burdone unit weight
γ_w (KN/m^3)=	10	
Bearing Pressure, q (Kpa)=	594	
Effective stress caused by over burdone, σ'_{zD} (KPa)=	135	
Effective stress at the middle of each layer, σ'_{z0} (KPa)		
Thickness of each layer, H (M)		
Hydrostatic pressure, U (Kpa)=		
Induced stress at the middle of each layer, $\Delta\sigma_z$ (Kpa) =		
Final effective stress at the middle of each layer, $\Delta\sigma_{zf}$ (Kpa) =		
Over consolidation margin, $\Delta\sigma_m$ (Kpa) =		

Equations used are given below

1 $\sigma'_{z0} = \gamma_{soil}\,H - u$

2 $\Delta\sigma_z = \{1 - (\frac{1}{1+(\frac{B}{2Zf})^2})^{2.6}\}\,(q - \sigma'_{zD})$

3 $\sigma'_{zf} = \sigma'_{z0} + \Delta\sigma'_z$

Layer No.	H	Z_f (m)	Dw	σ'_{z0} (KPa)	$\Delta\sigma_z$ (KPa)	σ'_{zf} (KPa)	σ'_c (KPa)	Case	$C_e/(1+e_0)$	$C_r/(1+e_0)$	δ (mm)
1	3.0	1.50	0.00	152	459.05	611.30	452	OC-II	0.13	0.04	92
2	3.0	4.50	0.00	187	459.01	645.76	487	OC-II	0.13	0.04	83
3	6.0	9.00	0.00	241	457.83	698.58	591	OC-II	0.13	0.04	128
4	8.0	16.00	0.00	325	444.53	769.28	675	OC-II	0.13	0.04	137
5	10.0	21.00	0.00	435	421.17	856.42	835	OC-II	0.15	0.05	134
										Total settlement=	**573**