NTNU – Trondheim
Norwegian University of
Science and Technology

THROUGHFLOW CAPACITY OF DOWNSTREAM SLOPE AND RIP RAP STRUCTURE OF KULEKHANI DAM

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Hydropower Development
Submission date: June 2013
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Norwegian University of Science and Technology
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MASTER THESIS

NTNU
Norges teknisk-naturvitenskapelige universitet

Fakultet for ingeniørvitenskap og teknologi
Institutt for vann- og miljøteknikk

MASTER THESIS

Student: *Pujan Bajracharya*

Title: THROUGHFLOW CAPACITY OF DOWNSTREAM SLOPE AND RIP RAP STRUCTURE OF KULEKHANI DAM

1 BACKGROUND

The KuleKhani reservoir in Nepal is the only seasonal reservoir for storing water for hydro power production. The dam is constructed as a rockfill dam with impervious central core slightly inclined upstream. The dam is 114m high and the crest length is 397m long and 10m wide. The embankment work was launched from the upstream rock fill in late March 1979 and whole embankment was completed in early June 1981. It has been into operation for Kule Khani HPP without any major complications. The dam owner is Nepal Electricity Authority NEA. The reservoir is estimated to have an expected life time of 100 years. During the years, sediments have filled a lot of the available reservoir capacity and during its 22 years of operation (1982-2004) it has lost more than 21 million m^3 of its capacity. It is not mentioned that the sediments influence the dam safety.

The dam is designed and constructed with a separate spillway section and no overtopping or large throughflow is accepted for the dam. Even though the dam is constructed with a randomized rip rap structure fully covering the rockfill. So far no accurate design criteria for the rip rap is explored but it is from experience proved that such rip rap structures has a significant influence of the dam safety. Former research has indicated the specific influence of the drainage capacity but results differ much depending on a wide set of parameters. The parameters could be found from the surface of the rip rap and such monitoring exists for the KuleKhani dam.

A research program for design of rip rap structures financed by Energy Norway has been run for the last two years at NTNU. Some of the parameters representing the major influence on dam safety are discovered through laboratory experiments and a detailed research program for full scale field experiments is coming up soon. With information from already carried out experiments it is now possible to find the capacity for an existing rip rap structure.

2 MAIN ISSUES IN THE THESIS

Based on available literature, laboratory facilities and current project data I would be doing the following:

1. A literature study of rip rap and design on downstream slopes of rock fill dams.
2. A review of available data from the KuleKhani dam as project reports and field work in 2012.
3. Estimates of drainage capacity and strength of the rip rap from established calculation methods.
4. Design of a laboratory test program for the current rip rap at Kule Khani.
5. Laboratory experiments described in 4.
6. Evaluation of the results by numerical models for throughflow in rockfill dams or at least a study in available numerical models or any means possible.
7. Conclusions and proposals for further research.

The full content of the master thesis will be further decided in cooperation with the supervisor and the partners in the project

3 SUPERVISON, DATA AND INFORMATION

Main supervisor at NTNU will be Associate Professor Morten Skoglund and co-supervisor will be Professor Leif Lia.

The candidate is encouraged to search information through colleges and employees at NTNU and NEA (Nepal Electricity Authority), SINTEF, NVE and other companies or organizations related to this topic. Contributions from other partners must always be referred in a legal way.

4 THESIS REPORT, FORMAT, REFERENCES AND DECLARATION

The final report should be written as a scientific report. All figures, tables, pictures should be of high quality. All used reference should be listed at the end of thesis and continuously in the report. Methods of analysis and performance of test should be described in scientific manner. The final report must be submitted in pdf-format. Standard cover for master thesis is available online. If more than three copies are required the department will take the extra cost. Together with the final report a CD with online version of the report and all data files (pictures, excel work sheet and so on) should be delivered. A summary of maximum two pages should be placed first in the report. The summary should include a presentation of the assignment, work method, discussion of key results and a conclusion.
The master thesis must be submitted in DAIM not later than Monday June 10. 2013.

Trondheim, January 14. 2013

Morten Skoglund

Morten Skoglund

PREFACE

The thesis entitled ‘THROUGHFLOW CAPACITY OF DOWNSTREAM SLOPE AND RIP RAP STRUCTURE OF KULEKHANI DAM’ is carried out for the study of the rip rap as the downstream protection measure in the rockfill dam. This is a part of the research group. The research group consists of the various Master’s students, PHD, Post Doc. and the professors of the NTNU.

The report presents the results and the literature study of the riprap, its performance on various angle of inclination with the dam slope. It also presents the results of the laboratory tests carried out in the NTNU. As an another major part of the thesis, it also presents the performance of the existing rockfill dam of Nepal named Kulekhani dam during the overtopping condition and its throughflow capacity of the downstream slope. It also incorporates the valuable comments and the suggestions obtained from the research team.

The report is prepared as the part of the Master’s thesis. The research team contributes in getting into the good track with the thesis of Masters Student. It can also be useful for the study of the existing rockfill dam, Kulekhani. It will be the good reference for future study to prevent the overtopping conditions in Nepal, which can be disastrous if allowed to overtop. The whole research team involved in this research, the Nepal Electricity Authority and the NTNU laboratory staffs have helped in all phases of the thesis.

Pujan Bajracharya
June 2013
Trondheim, Norway

ACKNOWLEDGEMENTS

It is my immense pleasure to express my deepest appreciation to the Department of the Hydraulics and Environmental Engineering and my course coordinator, Mrs. Hilbjørg Sandvik without whom this thesis couldn't have been possible.

I would like to owe my deepest gratitude and thanks to my thesis supervisor, Førsteamanuensis II Morten Skogland for giving the valuable guidance, feedback and constructive suggestions. I also owe my deep gratitude to my co-supervisor Prof. Leif Lia who gave me chance to work in the thesis. In addition, I would also like to thank my assistant supervisor Priska Hiller for giving guidance and encouraging suggestions to work more in the thesis. I would also like to thank Mr. Hibber Campos for giving his ideas and Antonia Larese as helping hand to give information regarding the software and giving so much help in my work.

I would also like to owe my heartily gratitude to Prof. Ånund Killingtveit, Professor in-charge and all the working crew of NTNU laboratory for giving helping hands to my lab works. In addition, also a big thank you to whole members of Nepal Electricity Authority team to give me more information about the project and accessing the information regarding the project dam.

I would not like to miss my thanks to my colleagues, research team members and my group member Ole Kristian Langekar for collaboration and cooperation in suitable time in laboratory. Last but not the least; I am indebted to my families and my friends who supported me all the time with the positive attitude.

ABSTRACT

Overtopping of the embankment dams has been the major cause of failure of the dams that has resulted much damages of the lives and property in the recent years. There are several accidental incidents of failure. As far as the extreme flood situations in Nepal, people have to fight with many disastrous and damages caused by flood. During such extreme scenario, the passage of such floods mainly relies upon the gate operation rule and the spillway operation rule. Many disastrous floods are unpredictable and no such planned procedures are applied due to the poor availability of the data and the forecasting systems. Due to these reasons, many damages occur regarding the loss of lives and damages of properties, infrastructures and sufferings till now.

According to the Norwegian rules and codes, the dams must be designed for the 1000 years of flood. Besides that, the dam should also be able to resist the probable maximum flood without any failure. Considering these criteria, the thesis is most focused with the throughflow capacity of the existing dam in Nepal. The existing dam project named Kulekhani is only the high rockfill dam of Nepal. The dam is checked against the overtopping in the probable maximum flood condition and found to be safe under these criteria. But when the gates of spillway are closed, the overtopping is occurred. The overtopping condition is tested by carrying out various lab tests in the flume of NTNU and the drainage capacity of Kulekhani dam is thus determined.

From the test results, the existing condition of Kulekhani with the random riprap placement is not able to withstand the vulnerable condition of overtopping. Thus, the dam needs to be reinforced with the riprap with the proper orientation. The design and the sizing of the rip rap are done with the available methods used. The methods are available from various literature and the practices existing so far. From the designed sizing, the mean diameter of the rock size is obtained.

For the orientation of the designed rip rap, the various orientations are checked to get the good performance of the riprap. The orientation mainly means the angle of the longest axis of the rip rap with the downstream slope of the dam. The test results show that the discharge that the dam can take before failure, i.e. failure unit discharge varies remarkably with the change in the orientation. The various lab test results are compared and the best orientation is chosen and proposed for the Kulekhani dam. Then dam is test against the overtopping with this proposed orientation and designed size by conducting the various similar tests in the lab. In addition, the strength of the rip rap is also checked. Eventually from the test results, it is observed that with this orientation and placement, it will able to resist the discharge about twelve times than the existing failure unit discharge for the dam. Thus the current placement of the rip rap in the

Kulekhani should be replaced with the proper orientation in order to be safe against the condition of overtopping under any circumstances.

Similarly the various parameters influencing the performance of the rip rap is studied.The results show that the drainage capacity of the dam is greatly affected by the orientation and the use of the bigger stones. The capacity is improved when the angle of the riprap is greater with the dam slope. The coefficient of the uniformity and the angle of repose of the rip rap are the also interesting parameters observed that affects the test results. The packing factor which is mentioned in the Oliver as one of the important factor to affect the threshold is also studied. It shows that the packing factor doesn’t have direct influence in test results. On the other hand, the numerical simulation is also performed by using the open source C++ programming language course called Kratos. Two models are used for the simulation one with the whole geometry and one with the rockfill only. It shows that the flow cannot be exactly pictured in the modeling of the whole dam. However the model with the rockfill only shows some realistic condition of the rockfill in the dam. It is necessary to do detail studies on both the numerical models and the physical models to give the true pictures of the throughflow capacity of the dam

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Chapter 1

1 Introduction

The dams are not generally designed for the overtopping. However, they may take some overtopping loads but as a whole they might not manage to take upto the threshold load. [1].Overtopping occurs as a result of the rise in the water level, which can occur due to the following possible main reasons:

- Seiches in the reservoir, the long periods of the oscillation usually generated by the wind
- Slides or rock falls into the reservoirs
- The continuous rainfall or melting of snow and consequent runoff into the reservoir
- Upstream water release for e.g. when the upstream embankment fails, the water is released into the reservoir.
- Mechanical failures

In addition, the climate change is also responsible for unpredictable high floods than previous years. Most of the rockfill dams and the earth fill dams are vulnerable to the overtopping. Therefore, it is necessary to consider the hydrological adequacy during the design of the dams and the construction of the dams for the safety purposes. It has been the social demand. Thus, it is important to take this into consideration and avoid the embankment failures and rock fill dam failures by the protection techniques combined with different traditional solutions.[2]. A rip rap as the protection measure is a promising measure for overall safety dam. [3]

1.1 Purpose and scope

This report describes a study, which is undertaken for the following purposes:

- Determine the cases of the overtopping
- How the dam overtops and to analyze which condition may be vulnerable to overtopping
- Based on the data calculation of the overtopping, the overtopping performance of the existing rock fill dam
- Recommend the protection measures that will prevent the failure of the dam
- Propose the proper orientation of rip rap and its performance in the existing dam by the help of laboratory tests.

The report mainly deals with the riprap study and the throughflow capacity of the Kulekhani dam. The Kulekhani dam, which is considered as the only high rockfill dam, is the part of the main storage project Kulekhani that prevents the load, cut off in the dry period to the extent. The Kulekhani dam is the main asset for the hydropower production. The main scope of study is to make the better option for the protection of the dam against any vulnerable conditions. The report therefore presents the laboratory testing of the dam and the general study of the riprap performances under different parameters. All the data are taken from the design reports and the measurements taken during the field visit.

1.2 Objectives

The main objectives of the thesis are summarized as follows:

1. A literature study of riprap and design on downstream slopes of rock fill dams.
2. A review of available data from the Kulekhani dam as project reports and fieldwork in 2012
3. Estimates of drainage capacity and strength of the riprap from established calculation methods
4. Design of a laboratory test program for the current riprap at Kulekhani
5. Laboratory experiments described in 4
6. Evaluation of the results by numerical models for throughflow in rockfill dams or at least a study in available numerical models or any means possible
7. Conclusions and proposals for further research

1.3 Structure of the thesis

The report of the thesis is structured with the following chapters.

Chapter1: Chapter 1 deals with the introduction, purpose and scope of the study. It gives the general objectives of the study.

Chapter 2: Chapter 2 deals with the background of the study area, general review of the data available from the Kulekhani dam and field work

Chapter 3: Chapter 3 deals with the rockfill dam, causes of the general failure of such dams and the forces involved when subjected to the flow

Chapter 4: Chapter 4 deals with the study of the rockfill hydraulics, its characteristics, throughflow and the overtopping flow. It also includes the calculation of the drainage capacity of the existing dam from the established calculation method. .

Chapter 5: Chapter 5 deals with the literature study of the riprap, its design from various design methods and its strength.

Chapter 6: Chapter 6 deals with the laboratory test program of current rip rap structure at Kulekhani and different test programs adopted.

Chapter 7: Chapter 7 deals with the results and discussions useful for further conclusion

Chapter 8: Chapter 8 deals with the study of the numerical methods, problems and some of the results given by the numerical model.

Chapter 9: Chapter 9 deals with the conclusion from the experiment and the recommendation for the future and the research possibilities.

Chapter 2

2 Kulekhani hydropower project

The Kulekhani Hydroelectric power plant is the major supplier of electricity to the Kathmandu. It is located 25 to 40 km southwest of Kathmandu around the Mahabharat ranges. It is a storage plant consisting of the rockfill dam called Kulekhani dam.

2.1 Review of Kulekhani dam from project report

The Kulekhani dam is a rock fill dam with the impervious core slightly inclined upstream. It has maximum height of 114m. Its crest length is 397m and the width is 10m. The upstream slope is 1:2.35 steepened upto 1:2. The slope is steepened near the top of the dam to allow the sufficient camber for the post construction settlement. The downstream slope is 1:1.8. The crest level is at 1534m with the free board of 4m during the highest regulation level.

The axis of the dam is skewed about 45 degree to the river channel**.** The rock formation at the Kulekhani dam site consists of the interbedded schist, phyllite and quartzite, striking generally in a direction sub-parallel to the dam axis dipping upstream at an angle of 30 to 40 degrees. The left abutment is quite steep with the slope of 1:2/3 and the right abutment with the slope of 1:2.

The main dam body is a zonal rock fill dam containing the following main zones, which can be shown in the Figure 2.2 Plan view of Kulekhani dam and Figure 2.2.[4]

- main inclined core
- random zone
- rock quarry zone
- the filter zone and
- Upstream rip rap

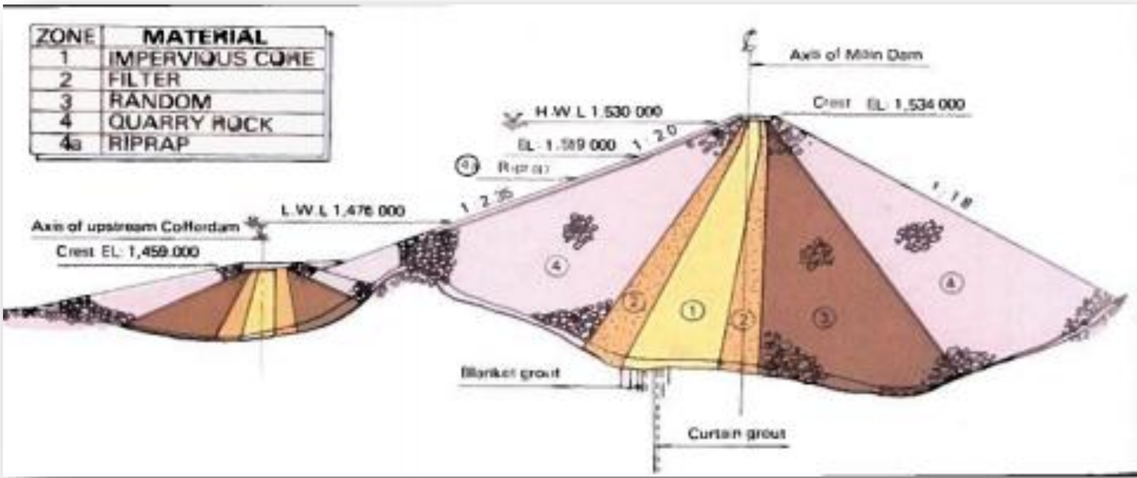


Figure 2.1 Cross sectional view of Kulekhani dam

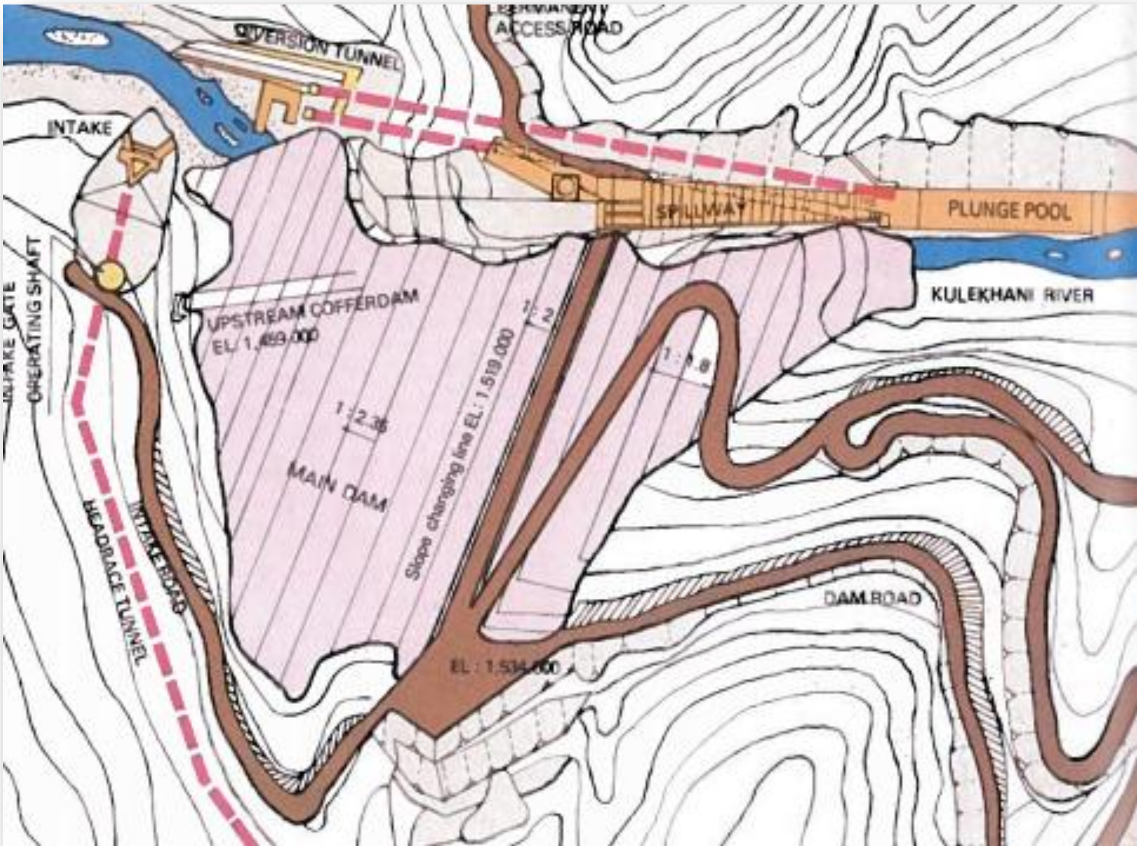


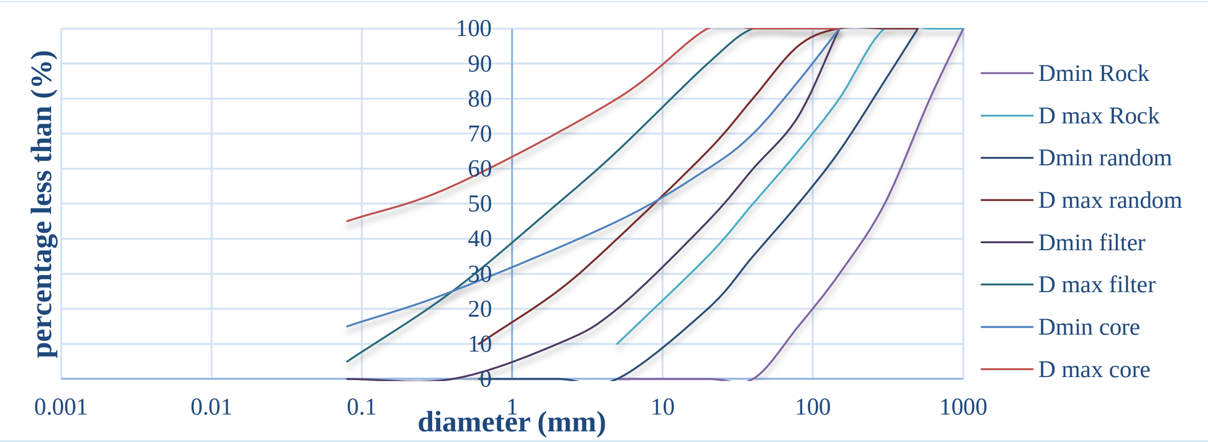
Figure 2.2 Plan view of Kulekhani dam

2.1.1 Main core

The main material used in the core impervious zone is the silty soil of low plasticity but with high permeability. The core material is at 40m above the riverbed. It has been placed at the moisture content of the ½ % below the optimum. The materials in the core in contact with the abutment is placed at higher moisture content so that the soil will be in the plastic condition and fill the irregularities in the core foundation.

The optimum moisture content of the core is slightly less than the optimum specified within the range of optimum moisture content by plus minus 2%. During the construction, the core material was filled in the layer with the uncompacted thickness of 20cm and was compacted by 15 tone-tamping rollers. 93% of the maximum dry density is almost obtained in all the cases with the average dry density of1.75t/m3. The positive pore pressure would not endanger the stability as it ensures there is no leakage developed through the core during the filling of the reservoir. The volume of the reservoir is not so large as it will be possible to fill within the short time during the heavy rainfall. Therefore, the core should be of good quality filters in both upstream and the downstream of the dam. The downstream filter is necessary for preventing the core materials to be washed away even if the local cracking occurs. The relatively dry moisture content averaging the optimum minus 0.5 % for the core material is considered satisfactory for lower 40m of the dam.

More plastic soil is used for the core adjacent to the abutment having a plasticity index of 16 to 20 and optimum moisture content is plus 2 % or more to ensure better contact of soil with abutment. The material used in the core of the dam is essentially silty material with the low plasticity and although having high permeability is nevertheless a satisfactory material. The gradation and the properties of the main core are shown in Graph 2.1 and in appendix.



Graph 2.1 Particle gradation curves of different zonings of dam

Filter zone

The filter zones material are obtained from the sand and gravel deposits in Kulekhani River. The filter material placed in the embankment is within the specified gradation limits. The maximum size of the particle is 150mm. During the construction, the filter zones are placed in approximately horizontal layer with thickness not exceeding 400mm before compaction. Each layer is compacted with the smooth drum vibrating roller having static weight of 15 tones with 6 passes after wetting. The well graded material in the filter zone would have highest modulus of all the zones in the dam and considerable differential settlement normally occurs between the core and the filter zones. Therefore, the heavy compaction is not necessary so the compaction is reduced to 4 passes of smooth drum vibrating roller. The material being placed in the filter zone is of adequate quality and uniform throughout the zone. The gradation and the properties of the filter zone are shown in Graph 2.1 and Table 10.1in appendix.

Random zone

The random zone is from the excavated rock, excavation from the spillway and dam foundation. The random zone consists of schistose, sandstone and mica schist.

The random zone during construction is placed in approximately horizontal layer of 600m. They were compacted with 4 passes of a smooth drum-vibrating roller of weight 15 tones. In the random zone, the trial pit was dug to see if it was free draining. Gradation tests done previously show that the material is slightly on the fine side and outside the specified limit. Due to the fineness in content, the water will not drain freely as it should be in the random zone. It is suggested 2-3 m thick layer of the free draining and used every 10m. . The gradation and the properties of the filter zone are shown in Graph 2.1 and Table 10.1in appendix.

Rock fill zone

The rocks used in the rock fill zone are specified to be from granite quarry which is highly denser than other materials. They are of well graded with the maximum size of 1000mm and ranging down to between 0 and 10% smaller than 5mm.They are of adequate strength and good quality. They are placed in the approximate horizontal layer not exceeding 1.5m in thickness with 4 passes of smooth drum vibrating roller weighing at least 15 tones. This zone both in the upstream and downstream are of good quality and adequate strength. . Wetting of rock fill of granite is not necessary. It is necessary if unconfined strength of saturated rock is less than unconfined strength of dry rock. They are additionally installed with the piezometer to check the buildup of the pore pressure.

During field visit, this zone was well inspected and measurements were taken for the rocks which will be discussed in the section 2.2. The gradation and the properties of the filter zone are shown in Graph 2.1 and Table 10.1 in appendix.

2.1.2 Embankment work and method

Gradation of the zone materials were designed taking into consideration of the actual materials gradation in each borrows area, design conditions, spreading thickness, etc. The moisture content was also specified overall ranging from optimum minus 3% to optimum plus 3%.

In the impervious core embankment, special material of the high plasticity (PI$\geq$12%) was used in the portions in contact with the rock or concrete of foundations and abutment. This is to follow up the deformation due to the settlement and to ensure the imperviousity of contact portions, since the main core materials were of low plasticity with the PI of 0 to 8%.

In the filter materials in contact with the abutments, the selected materials free from the boulders were used in order to avoid the large void created by the segregated boulders. The riprap was placed by labor and backhoe.

2.1.3 Spillway

The spillway is located adjacent to the dam embankment on the left abutment side. It consists of the gate controlled and uncontrolled section. The gate-controlled section, in immediate adjacent to the dam embankment is controlled by two gates, each 9m wide and 11.5 m high as shown in Figure 2.3. The uncontrolled section consists of a side channel spillway having the length of 65m and extending to upstream. The crest level of the uncontrolled section is at 1530m, which is the full supply level of the reservoir. The uncontrolled section of spillway works after the water level exceeds 1530m .The water from both spillway discharges down the slope of 1:2 and tapers in width from 33m at the top to 21m at the bottom.

Figure 2.3 Spillway controlled by two gates

The uncontrolled side channel section has the capacity of the 800m^3/s with the reservoir level at 1534m which is the crest level of the dam embankment. It is capable of discharging the 100 years flood without the control gates into operation. This existing spillway passes this substantial flood through it even when the spillway gates are not opened which is the excellent safety measure.

The probable maximum flood determined from the study had a peak of 2720 m^3/s.

2.2 Field visit

The field visit was done to the proposed site in the summer 2012. Information regarding the overtopping, the dam site, project and construction of existing dam was collected. The group from NEA (Nepal Electricity Authority) was taken as the source of the information. The published reports, the report collection and the interviews were carried out to know the general site conditions. Most of the data were difficult to collect and some lack qualitative details. Some measurements were taken in the site to collect general existing condition of the dam.

The whole field visit plan is divided into two plans:

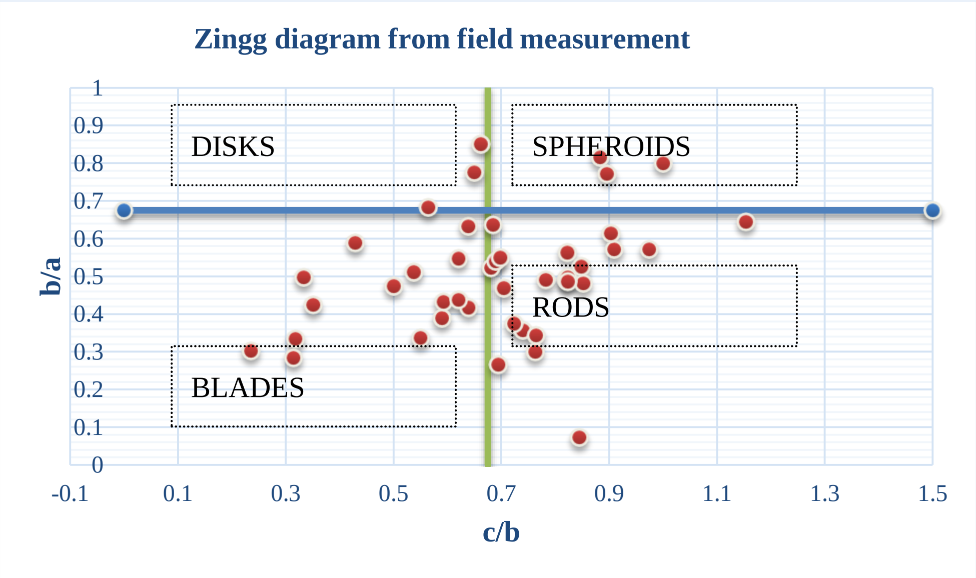
- Visit to the NEA (Nepal Electricity Authority)
- Field visit to the main dam

Visit to the NEA

According to the first plan, the short visit to the office was made on June 25th 2012. The visit was done to know the people and to gain some information about the working systems and the arrangement for the site visit. After submitting the official letter from NTNU, the library was accessed for the report and the various literatures. But the existing data necessary was not found. Therefore for the few days studying on the report, the planning of the site visit was made.

Site visit

The site visit was done on 1st of July 2012 for about a week. The place was studied carefully and the measurements necessary were taken. The existing hydropower project was studied more in detail. The previous design reports and drawings were searched and studied. But it was not possible to get much relevant data. Therefore, the field measurements were done both on the upstream and the downstream rockfill section. The data were obtained regarding the size of the stone, the longest axis of the stones (a), the shortest axis (c) and the axis perpendicular to both axis(b) were taken The inclination of the long axis of the stones with the dam slope alignment was also observed. Thus after the completion of the measurements the rest of the days were spent for the literature review and the detail study of Kulekhani dam and the project. The data obtained in the site is shown in the appendix Table 7.4 and the Zingg diagram is shown in Graph 2.2.



Graph 2.2 Zingg diagram from field measurement

2.3 Conclusion

Studying the available data and the project site, the spillway capacity is quite enough to pass the probable maximum flood. The probable maximum flood at Kulekhani has been determined from the recorded storms throughout Nepal and transporting the storms into the Kulekhani catchment. The storms of 1958 and 1974 are used for the PMF and very little information regarding the flood hydrographs at the dam site is available. Therefore, there is uncertainty related to the data used in the study. From the studies made so far, dam break of Kulekhani is unlikely. The other details would be studied in the hydraulic computation in the later chapter. The other possible condition of failure is by seismic activity, rock soil slide. The dam itself has been constructed along the seismic fault (not active) and requires the special attention.[5] The longitudinal crack in the central core was observed in 1985 which was repaired later. The detail study of other probable condition should be carried out to find the probable condition of defects and failure but this report presents the only particular condition of the overtopping.

Chapter 3

3 Rock fill dams

According to the ICOLD definition, the rock fill dams are the ones, which contain more than 50% of the volume of the rock fill obtained from the rock quarry site or any site containing the natural stones and the boulders. The typical rock fill dam consists of the main central core, which is impervious in nature generally formed from moraine core or the silty clay etc., filter zones of the sandy gravel, transition zones that is mainly from the fine blasted rock and the supporting rock fill that is generally from the blasted rock. Riprap structures are mostly used for the protection of the upstream and the downstream slope. In the upstream slopes, the embankment dams are mainly protected by the riprap for the effects of the wave action, surface runoff, floating debris, and weathering and in Norway; it is mainly due to the icing problems. In the downstream slope, most of the rock fill dams are vulnerable to failure by overtopping and the stability problems in the slope. In that case, the riprap could be the reliable measure for the safety of the dam.

3.1 Failure of the rock fill dams

ICOLD has defined the failures and accidents in two different terms. Accident is defined as the incident that could be observed during the initial filling of the reservoir and that has been prevented from failure by the immediate measures possibly by the immediate drawing of the water. Accidents can be during the construction stage as if the settlement of the foundation, slumping of the side slopes that have been observed initially before the impoundment of the water and later the remedial measures have been done.

The movement or the collapse of the dam is such that it is not able to retain any water causes the failure of the dam. This failure induces the lot of water to release downstream causing the risk in the property and people downstream. Two types of failure are categorized by the ICOLD (1974). [6]

Failure by overtopping

Schintter (1979) summarized that out of the analysis of 216 dams, 34 % dams failed only due to the overtopping and one third of the total dam failures is by the overtopping. The prominent reason of the failure is the inadequate spillway discharge capacity besides the malfunctioning of the equipment and the errors in the operational management. Spillway discharge capacity is one of the important factors that help the dam to pass the floods during the extreme flood conditions or the probable maximum flood condition. The reservoir water level is influenced by the spillway discharge.

The discharge increases as the function of the (3/2) power of the reservoir water head over the spillway crest elevation. For the gated spillway, the discharge will be function of the reservoir head at the centerline of the opening to the power of ½. Generally ungated spillways are not the most likely to fail to pass the adequate discharge even the severe conditions arises such as the earthquake damage, power failure, broken communication, or any conditions during the extreme failure. The failure occurs mostly with the gated spillway. In case of the gated spillway any operational or mechanical failure is expected, then the full capacity of the spillway will not be available and then that might bring the severe conditions of the overtopping. [7] .

In the accidental cases like the closing of the gates during the power failure, the radial gates may be overtopped and might cause the problems regarding the flow instability and the vortices as mentioned in Kjellesvig. [8]

Depth and duration of the overtopping

The probability that the dam failure will occur during the overtopping is more dependent on the depth and the duration of the overtopping. [9] The failure of the dam is more site specific and dependent on the zone and the details of the dams. The provision of the downstream protection and plastic materials in the downstream slope helps to make it more resistive to the erosion and thereby preventing the failure.

Failure by the leakage

Due to the insufficiency in the permeability and the filter criteria in the rockfill dams, the leakages may be occurred progressing into the pipe during the erosion in the embankment dams. The embankment filters becomes not able to stop the erosion of the particles and the hydraulics of the flow during erosional process.

3.2 Forces on the rock fill when subjected to the water flow

The failure of the dam is mainly due to the positive or negative hydrodynamic pressures acting in the rock fill in the downstream slope. The pressure increases with the increase in the velocity of flow. They are more exposed to the single stones of the riprap of the placed stones. The resultant lifting forces on the single stones when exceeds the gravitational forces, G_s and other stabilizing forces together, then it will be unhinged from the protection layer and brings movement. This brings loss of large number of particles and transported to the downstream of the toe, causing the instability of the entire dam. The rock fill particles are dragged with the erosive effects or the part of the downstream region loses its stability due to the landslide.

Regarding the forces in the rockfill, there are following primary forces, relevant to the particle residing within the seepage face[10]:

- Drag force due to the overflow
- Shear due to the overflow
- Seepage forces
- Forces derived from the submerged weight of the particle

Besides these, the other forces are

- Hydrodynamic lift

- The frictional resistance underneath the particle
- The pinching or constraining effect of the neighboring particles

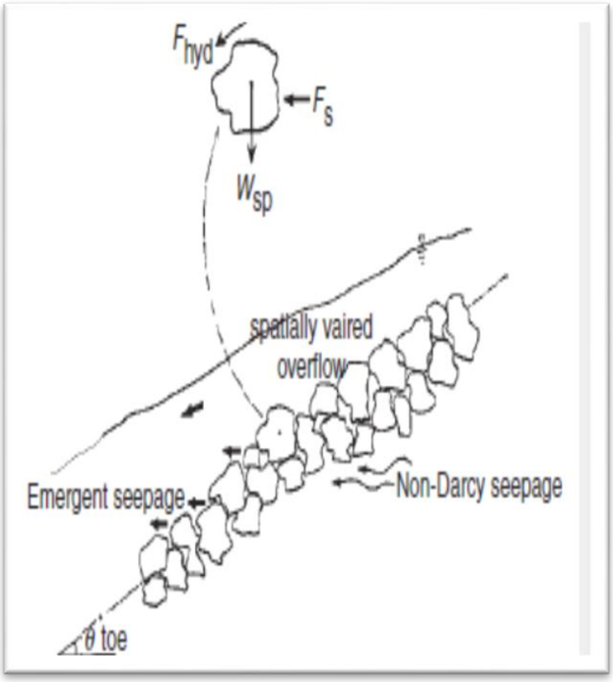


Figure 3.1 Forces in the rockfill downstream[11]

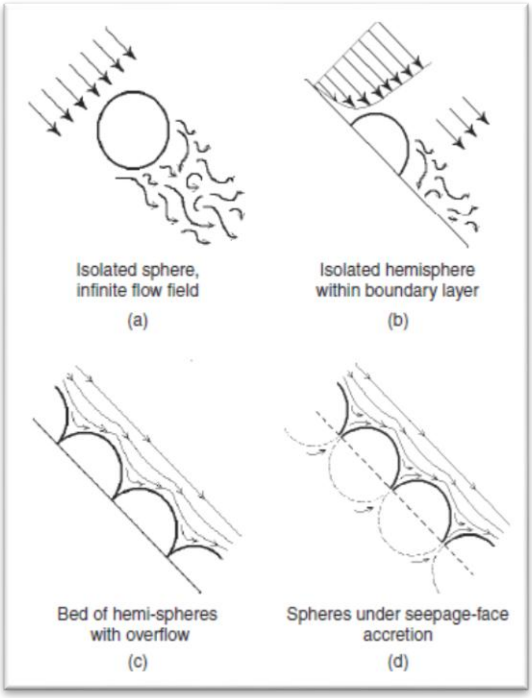


Figure 3.2 Rockfill under various conditions of flow[11]

For the stability of the rock fill under the flow, the equilibrium should be maintained between these forces in the critical conditions i.e. incipient motion conditions.

Drag force due to overflow

When the overflowing water contacts with the exposed frontal surface of the rock particles, the destabilizing hydraulic force F_{hyd} called drag force acts in the direction of the flow. The force is imparted against the face of the rock due to the change in the momentum and the rock comes out from the downstream face and fluid loses its momentum. This is more associated with the classical drag equation given by Figure 3.1

$$F_{hyd} = C_D A \rho \frac{U^2}{2} \qquad \textbf{Eq. 3-1}$$

Where,

C_D = dimensionless drag coefficient that depends on the velocity of the flow and the particle dimensions

ρ = density of the water (1000 kg/m3)

U = velocity of the uniform infinite flow field impinging upon the object that varies from zero at the point of the emergence at the top of the seepage face to the high value at the downstream of the toe.

A = area of the particle projected forward on a plane normal to the flow

Shear due to overflow

When the water overflows over the downstream the quantity of the flow is completely overflowed at the toes of the dam. The over flow at the seepage face through rock fill is spatially varied flow under relative roughness conditions. The seepage face, where the flow is spatially varying occurs at the downstream slope where the bed refers to downstream face of the dam not the foundation of the dam.

The shear force due to this is given as Eq. 3-2 and Eq. 3-3.

$$F_\tau = \tau A = \tau \pi r^2 \qquad \textbf{Eq. 3-2}$$

$$\tau = \gamma_w R S_f \qquad \textbf{Eq. 3-3}$$

Where S_f is determined by spatially varied conditions. For the determination of the Sf, Chow (1959) [12] suggested that the standard form of the equation for the friction slope needs to be modified for the steep slopes. The modified form of the equation is given by

$$\frac{dd}{dx} = \frac{\frac{S_o - S_f}{\cos\theta} - \alpha \frac{2Qq_*}{gA^2 \cos\theta}}{1 - \alpha \frac{Q^2}{DA^2 \,\mathrm{g}\cos\theta}} \qquad \textbf{Eq. 3-4}$$

$\frac{dd}{dx}$ = local depth perpendicular to the bed

S_o = bed slope

S_f = friction slope which is not equal to the absolute value of the rate of change of total head with distance along the bed

α = velocity head correction coefficient

θ = angle of toe of dam as shown in Figure 3.1

q_* = local rate of change of discharge with distance

D = hydraulic depth by Chow which can be taken as the flow depth for wide channel but for narrow channels it is the cross sectional area divided by top width.

Seepage force

Seepage flow is another destabilizing force that emerges from the downstream slope. It has hydraulic gradient associated with it but in case of the rock fill, which does not follow the linear Darcy equation, the critical hydraulic gradient is not clear. However, if the critical hydraulic gradient is unity and factor of safety is 2 then the piping velocity given by Wilkin [13] can be represented as

$$V_p = 16m^{0.5} \qquad \textbf{Eq. 3-5}$$

From the experimental results made by Hansen (1992)[9], it is seen that the head of the water flowing out of the dam is more dependent on the ratio of h_e/H where he is the height of the point of the exit gradient and H is the height of the dam during the overtopping condition. At low discharges with small h_e/H, the phreatic line is almost flat as it approaches the downstream face and for the high value of h_e/H, exit gradient is steeper almost approaching to the tangent of the slope of downstream shown in Figure 3.3. It can be seen that the ratio considerably changes with the slope of the downstream.

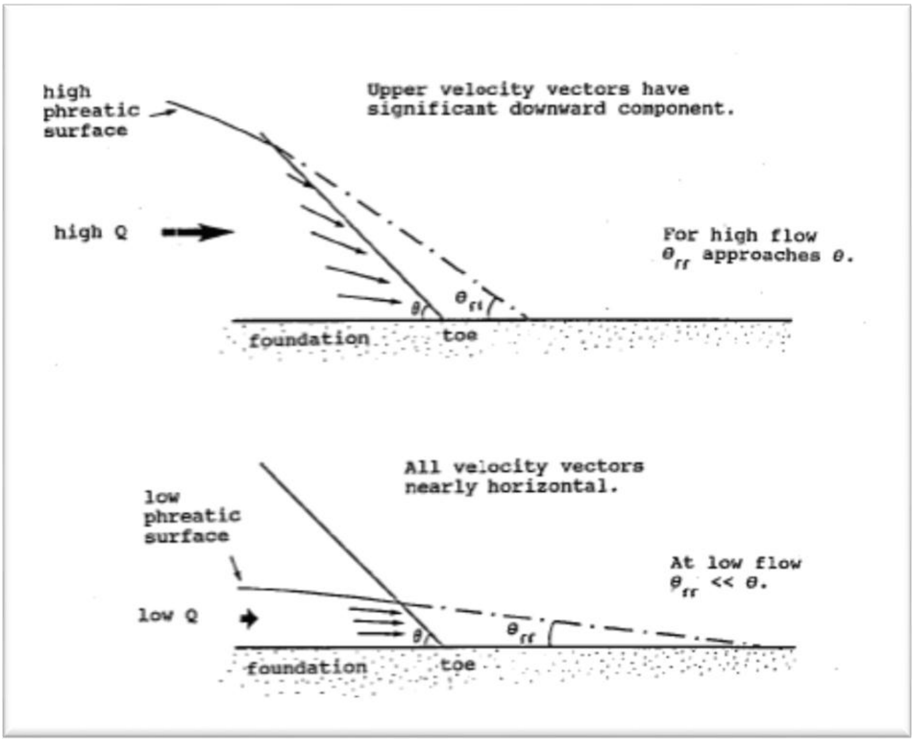


Figure 3.3 Concept of the angle of the emergent flow field[9]

The angle of the emergence of the water from the dam is different from the angle of the toe of the dam, which can be related as by given equation:

$$\frac{\theta_{ff}}{\theta} = 1.41\frac{h_e}{H} + 0.17 \qquad \textbf{Eq. 3-6}$$

Once the hydraulic gradient is known somehow, then the force per unit volume due to the seepage can be estimated as

$$\frac{F}{v} = \gamma_w i \qquad \textbf{Eq. 3-7}$$

In relation with the bulk volume, the seepage force per particle can be calculated as given by equation:

$$F_{seep} = V_p(1+e)\gamma_w i_H \qquad \textbf{Eq. 3-8}$$

Where

V_p = representative volume of the particle

e = void ratio

i_H = hydraulic gradient

The vertical component of the seepage force is neglected because the arrangement of the particles on the downstream face is quite erratic and it is found to be very small. The destabilizing effect of the seepage force is about 36 to 378 times larger than the overflow effect for the particles on the exit face.

The force due to the submerged weight

The submerged weight of the particle provides the stabilizing force, which acts opposite to the direction of the moment created by forces described previously. It is given by the equation

$$W_{sp} = V_p(\gamma_p - \gamma_w) = (\gamma_p - \gamma_w)\pi\frac{d^2}{6} \qquad \textbf{Eq. 3-9}$$

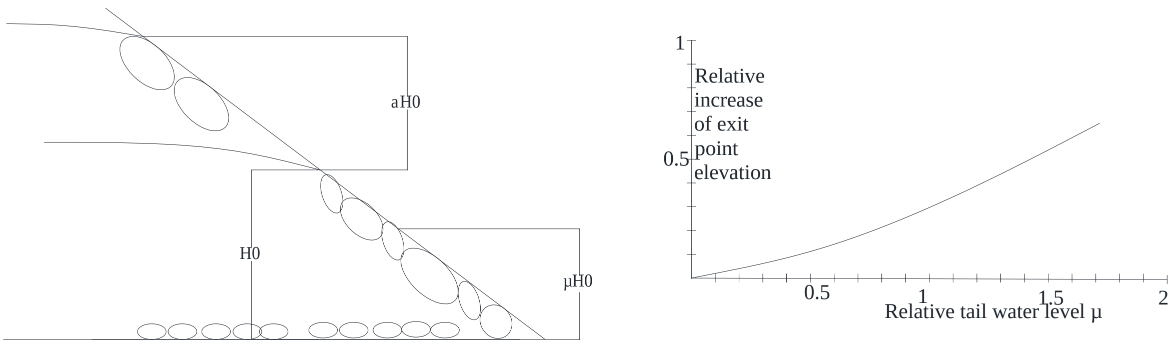
3.3 Stability of the downstream face and the drainage capacity of dam

In the real conditions, the events of abnormal conditions like accidents, natural disasters or mechanical failures in the operations may occur. This leads to the overtopping of the dam that the dam eventually should be able to withstand without failure[14].The crown of the rock fill dam can usually withstand the overtopping head of about 1,2 times the diameter of the protecting stone on the crown but that is not necessarily means that it can take the corresponding discharge. The flow inside the dam is three dimensional so water when flows through the dam, its level concentrates the valley bottom. Therefore, the topography of the dam foundation also leads the out flowing water to concentrate to the dam toe making it the weakest point of the dam. The flow through the dam and exit of the water through the dam is more dependent on the geometry of the dam's profile.

From the regulations, the dam toe, the dam foundation, the downstream face and the top of the dam should be designed in order to withstand the impacts. Special attention should be given to the last row of stones of the toe of the dam. These are the key stones that also bring instability of the next row stones if not strengthened well or if removed. Besides that, the slope of the foundation, angle of repose, the shape of the stones, orientations of the stones also affect the stability of the stones.

It is not possible to predict the movement of the stones at the downstream slope, which is stochastic process. The turbulence of the overtopping flow and the seepage flow makes the flow more complicated.[9]Erosion of single stones of protection layers during the overtopping cannot be compared to erosion processes in riverbeds. The form of the stones is more sharp edge so the retaining grouting forces are much higher. Here we consider only the stability of the individual key stones or the row of the stones than the deep-seated slides in the rock fill. Because of the high drainage capacity the deep-seated slides are unlikely in supporting fill. The deep-seated slide may occur when the supporting fill is very fine grained and low permeable.

The stability of the downstream is also improved by raising tail water level taking into the consideration that the point of exit is lifted. [14] By doing so the downstream face is more exposed to the out flowing water.



H0=position of the exit point for the negligible tail water

Figure 3.4 Influence of the tail water on exit point elevation and position of exit point[14]

The erosion in the steep slope is not similar to the erosion in the typical subcritical river flows. The hydraulic steep river will result in the spatially varied supercritical nature of flow with air entrainment. The bed loads transport calculated by the sever formulas given by Meyer's and Peter formula, Muller and Einstein gives the higher erosion rates by factor of 10 to 100. Even the Shields provide very less velocities than the experimental values of the velocities.

The flow of the water at the downstream slope has the higher velocity and the higher intensity than over the top of the core due to the gorge effect. The flow intensity of the water also increases linearly from the exit point to the toe of the dam. The linear variation can be described from the depth estimated at the toe and the total diagonal length of the seepage face. The depth of the flow at which the water emerges from the dam is given by using the formula [11]

$$h_t = \frac{q}{\sqrt{k_t \sin\theta}} \qquad \textbf{Eq. 3-10}$$

Where

h_t = head of the water flowing out of the dam

α = the slope of the landscape

q = the outflow discharge from the dam

3.4 Failure mechanisms

The safety factor needs to be greater in order to prevent the unraveling failure process, which is given by

$$F.S = \frac{stabilising\ moment}{\sum destabilising\ moment} \qquad \textbf{Eq. 3-11}$$

The expression for F.S for the moment based initiation of motion or the unraveling failure under the seepage face for a single particle is given by

$$F.S = \frac{W_{sp}\cos\theta}{0.5F_{hyd} + F_{seep}\sin(\theta - \zeta)} \qquad \textbf{Eq. 3-12}$$

Most of the rock fill dam is failed due to the erosion of the dam during the transport of the cohesionless materials downstream and the slope failure as following ways.

- internal erosion

- external erosion
- geotechnical failure (mass sliding)

Internal erosion occurs when the rock particles are washed away consequently from the surface of the dam or from one zone to another zone by the flow leading to the failure of the dam. The geotechnical failure or the slope slippage can occur in the upstream or the downstream. The slope slippage is more dependent on the average size of the particles and the value of the slope in the case of the cohesionless soils

The focus should be given to the downstream toe which is more vulnerable to the erosion as the erosional process occurs from the toe of the dam upwards. The mass sliding failure is distributed among the whole width of the dam slope including the toe[15]. Due to the high pore pressure, the water is forced from the core zone and the sinkholes or any paths will be formed and there would be problem regarding the toe vulnerability.

3.4.1 Internal erosion

The internal erosion can occur under the four conditions

- When there exist the seepage flow path and a source of water
- When there are materials within the flow path and carried by the flow of water
- When there is unrestricted the exit from where the eroded material can easily washed away
- For piping to occur, the material, which is being piped, or the material above the pipe should be able to support for the pipe.[16]

In the case of the rock fill dam, the three main processes that initiate piping are

- progressive erosion at the exit point of the seepage which starts from the backward
- concentrated leak that initiates at the small crack or soft zone
- Suffusion, which is the process of the small particles being washed away through the voids of the bigger rocks.

Erosion develops into the pipe that leads to the washout of the section of a dam, which can be explained in the Figure 3.4.

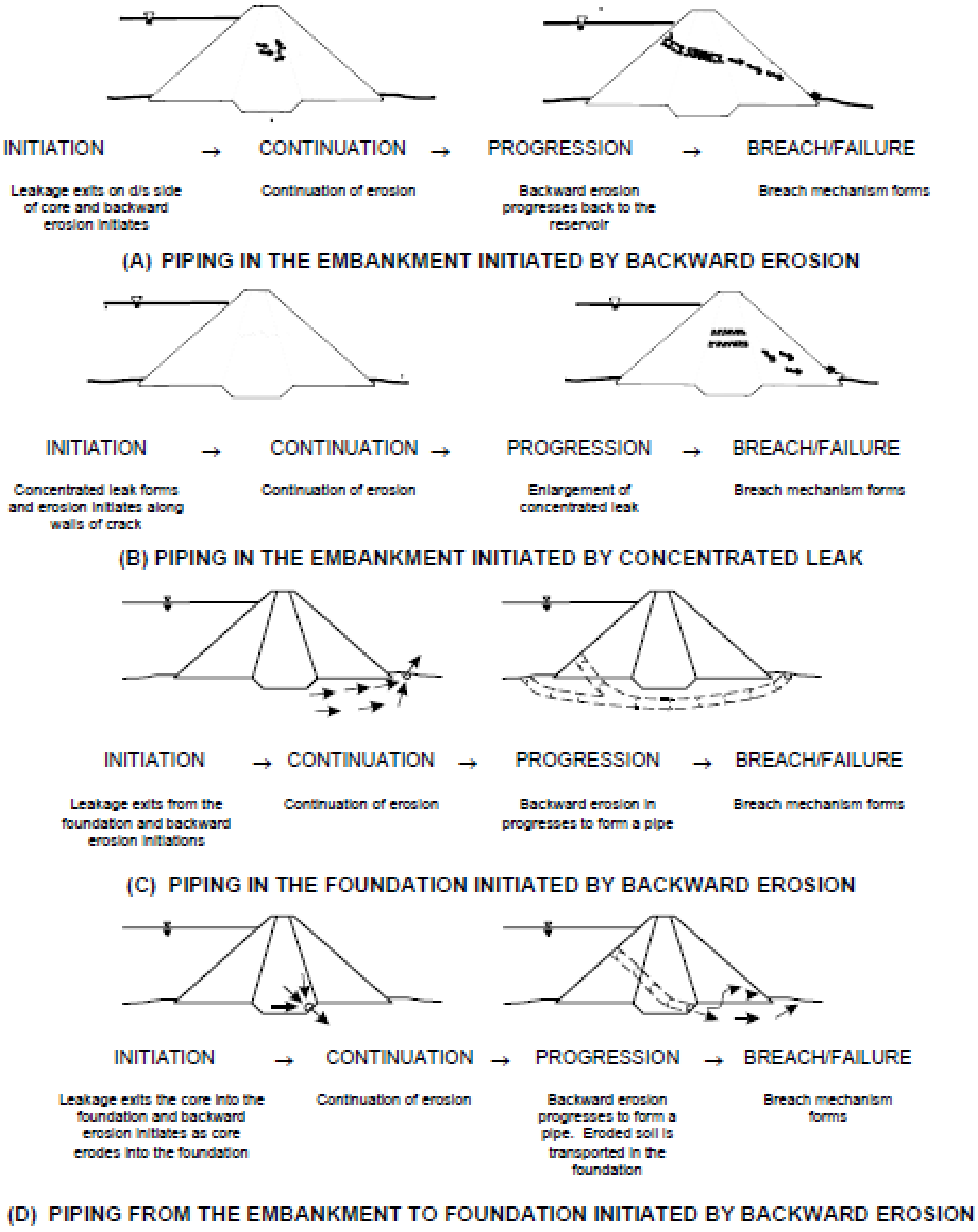


Figure 3.4 Internal erosion process in the rockfill dam [16]

3.4.2 External erosion

External erosion occurs due to the drag forces exerted by the seepage flow leaving the dam body or protection flowing along the downstream slope eventually leading to the failure of the protection. In the case of the external erosion, the four types of different stones need to be checked. These are

- Stones in the inflow zone
- Stones in the outflow zone
- Dam toe stones in the bottom of the valley
- Dam toe stones on the sides of the valley

The stones in these places are more important that are exposed to the external erosion. Regarding these stones, the toe stones rest on the foundation whereas other stones are resting with other stones within the dam. The stones on the outflow zones and the dam toe stones can be designed if the drainage capacity of the dam is known. This is important because it gives the spatial condition for the unsteady flow travelling through the dam. The sizing of the external surface of the stones needs to large enough to resist the drag force as 3.2

3.4.3 Mass sliding

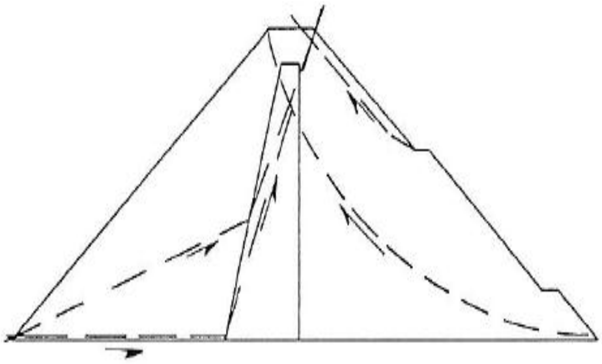


Figure 3.5 Different sliding planes of failure

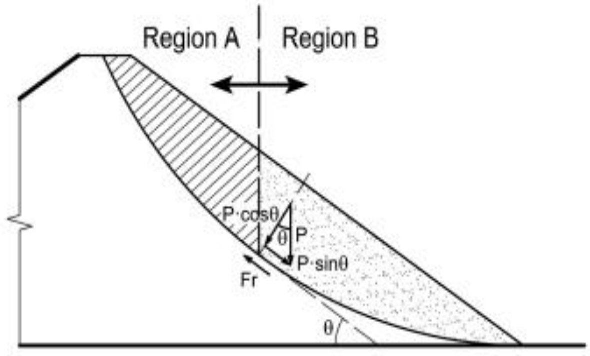


Figure 3.6 Stability areas in a particular slip circle [2]

When the sum of the destabilizing moment exceeds the sum of the stabilizing moments, the failure occurs. [17]When the rock fill is subjected to the overtopped flow, seepage flow contributes in the development of the pore pressure leading to the deeper slides at the downstream slope which is observed by Parkin [18]. The mass sliding process is more dominant for the slopes steeper than 2.

The factor of safety is more influenced by the internal angle of friction. Many methods are being used in the design of the rock fill dams like Bishops simplified method, Spencer's method, Janbu, Morgenstem price method and wedge method.

The mass sliding can be prevented simply by adding the weight over the potentially unstable slope at the lower most area shown in the Figure .It gives the additional friction force at the critical point and improve the stability of the dam body. [2] The downstream slope can also be stable if the water is freely draining i.e. decreasing pore pressure. It can be done by increasing the permeability of the rock fill. The increment of the permeability will depress down the saturation line and affect the small portion of the dam body.

Chapter 4

4 Rock fill hydraulics, throughflow and overflow

The water flowing over the rock fill is quiet complex as it involves large volume of water passing through the wide profile and over the permeable beds, and the flow is quite turbulent resembling to the overtopping open channel flow, seepage flow.[19].

The flow through the rock fill has quite large exit gradient and high exit flow velocity on the downstream slope that can lead to the initiation of the movement of the particle. This is quite complicated to deal with. Another complexity of the rock fill dam is to define the rock fill properties.[20]The hydraulic performance of the rock fill is influenced by many factors like the physical properties of the material.

- Porosity
- Shape factor
- Roughness
- Packing factor
- Void ratio
- Contact surface(curved /angular)

The parameters like the friction angle, permeability and specific weight of the materials are most important parameters of the rock fill.[9] Generally, the rock fill hydraulics mainly includes flow through the high velocity zone during the seepage through the rock fill that generally involves the infringement of the linear Darcy Law.[21].Regarding the general principle of rock fill hydraulics, it deals with the friction head losses, stability under flow and the seepage flow and the characteristics of the rock fill , which will be discussed further.

4.1 Characteristics of the rock fill

The flow through the rock fill can be simplified as the flow above the rock fill as the open channel flow and the seepage flow through the rock fill below where the normal pressure seepage exists [19].In addition to this, three-phase flow also exists in case of the voids. Kjellsvig [8] also divided the different types of the zone in the rockfill during the throughflow and the overtopping conditions. Besides that, the shape roughness is more considered to be dominant than the surface roughness of the material.

Particle Size

The characteristic dimension is required to represent the block as the individual size is not regular Therefore, it can be expressed in three terms:

- ***Nominal diameter***
- ***Mean diameter***
- ***Equivalent diameter***

For the given set of blocks, the most reasonable way of representing the characteristic dimension is the classical sieving, as it is not possible to measure the blocks individually in the rock fill. Leps [10] also suggests that it is also possible to estimate the most dominant diameter for a given set of blocks by visual inspection, which will be more or less to d_m, or d50.

Shape

Stephenson classified the shape of rock being varied in shape and size into spherical, rounded, cubical angular and the elongated. However, Scott quotes that the most frequent shape is the angular and the rounded shapes. In case for the irregular shapes, shape coefficient is necessary to give qualitative description of the shape.

The shape can also be represented in the Zingg diagram which is the plot of the ration c/b and b/a. It is generally used for characterizing the individual particles. [17]

Porosity

Porosity and void ratio both represent the volume fraction of the pore space, which are the main parameters for the structuring of the rock fill. Any rock fill having the same void ratio can have different arrangements and each disposition will have different behavior depending upon the block shape, gradation and the degree of the compaction. The void ratio is more in the uniformly graded material than the well-graded materials, and it is for the angular particles than the rounded ones.

In case of the consolidated media, it is important to distinguish the two types of the pores in the pore space in which one is called the interconnected or effective pore space and the other is called the dead pore space. The dead pore space consists of the isolated or non-interconnected pores. Only the effective or interconnected pore contributes to the flow of the fluids through the porous medium. [22]

Specific gravity

The specific gravity is defined as the ratio of the density of a substance compared to the density of the water at 4℃ i.e. it corresponds to the values of the apparent dry unit weight.

Internal friction angle

The internal friction angle also parameterizes the rock fill and it is more related with the geotechnical areas. The friction angle changes with the change in the compaction energy, confining pressure and the wetting during the construction. Brauns (1968) found that in the case of the well-graded materials, there is less percentage of the crushed rock and the angle of the friction is higher. Leps[10](1970) found that the confining pressures has shown variation of the friction angle up to 10 degrees since the higher confining pressure yields lower friction angles. Besides that the particle breakage strength, the maximum size, and the shape factor also influence the internal angle of friction value.

When the rock fill is not subjected to the pore water pressure, the shallow slide may occur. While the mass sliding pattern occurs, it can be defined by Eq. 4-1.

$$\mathrm{F} = \frac{N}{\tan \emptyset} \qquad \textbf{Eq. 4-1}$$

Where F is the safety factor during the mass sliding, N is the downstream slope and Φ is the internal angle of friction.

Hydraulic mean radius

Hydraulic mean radius is the measure of the representative diameter of the average pore within the porous media.[17] This is important for the study of the seepage flow through the rock fill as the water flows through the voids. The hydraulic mean radius is defined as the volume of the voids within a control volume containing a porous media to the surface areas of the voids having the total volume V_v as defined by Eq. 4-2. The practical determination of m is uncertain for the well-graded or non-homogenous rockfill.

$$m = \frac{V_v}{Sa} \qquad \textbf{Eq. 4-2}$$

4.2 Permeability

The permeability is one of the most important parameter while dealing with the through flow of the rock fill dams. The flow condition varies greatly in the rock fill dams. [23]The grain size and the velocity, which are the two main factors, mainly affect the flow condition. In case of rockfill, the linear Darcy law cannot be applied as the flow being turbulent.

If the flow is laminar, the coefficient of permeability can be expressed as equation

$$\mathrm{k_l} = 1 + \frac{1}{\alpha_0} . \frac{\mathrm{n}^2}{(1-n)^3} \frac{\mathrm{gd_l^2}}{\mathrm{Y}} \qquad \textbf{Eq. 4-3}$$

If the flow is turbulent, the coefficient of the permeability can be expressed as equation given by Engelund[24]

$$\mathrm{k_t} = 1 + \frac{1}{\beta_0} . \frac{\mathrm{n}^3}{(1-n)} \mathrm{gd_t^2} \qquad \textbf{Eq. 4-4}$$

α_0	=	grain shape coefficient for the laminar flow
	=	780 for spheres and
	=	1000 for uniform and rounded
β_0	=	grain shape coefficient for the turbulent flow
	=	1.8 for the spheres and
	=	2.8 for the uniform and rounded
n	=	porosity
d	=	grain size
g	=	acceleration due to the gravity

ϒ = kinematic viscosity

In the following case above if the Reynolds number is less than 600, then the turbulent permeability should be corrected using the permeability[25]

$$k_c = v.k_i + \frac{k_t}{(v.k_l + k_t)}$$ **Eq. 4-5**

Determination of the grain size

While determining the grain size three methods can be used

- Method 1

The grain size is determined based on one point d_n on the sieve curve multiplied by factor x

$$d_t = x.d_n$$ **Eq. 4-6**

where d_n is the sieve opening through which 10 percent weight of the grains will pass.

- Method 2

$$\frac{100}{\mathrm{d}_\mathrm{t}^{\alpha}} = \frac{\Sigma\Delta n}{\mathrm{d}_\mathrm{n}^{\alpha}}$$ **Eq. 4-7**

- Method 3

This method not only involved one fixed point d_{n+1} on the sieve curve but also the slope defined by $\frac{1}{d_{n+1}-d_n}$

$$d_t = \frac{d_{n+1}}{d_{n+1} - d_n}$$ **Eq. 4-8**

4.3 Friction head losses

The friction head losses generally are based on the uniform flows. However the flow is not uniform in the case of the rock fill as the relative roughness is high than 0.05. The cross section of the flow is not constant and the boundary changes with the along each length of the flow due to the rockfill boundary. Figure 4.1 depicts that the effective cross section of the flow is not constant along the section of the flow. It gives the condition of the non-uniform flow in rockfill. In general the flow depth is measured from the impermeable bed. However, this is not relevant to the rock fill as the flow also occurs through the rock fill as the seepage flow.

Friction head loss is calculated from the Darcy Weisbach equation

$$i = f\frac{1}{D}\frac{U^2}{2g}$$ **Eq. 4-9**

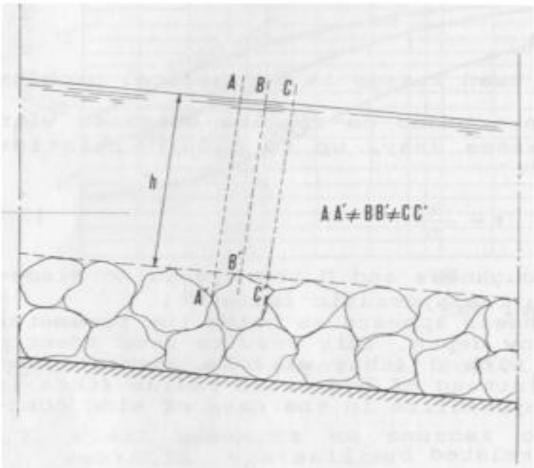


Figure 4.1 Variable cross section of the flow[21]

4.4 Throughflow and overflow in rockfill dams

In the rockfill dam, the condition of the throughflow and overflow both can be possible under the conditions. In case of the rockfill dam with the impermeable core, when the water level exceeds the top of the core and lesser than the crest of the dam, the flow through conditions prevail. When the water level exceeds the crest of the dam and overflow over the embankment slope both the overtopping and the flow through condition occurs. When the flow exceeds the drainage capacity of the dam, it leads to the failure which is possible both by flow through or the overtopping flow or combined flow.

It is very important to use the reliable non-Darcy equation for the seepage discharge through the rockfill dams as the flow through the rockfill varies with the different zones of the rockfill dam. In the case of the throughflow during the overtopping condition, the flow is extremely turbulent. The failure mechanism that occurs in the flow through the rockfill dam is the unraveling of the downstream slope and the massive stability failure of the slope. The drainage capacity of the dam is influenced greatly by the flow through and the overtopped flow.

From the available prototype structures, the available expression for the non-Darcy flow and the failure patterns can be examined in the practical way. Most of the field observations of the embankment dam failure show that the initiation of the failure occurs by the head cutting near the toe of the dam[26]. Later, it advances to the upstream until the crest of the dam is breached. In some cases it also occurs from the knick point present at the downstream edge of the crest.

4.4.1 Different theoretical estimation of the flow

The term throughflow is the flow of the water through the rock voids. The understanding of the flow in the turbulent flow medium is necessary[27] for the throughflow study and analysis of the downstream slope. The throughflow is created due to the seepage flow through the rock fill dams. This flow is mainly governed by the nonlinear equation or non-Darcy flow. The flow through rip rap can be represented by the two different laws represented by exponential law given by Eq. 4-10 and the quadratic law [28] represented by Eq. 4-11. They can be used to make the connection with the bulk velocity too if the velocity of the void is replaced by the bulk velocity.

$$i = cV^m \qquad \textbf{Eq. 4-10}$$

Or the parabolic equation

$$i = aV + bV^2 \quad \textbf{Eq. 4-11}$$

where c, m, a, b are the coefficients depending upon the material, I is the hydraulic gradient and V is the seepage velocity(Volker 1969)[2].

The hydraulic performance of the rockfill is affected by the physical properties of the material as mentioned before. Therefore, the analysis of the hydraulic performance of rockfill requires examining the level of the turbulence for the hydraulic gradient field. Numerous approaches are forwarded for the analysis of the turbulent throughflow like Weiss(1951), Escande (1953), Olivier (1967), and Stephenson (1979).

Wilkins

Wilkin[29] proposed the equation for the uniform velocity and the flow having the phreatic surface not equal to the bed slope given by

$$V_v = W m^{0.5} i^{1/N} \quad \textbf{Eq. 4-12}$$

W = 32.9 for inch-sec units, 5.243 for m-sec units

m = hydraulic mean radius which is the representative pore diameter that can be estimated using Eq. 4-2.

V_v = average velocity in the voids called average linear velocity (Freeze and Cherry 1979)

$$V_v = \frac{V}{n} \quad \textbf{Eq. 4-13}$$

Where V is the bulk velocity and n is the porosity.

The viscosity of the fluid and the wall effect is not taken into account.

Ergun

Ergun in 1952 modified the Darcy law by introducing the porosity. This equation is also used in the Numerical method of analysis which is discussed in the chapter 8.

$$\frac{\mathrm{i.d}}{\mathrm{v}^2/\mathrm{g}} . \frac{\mathrm{n}^3}{(1-n)} = 150 \frac{(1-n)}{Re_p} + 1.75 \quad \textbf{Eq. 4-14}$$

Ergun and the Orning took into account of the wall effect in the nonlinear Darcy regime. He defined Reynolds number to take in account of the viscous forces in the low velocity flow, kinetic effects in the highly turbulent flow and smooth transition in flow in between. In order to corporate the wall effect the Ergun-Reichelt suggested the equation

$$\mathrm{i} = \frac{\mathrm{M}^2\mathrm{v}}{\mathrm{d}^2 g} . \frac{(1-n)^2}{\mathrm{n}^3} V + 1.57 \frac{\mathrm{M}}{\mathrm{dg}} . \frac{(1-n)}{\mathrm{n}^3} \mathrm{v}^2 \quad \textbf{Eq. 4-15}$$

Where

$$\mathrm{M} = 1 + \frac{2}{3} . \frac{\mathrm{d}}{\mathrm{D}(1-n)} \quad \textbf{Eq. 4-16}$$

D = diameter of the permeameter.

Parkin

Parkin (1963) [18] performed the tests on the clean angular gravel and derived the equation of the hydraulic gradient as

$$i = 1.86{V_v}^{1.86} \qquad \textbf{Eq. 4-17}$$

where $V_v$is the average throughflow velocity in feet per second.

Leps

Leps used the combined concepts of Escande, Wilkins, and Parkin and derived the average flow through the rock fill for the turbulent conditions.

$$V_v = Wm^{0.5}i^{0.54} \qquad \textbf{Eq. 4-18}$$

Ward

Ward (1964) suggested the equation for the hydraulic gradient as the function of the intrinsic permeability, density and the absolute viscosity of the fluid.

$$\mathrm{i} = \frac{\mu V}{k} + \frac{0.55\rho V^2}{k^{0.5}} \qquad \textbf{Eq. 4-19}$$

k = intrinsic permeability associated with the non-Darcy flow

ρ = density of the fluid

μ = absolute viscosity of the fluid =

Martin

Martins in 1990 introduced the velocity in the voids as the function of the uniformity coefficient and the particle gradation given by

$$V_v = \frac{K_m}{\mathrm{C}_u^{\propto}} . \sqrt{2\mathrm{gedi}} \qquad \textbf{Eq. 4-20}$$

Where

C_u = Coefficient of the uniformity

K_m = Martins empirical coefficient

$\propto$ = expirical exponent for C_u

$d_{60,}d_{10}$= rock diameter such that 60 % (10%) is finer

K_m = Martins empirical coefficient

For the fully developed turbulence

$$V_v = \frac{K_m}{\mathrm{C}_u^{\propto}} . \sqrt{2\mathrm{gedi}} \qquad \textbf{Eq. 4-21}$$

The equation suggested by Martin is used for the fully developed turbulence.

McCorquodale et al (1978) used the Reynolds number and the flow was found to be fully turbulent when the Rep exceeds 500.

Scheidegger (1974) states highest Reynolds number, which must be exceeded before fully developed turbulence, is 750.

$$Re_p = \frac{Vm}{\gamma n} \qquad \textbf{Eq. 4-22}$$

Where

V = bulk velocity
m = effective hydraulic radius without considering the wall effect.
n = porosity

Hansen

Hansen focused the three main criteria for the flow through the rockfill structures, which are namely

- volumetric rate of flow through the structure,
- height of the seepage face
- the most unstable particle within the seepage face

Non-linearity and turbulence regime in the rockfill

The flow condition in the rockfill at the downstream face is turbulent which is characterized by eddies and the instantaneous fluctuations from the mean velocity. This is introduced in the typical Navier Stokes equation as additional terms. The various term that is used in the Navier stokes equation needs to be averaged to obtain the equation of the turbulent form. The equation given by Hinze in 1959[30] as follows:

$$\left(\frac{\partial \overline{u_i}}{\partial t} + \overline{u_j}\frac{\partial \overline{u_i}}{\partial x_j}\right) + \rho\overline{u_j{}'\frac{\partial u_i{}'}{\partial x_j}} = -\frac{\partial \overline{p}}{\partial x_i} + \overline{B_i} + \mu\overline{\frac{\partial^2 u_i}{\partial x_i \partial x_j}} \qquad \textbf{Eq. 4-23}$$

where

u_i = instantaneous value of the component of the velocity in the i direction
$\overline{u_i}$ = mean value of the velocity in the i direction
$u_i{}'$ = instantaneous variation of the actual velocity from the mean velocity

$$u_i = \overline{u_i} + u_i{}' \qquad \textbf{Eq. 4-24}$$

$\rho\overline{u_j{}'\frac{\partial u_i{}'}{\partial x_j}}$ = extra term due to the turbulence

[31]The extra term $\rho\overline{u_j{}'\frac{\partial u_i{}'}{\partial x_j}}$ is the Reynold stress indicates the loss of head due to the turbulence.

Oliver

Oliver [32] defined two types of the flow for which the movement of the individual stones begin. The first type of flow, threshold flow gives idea of unraveling failure, which is the initiation of motion causing the particles to move further downstream, destabilizing into the motion. Before the movement, firstly some of the stones vibrate until the flat characteristics of the stones with their flat surfaces horizontal tends to be turned through 90 degrees and to stand on the edge. Then the less supported stones get carried to the downstream to the second more stable condition which cause the increase in the permissible flow condition. The first failure mode here is the unraveling failure or the progressive erosion of the downstream toe than the pseudo rotational failure. This is due to the large amount of the seepage water and the flow on the seepage face causing the rocks to dislodge and move down the face.

The second type of flow, collapse flow where the movement of one stone causes the mass movement of other stones resulting uneven position of the bed, is the distinctive stage at the rock fill. The motion can be disastrous if there is nothing to prevent the last row of particles from moving downstream. This makes the protection of the toe to be the primary concern. Then after this stage, the flow if is increased the final stage will occur enduring the collapse of the rock fill.

Generally the collapse flow is 20 percent higher than the threshold flow in the case of the gravel and 80 percent greater in the case of the large granite. [32]. Based on the tractive theory, Oliver proposed equation for the threshold flow that takes in account of the apparent weight of stones in water, angle of inclination, repose of the stones, equivalent diameter and the packing factor which relates the number of stones of given size within given area or given volume.

The equation suggested by Oliver for the threshold discharge of the rock fill slope during the overflow is given by

For crushed granite,

$$\mathrm{q}_{oT} = 0.423(\mathrm{G}_s - 1)^{5/3} {d_s}^{3/2} \tan\theta^{-7/6} \qquad \textbf{Eq. 4-25}$$

For gravel and pebble,

$$\mathrm{q}_{oT} = 0.344(\mathrm{G}_s - 1)^{5/3} {d_s}^{3/2} \tan\theta^{-7/6} \qquad \textbf{Eq. 4-26}$$

Where

q_{oT}	=	observed threshold flow (cfs per ft of width)
d_s	=	equivalent diameter of the rock
G_s	=	Specific gravity of stone particles.
$\tan\theta$	=	downstream face slope

The equations proposed by the Oliver are for the study on rock slopes, which are flatter than typical rock fill downstream slope. This gives the rock volume of the rock very large which reduces the economic advantage of rock fill dams.

Prajapati

Prajapati [33] defined the threshold flow that initiates the failure of downstream slope of rockfill under the submerged condition to be the function of the downstream tail water depth and the diameter of the stone given by following.

$$\mathrm{q}_{oT} = 0.68.K^t \mathrm{g}^{1/2} {d_s}^{7/6} {\mathrm{H}_d}^{1/3} \qquad \textbf{Eq. 4-27}$$

$$\frac{\mathrm{H}}{\Delta D} = 2.78 + 0.71\frac{\mathrm{h}_b}{\Delta D} \qquad \textbf{Eq. 4-28}$$

Where

K^t = dimensionless coefficient depending on the properties of rockfill

H_d = downstream head

Toledo

Toledo studied the downstream and focused more on the toe of the dam as it is much vulnerable to the erosion[15]. He mentioned two ways of the failures mechanisms one is by the mass slip mechanism and other one is the scouring. In some cases the combined mechanism also occurs.

4.5 Calculations of the estimation of the drainage capacity from existing method

The main objective of this study is to analyze if the downstream slope rockfill of the current Kulekhani rockfill dam is safe against the overtopping failure. The drainage capacity of the dam should be known in order to design and calculate the strength of the rip rap of downstream. The efficiency of the rock fill and its effectiveness of the random placement of the rockfill are to be studied as the part of the thesis.

The main parameters that are needed to be considered are

- Hydraulic parameters
    - the overtopping unit flow q_s
    - hydraulic gradient
    - upstream level
    - hydrological data
    - discharge coefficient
- particle size distribution
- downstream slope
- height of the rock fill

The goal is to develop the design as the rockfill protection to withstand the mass slide and the external erosion during the overtopping of the rock fill dams. The main parameters involved in the analysis are based on the dam and protection geometries, particle size distribution and the geotechnical properties and the hydraulic conditions.

The whole methods in the laboratory can be divided in the various stages.

- Hydraulic computations of the condition of overtopping height
- Discharge over the dam during the overtopping
- Unit discharge outflow from dam
- Calculation of the depth of the water emerging out from downstream

4.5.1 Hydraulic computations

The hydraulic computations mainly contain the computations of the condition of the overtopping. In the first phase of computation, the dam is checked against the overtopping conditions. The overtopping is mostly due to the inadequate spillway capacity and the inability of the opening of the gate due to the high flood conditions. The reservoir water level in the gates is one of the main parameters. Generally, the radial spillway gates are generally operated during the monsoon and under the flood condition. In such condition, and when the potential failure mode is triggered in the gate, mostly the reservoir water level is at the top of the gates before the operation of the gate normally.

In case of the Kulekhani dam as mentioned above contains two spillways:

- controlled or gated spillways and
- uncontrolled spillways

Discharge over the radial gates and controlled spillway gates

Normally the radial gates are used for controlling the water level. The discharge through the gate is the function of the square root of the head below the gate.

$$Q=kab(2.g.H^{\frac{1}{2}}) \qquad \textbf{Eq. 4-29}$$

However, in accident situation, the malfunctioning of the gates may occur due to the power failure, or mechanical failure or gate failure. It is assumed that the failure occurs at the different number of slots during the flooding condition[34]

When the gates are closed during the flooding condition, the water flows over the gates as shown in the Figure 4.2 and the discharge capacity is the function of the 1.5 times the head over the top of the gate. Kjellsvig [8] defined the coefficient of discharge to be ranging from 1.6 to 1.8 during the overtopping of the radial gates.

$$Q = C.L.H^{\frac{3}{2}} \qquad \textbf{Eq. 4-30}$$

This occurs when there is free surface between the water level and the top of the platform or bridge in the real case. The flow can sometimes become high flow, which means the flow where the water surface elevation exceeds the lower chord of the bridge deck especially when the bridge is above the gated spillways. In the absence of more reliable data, the US Army Corps of Engineers (2002) [35] suggests weir coefficients of about 0.32 for flow over the bridge deck and 0.37 for flow overtopping the roadways approaching the bridge.

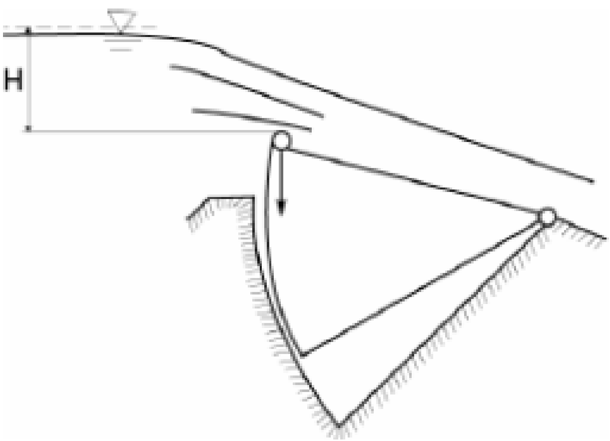


Figure 4.2 Flow over the radial gate[8]

Depending upon the water surface elevation, the crest level of the road embankment and the elevation of the high and low chord of the bridge deck three types of the flow can occur during the flooding conditions.[35]

- Sluice gate type of flow
- Orifice type flow

- Weir type flow

- ***Sluice gate type of flow***

When the flow comes into the contact with the bridge in the upstream side but at the downstream sides level is below the lower chord, the hydraulic behavior at that time resembles with that of the flow under the sluice gate. This type of condition persists when the flow height is less than the road level which can be shown in Figure 4.3. The flow is calculated as given by the equation

$$q = \frac{2}{3} C \sqrt{2g(d_1 - \Delta z)^3} \qquad \textbf{Eq. 4-31}$$

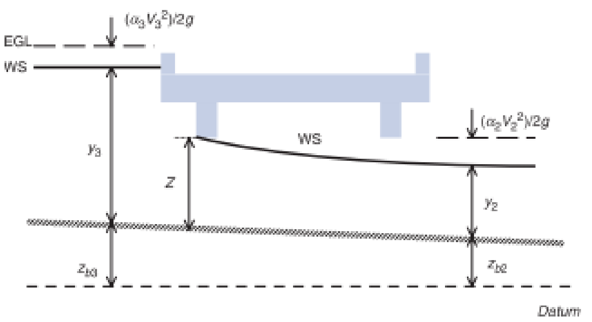


Figure 4.3 Sluice type of flow[35]

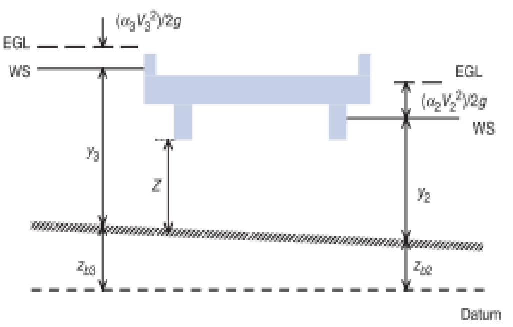


Figure 4.4 Orifice type of flow[35]

- ***Orifice type flow***

This type of orifice flow occurs when the upstream and the downstream sides of the bridge are submerged shown in Figure 4.4.This type of the case does not persist in our case as the difference in the upstream and the downstream level is very high.

- ***Weir type flow***

Generally the weir type of flow occurs when the flow overtops the roadway approaching the bridge and the bridge shown in the Figure 4.5. Weir type of flow is taken into the consideration and the calculations are discussed in the Table 4.1 Calculation showing the discharge during the PMF when both gates are closed.

The total flow in the upstream will be equal to the sum of the flow over the road or the bridge, the flow through the bridge opening. The flow over the bridge and the road can be considered as the free flow equation by the use of the weir flow equation. The flow through the bridge opening between the gate and the bridge can be calculated as the sluice gate equation or the orifice equation depending upon the downstream level of the bridge.

When the level is above the road level the pressure flow occur through the opening between the bridge lower deck given by Eq. 4-32 and the top of the gate and the free weir flow occur above the top of the bridge or the road given by equation of the flow over the gate.

$$Q = \frac{2B}{3} C_d \sqrt{2g} \left(H_2^{3/2} - H_1^{3/2} \right) \quad \textbf{Eq. 4-32}$$

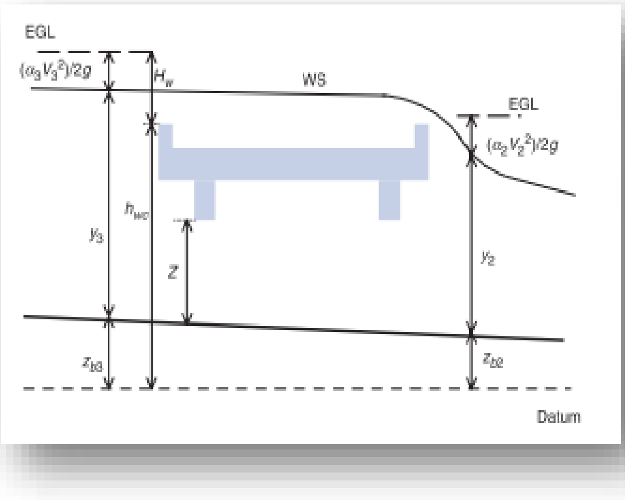


Figure 4.5 Weir type flow[35]

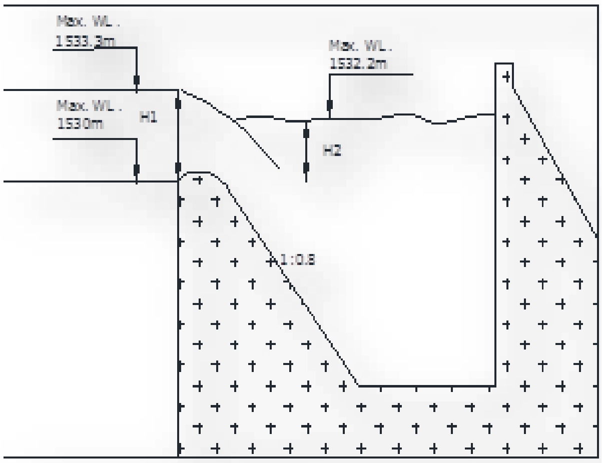


Figure 4.6 Cross section of troughway

Discharge capacity of the uncontrolled spillway side channel

The discharge capacity of the uncontrolled spillway is 800m^3/s and the elevation of the maximum spillway level is 1534m, which brings the discharge coefficient of the spillway side channel as 1.538. The discharge coefficient of the side channel spillway is less because of the submerged condition of the water that may be created during discharging of the water downstream shown in Figure 4.6.

4.5.2 Discharge over the dam during the overtopping

Stephenson 1979 stated that the rock fill are considered to be more stable and less erosive in the overtopping conditions than the earth fill. Thus, the rock fill can withstand relatively high values of discharges without the failure. For the computation of the flow over the crest of the dam the formulas and the methodology presented by Martin (1984) is used. The discharge coefficient is calculated using the regression equation obtained from his experimental results[36].

$$Q_c = C_c L'_c \sqrt{2g} (N_R - N_C)^{1.5} \quad \textbf{Eq. 4-33}$$

$$C_c = 0.333 + 0.132 \frac{N_R - N_C}{\text{B}} \quad \textbf{Eq. 4-34}$$

C_c = crest discharge coefficient
L'_c = overtopped crest length
N_C = crest level
N_R = water level in the reservoir.

This formula neglects the initial head effects caused by the propagation in the reservoir.

During the design of the kulekhani dam two types of the flood are considered. One is extraordinary flood and another is probable maximum flood. The extra ordinary flood is1.2 times bigger than the spillway design flood computed. The calculation of the overtopping during the extreme flood calculation is analyzed. The spillway capacity is quite enough for both the conditions.

After the calculation of the discharge through the main controlled spillway, side spillway and uncontrolled and the discharge over the crest, the total outflow discharge obtained during the probable maximum flood is equated to the sum of these three discharges as shown by Figure 4.7. Normally when the water level exceeds the core level of the rock fill dam in typical cases, the water begin to flow through the different zones outside the core. However, in case of the Kulekhani dam, the core level is almost at the top crest level of the dam, the overtopping occurs when it exceeds the crest of the dam. The discharge over the dam crest gives the overtopping height above the dam. This is of main concern for the drainage capacity computation of the dam.

Figure 4.7 the discharge over the dam crest

The height of the overtopping above the crest level during the probable maximum flood condition is obtained to be 1.377m which is shown in Table 4.1.

Table 4.1 Calculation showing the discharge during the PMF when both gates are closed

Calculations

Spillway design flood	=	1150	m^3/s
Extraordinary flood	=	1380	m^3/s
Probable maximum flood	=	2720	m^3/s

Inflow discharge (m^3/s)	Outflow discharge(m^3/s)	Max. water level (m)
1380	1270	1530.3
2720	2540	1533.3

Spillway

Design peak flow	=	2720	m3/s
Overflow crest elevation	=		

Gate controlled weir	=	1519	m
Side spillway	=	1530	m
Top of the dam crest	=	1534	m
Top of the bridge	=	1534	m
bottom of the bridge	=	1533	m
Effective overflow weir	=	18	m
Side spillway	=	65	m
Gates	=	2	

radial gates each 9m wide and 11.5 m high

Flow over the radial gate

Q =	discharge gate flow
Cd	coefficient of discharge
G0	vertical gate opening
B	gate width
g	gravitational constant
H	head term

When both gate is closed

Calculation for the overflow height

height of the weir Δz 16m

For the effective length of the crest

Leff=L-Ha(np.ep+ne.ee)

L =	Open crest length	
np =	no of round pile sides	
ne =	no of 90 edges	
ep =	parameter for round piles	
ee =	parameter for 90 edges	
C =	1.6-1.8	1.7

The flow over sharp crested weir = $CLH^{1.5}$

Discharge when the water level reaches the bridge level exceeds the 1533m

Top of the gate	=	1530	m
Effective overflow weir	=	18	m
Side spillway	=	65	m
Number of radial gates each 9m wide and 11.5 m high	=	2	
height of the opening under bridge (y_g)	=	3	m
Cd for the flow over the bridge	=	0.32	
Length of the waterway over the bridge	=	20.5	

Discharge through the side channel spillway

the discharge capacity of the side channel spillway	=	800	m3/s
The elevation of maximum spillway level	=	1534	m
crest level of the spillway	=	1530	m

The head above crest during maximum	=	4	m
crest length of the spillway	=	65	m
The discharge coefficient of the spillway side channel	=	$Q/(LH^{3/2})$	
	=	1.538461538	
Level of the bridge	=	1533	m

Discharge over the dam (section Error! Reference source not found.**)**

overtopped Crest length (L'C)	=	397	m
crest level (Nc)	=	1534	m
Crest discharge coefficient (CC)	=		
Crest width (B)	=	10	m

4.5.3 Unit discharge outflow from dam

The downstream depth of the water emerging from the dam is calculated by hit and trial shown in and the depth is obtained to be at the elevation of 1439.383m shown in Figure 4.9 and Figure 4.9. The methodology is summarized as follows:

- The height is assumed at initial stage.
- The whole section is divided into equal sections
- The discharge per unit section is calculated for each section divided
- The discharges is summed for the given assumption of the height
- If the summed discharge is equal to the inflow discharge over the dam, then the given height assumed is ok but if it is not then another height is assumed and the process is repeated.

From the given calculated outflow discharges, the maximum discharge is selected and the design is done for the given discharge.

Height of overtopping from dam when the two gates are closed	=	1.38	m
The discharge that is passed through the spillway		1541.98	m3/s
The discharge total outflow during the probable maximum flood (PMF)	=	2540	m3/s
From the flood routing			
The discharge that overflows through the dam	=	998.02	m3/s
crest length of the dam	=	397	m
Unit discharge over the dam at the top	=	2.514	m3/s/m

4.5.4 Calculation of the depth of the water emerging out from downstream

The total amount of the water that outflows from the dam is shown in

Table 4.2. During the flow downstream, the water is assumed to flow down and it emerges outward at the certain height downstream as in Figure 4.8. The height of the water that emerges out of the toe is given by the equation as explained which is observed to be 1439.383m by using the Eq. 4-35.

$$h_t = \frac{q}{\sqrt{k_t \sin\theta}} \qquad \text{Eq. 4-35}$$

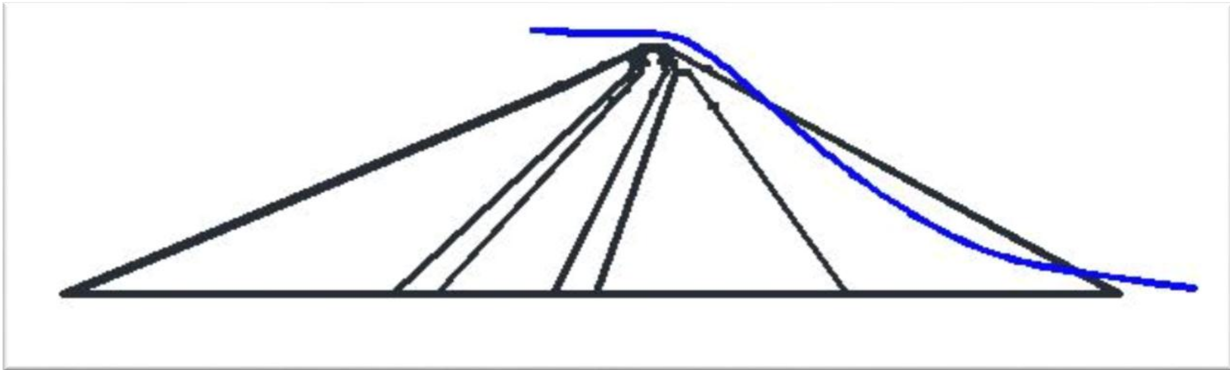


Figure 4.8 Assumed flow pattern during overtopping

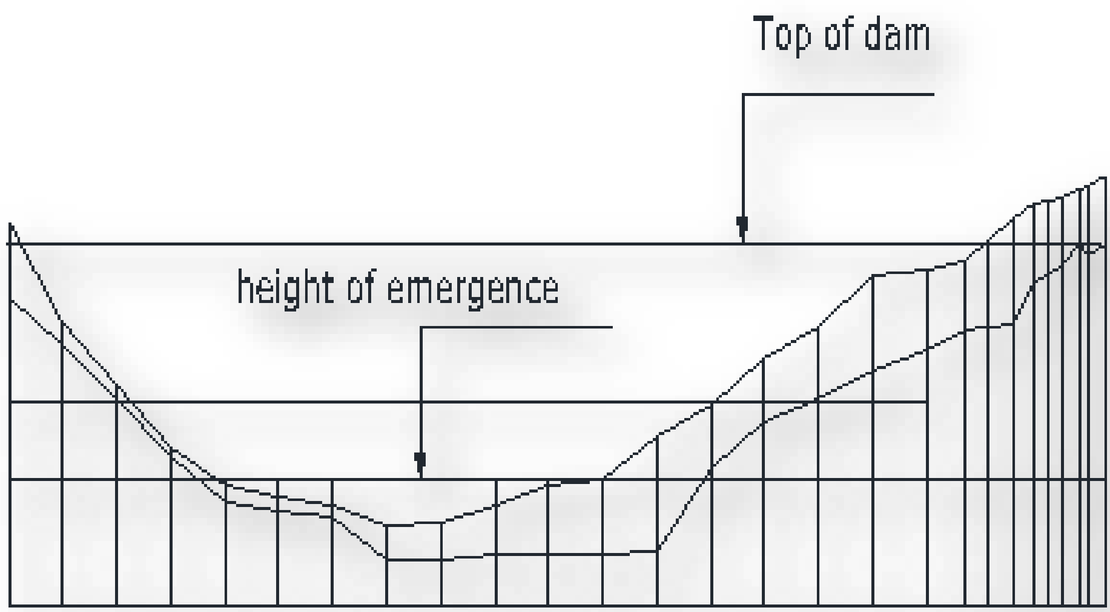


Figure 4.9 Calculation of the height of emergence

Table 4.2 Calculation of the outflow from the dam during the overtopping

B(m)	Elevation (m)	height (m)	mid ht of strip (m)	$q=h*(k_t \sin\alpha)^{0.5}$ (m3/s/m)	Total discharge(m3/s)
0	1513.50	0.00	0.00	0.00	0.00
0	1497.00	0.00	0.00	0.00	0.00
0	1477.00	0.00	0.00	0.00	0.00
0	1455.00	0.00	0.00	0.00	0.00
20	1438.50	0.88	0.44	0.02	0.02
40	1435.00	4.38	2.63	0.11	4.23
60	1433.00	6.38	5.38	0.22	12.98
80	1417.00	22.38	14.38	0.58	46.25
100	1417.00	22.38	22.38	0.90	89.97
120	1418.50	20.88	21.63	0.87	104.34
140	1418.50	20.88	20.88	0.84	117.51
160	1419.00	20.38	20.63	0.83	132.69
180	1420.00	19.38	19.88	0.80	143.85
0	1451.50	0.00	9.69	0.39	4.79
0	1468.00	0.00	0.00	0.00	0.00
0	1477.00	0.00	0.00	0.00	0.00
0	1487.50	0.00	0.00	0.00	0.00
0	1495.00	0.00	0.00	0.00	0.00
0	1502.00	0.00	0.00	0.00	0.00
0	1503.00	0.00	0.00	0.00	0.00
0	1504.00	0.00	0.00	0.00	0.00
0	1520.00	0.00	0.00	0.00	0.00
0	1522.00	0.00	0.00	0.00	0.00
0	1527.00	0.00	0.00	0.00	0.00
0	1534.00	0.00	0.00	0.00	0.00
0	1530.00	0.00	0.00	0.00	0.00
0	1534.00	0.00	0.00	0.00	0.00
180	**Σ**			**5.54**	**998.02**

The height of the outflow from the dam = 1439.383m

For the design of the rip rap, the region with the greatest outflow discharge is taken as the design outflow discharge. In this given case, it is 0.899m3/s/m.

4.6 Discussion of Overflow and throughflow in context with the present study

When the water overtops the embankment, large portion of water overflows with the combination of the throughflow. All the water flowing through the rockfill slope emerges above the toe. The flow of water, which passes through the embankment slope, produces the pressure and the drag force on the elements within the body of the slope and the seepage face. Hauser and Michael presented the velocity of the surface current at the downstream slope of the rockfill based on the arbitrary roughness equation similar to Chezy equation. [37]The effect of the seepage is more critical when the water exits out of the dam at the downstream during the overtopping. For calculating the velocity simply, the Chezy formula can also be used.

Overflow is characterized when the water level exceeds the crest of the dam whereas the throughflow occurs in both exceeding and non-exceeding of the crest. The modeling should involve the modeling of the existing condition of the throughflow and overflow. Two approaches are highlighted for the modeling,

- One permitting only overflow and not the overflow condition
- Other permits both the overflow and the throughflow conditions. This approach will give truer picture because the dam under the topic is subjected to throughflow condition and the overtopping condition under the exceeding of the water level.

This overflow and throughflow condition is used for the study of the laboratory testing in the rockfill.

Chapter 5

5 Rip rap study literature review

The safety of the complete dam depends on the safety of the protection layer, thus it is important to have protection layer in order to protect the dam. The flow of water over the embankment results in the strong erosion forces that cause the breaking of the flow surface. The flow becomes more complex due to the possible air entrainment, the shear stresses and the high velocity. During the erosion process, the strong erosion forces typically occur at the downstream shoulder and on the embankment slope, which needs to be withstood. [22] Thus if the downstream face of the overtopped zone is not applied with some slope protection measures, then the dam would easily be eroded starting at the toe of the dam.

According to the Powledge, placing riprap over the embankment downstream face is one of the promising measures for the safety of the dam. Rip rap is used as the protective layer of rock fragments or the blocks for absorbing the energy. It prevents the erosion and protects the slope so that no initiation of the movement occurs due to the reduced the shear stress and the velocity of the flow. The term riprap is used to refer as the assemblage of the rocks nested together to protect the structure and to rock riprap.

5.1 Evaluation of the rip rap

The rip rap is divided into different types depending upon the placement method like placed riprap, hand placed rip rap and the plated rip rap.[38].The stability of the riprap is the function of the individual rock size, shape of the rocks, weight and the durability. The placing of the stones depends on the characteristics of the individual rocks for the stability, gradation of the stones, thickness of the layer and on the site conditions. The placement of the rock is to be done to minimize the voids and thickness to keep the volume as low as possible. Under sizing of the thickness of the riprap protection can lead to the fluidizing of the protective layer causing the embankment to the severe erosive process.

Dimensions of the rip rap

The dimension of the riprap is expressed by the dimension of the three axes. The longest axis is denoted as a, the intermediate axis, where b is the maximum width, is perpendicular to the long axis and the thickness of the stone perpendicular to a and b axes is denoted by c. The stone size is measured in the following way in terms of the dimensions.

$$d = (axbxc)^{\frac{1}{3}} \qquad \textbf{Eq. 5-1}$$

While dimensioning, weight is also commonly used by using the equivalent spherical diameter[39].

$$D = 2\left(\frac{W}{691.15}\right)^{1/3} \qquad \textbf{Eq. 5-2}$$

The weight of the riprap depends on the shape of the stone, specific weight, density of the rock. The riprap to be used is not spherical and the shape is generally in between the sphere and cube. The sphericity is computed as

$$\text{Sphericity} = \left(\frac{V_p}{V_{Sp}}\right)^{1/3} \qquad \textbf{Eq. 5-3}$$

Where

V_p = volume of the particle given by mass/density
V_{Sp} = volume of the circumscribing sphere
Sphericity is the measure of the shape of the particle to see how closely the major axes of the particles are closely to the sphere.

SF shape factor is the measure of the oblateness of the particle given by

$$SF = (\frac{c}{\sqrt{ab}}) \qquad \textbf{Eq. 5-4}$$

The drag force on the rounded stones is less than that of the angular rock fragments and the interlocking is poor due to which angular stones nested together resist the movement from hydraulic forces and gravitational forces and make best riprap.

Durability

The riprap should be durable and should not be susceptible to displacement by the hydraulic forces as it affects the ability to maintain the shape, size and gradation and the ability to resist the weathering and other influences. Durability is the function of the rocks mineralogy, porosity, weathering, discontinuities, and the conditions of the site.

Rip rap thickness.

The thickness of the riprap should be enough in order to protect the slope from severe condition to improve the stability. The riprap thickness should not be less than maximum of the spherical diameter of the D100 stone or less than the equivalent spherical diameter of D50 stones. The under sizing of the thickness is not good for the resistance whereas the oversize is also not good as it might contribute to the failure by creating the local turbulence and removing the smaller stones[40].Various literatures propose the different thickness of the riprap. NVE guidelines state that the thickness of the riprap cover needs to be the two times the diameter of the stone size.

Particle gradation

The gradation design of the riprap is based on the ability of the stones to produce the appropriate sizes, which is controlled, by the rock mineralogy, cleavage and the fractures in the rock. The gradation will be appropriate if the particles can nest together and withstand the environmental conditions.

5.2 Role of packing factor and angle of repose

Packing factor is also important factor for maintaining the stability of the bed condition. The packing factor is defined as the number of the stones in the unit area and divided by the area of the average stone. If N is the number of stones in the unit area and $\mathrm{d_s}$ is the mean diameter of stones then the packing factor if the exposed surface is assumed to be the square is given by

$$P_c = \frac{4}{\pi \mathrm{d_s}^2} \quad \textbf{Eq. 5-5}$$

The packing factor has the great impact on the threshold flow given by the Oliver. The packing factor for the manually placed on the edge and the flat varies a lot. The manually placed stone on edge corresponds to the higher threshold flows than the stones placed on the flat edge or the random placed[41].

The angle of the repose also has the direct influence on the threshold flow. When the stone is placed in the inclined bed and subjected to the force of the flowing water F and bed is placed at angle with angle of the repose. $f(x) = \mathrm{F} = \cos\emptyset\,(\tan\emptyset - \tan\theta) = 0$

Under the high flow condition, the shear stress also acts and F can be expressed as the differential force acting on the stone.

$$\tau = 0.524\,\mathrm{d_s}\cos\emptyset\,(\tan\emptyset - \tan\theta)(\mathrm{w_s} - \mathrm{w}) \quad \textbf{Eq. 5-6}$$

The shear stress in terms of the experimental constant C and packing factor$\mathrm{p_c}$ is given by

$$\tau_0 = 0.524\frac{\mathrm{C}}{\mathrm{P_c}}\mathrm{d_s}\cos\emptyset\,(\tan\emptyset - \tan\theta)(\mathrm{w_s} - \mathrm{w}) \quad \textbf{Eq. 5-7}$$

The threshold flow is expressed in the terms of the packing factor and angle of repose as

$$\mathrm{q}_{oT} = 11.84\left(\frac{\mathrm{w_s} - \mathrm{w}}{\mathrm{w}}\right)^{5/3} d_s^{\,3/2}\tan\theta^{-7/6}\left(\frac{\mathrm{C}}{\mathrm{P_c}}\right)^{5/3} E^{\frac{2}{3}} \quad \textbf{Eq. 5-8}$$

$$\mathrm{E} = \cos\emptyset\,(\tan\emptyset - \tan\theta) \quad \textbf{Eq. 5-9}$$

Where

C	=	experimental coefficient
$\emptyset$	=	angle of repose
θ	=	slope angle

The threshold flow calculated using the Eq. 5-8 and compared with the laboratory tests but no correlation is found to exist between the experimental and the calculated value.

5.3 Methodology for riprap filling

Rip rap layer should be filled in such a way that it becomes able to withstand the erosive forces and different instability conditions. [42]. The stones should also fulfill the following parameters for the placement.

$$\frac{3}{2} < \frac{a}{b} < \frac{3}{1}$$

$$\frac{b}{c} < \frac{3}{2}$$

$$\frac{a}{c} < 3$$

The four rules of thumb given by NVE guidelines can be applied for the placement of the riprap.

Inclination:

The inclination of the stones with respect to the longest axis should be placed with the inclination of the stones.

Axis

As far as possible the longest axis of the stones with the dimension represented by a should be perpendicular to the axis of the dam. The orientation and the strength of the riprap on different orientation will be studied in detail.

Bond

While placing the layers, the riprap should be placed in such a way that they do not form the vertical and the horizontal bonds and joints in the rip rap face.

Interlocking

The stones should be placed interlocking pattern so that the hydraulic forces and other forces of instability do not easily move them.

Rip rap flow conditions

The flow through the riprap placed on the embankment dams is the function of the rock size, embankment, slope and discharge. When the flow is low, the flow occurs over the crest and enters into the riprap layer with no visibility of the water over the surface of the rock layer and the flow is entirely interstitial. When the flow increases, the flow cascades over the surface and it tries to penetrate the rock layer. With the continual increase in the velocity, the instability occurs and the forces will eventually move the surface rocks from the protective layer.

5.4 Design and sizing of the rip rap

Sizing of the stone is the first step in order to make more comprehensive design and the successful riprap armoring system. Various literatures give the different aspects of the design and sizing of the riprap. Most of the methods are based on the methods presented by the Isbach or Shields. The proper riprap design criteria should prevent the stone movement and should ensure the riprap layer does not fail. The main sizing of the riprap is dependent on the material properties, embankment angle and the unit discharge. Some of the concepts and the criteria proposed can be summarized as follows:

Isbach (1935)

Isbach (1935) studied the sizing of the stones placed on the downstream of the dam slope to resist the movement during the overtopping flow and the throughflow through the dam body [43]. He deduced the expression for the critical transport velocity for displacing the rounded stones. He further expressed the velocity through the rock layer[44]. Based on that, he proposed the methods for dumping and stabilizing the stones in flowing water.

Witters

Witters [45] expressed the forces on the riprap particle by using Shield parameter, which is the dimensionless ratio of inertial forces due to bed shear and gravitational forces. He hypothesized the value of Shields parameter as 0.047 even for shallow highly flows down the steep slopes. Studying the influence of the riprap gradation uniformity, the well-graded mixtures had significantly lower failure thresholds.

Hartung and Scheuerlein

Hartung and Scheuerlein [46] obtained the relation of the maximum unit discharge that would resist the movement of the stone for the steep slopes. They suggested that fully developed flow velocity over the downstream face of a rock fill dam is the function of the flow depth, slope, rock size, particle-packing factor, and aeration factor, which is the function of the mean roughness height, mean water depth and angle of the slope.

Knauss

Knauss [47] compared Oliver's method and that of Hartung and Scheuerlein and proposed the expression for overtopping flow conditions for both angular and the crushed stones. He developed the rock stability function based on the unit discharge, slope, rock-packing factor and the air concentration for sizing riprap. Stephenson performed the stability analysis of the angular stones on the downstream slope of the rock fill dam subjected to overtopping.

$$\mathrm{F}_c = \frac{\mathrm{q}_c}{\sqrt{g}{d_s}^{3/2}} = \frac{\mathrm{C}}{\sqrt{g}} = (\tau_c^*)^{\frac{5}{3}}(\frac{\rho_s - \rho_w}{\rho_w})^{5/3}{S_0}^{-7/6} \quad \textbf{Eq. 5-10}$$

For the slope steeper than 0.2, he proposed the equation as followed for

$$\mathrm{d_s}=(\frac{\mathrm{q_c}}{\sqrt{\mathrm{g}}(1.9+0.8\emptyset\text{-}3\sin(\arctan \mathrm{S_0}))})^{2/3} \quad \textbf{Eq. 5-11}$$

The aeration of the flow increases the critical velocity at which the stone starts to move which makes the riprap on the steep slope stable. (Oswalt et. al)

Abtet. Al

Abt. Et al and Johnson found out the sizing of the angular stones as the function of the specific design unit discharge. It does not take in the account of the effect of rock gradation but used 1.35 load factor for the design. The equation is given by

$$\mathrm{D}_{50} = 5.23S^{0.43}q_{design}^{0.56} \quad \textbf{Eq. 5-12}$$

$$q_{design} = 1.35q_f \quad \textbf{Eq. 5-13}$$

The equation is written as

$$\mathrm{D}_{50} = 0.503i^{0.43}q_f^{0.56} \quad \textbf{Eq. 5-14}$$

The unit discharge q_f is determined by

$$q_f = c_f q \quad \textbf{Eq. 5-15}$$

Where

D_{50} = median stone diameter
S = embankment slope
q_f = design unit discharge
q_{design} = specific design unit discharge

The round stones failed at the design discharge of about 45 % less than that for the angular stones so the design requires over sizing of about 40%.

The depth of the protection layer is given by Abt.'s formula

$$t_t = \max(1.5D_{50}, D_{100}) \quad \textbf{Eq. 5-16}$$

Robinson and Rice

Robinson and Rice used the riprap of angular in shape with coefficients of the uniformity varying from 1.47 to 1.73 and proposed that the rock sizing relation as the function of the unit discharge and the chute slope. The slope varies for 17 degree to 40 degrees.

$$\mathrm{d}_{50} = 0.5{S_0}^{0.31} q_t^{0.53} \quad \textbf{Eq. 5-17}$$

Mishra

Mishra concluded that the stable size of the rocks is the function of the stone coefficient of the uniformity, unit discharge and the slope of the embankment.

Rui Martins

Martins suggested formula for the embankment slope varying from 1.1 to 1.3 and introduced the height of the dam as the parameter to influence the stone size. But the variation of the dam height doesn't vary much the result.

$$\mathrm{d}_{50} = 0.64 \frac{q_t^{0.94} S_0^{0.86}}{H_d^{0.41}} \quad \textbf{Eq. 5-18}$$

5.5 Prediction of the stone size and analysis

Various guidelines and methods are used for the designing the rip rap as protection methods. Some are based on the calculations as the function of the various parameters influencing the characteristics of rip whereas some are based on the different sets of curves from various investigations and research which combines the rock properties of the rock material with the embankment slope, discharge. But these methods and the factors used are varying for various slopes of the rockfill and types. Some of the sizing methods are already discussed and some of the major methods are described in the following section.

5.5.1 NVE Guidelines

According to the Norwegian dam safety authorities, the Norwegian Water resources and Energy Directorate (NVE) the downstream slope of embankment dams should be able to withstand the certain overflow. NVE specifies that the minimum allowable size should be calculated by using Solvik formulae

$$D_{50} = 1q^{2/3}S_0^{7/9} \qquad \textbf{Eq. 5-19}$$

where

q = flow per unit width
S_0 = slope of downstream dam face

Norwegian rockfill dams have generally the downstream slope of 1:1.5, which can be considered as the steeper in consideration to the other used in the experiments. The calculations showed in the Graph 5.1 in results shows that the size given by NVE for flatter slope is quite oversized.

The NVE guidelines gives the thickness of the riprap needed to be twice the median diameter of the stones or in some, it is specified as the D100 size rock in the layer.

Oliver proposed a relationship of rock size d_s as the function of slopeS_0, rock density ρ_s and threshold flow q.

$$d_{50} = 2.63{q_t}^{2/3}\left(\frac{\rho_s - \rho}{\rho}\right)^{-10/9}{S_0}^{7/9} \qquad \textbf{Eq. 5-20}$$

5.5.2 Graphical methods for prediction of the stone sizing and safety analysis of the dams

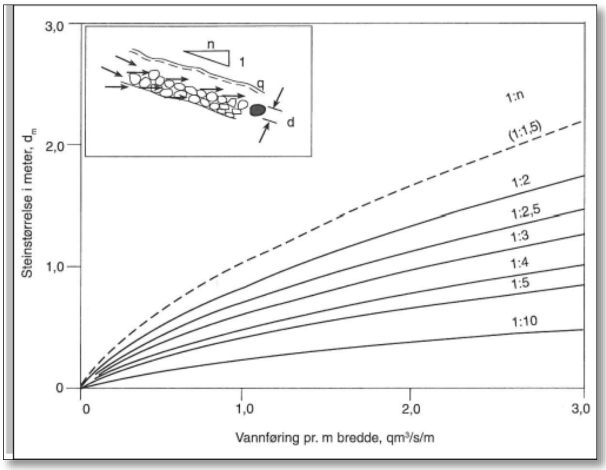
Solvik in 1991 presented the dimensioning criteria for the overtopping conditions as given in the Figure 5.1 and Figure 5.2. He simplified the graph for the stone size required for different sloping conditions for the top of the dam, downstream face and the important key stones at the end of the dam toe. The dimensioning criteria provided by Solvik may be used for the safety analysis of the dam for the overtopping situations.

The design of the toe of dam is mainly affected by the following factors according to Solvik.

- The unit discharge out of the dam
- The permeability of the dam

He also emphasized considering the different designs at the inflow area and the toe of the dam. He pointed out the toes or the key stones are vulnerable to the dam so the stability needs to be increased by using the larger stones. The dimensioning of the toe stones are given by Figure 5.2

The stones in the outflow can also be dimensioned as the function of the discharge per unit width and the slope of the embankment as given by Figure 5.1.



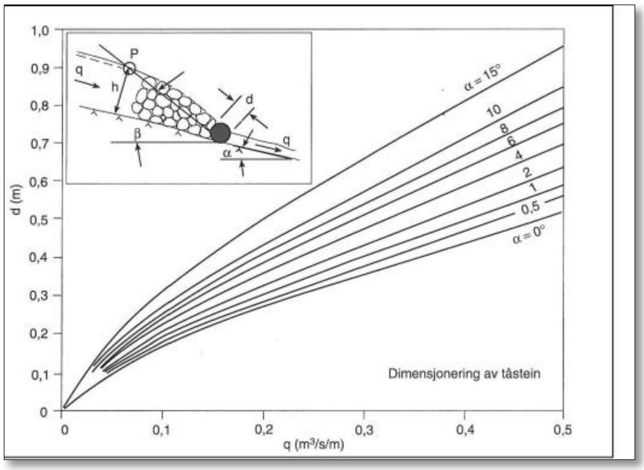


Figure 5.1 Design of the stone in the outflow area **Figure 5.2 Design of the toe stone in the foundation**

5.5.3 USBR (United States Bureau of Reclamation) method

USBR and Colorado State University (CSU) in the cooperative agreement built the prototype-sized embankment and stated the velocity of water, which flows through the rockfill voids to be governing factor in the riprap design shown in Eq. 5-21. It is used for determining the depth of the water flowing through the riprap.

$$\frac{v_i}{\sqrt{gD_{50}}} = 2.48S^{0.58}C_u^{-2.22} \qquad \textbf{Eq. 5-21}$$

Where

v_i = interstitial velocity

The average velocity is determined from the velocity in through the rock voids by taking into account of the porosity. The required thickness is determined by using the flow depth given as $y=q/v_{ave.}$ In addition, relationships between the embankment slopes, median rock size D50 and allowable surface flow. The thickness of the rip rap to be used is completely dependent on the interstitial velocity.

In general, for steeper slopes, the flow will be interstitial easily but for the milder slopes, thickness can be increased to the placement limit of 4D50. In the mild slope, where the depth of flow exceeds the 2D50, discharge that can safely pass the riprap must be determined. The flow depth needs to be determined by using the standard flow equations and roughness by Manning's equation.

$$n = 0.0414D_{50}^{1/6} \qquad \textbf{Eq. 5-22}$$

This approach gives the design where the riprap layer is at the point of failure for design discharge. Thus, the factor of safety is required if the design is for the design flood other than probable maximum flood. In the case of kulekhani dam, the factor of safety is not considered for the accidental case.

The universal riprap design is based on the Shield's parameter and it takes in account of the effects of the gradation of the rock layer, slope of the embankment and the unit discharge. The equation for determining the size of the rock is given by

$$d_{50} = K_u q_f^{0.52} S^{-0.75} \left(\frac{\sin\theta}{(S_s\cos\theta - 1)(\cos\theta\tan\phi - \sin\theta)}\right)^{1.11} \quad \textbf{Eq. 5-23}$$

5.5.4 Stephenson method

Stephenson.[30] developed this procedure for the resistance of the rock fill to the through and the surface flow. The stone size D50 can be expressed in the term of following equation Eq. 5-24. Here the value of c varies for different types of rocks. This method does not accounts for the uplift of the stones due to the emerging flow and is developed for the overtopping flow and the through flow at the rock fill of steep slope. The q is the unit discharge at failure and it is to be multiplied by Oliver's constant for the stability.

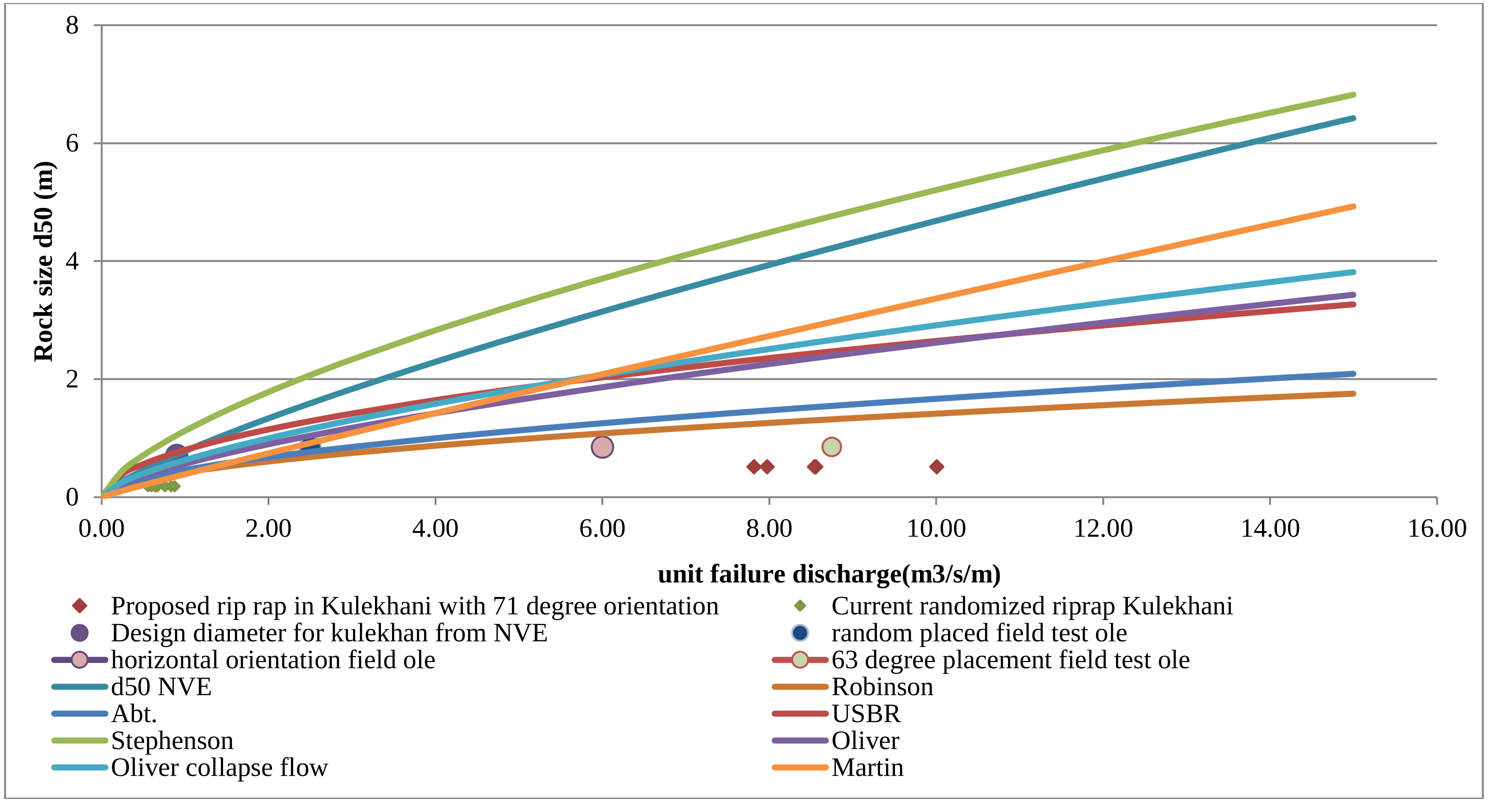
$$d_{50} = \left(\frac{q(\tan\theta)^{7/6} {n_p}^{1/6}}{C g^{1/2}((1-n_p)(S_s - 1)\cos\theta\,(\tan\phi - \tan\theta))^{5/3}}\right)^{2/3} \quad \textbf{Eq. 5-24}$$

5.6 Riprap design results and discussion

For designing the downstream toe, many guidelines have been presented by many authorities, which are discussed in the previous sections. For the design of the rip rap, the maximum unit discharge outflow from the dam is taken as the design unit discharge. First the design curves given by the Solvik [14] is used for the design of the stone at outflow area. In addition, various methods given are used to check the variability in the methods and results. The results from the various methods and designs are shown tabulated in Table 5.1. The various methods can be compared in the Graph 5.1 and the strength can be checked with the designed.

Table 5.1 Calculation from various methods

Methods	Diameter d50(m)
d50 NVE	0.72
Robinson	0.39
Abt.	0.43
usbr	0.76
stephenson	1.05
oliver	0.53
collapse oliver	0.58
martin	0.35
Solvik curve	0.7



Graph 5.1 Rock size at failure for slope 1:1.8 and strength from experiments

The methods proposed by Oliver are generally used for the flat slopes. Oliver defined the side slopes if greater than 1:5 then the downstream toe is more stable to the vulnerable flows. However, the disadvantage of the using the flatter slope is that most volume of the rock fills is necessary which can prove to be the uneconomical**.**

NVE suggests the design should be able to resist the leakage or an overtopping that causes the throughflow of 0,5m3/s per meter width or 10m3/s for dams in narrow valleys. The NVE refers to Solvik et al 1992 when considering the design requirement for the downstream toe. NVE gives the minimum dimension of the stones to be used as the riprap according to Eq. 5-19. The guidelines are generally used for the slopes upto 1:1.5 and the results are quite oversized for the flatter slopes than 1:1.5.Therefore, for the design of the rip rap, the size of available stones of the Kulekhani dam used is quite less than the design provided by NVE. The strength of the rip rap used is evaluated from the experimental results which are also shown in Graph 5.1 Rock size at failure for slope 1:1.8 and strength from experiments

Stephenson method is generally used for the emerging flows as discussed in the sizing of the stones. It does not account for the uplift of the stones caused by the emerging flow. It is generally used for the stabilization for the overflow but still alternative riprap rockfill design procedures need to be considered for the additional toe protection.

USBR method provides the sizing of the stone based on the conservative approach based on the velocity and the flow impingement at the toe. They take the gradation of the stones into the consideration unlike other formulas.

The Abt method and Robinson method were based for the slopes greater than 1:2.5. They gave more undersized results for the sizing of the riprap. In the case of Kulekhani, the riprap is proposed taking into the consideration of the NVE and Solvik curves. It is proposed to use the existing bigger sized stones available in the downstream dam and to meet the minimum criteria of the diameter of the stones from NVE guidelines. The use of the existing stones, which are varying in size and bigger than considered in the tests, can be good for both economical point of view and for the safer dam itself. From the Graph 5.1 Rock size at failure for slope 1:1.8 and strength from experiments it shows that the strength of the rip rap is quite good.

The choice of the method for design should be done based on the existing condition of the rockfill, the rockfill properties, the hydraulic parameters and the geotechnical consideration. Those factors greatly affect the design of the rip rap conditions.

Similarities

These formulae are generally used for the angular riprap and based on the empirical and the experimental results. The bedding material satisfies the filter criteria and the riprap layer thickness used is 2*D50 or greater. In the case of the Stephenson method, the rock fill layer should be well graded and the layer thickness is same as mentioned.

Differences

Traditional procedures such as the USBR determined the median size of the rock by utilizing the flow velocity estimated at the transition for the toe stones. The design proposed by Abt and Johnson are used for the flatter slopes whereas the Stephenson method is generally used for the steep slopes.

5.7 Rip rap failure

When the riprap layer is removed enough so that the rock fill material gets exposed then we can say that the rip rap slope fails. The different tests are also conducted to analyze the procedure of how rip rap fails during the overtopping and the failure. Most of the failure starts with the dislodging of the stones. The main four types of the failure occur in the rip rap from (Blodgett 1986) are as follows

Particle erosion

The particle erosion starts when one of the stones from the downstream slope is dislodged and displaced as a result of the undersized rock or impact of the flow in that stones. Some of the toe stones are highly exposed and the particle starts to be eroded from that situation. The particle erosion can also occur in the case of the side slope being steep than the angle of the repose of the riprap material.

Translational sliding:

The translational sliding occurs when the downstream slope gets scour and generally loses the support along the downstream base occurring the sliding parallel to the side slope.

Modified slumping:

This slumping generally is characterized by the movement of the whole riprap without the toe failure. This occurs mainly in the steep slope than the flatter ones. It is similar to the translational slide but the geometry is similar to that caused by the particle erosion.

Slumping

Slumping denotes the rotational failure generally occurring at the high unstable banks. The main cause of slump failure is related to shear failure of the base material that is supporting the riprap.

5.8 Failure condition and criteria

Before the main whole riprap fails, several individual stones moved or rearranged the positions throughout the test period. This movement is the incipient motion. It occurs when the stabilizing moment or resisting moment is exceeded by the displacing or destabilizing moments. In the steeper slope, the riprap fails more by the whole mass sliding than the erosional process.

EBL [48] considered three failure criteria in the experiment namely first movement of block(movement of the stone), first surface slide(movement of few particles) and the incipient failure(slide or erosion or combination) Oliver defined the failure into threshold and collapse as mentioned previously. Prajapati also defined the threshold flow where the stones in the seepage region start eroding under the increment of the discharge without tailwater fluctuation. Toledo mentioned that the reduction of rockfill vertical permeability caused by the compaction caused the failure mechanism which occurs in two ways scouring and sliding. The mode of the failure will be discussed in the results and discussion chapter.

Chapter 6

6 Laboratory methods

This section mainly deals with determining the throughflow capacity and performance of current rip rap structure of Kulekhani dam with the help of various lab test programs. The laboratory tests were carried out in the flume in the Norske laboratory in Vassbygget for the modeling of the downstream slope of the rock fill dam.

6.1 Physical scale modeling

All the physical processes and the form of the structure could not be dealt singly by analytical techniques. Physical modeling provides the comprehensive study and the complementary techniques. It can be the good approach to solving many engineering problems, which may not be amenable to exact mathematical solution.

Physical models provide and simulate the complex process and the results in many geomorphic phenomena (Peak al et. al, 1996). This can be observed in a reduced time length under the controlled laboratory environment. It allows the incorporation of many variables where were not known at the initial stage and which have much nonlinear effects on the resultant dynamics or morphology. [49]

6.1.1 Model similitude

During the modeling, similitude should exist between the prototype and the model scale, which means that there should be dynamic, kinematic and geometric similarity between the model and the prototypes. Many variables, which are not known from the beginning, can be incorporated and the non-linear effects of each variable can be studied.

Similarity

Any physical model would only give the realistic results only if the similitude requirements are properly chosen. Formal mathematical definitions must be met by the scale ratios between the prototype and model. There are three laws of similitude between the model and the prototype.

Geometric similarity

The ratio of the length of the scale and the prototype is equal. This relationship is independent of the motion and involves only similarity in the form.

$$L_r = \frac{L_m}{L_p} \quad \textbf{Eq. 6-1}$$

Kinematic similarity

It indicates the similarity of the motion between the fluid particles. It is achieved when the ratio between the components of all the vectorial motions is the same in the model and the prototype. It means the streamlines shape at any particular time is the same in the model and the prototype. For this, the ratio of the flow velocity at any point in the model and the prototype need to be similar.

$$V_r = \frac{V_m}{V_p} = \frac{V_{1,m}}{V_{1,p}} = \frac{V_{2,m}}{V_{2,p}} \quad \textbf{Eq. 6-2}$$

Dynamic similarity

It indicates the system with the geometrical similar boundaries should have similar flow patterns at corresponding instants of time. Therefore, all the forces acting in the corresponding elements of mass must have the same in the two systems. This is the important prerequisite for the physical models as it ensures that there is constant prototype to model ratio of all the masses and forces acting on the system. For the fluid mechanics the Newton's law can be written as

$$F_i = F_g + F_\mu + F_\sigma + F_e + F_{pr} \quad \textbf{Eq. 6-3}$$

The overall dynamic similarity

$$\frac{(F_i)_m}{(F_i)_p} = \frac{(F_g + F_\mu + F_\sigma + F_e + F_{pr})_m}{(F_g + F_\mu + F_\sigma + F_e + F_{pr})_p} \quad \textbf{Eq. 6-4}$$

F_i = vector sum of the inertial forces
F_g = vector sum of the gravitational forces
F_μ = vector sum of the viscous forces
F_σ = vector sum of the surface tension forces
F_e = vector sum of the elastic compression
F_{pr} = vector sum of the pressure force

For the perfect similarity

$$\frac{(F_i)_m}{(F_i)_p} = \frac{(F_g)_m}{(F_g)_p} = \frac{(F_\mu)_m}{(F_\mu)_p} = \frac{(F_\sigma)_m}{(F_\sigma)_p} = \frac{(F_e)_m}{(F_e)_p} = \frac{(F_{pr})_m}{(F_{pr})_p} \quad \textbf{Eq. 6-5}$$

It is not possible to satisfy these all the forces described unless the model scale is equal to the prototype scale.

6.1.2 Model laws

There are several laws of similarity regarding the physical modeling of the hydraulic structures given as follows:

***Froude similarity* (Inertia and gravity forces are dominating)**
***Reynolds similarity* (Inertial and viscous forces are dominating)**
***Euler similarity* (Pressure forces are the dominant force)**
***Weber similarity* (Surface tension is dominating)**
***Cauchy similarity* (Elasticity is dominating)**
***Mach similarity* (Elasticity is important)**

Our physical model has to deal with gravity driven flow. It deals with highly turbulent flow and the viscous forces are very small compared to the gravitational forces of the water. Since it deals with the high Froude number, the viscous forces shall not be taken into account. Therefore, Froude's law is used for further processing.

6.1.3 Froude law of similarity

This Froude law is mainly for the open channel and spillways, which are gravity driven flow where the inertial and the gravity forces are dominating. The Froude model law is given as

$$\sqrt{\frac{F_i}{F_g}} = \sqrt{\frac{Inertial\ force\ F_i}{Gravity\ force\ F_g}} = \sqrt{\frac{\rho L^2 V^2}{\rho L^2 g}} = \sqrt{\frac{V}{gL}} = Fr_m \qquad \textbf{Eq. 6-6}$$

Some principal formulas from the Froude law that can be used for the modeling are tabulated as follows:

$$L_r = \frac{L_m}{L_p} \qquad \textbf{Eq. 6-7}$$

$$A_r = \frac{A_m}{A_p} = L_r{}^2 \qquad \textbf{Eq. 6-8}$$

$$V_r = \sqrt{L_r} \qquad \textbf{Eq. 6-9}$$

$$\frac{q_m}{q_p} = L_r{}^{1.5} \qquad \textbf{Eq. 6-10}$$

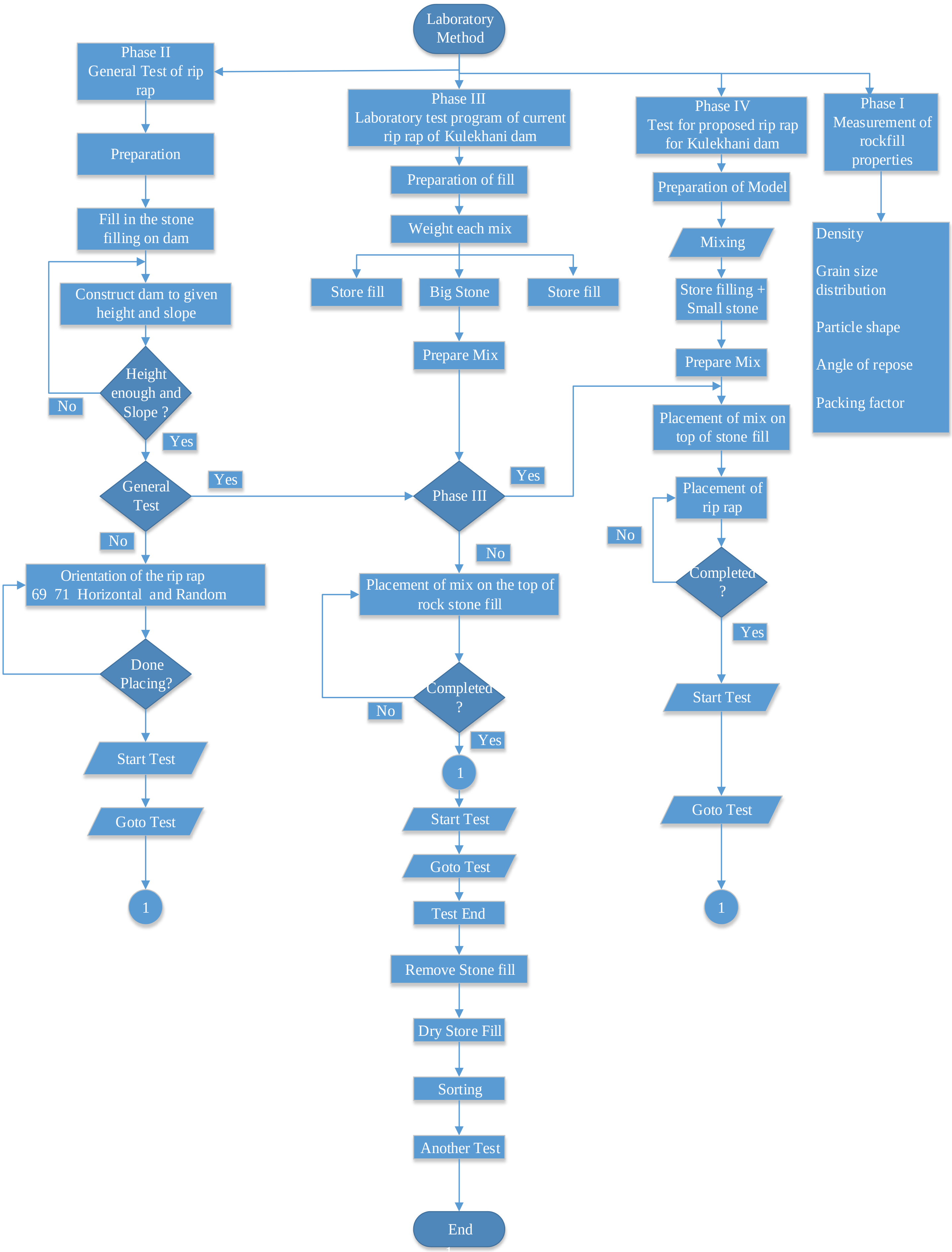
$$\frac{Q_m}{Q_p} = L_r{}^{2.5} \qquad \textbf{Eq. 6-11}$$

6.2 Laboratory test program and methodology

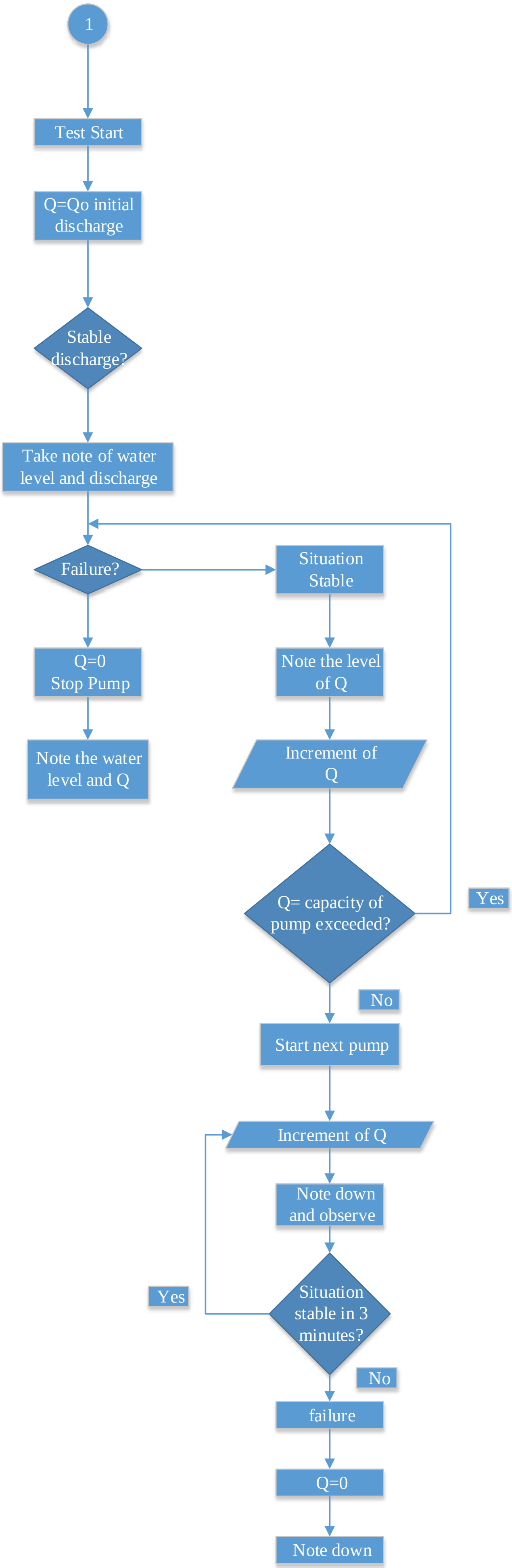
The main test program in the laboratory was divided into four different phases:

- ***Phase I*** Measurement of the rock fill properties
- ***Phase II*** General test of the rip rap of the downstream dam
- ***Phase III*** Laboratory test program for current rip rap structure of Kulekhani
- ***Phase IV*** Laboratory test program of the proposed rip rap on the downstream of Kulekhani dam

The whole test program of the laboratory can be represented in the flowchart as follows.



Laboratory Method
Phase II General Test of rip rap
Phase III Laboratory test program of current rip rap of Kulekhani dam
Phase IV Test for proposed rip rap for Kulekhani dam
Phase I Measurement of rockfill properties
Preparation
Fill in the stone filling on dam
Construct dam to given height and slope
Height enough and Slope ?
No
Yes
General Test
Yes
No
Orientation of the rip rap 69 71 Horizontal and Random
Done Placing?
Start Test
Goto Test
1
Preparation of fill
Weight each mix
Store fill
Big Stone
Store fill
Prepare Mix
Phase III
Yes
No
Placement of mix on the top of rock stone fill
Completed ?
No
Yes
1
Start Test
Goto Test
Test End
Remove Stone fill
Dry Store Fill
Sorting
Another Test
End
Preparation of Model
Mixing
Store filling + Small stone
Prepare Mix
Placement of mix on top of stone fill
Placement of rip rap
No
Completed ?
Yes
Start Test
Goto Test
1
Density
Grain size distribution
Particle shape
Angle of repose
Packing factor


1
Test Start
Q=Qo initial discharge
Stable discharge?
Take note of water level and discharge
Failure?
Situation Stable
Q=0 Stop Pump
Note the level of Q
Note the water level and Q
Increment of Q
Q= capacity of pump exceeded?
Yes
No
Start next pump
Increment of Q
Note down and observe
Yes
Situation stable in 3 minutes?
No
failure
Q=0
Note down

6.2.1 *Phase I* Measurement of properties in laboratory

Different types of materials and different colored stones were used for carrying out the laboratory tests. The different parameters like the Grain size distribution of the different stones, the density, quasi angle of repose, packing factors were measured in this phase for each type of the materials.

Density

The density measurements were carried out for each color of stones. A sample of about 30 stones were taken and each stones were weighed and then few stones were immersed in water so that the water completely covered the material and the volume of the water displaced by the stones was read. Applying Archimedes principle, the average density was found for each stones which is shown in Table 6.1

Table 6.1 Average density of the stones used

Color of stones	Average density(gm/cm^3)
Red stones	2.9955
Blue stones	2.7008
Yellow stones	2.8488
Grey stones	2.6921

Grain size distribution

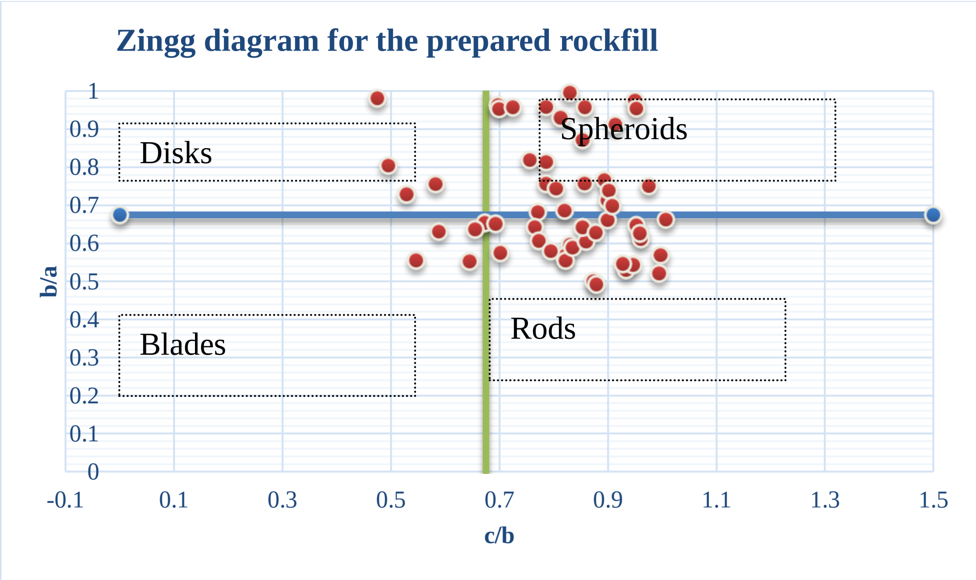
The grain size distribution of the materials with different stones were found and plotted in the Graph 10.1 to Graph 10.2 in the AppendixI. The calculations for the grain size distribution are shown in the AppendixI from Table 10.4 to Table 10.7

For the rockfill to be prepared for the Kulekhani dam. The two types of the mixture were used in the model itself. One was the stone filling available in the lab and the other was the mixture with the particle size distribution representing the particle size distribution of the Kulekhani dam. The particle size distribution of the rock fill of the Kulekhani dam contains the various ranges of the particles, which can be shown in Table 10.7 in the AppendixI. The similar particle size distribution of the prepared mixture can be shown in Table 10.8 in the AppendixI.

Particle shape

The shape of the particles can be characterized with the help of the Zingg diagram. The Zingg diagram for each of the stones was calculated. For the existing rockfill and prepared mix, the diagram is represented in Graph 6.1and Graph 2.2 in previous field study chapter.

Graph 6.1 Characterization of the particle shape for prepared rockfill



Angle of repose

The angle of repose of material is the steepest angle at which the materials remain stationary without sliding down the slope. It is not identical to the internal angle of friction. For the riprap, it varies with the inclination of the longest axis with the dam slope. Thus we named it as quasi angle of repose not totally resembling to the angle of repose.

The quasi angle of repose was determined for the riprap with various orientations. The riprap was placed in the similar way as placed during the lab tests in the box and tilted down to the angle until the stationary riprap starts to move as shown in Figure 6.1. The angle was then measured with the angle meter to give tentative measurement of the angle of repose, which is shown in the table. This method is not standardized. However, it should give good approximation of the angle of repose and variation of the angle with the orientation of the riprap with the dam axis.

Figure 6.1 Experimental setup for the angle of repose measurement

Table 6.2 Calculation of quasi angle of repose

Friction tests	Orientation	Test 1	Test 2
Stone size 41 mm (red stones):	Horizontal placement	68	76
	63 degrees	83	
	71 degrees	89	
	Random	62	57
	Dumped	43	
Stone size 31 mm (blue stones):	Horizontal placement	78	
	63 degrees	Did not test	
	71 degrees	88	
	Random	59	58
	Dumped	44	
Stone size 26 mm (yellow stones):	Horizontal placement	78	75
	63 degrees	83	
	71 degrees	85	
	Random	59	53
Prepared mix of Kulekhani dam	Random	45	

Packing factor

The packing factor is defined simply as the number of the stones in the unit area and divided by the area of the average stone. For the measurement of the packing factor, one rectangular wooden plank of 20cm x 30cm was prepared and was placed in the top of the downstream slope to give the idea of the packing factor. This is not done for all the tests carried out but done for few tests to see the variation of the packing factor in the different orientation of the stones and variation of the discharge.

The wooden plank so prepared was placed simply on the top of the stones before carrying out the experiment and then the number of stones was counted within that rectangular wooden plank. The sample of tests done can be shown in the Figure 6.2 to Figure 6.4. The results are discussed in the results and discussion chapter.

Figure 6.2 Packing factor for yellow stones

Figure 6.3 Packing factor for blue stones

Figure 6.4 Packing factor for red stones

6.2.2 *Phase II* General test program of the rip rap test downstream

The general test is based on the real field scale model and it was done together with another Master student Ole Kristian Langekar. The experiment is mainly based on the orientation of the stones with the slope of the downstream. The riprap was placed varying the angle of longest axis with the dam slope angle. The different color stones were used to analyze the effect of the scale results in different scale. Here two different hand skills were used to see how the results could vary with the people placing the riprap in the downstream slope. The slope of the downstream was taken as 1:1.5. The test results and the plan can be seen in the form of the Table 6.3.Table 6.3The whole test program can be summarized as follows and in the Figure 6.5 Arrangement of the riprap in different orientation for the experiments

- *Horizontal to the slope of the dam*
  The horizontal riprap placement of the stones with the dam slope, which is approximately at an angle of about 33.7 degree with the horizontal for the slope of 1:1.5
- *At an angle of 71 degrees*
  The placement of the rip rap at an angle of about 71 degrees with the longest axis of riprap placed 71 degrees with dam slope approximately 105 degrees with the horizontal for 1:1.5.
- *At an angle of 63 degrees*
  The placement of the rip rap at an angle of about 63 degrees with the longest axis of riprap placed 63 degrees with dam slope approximately 96 degrees with the horizontal for 1:1.5
- *Random placement of the stones*
  The placement of the rip rap is not particularly aligned at particular angle but randomly placed directly into the downstream slope. The direction of the longest axis is also not considered during the placement.

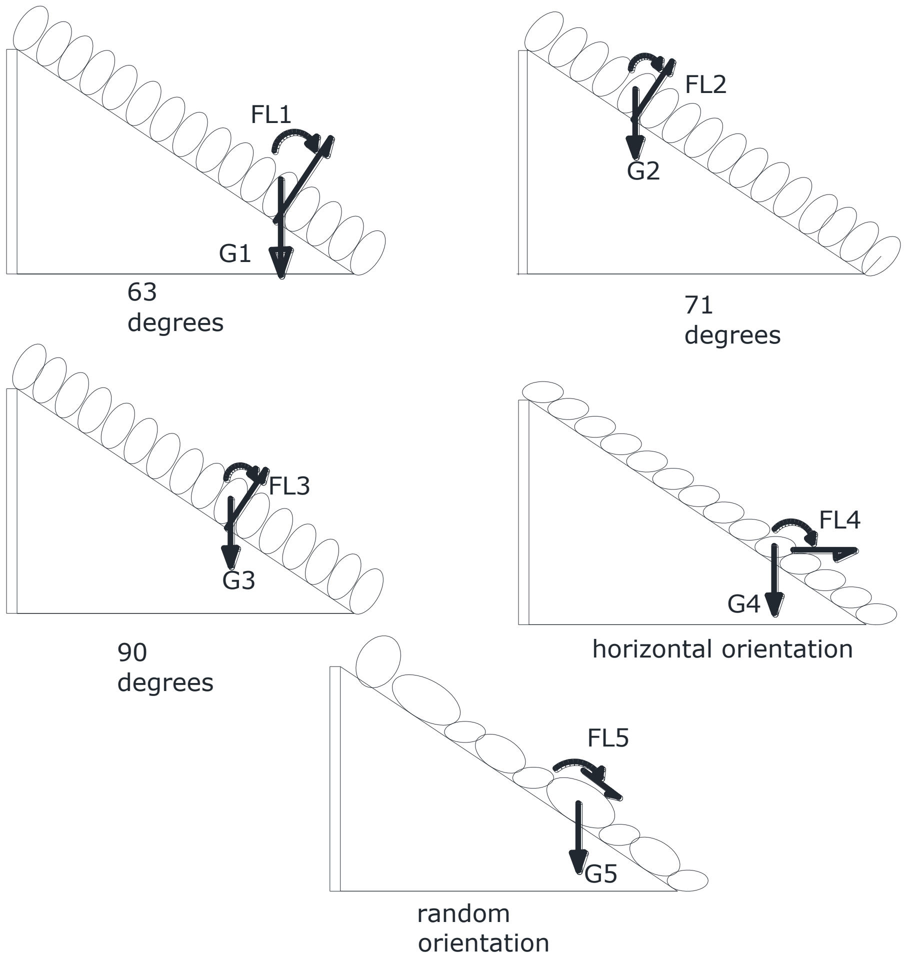


Figure 6.5 Arrangement of the riprap in different orientation for the experiments

Table 6.3 General test plan for general tests

Stone color	Stone sizes	Inclination with the dam slope			
	mm	Horizontal	71 degrees	63 degree	Random placement
Red	41	x	x	x	x
Blue	31	x	x	x	x
Yellow	26	x	x	x	x

The normal orientation with the longest axis of the rip rap was not carried out as per the field scale dam and it is also not possible to attain the exact normal orientation of the stones in the dam slope economically. In addition, the variation of the failure of the riprap introducing the roughness in the foundation and simply on the wooden plank of the flume was also studied. It can be referred from Masters Student Ole Kristian Langekar along with the variation of the discharge in the horizontal orientation. He focused more on the horizontal orientation and angle of 63 degrees with the slope. Whereas, this report is more focused in the random orientation and the proposed orientation, which is 71 degrees with the dam slope. The general tests were also made for two different slopes of the downstream. One was for 1:1.5 and another was 1:1.8. The variation of the discharges at failure observed is discussed in the section 7.4

The height of the model was varied in different tests and the scale was maintained to match the original field scale model. The variation of the discharges and the discharges at which it fails are noted. The performance of the riprap with the variation in the inclination of the longest axis of the riprap with the dam slope was noted. The results are discussed in the section 7.1.

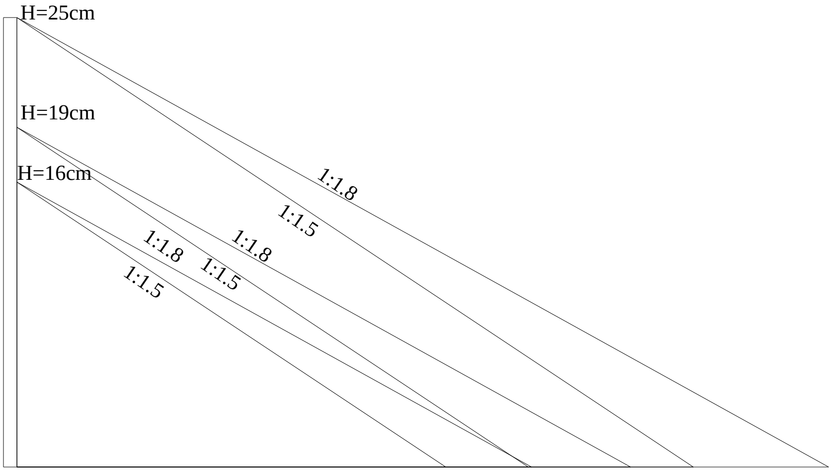


Figure 6.6 drawing of the side view of model dams

Figure 6.7 Horizontal arrangement

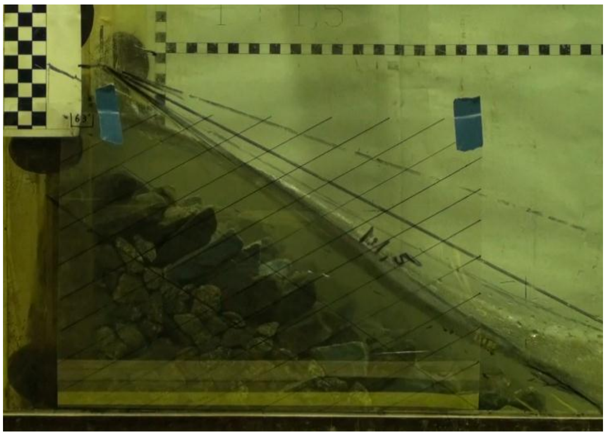


Figure 6.8 63 degrees with the dam slope

Figure 6.9 71 degree orientation

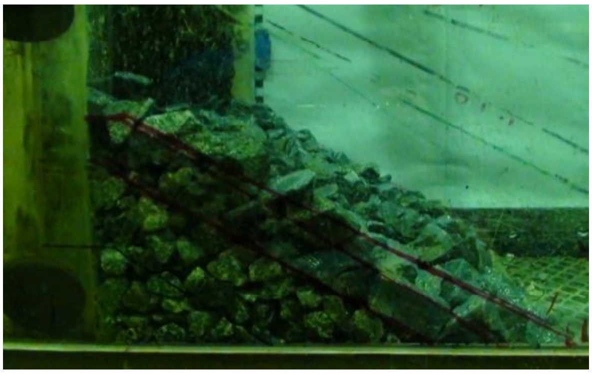


Figure 6.10 Random orientation

6.2.3 *Phase III* Laboratory test program for current rip rap structure of Kulekhani

The second part of the laboratory test is to design the test program for the current structure of the existing kulekhani rock fill dam. It is not possible to model the whole part of dam and so only downstream part of the dam is modeled for checking if the existing dam with the existing rock fill structure is able to take the throughflow and the overtopping flow during the abnormal conditions. The kulekhani rock fill dam has the downstream slope of 1:1.8, which is flatter than the general model tests that are carried out in the phase II.

The whole test program in this phase III can be shown in the Table 6.4 and be divided into following

- First test models
- Second test models
- Scale dams

Table 6.4 Laboratory test program for current structure of Kulekhani dam Phase III and IV Test program

Test number	diameter	Orientation	Height of dam(cm)	slope (1:n) n	Scale
FT1	15.84	Stone filling	20	1.8	10
FT2	15.84	Stone filling	20	1.8	10
FT3	15.84	Stone filling	20	1.8	10
FT4	15.84	Stone filling	20	1.8	10
ST1	15.84	Stone filling	20	1.8	12.5
ST2	15.84	Stone filling	20	1.8	12.5
ST3	15.84	Stone filling	20	1.8	12.5
SC1.8a	14.4	Random prepared	25	1.8	12.5
SC1.8b	14.4	Random prepared	25	1.8	12.5
SC1.8c	14.4	Random prepared	25	1.8	12.5
SC1.8d	14.4	Random prepared	25	1.8	12.5
SC1.5a	14.4	Random prepared	25	1.5	12.5
SC1.5b	14.4	Random prepared	25	1.5	12.5
SC1.5c	14.4	Random prepared	25	1.5	12.5
SC1.5d	14.4	Random prepared	25	1.5	12.5
RY1b	26	Random	16	1.80	19.7
RY2b	26	Random	16	1.80	19.7
RY3b	26	Random	16	1.80	19.7
RR1b	41	Random	25	1.80	12.5
RR2b	41	Random	25	1.80	12.5
RR3b	41	Random	25	1.80	12.5
RR4b	41	Random	25	1.80	12.5
RB1b	31	Random	19	1.80	16.5
RB2b	31	Random	19	1.80	16.5
RB3b	31	Random	19	1.80	16.5
RB4b	31	Random	19	1.80	16.5
71B1b	31	71 degree	19	1.80	16.5
71B2b	31	71 degree	19	1.80	16.5
71B3b	31	71 degree	19	1.80	16.5
71B4b	31	71 degree	19	1.80	16.5
71B5b	31	71 degree	19	1.80	16.5
71R1b	41	71 degree	25	1.8	12.5
71R2b	41	71 degree	25	1.8	12.5
71R3b	41	71 degree	25	1.8	12.5
71R4b	41	71 degree	25	1.8	12.5
71Y1b	26	71 degrees	16	1.5	19.7
71Y2b	26	71 degrees	16	1.5	19.7
71Y3b	26	71 degrees	16	1.5	19.7
71B1a	31	71 degrees	19	1.5	16.5
71B2a	31	71 degrees	19	1.5	16.5
71B3a	31	71 degrees	19	1.5	16.5
71R1a	41	71 degrees	25	1.5	12.5
71Y1a	26	71 degrees	16	1.5	19.7
71Y2a	26	71 degrees	16	1.5	19.7

71Y3a	26	71 degrees	16	1.5	19.7
RY1a	26	Random	16	1.50	19.7
RY2a	26	Random	16	1.50	19.7
RB1a	31	Random	19	1.50	16.5
RB2a	31	Random	19	1.50	16.5
RR1a	41	Random	25	1.50	12.5
RR2a	41	Random	25	1.50	12.5
71R1a	41	71 degree	25	1.50	12.5
71B1a	31	71 degrees	19	1.50	16.5
71B2a	31	71 degrees	19	1.50	16.5
71B3a	31	71 degrees	19	1.50	16.5
71Y1a	26	71 degrees	16	1.50	19.7
71Y2a	26	71 degrees	16	1.50	19.7
71Y3a	26	71 degrees	16	1.50	19.7

- ***First test models***

The first four tests on the rock fill dams were not much extensively monitored as they were used for the examination of the flume operation. The gradation was not proper and the stone fillings were only used as the material for the rock fill .The material i.e. the stone filling used in this test was poorly graded .The compaction was done by the bucket compaction and hand placing method of the rockfill manually and the model was in the scale of 1:10. The height of the model was 20cm. They are named as FT1, FT2, FT3, etc.

The hand placement method enables the high void ratio and increases the potential of flow through failure. Slump and the movement of the rock from the dam crest in the downstream face when the discharge exceeded 6l/s. Most failure of the dam was by the movement of the rock in the downstream face resulting in the slumping of the dam crest. As the result of the overtopping, the flow channel through the section of the dam resulted. The discharge at failure was in between 6 and 7. The first movement occurred carried away few stones and tried to be unstable and after that again it became stable for certain discharges and then failure occur at higher discharge.

- ***Second test models***

In this series of tests also the poorly graded rock fill materials were used as the rockfill materials and the compaction was done by both hand and bucket placement method. The same mass of the rock were used approximately. This ensures more or less the same void ratio in the model. The model is constructed in the scale 1:12.5 with the downstream slope of 1:1.8. They are named as ST1, ST2, ST3, etc.

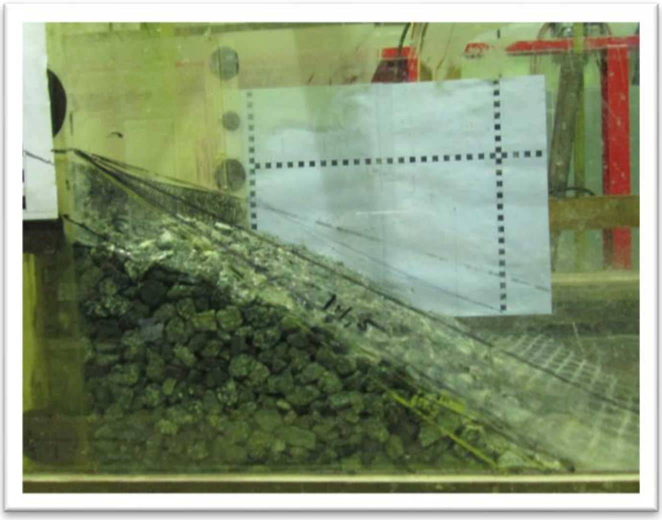


Figure 6.11 Second test model side view

Figure 6.12 Second test model front view

- ***The first movement***

The first observed movement that started from the toe by sliding few stones from the toe causing it instable and then slided down.

- ***Second movement***

The toe sliding caused the change in the toe slope. Later on in increasing the discharge, the surface movement of the rock fills occurred from the crest of the dam

- ***Third movement***

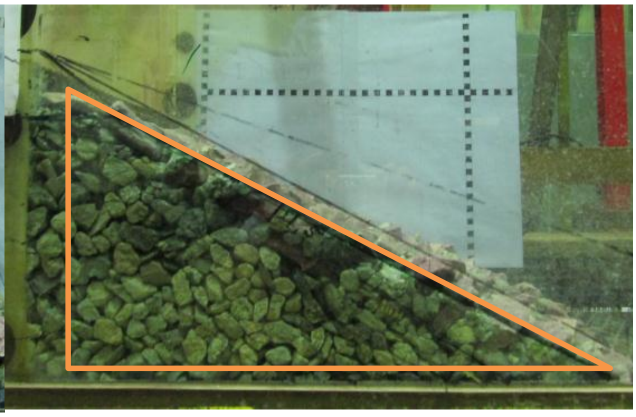
In the final stage of the movement, the dam attained the flatter slope at the downstream, which gives the erosional process of movement of the stones for increasing discharge. The movements resulted in the decrease in the height of the downstream crest. The decrease in the height of the dam induces more overtopping.

- ***Scale dams and real prototype***

For modeling the real field scale dam, it is tried to resemble with the real field dam. The methodology is more described more in detail below in the section below. Two relatively models of the same scale differing only by the downstream slope were constructed to know the difference in the behavior of the rock fill. The slope of one rock fill was 1:1.8 and that of another was 1:1.5.

Most of the general arrangement of the rip rap was carried out in the slope of 1:1.5. The riprap is arranged at different angle with the dam axis for the same size and scale of the stones. The test is made possible for the arrangement and the best arrangement was carried out for the final test of the given thesis topic dam .This scale test is carried out to evaluate the flow conditions, the failure conditions, the mode of failure and the comparative study of the rock fill dam and rip rap performance.

Front view of 1:1.8 scale dam



Side view of 1:1.5 scale dam

Front view of 1:1.5 scale dam

Side view of 1:1.5 scale dam

Figure 6.13 Scale model dams

Relevant number of tests was carried out. The scale dams are named as ‘SC1a’ and ‘SC1b’where SC represents the scaled test dams, ‘a’ and ‘b’ represents the slope of downstream and ‘a’ for 1.5 and ‘b’ for 1.8 and 1, 2, 3, etc. represents the number of tests for the ease representation. The overall process of sieving and drying and the arrangement took quite long time than expected.

- ***Modeling representative with the real prototype***

As discussed previously as well, the real scale model was prepared in the lab. The downstream was modeled with the scale of 12.5. The existing kulekhani dam has the randomized riprap and the particle sized distribution curve used in the randomized riprap contains the varying size of the particle given by Table 10.8. Therefore the model has to be made complying with the existing structure of the rock fill of Kulekhani dam. For the complying the main dam with the model dam following criteria are used for the similarity and physical modeling.

  - ***Particle size distribution curve preparation***

For the model preparation of the particle size distribution for the existing dam, two types of the rock mixture were used for the modeling of the rock fill dam. One mixture is the stone fillings, which are the general stone fillings, which are used in the general tests as well. In addition to it, there are other mixtures for representing the existing rock fill of the Kulekhani dam. For this rockfill, the mixture was prepared from the three different types of the rocks of different diameters by sieving. The particles sizes are as follows

- stone fillings of diameter 15mm that is used as the stone filling inside the rock fill dam
- the stones sizes of about 4mm
- the stones sizes of diameter 41mm in the model

In the Figure 6.14, the red line represents the minimum diameter present in the rock fill, dark blue line represents the maximum diameter of the rock fill and the light blue represents the average rock fill that is prepared for the laboratory tests.

These stones of varying diameters were mixed and proportioned as Table 10.8 Particle sized distribution for the rockfill used in the laboratory tests. This mixture was placed in the stone fill for representing the random riprap. The particle sized distribution curve of the prepared rock fill has the range of diameters and the model was created with the average size of the rock fills such that the distribution of the rock fills lies within the curve of maximum and minimum diameters. The amount of these stones is proportioned to give the similar PSD curve as shown in the Figure 6.14.

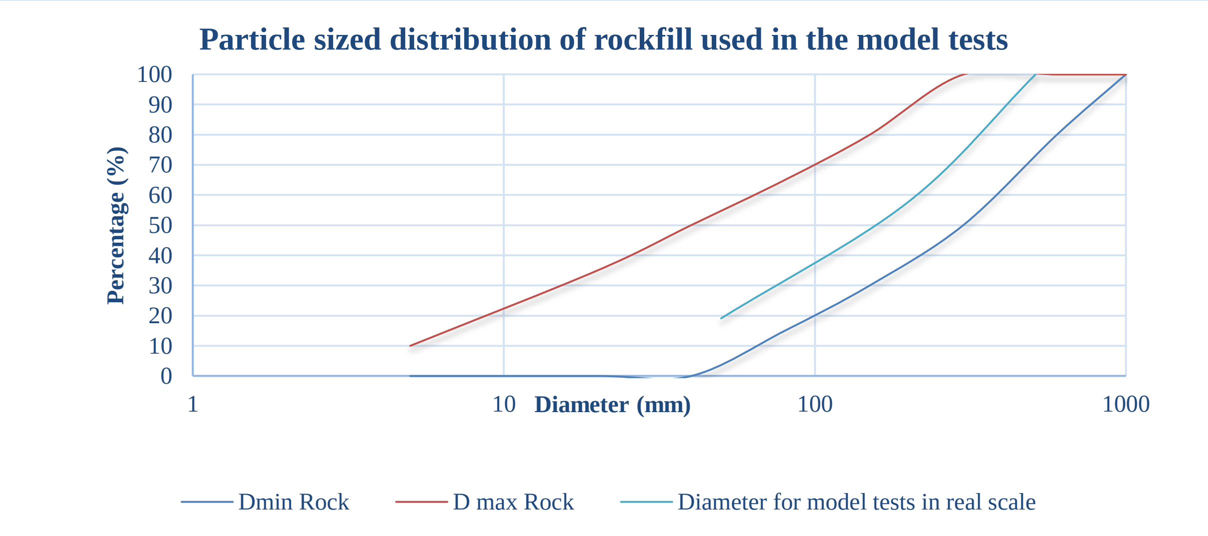


Figure 6.14 Particle sized distribution of rockfill used in the model tests and real rockfill

The tests were done with this prepared rock fill. Similar tests were done for the scale dam tests. It was not possible to do number of tests in the real field scale as it had long procedure. For each test, amount of the rock fill for the randomized layer was weighed, and proportioned according to the PSD curve and placed above the stone fillings. The drying of the stones was done for weighing each time except bigger stones and the stones are sorted out after each time for making the new model again. The dam has to be built and proportioned for each test. Thus it is time consuming so few but relevant tests were done to see if the existing rock fill is able to sustain the exceptional load of overtopping and throughflow during the extreme conditions.

After the test, the same materials were used for the next test to make the scale dam tests with the same type rockfill, same height but the difference was with the slope of the dam. This was done to see if the difference in the behavior and the discharge that it could take under the throughflow and overtopping condition. The test results are present in the Graph 7.18 in results and discussion chapter. The instrumentation was done with the help of the video camera and the photos taken from the model. The rockfill in the real prototype is shown in the Figure 6.15 and Figure 6.17 and the test model dam is also shown in the Figure 6.16and Figure 6.18.

- ***Slope of the dam***

The slope of the downstream of the dam in the real rock fill dam 1:1.8 was used in the model for carrying out the test. For the scale dam test, the slope of 1:1.5 was also used to observe the result and the behavior of the dam in that slope. This is done for the interpretation of the general test with the tests obtained with the real model scale.

- ***Scale***

The scaling is done with the help of the stone size available in the laboratory. In the first test model described earlier used the scale of 1:10 is used and in the later second test models all the scale used is 1:12.5. After the proposed rip rap is placed in the real scale dam, firstly the scale of 12.5 is tested and again in the second phase due to the limitation of the pump capacity the 1:16.5 is chosen.

Figure 6.15 Existing rock fill of prototype Kulekhani of downstream slope

Figure 6.16 Scaled Model of the prototype dam

Figure 6.17 Existing rock fill of prototype Kulekhani of downstream slope

Figure 6.18 Test model dam of the prototype

6.2.4 *Phase IV* Laboratory testing of the proposed rip rap on the downstream of Kulekhani dam

From the laboratory tests, it showed that the existing rockfill is not able to withstand the overtopping condition of PMF which shows that it is not safe during when both of the gates are closed due to the mechanical failure. The protection measures are to be taken for preventing the dam against such overtopping load.

Therefore the new orientation of the riprap model is proposed for the existing dam with the proper orientation of the rip rap design. The models are represented as RR1a, RR1b, 71R1a, 71R1b, etc. where first R and 71 represents the orientation of the stones either random or 71 degrees. Second R, B, Y represents the stones color of different diameter d50, R for red of 41mm, B for blue of 31mm and Y for yellow of 26mm. And 'a' and 'b' represents slope of 1.5 and 1.8 and 1, 2, 3, etc. represents the number of different tests. The orientation of the proposed rip rap is based on the previous general tests made discussed in section 6.2.2 and several tests made earlier years with the slope of 1:1.5. It is assumed that the cases safe for the slope of 1:1.5 is also safer for the slope of 1:1.8, which is comparatively flatter than the former one. However the tests were carried out for both the slope with the same orientation as the scale dam tests.

During the preparation of the model in this phase, the rock fill was proportioned as same as the existing dam. The bigger stones, which were used in the proportioning of the dam, were removed from the mix and they were used as the rip rap. The design is done for the rip rap. The design of the rip rap as mentioned in the earlier chapter shows the dimensions of the rip rap to be used. In the model, the small stones are placed as it is in the real case. This is because practically it will be difficult to remove all the stones and replace new one all and the bigger stones were proposed to be used as the riprap. In the real case, bigger stones than the model can also be found that will give safer dam.

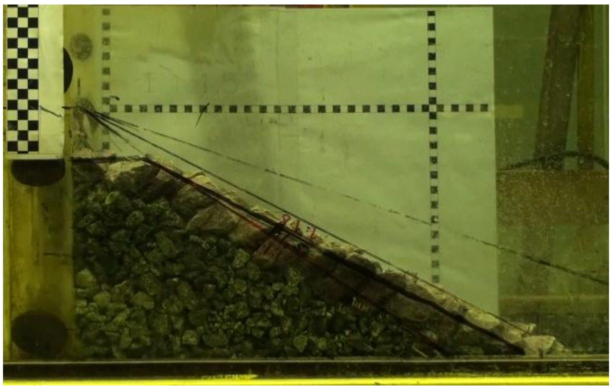


Figure 6.19 71 degree orientation for red stones

Figure 6.21 71 degree orientation for blue stones

Figure 6.20 71 degree orientation for the yellow stones

During the modeling, only one type of available stones was used to ensure the dam to be safe even at the minimum size of stones available there. The orientation of the riprap was placed at 71 degrees with the slope of the dam with few degrees more or less. This orientation is based on the results of the general tests used and performance of the riprap under different orientation. The test was carried out for various scales as shown in Table 6.4 above. Thus during the test, the scale of the test was changed from 12.5 to 16.5 due to the limited capacity of the pump and the inability to visualize the whole collapse flow. Overall discharge at which dam failed was noted and the strength of the existing rip rap was noted. The results are presented in the next results and discussion chapter.

The additional tests were also carried out for the detail study of the parameters influencing the rip rap performance. The later tests were more focused on the random and the 71 degree orientation. The height of the dam is also varied for different sizes of stones.

6.3 Instrumentation

The scale dam instrumentation was done to compare and relate the general topic with the specific topic. This also gives the overall picture how the change in the downstream slope influences in the failure of the dam and its performance in the failure. In order to know the variation of the upstream level, the measurement scale was fixed at the upstream. Two video cameras were put on the both the side and front of the model to capture the movement of the stones and the process of the failure and the initiation. The video cameras also helped to note the failure at the time when one is not able to note down the failure mode during the increasing of discharge from the pump.

During the construction of the model, the particle size distribution of the rock fill in the real scale is to be matched with the particle size distribution of the model scale. The proportion of each stones is weighted during the initial building of the rock fill layer in the rock fill dam. The inner layer of the rock fill was built from the same stone fillings, which were done in the previous tests. The stone fillings are taken out each time, dried and sorted out to get same void ratio of the initial test. The drying was not done for the bigger stones. The approximately more or less the same amount of the rockfill was used as well. From the scale test models, it is possible to know how much flow is needed to fail the dam downstream.

6.4 Flume

All the experiments were carried out in D yellow flume in NTNU laboratory shown in Figure 6.22. The flume is 12 m long, 0.61m wide and 0.75 m high. It is located in the Vassdrag Laboratory of Norwegian Institute of Science and technology (NTNU) in Trondheim. The flume is made of the ply wood and supported by the steel framework. The framework is covered with the glass for the visibility on both sides of the flume for the central 6 m.

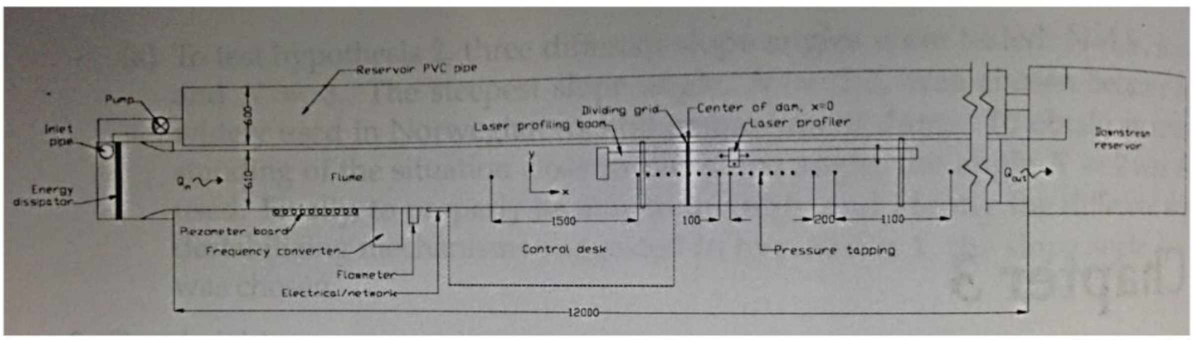
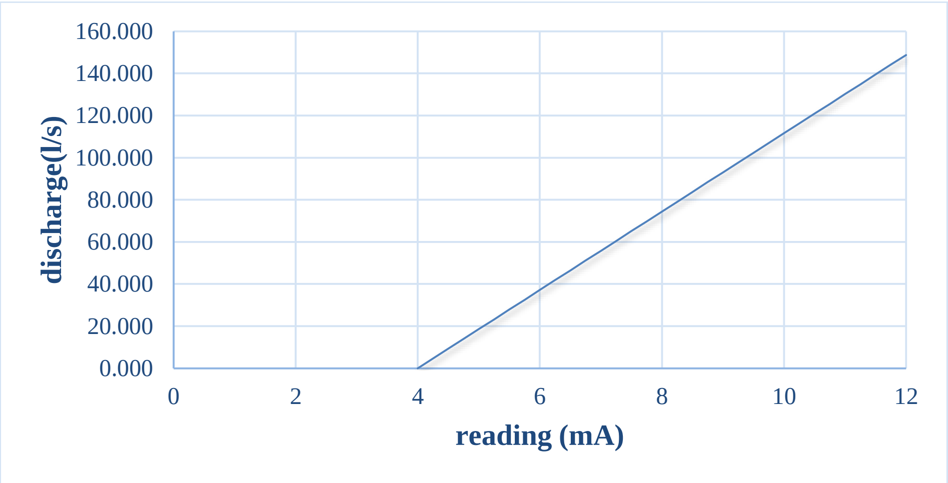


Figure 6.22 Drawing of the D flume from above[50]

It consists of two pumps one is of capacity of 40l/s and other is of 200l/s. But the bigger pump cannot be increased more than 130l/s. The pump is the closed water circulation system. The water is pumped from the small reservoir at the end of the flume which receives the same and collected water resulting from the experiment.

Figure 6.23 D flume in the laboratory

The reservoir is also used for collected the stones and the rock fill that flow with the water during the movement of stones and failure of dam. The downstream reservoir is made by the ply wood box. The far end of the pipe which carries the water is mounted with the pump which is managed by the feedback control system. The flume is easily accessed by the help of the wooden framework steps. The control of the bigger pump is given by the following Graph 6.2.



Graph 6.2 Graph for the bigger pump in D flume

6.5 The time of discharge

During running of the tests, the discharge was increased at certain interval of time. It is necessary to have the uniform flow on the model during the experiment, therefore, the discharge was increased and let it stay stable for certain interval of time. The time was taken approximately about 3 to 4 minutes for each increment of the discharge. The change in the discharge was made more or less constant. But for the rip rap, the discharge was changed in a great interval at the initial time and later when it approached to failure, the increment value in the discharge is made small. The influence of the duration of the discharge is not taken into the consideration and the study of this effect of change in time is not included in this report.

Chapter 7

7 Results and Discussions

The results and the discussions of the test program which is carried out in several phases are discussed as below.

7.1 Drainage capacity and hydraulic computation

For the calculations, the hydrological data and other relevant data are obtained from the design report of Kulekhani Hydropower Project. All the calculations are shown in the previous computation chapter. The main results are as follows.

Height of overtopping from dam when the two gates are closed	=	1.377	m
The discharge that is passed through the spillway	=	1541.975	m^3/s
The discharge total outflow during the probable maximum flood (PMF)		2540	m^3/s
From the flood routing			
The discharge that overflows through the dam	=	998.024	m^3/s
The total discharge over the top of the dam	=	998.024	m^3/s
Crest length of the dam	=	397	m
q over the dam	=	2.514	$m^3/s/m$

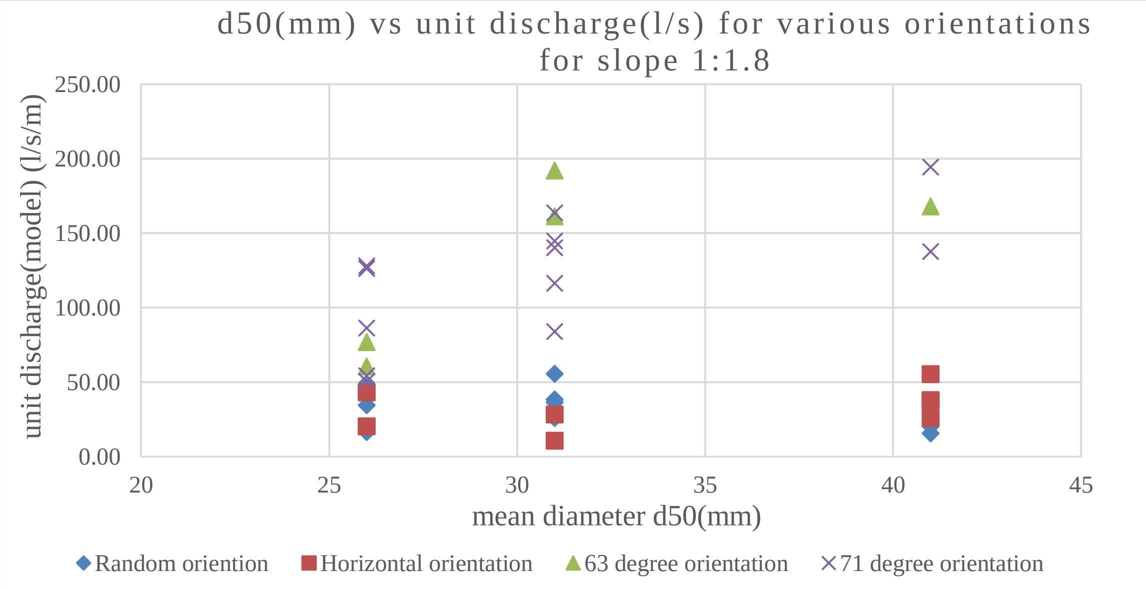
The height of the outflow from the dam is 1439.383masl (meters above the sea level) which is about 9.383m from the bottom of the dam.

The value is obtained for the two gates being closed. This is just one case of overtopping. There might be several other conditions of failure besides the mechanical failure. There might be the settlement of the dam, or landslides or seismic activity. Nepal is also earthquake prone zone and due consideration should be paid to this present context which might cause the overtopping condition. Those conditions need to be studied carefully and those lack the relevant geotechnical and other required data so the existing condition is assumed.

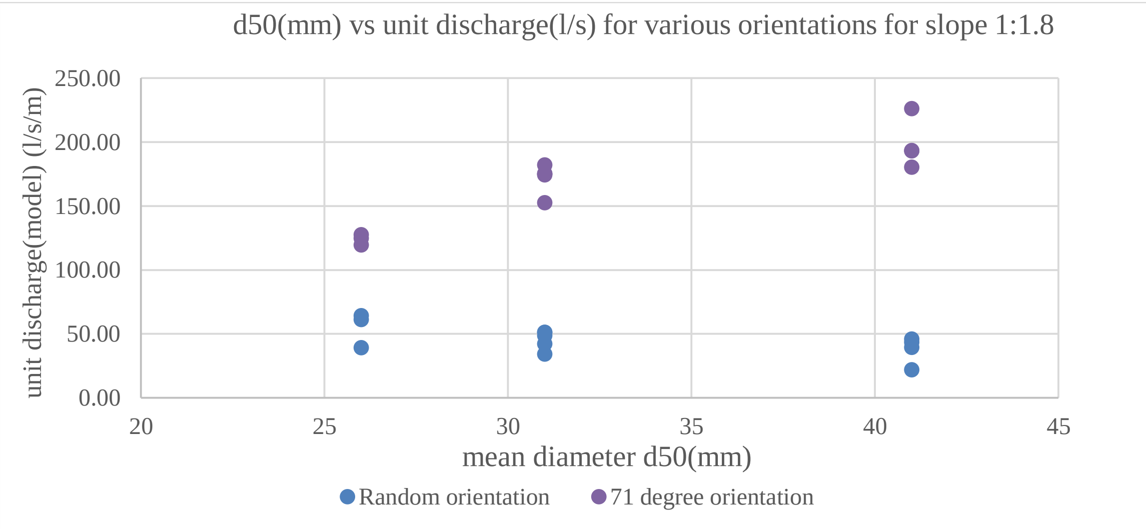
During the calculation, the discharge capacity of the side channel which acts as the uncontrolled spillway is found to be 1.538 which is comparatively low. The side channel spillway has a bad coefficient of discharge because the water instead of passing directly to the downstream, it is first submerged and then carried out downstream.

7.2 General tests

From the general tests done as described in the methodology, the discharge at which the dam breaks is greatly varied with the change in the orientation and the arrangement of the riprap which is shown in the Graph 7.1. The random placement of the riprap gives poor results in compare to the other arrangement of the stones. The results of the different orientation as discussed in the methodology are tabulated in the Table 7.1. In the later phase more tests were done on the orientation of 71 degree and the random orientation more on the slope of 1:1.5 and the slope 1:1.8.



Graph 7.1 Unit failure discharge (l/s/m) vs. mean diameter d50 (mm) the slope of 1:1.5



Graph 7.2 Unit failure discharge (l/s/m) vs. mean diameter d50 (mm) the slope of 1:1.8

The general results with the variation on the orientation can be seen in the Graph 7.1 and Graph 7.2 above. This is observed for both the slopes. From Table 7.2 also, it can be seen that the same rip rap if placed in the greater angle with the dam slope, the strength is increased to great extent.

Table 7.1 Tabulated results of the general test (combined from my tests and Ole kristian)

SN	Diameter	Date	Orientation	angle of repose	slope	scale	Q start l/s	Q collapse	ht cm	lm (cm)
71R1G	41	21.02.2013	71 degree	89	1.5	1:12.5	84.00	120.70	25	61
63R1G	41	25.02.2013	63 degree	83	1.50	1:12.5	102.40	106.00	25	61
RR1G	41	01.03.2013	Random	59.5	1.5	1:12.5	9.50	14.20	25	61
HR2G	41		Horizontal	76	1.50	1:12.5	23.10	23.10	23	61
HR3G	41		Horizontal	76	1.50	1:12.5	23.00	23.00	22	61
HR4G	41	21/22.02.2013	Horizontal	76	1.50	1:12.5	33.70	33.70	25	61
HR5G	41		Horizontal	76	1.50	1:12.5	15.80	15.80	40	61
71Y1G	26	12.03.2013	71 degrees	85	1.50	1:19.712	30.70	48.30	16	61
71Y2G	26	12.03.2013	71 degrees	85	1.50	1:19.712	33.16	33.16	16	61
63Y1G	26	11.03.2013	63 degree	83	1.50	1:19.712	46.80	52.60	16	61
63Y2G	26	11.03.2013	63 degree	83	1.50	1:19.712	36.85	36.85	16	61
HY1G	26	08.03.2013	Horizontal	76.5	1.50	1:19.712	26.30	26.30	16	61
HY2G	26	08.03.2013	Horizontal	76.5	1.50	1:19.712	12.35	12.35	16	61
RY1G	26	13.03.2013	Random	56	1.50	1:19.712	10.00	10.00	16	61
RY2G	26	13.03.2013	Random	56	1.50	1:19.712	21.00	21.00	16	61
63B1G	31	05.03.2013	63 degree	83	1.50	1:16.5	98.40	98.40	19	61
63B2G	31	05.03.2013	63 degree	83	1.50	1:16.5	117.05	Pump capacity	19	61
71B1G	31	06.03.2013	71 degrees	88	1.50	1:16.5	99.80	99.80	19	61
71B2G	31	06.03.2013	71 degrees	88	1.50	1:16.5	88.32	88.32	19	61
RB1G	31	07.03.2013	Random	58.5	1.5	1:16.5	22.26	22.26	19.5	61
RB2G	31	07.03.2013	Random	58.5	1.5	1:16.5	15.75	15.75	19.5	61
HB1G	31	04.03.2013	Horizontal	78	1.50	1:16.5	17.20	40.90	19	61
HB2G	31	04.03.2013	Horizontal	78	1.50	1:16.5	6.47	6.47	19	61

Table 7.2 Relative improvement with different orientation

	H=25cm d50=41mm		H=19cm d50=31mm		H=16cm d50=26mm	
Placement	Mean Failure discharge (l/s/m)	Relative improvement	Mean Failure discharge (l/s/m)	Relative improvement	Mean Failure discharge (l/s/m)	Relative improvement
Random	7404.4	1.0	26353.2	1.0	36879.3	1.0
71 degree	56568.5	7.6	119132.5	4.5	72044.6	2.0
63 degree	55937.7	7.6	87545.1	3.3	93553.4	2.5
horizontal	12733.4	1.7	13088.1	0.5	33287.8	0.9

7.3 Results from the tests of the Kulekhani dam

From the methodology described in the phase III, from the model, several model dam tests were run. The tests from the results can be tabulated in the Table 7.3.

Table 7.3 Results from the test dam from prototype

Description	Tests name	Dia d50 (cm)	ht (cm)	slope (1:1.8)	scale	Q f (l/s)	qf model (l/s/m)	qf Proto Type (l/s/m)
Preliminary test dams	FT1	15.84	20	1.8	10.00	7.49	12.28	388.3
	FT2	15.84	20	1.8	10.00	6.14	10.07	318.3
	FT3	15.84	20	1.8	10.00	7.27	11.92	376.9
	FT4	15.84	20	1.8	10.00	6.07	9.95	314.7
Stone fillings	ST1	15.84	20	1.8	12.50	6.57	10.77	476.0
	ST2	15.84	20	1.8	12.50	6.30	10.33	456.4
	ST3	15.84	20	1.8	12.50	6.38	10.46	462.2
actual random rip rap 1.8	SC1.8	14.40	25	1.8	12.50	11.50	18.85	833.1
	SC1.8	14.40	25	1.8	12.50	12.07	19.79	874.5
	SC1.8	14.40	25	1.8	12.50	10.48	17.18	759.3
	SC1.8	14.40	25	1.8	12.50	9.25	15.16	670.2
actual random rip rap 1.5	SC1.5	14.40	25	1.5	12.50	8.27	13.56	599.2
	SC1.5	14.40	25	1.5	12.50	8.87	14.54	642.6
	SC1.5	14.40	25	1.5	12.50	7.72	12.66	559.3
	SC1.5	14.40	25	1.5	12.50	9.04	14.82	654.9

The mean value and the standard deviation are observed in the Table 7.4. The value obtained is the existing drainage capacity of the dam. The mean value shows that the average value of the drainage capacity of the dam is 0.784m3/s/m and relative standard deviation is 11%. The same type of void ratio and the dam conditions is tried to be maintained for the consistency in the model results. Results also show that the discharge capacity of the dam of the slope 1:1.8 is 27% more than that with the slope of 1:1.5.

Table 7.4 Statiscal values for the Kulekhani dam

Statistical value of scale dam	Mean unit failure discharge l/s/m	Mean unit failure discharge (m3/s/m)	standard deviation (l/s/m)	Relative standard deviation%	Relative improvment
for slope 1:1.5	614.0	0.614	43.6	7	1
for slope 1:1.8	784.3	0.784	89.8	11	1.276

The dam will be safe if the existing dam model can withstand the unit outflow discharge from the dam during the overtopping condition. The ouflow discharge equivalent to the Froude's law calculation is shown below.

For the scale **1: 12.5**

For the prototype

The maximum discharge outflow	=	0.899	m^3/s /m
Length of the section	=	20.00	m
For the 20 m strip section	=	17.99	m^3/s
Outflow discharge at prototype for 20m	=	17.99	m^3/s

Scale **1: 12.5**

For the model

L_r	=	0.080	
L_m	=	LpxLr	
L_m	=	1.6	m

Froude's law $Q_m=Q_p*L_r^{2.5}$

For the model of 1.6 m	=	0.0203	m^3/s /m
$Q_m=Q_p*L_r^{2.5}$			
Q_m	=	0.033	m^3/s
For 1m width			
Q_m	=	0.020	m^3/s /m
Width of the flume	=	610	mm

For the model of 610 mm in the model

Q_m	=	0.012	m^3/s
	=	12.42	l/s

The failure may occur at this discharge in the model when the rip rap is randomly placed.

For the scale **1: 10**

For the prototype

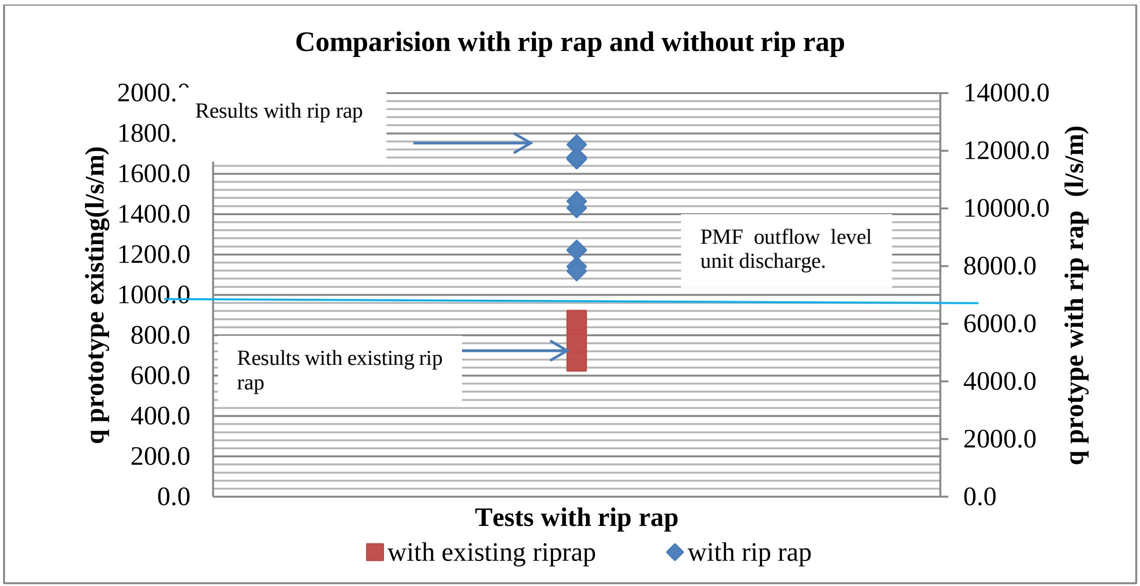
The maximum discharge outflow	=	0.90	m^3/s /m
Length of the section	=	20.00	m
For the 20 m strip section	=	17.99	m^3/s
Q_p	=	17.99	m^3/s
Height of the model	=	25.00	cm

The results after placing the rip rap with proper orientation in the existing Kulekhani dam are shown in the Table 7.5. Graph 7.3 shows the comparative results of the performance of the dams with the existing condition of the rip rap and properly oriented riprap under the overtopping condition of the probable maximum flood.

Table 7.5 Results after the placement of the riprap

Description	Tests name	Dia d50(cm)	ht (cm)	slope (1:1.8)	scale	q f (l/s)	qf (model) (l/s/m)	qf (prototype) (l/s/m)
proposed rip rap with 71 degree orientation	71R1b	41	25	1.8	12.50	138.10	226.40	10005.4
	71R2b	41	25	1.8	12.50	110.05	180.41	7973.1
	71R3b	41	25	1.8	12.50	118.1	193.61	8556.3
	71R4b	41	25	1.8	12.50	117.88	193.25	8540.3
	71R5b	41	25	1.8	12.50	107.89	176.85	7815.8

	71B1b	31	19	1.8	16.50	111.10	182.13	12207.0
	71B2b	31	19	1.8	16.50	106.46	174.52	11697.2
	71B3b	31	19	1.8	16.50	106.85	175.16	11740.1
	71B4b	31	19	1.8	16.50	93.2	152.79	10240.3
	71B5b	31	19	1.8	16.50	107.10	175.59	11768.3
					Mean value			**10054.4**
					Standard deviation value			**1732.97**



Graph 7.3 Comparison of the results with oriented riprap and with existing rip rap

7.3.1 Discussions

From the test results it shows that the proposed riprap in the Kulekhani dam if built in the proposed orientation, the drainage capacity of the existing dam will be increased to about 12 times shown in Table 7.6 and the dam will be safer to any susceptible conditions of the overtopping in any cases.

Table 7.6 Comparative strength improvement

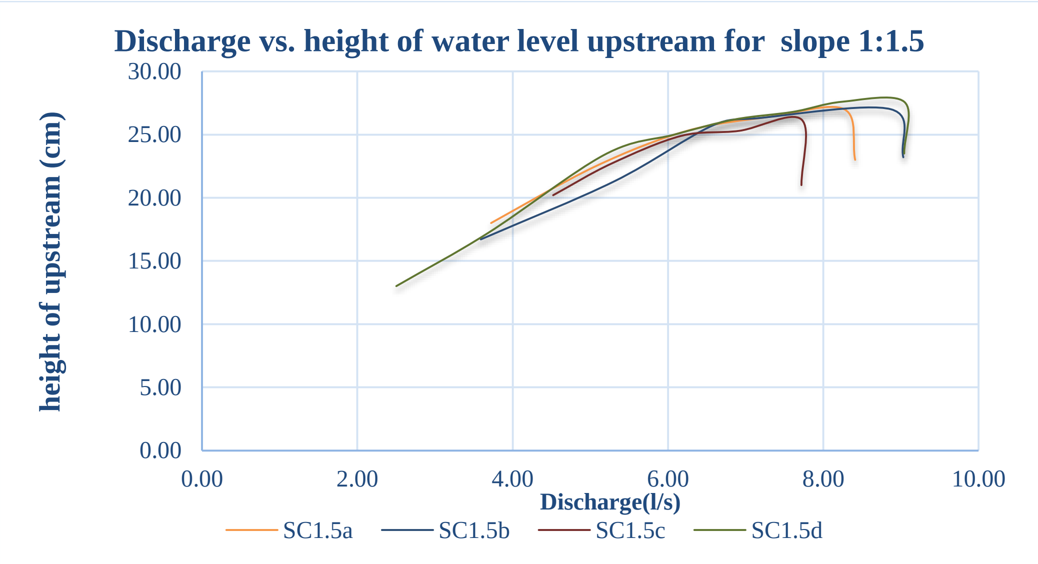
Rip rap	Drainage capacity	Unit	Improvement
Existing riprap	784.3	l/s/m	1
Proposed riprap	10054.4	l/s/m	12.82

The condition of the dam during each test is tried to be maintained in similar condition and same void ratio and permeability. The height of the water level upstream is noted at each test along with the discharge. Thus, the Graph 7.4 and

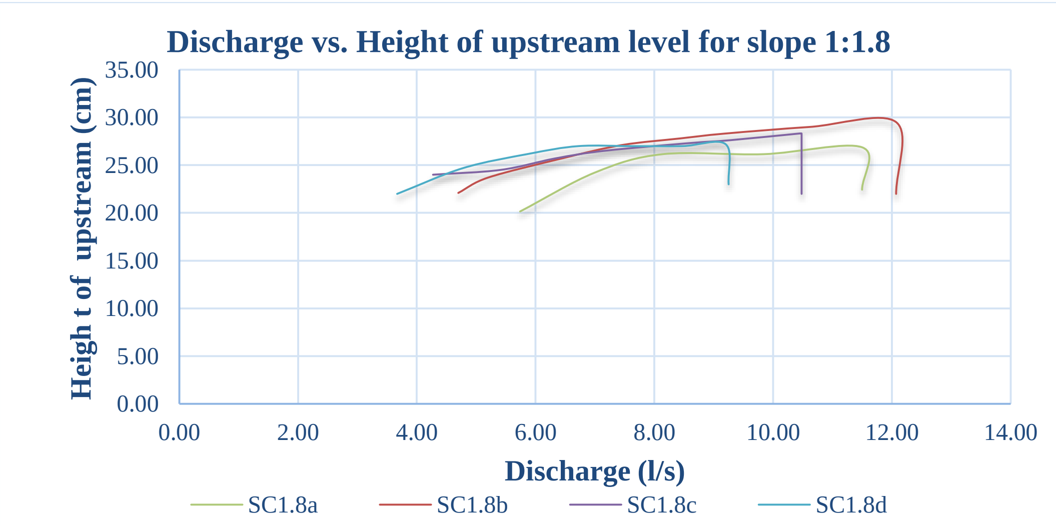
Graph 7.5 represents the variation in the water level in each tests carried out for the Kulekhani dam. The tests are tried to make it more or less similar.

Besides that the field visit, which was made during the summer, helped a quite lot to know the general information and the existing site condition of the Kulekhani dam. It also helped to visualize the dam condition from the existing report. During the placement it was not easy to place the stones as per site condition but efforts were made to do it as per the site condition by mtching the rockfill properties. The measurement carried out in the fieldwork is used as the reference to see the existing site condition and building the orientation of the stones in the test dam.

The sizing of the rip rap stones proposed on the Kulekhani dam is based on the NVE design method. Here only one type of stone was used for the riprap placement but in the real case, several stones will be available. However, the available bigger stones in the field site are proposed to be used for the placement of the riprap. The riprap in the real case if built by use of the available stones, it will give more safe result than the predicted result as the existing dam contains several bigger stones as well.



Graph 7.4 Discharge vs. upstream water level for slope 1:1.5



Graph 7.5 Discharge vs. upstream water level for slope 1:1.8

7.4 Results from the general and additional tests

The general tests were made at initial phase in which several orientations were focused. For the study of the variation of the parameters in the rip rap, the placement was focused more on the random orientation and the 71 degree orientation. The several test results done are tabulated in the Table 7.7 and studied in each heading.

Table 7.7 Results of the laboratory tests

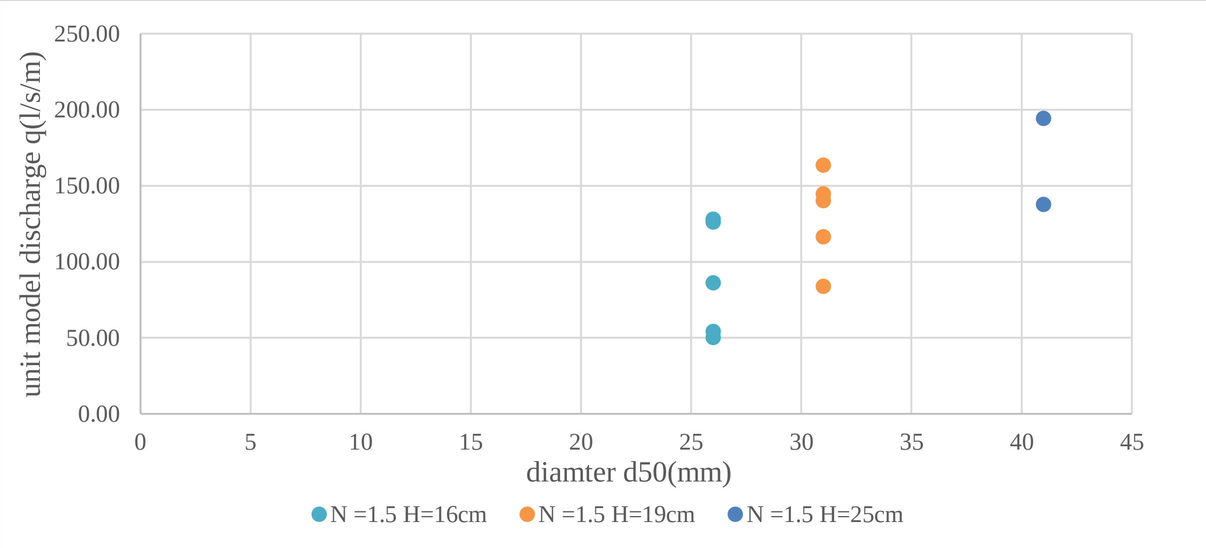
SN	Diameter	Date	Orientation	angle of	slope	scale	Q start l/s	Q collapse	ht cm	lm (cm)
RY1a	26	08.05.2013	Random	56.00	1.5	1:19.712	28.77	47.42	15.85	61
RY2a	26	08.05.2013	Random	56.00	1.5	1:19.712	25.87	25.87	15.85	61
RB1a	31	09.05.2013	Random	58.50	1.5	1:19.712	33.86	33.86	18.90	61
RB2a	31	09.05.2013	Random	58.50	1.5	1:19.712	23.45	23.45	18.90	61
RR1a	41	13.05.2013	Random	59.50	1.5	1:12.5	12.36	12.36	25.00	61
RR2a	41	13.05.2013	Random	59.50	1.5	1:12.5	18.35	12.36	25.00	61
71R1a	41	13.05.2013	71 degree	89.00	1.5	1:12.5	118.52		25.00	61
71B1a	31	09.05.2013	71 degrees	88.00	1.5	1:19.712	51.14	51.14	18.90	61
71B2a	31	09.05.2013	71 degrees	88.00	1.5	1:19.712	71.03	71.03	18.90	61
71B3a	31	09.05.2013	71 degrees	88.00	1.5	1:19.712	85.53	85.53	18.90	61
71Y1a	26	08.05.2013	71 degrees	85.00	1.5	1:19.71	52.63	52.63	15.85	61
71Y2a	26	08.05.2013	71 degrees	85.00	1.5	1:19.712	76.98	76.98	15.85	61
71Y3a	26	08.05.2013	71 degrees	85.00	1.5	1:19.712	78.09	78.09	15.85	61
RY1b	26	07.05.2013	Random	56.00	1.8	1:19.712	23.96	23.96	15.85	61
RY2b	26	07.05.2013	Random	56.00	1.8	1:19.712	37.39	37.39	15.85	61
RY3b	26	07.05.2013	Random	56.00	1.8	1:19.712	39.25	39.25	15.85	61
RR1b	41	13.06.2013	Random	59.50	1.8	1:12.5	26.56	26.56	25.00	61
RR2b	41	13.06.2013	Random	59.50	1.8	1:12.5	28.10	28.10	25.00	61
RR3b	41	13.06.2013	Random	59.50	1.8	1:12.5	24.14	24.14	25.00	61

RR4b	41	13.05.2013	Random	59.50	1.8	1:12.5	13.46	13.46	25.00	61
RB1b	31	10.05.2013	Random	58.50	1.8	1:19.712	20.93	20.93	18.90	61
RB2b	31	10.05.2013	Random	58.50	1.8	1:19.712	29.77	29.77	18.90	61
RB3b	31	10.05.2013	Random	58.50	1.8	1:19.713	25.90	25.90	18.90	61
RB4b	31	10.05.2013	Random	58.50	1.8	1:19.71	31.35	31.35	18.90	61
71B1b	31	17.04.2013	71 degree	88.00	1.8	1:16.5	111.10		18.90	61
71B2b	31	17.04.2013	71 degree	88.00	1.8	1:16.5	106.46		18.90	61
71B3b	31	17.04.2013	71 degree	88.00	1.8	1:16.5	106.85		18.90	61
71B4b	31	16.04.2013	71 degree	88.00	1.8	1:16.5	93.20		18.90	61
71B5b	31	16.04.2013	71 degree	88.00	1.8	1:16.5	107.11		18.90	61
71R1b	41	12.04.2013	71 degree	89.00	1.8	1:12.5	138.10		25.00	61
71R2b	41	14.04.2013	71 degree	89.00	1.8	1:12.5	110.05		25.00	61
71R3b	41	10.04.2013	71 degree	89.00	1.8	1:12.5	118.10		25.00	61
71R4b	41	22.03.2013	71 degree	89.00	1.8	1:12.5	117.88		25.00	61
71Y1b	26	06.05.2013	71 degrees	85.00	1.8	1:19.712	77.91	77.91	15.85	61
71Y2b	26	06.05.2013	71 degrees	85.00	1.8	1:19.712	73.07	73.07	15.85	61
71Y3b	26	06.05.2013	71 degrees	85.00	1.8	1:19.712	76.23	93.33	15.85	61
RB4b	31	10.05.2013	Random	58.50	1.8	1:19.713	31.35	31.35	18.90	61

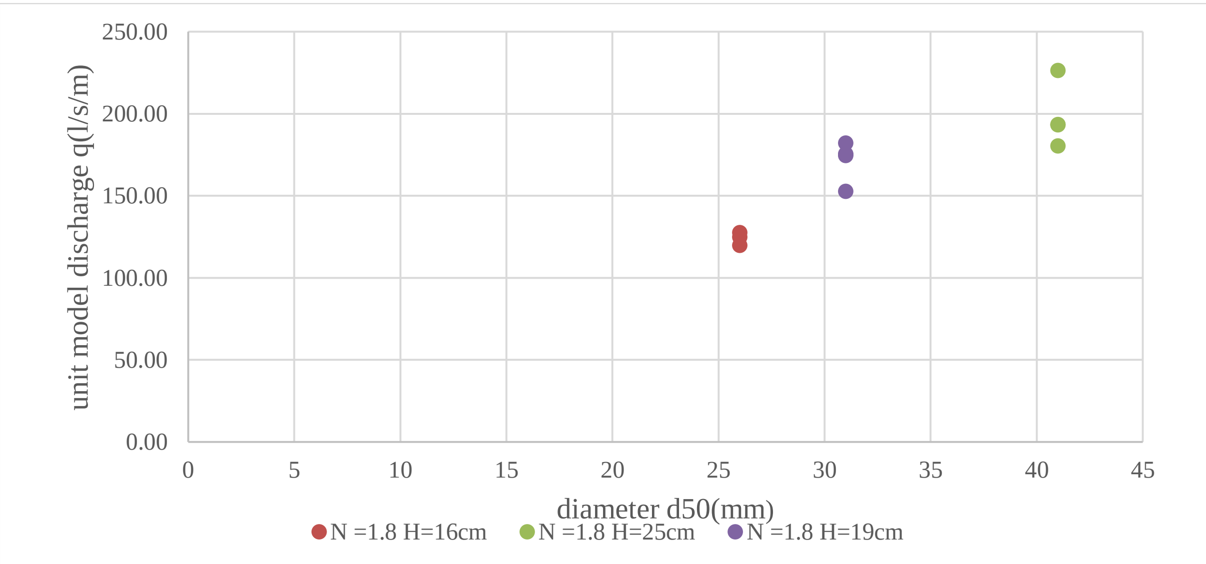
71 degree orientation of longest axis of riprap with dam slope

The various experiments were carried out for the riprap with the orientation of 71 degree with the dam axis. Graph 7.7shows the variation of the failure discharges as the function of the mean diameter of the stones for two different slopes of downstream 1:1.5 and 1:1.8. It shows the rising trend line for the discharge as the function of the mean diameter of stones in both slopes. The unit discharge at failure increases with the increase in the size of the stone diameter and the height of the dam. From the graph of two different slopes, it shows that the failure discharges is less for the steeper slope for the same diameter of the stones. The discharge is 1.29 times greater for the slope of 1.8 than for 1.5.

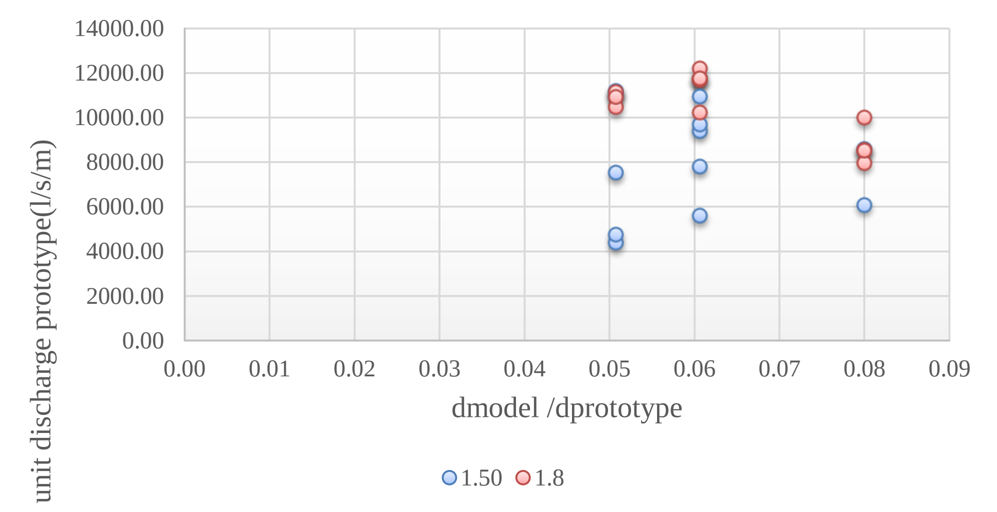
Graph 7.8 shows the relative variation of the stone size with the prototype and the failure discharge and Table 7.8 below the table shows the mean and standard deviation. It takes into the consideration of the Froude's law.



Graph 7.6 Failure unit discharge vs. mean diameter for 71 degree orientation N=1.5



Graph 7.7 Failure unit discharge vs. mean diameter for 71 degree orientation N=1.8



Graph 7.8 Relative stone size to the unit discharge prototype for 71 degree orientation

Table 7.8 Mean and the standard deviation for 71 degree orientation

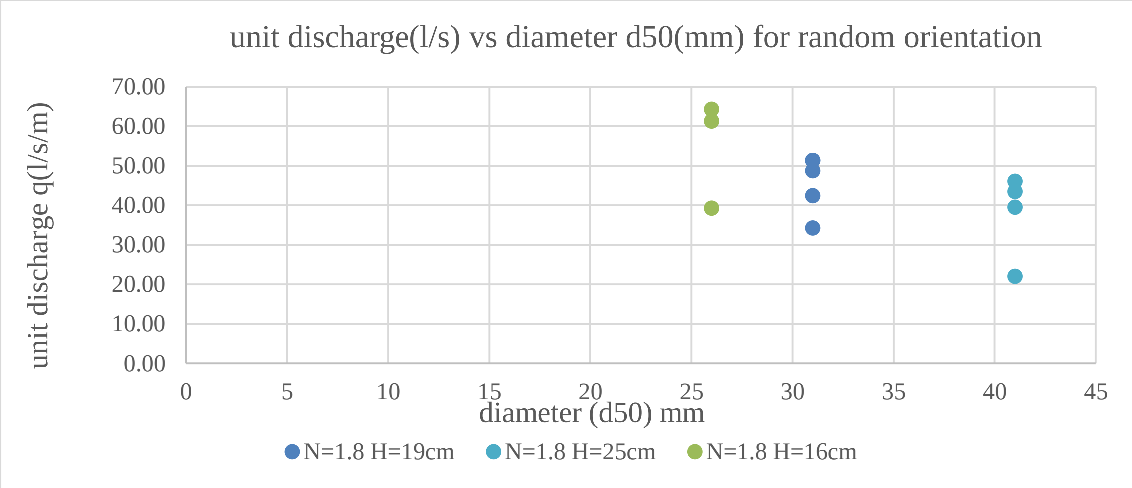
slope 1:1.8	**Mean**	**10441.44**	**l/s/m**
	Standard deviation	1426.91	l/s/m
slope 1:1.5	Mean	8090.62	l/s/m
	Standard deviation	2456.49	l/s/m

63 degree orientation of longest axis of riprap with dam slope

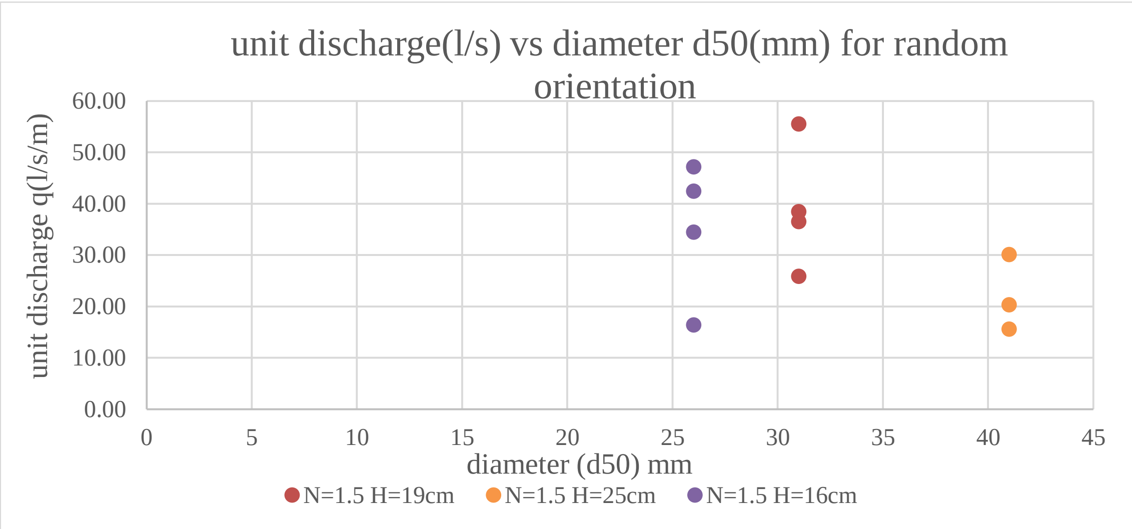
In the general tests, the longest axis of the riprap is also inclined with the angle of 63 degrees with the with the dam slope. The orientation of the angle is varied to know the difference in the performance of the rip rap under these conditions. Besides that the Norwegian dams in most of the places often varies to this degree and this orientation is chosen to resemble also the prototype field scale tests. There are not much tests done with this orientation but the test done is shown in the Graph 7.1.

Random Orientation

The number of the experiments based on the random orientation is performed. The prototype taken also has the random orientation so more experiments are done in the random orientation to study the variation and results. The variation of the results in the different sized stones and height of the model test dam is presented in the form of Graph 7.9 and Graph 7.10. In the following graph, the height of the dam is also varied which gives the change in the result of the tests. The graph shows the opposite trend than expected. The height is varied for the each set of experiments and the conditions are same as in the 71 degree orientation only the difference in the orientation. It shows the falling trend of failure discharge with the increase of diameter and height.

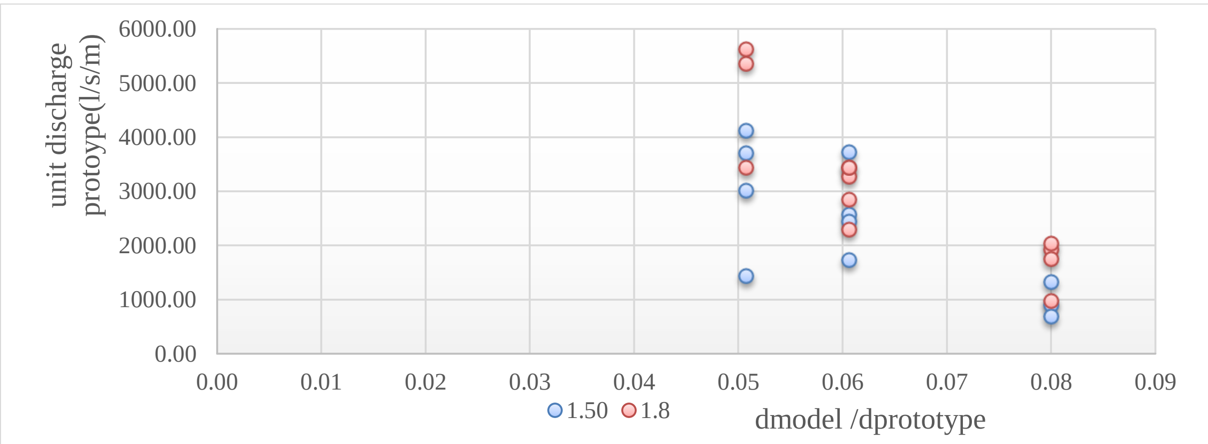


Graph 7.9 unit discharge (l/s/m) vs. diameter d50 (mm) for random orientation N=1.8



Graph 7.10 unit discharge (l/s/m) vs. diameter d50 (mm) for random orientation for N=1.5

The relative size of the stones with the prototype for the two different slopes carried out in the tests is also shown in Graph 7.11.



Graph 7.11 Relative sizes of the stone to prototype with the failure unit discharge for random orientation.

The standard deviation and the mean discharges for the experiments for the two different slopes are presented in the Table 7.9 which shows the discharge is 1.3 times greater for 1.8 slope than 1.5 slope.

Table 7.9 Mean and the standard deviation for random orientation

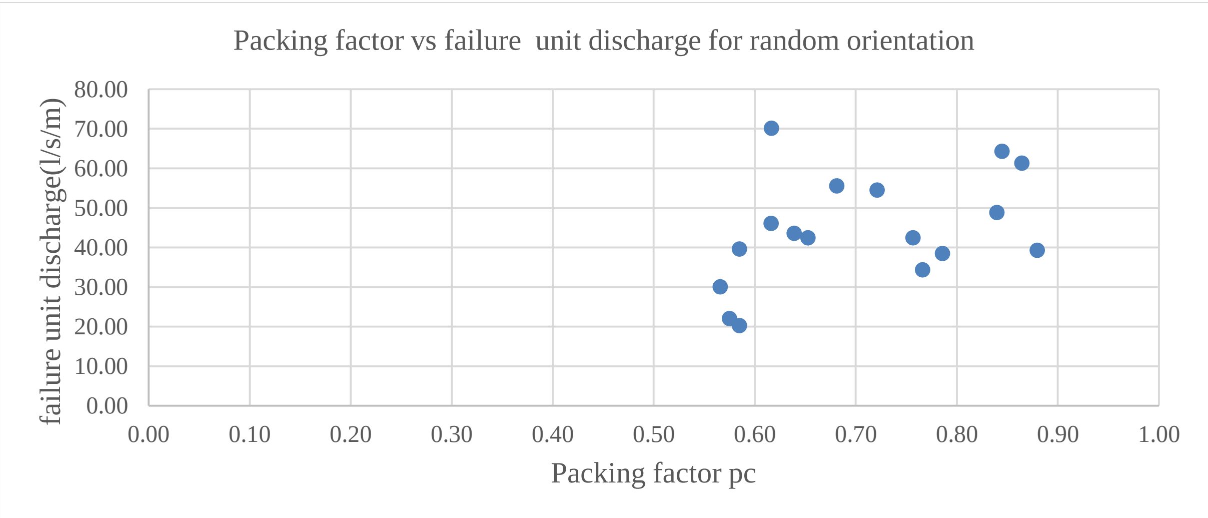
slope 1:1.8	**Mean**	**3034.049927**	**l/s/m**
	Standard deviation	1393.743849	l/s/m
slope 1:1.5	Mean	2332.92	l/s/m
	Standard deviation	1204.36	l/s/m

7.4.1 Horizontal orientation

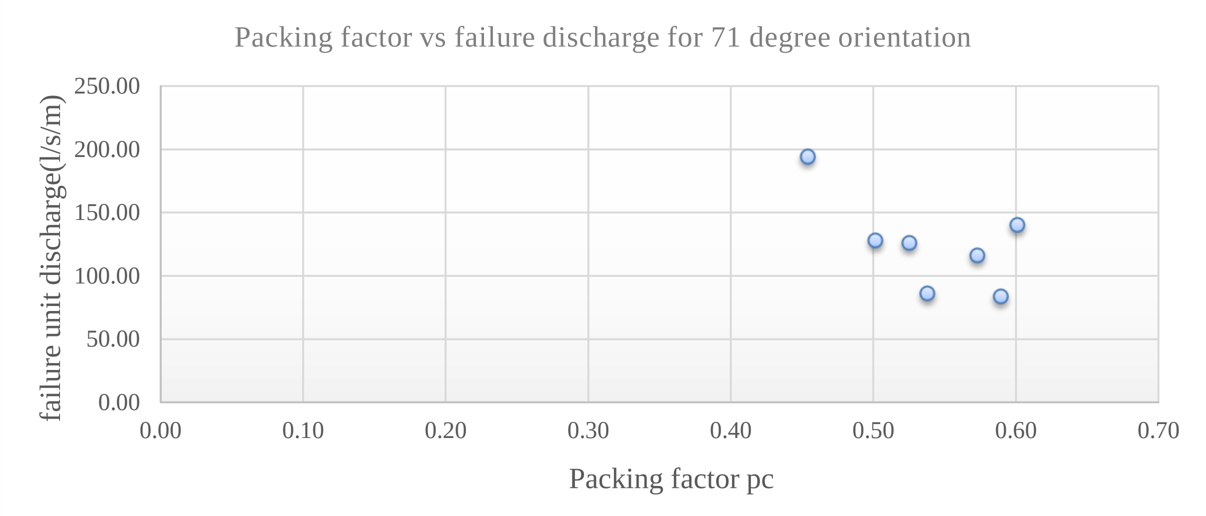
The results of the horizontal orientation are presented in Graph 7.1 for the slope of 1:1.5.There is not many experiments done with this orientation but it can be seen clearly the relation between the failure discharge capacity and size of the rip rap. It is not possible to clearly define the trend line. The Table 7.9 shows the standard deviation and the mean value for the horizontal arrangement.

7.4.2 Results from the packing factor

The packing factor was observed for couple of the tests before the experiment and it was checked to see if the packing factor influences the discharge at failure. The results are shown in Graph 7.12 for random orientation and Graph 7.13 for the 71 degree orientation. But the test results show that the packing factor has no specific trend line going with failure discharge. Thus it can be said that show that packing factor which is one of the rock fill properties has nothing to do with the failure unit discharge of the dam. But in Oliver and Knauss, the formula shows that the packing factor has great impact on the stability and the threshold flow.



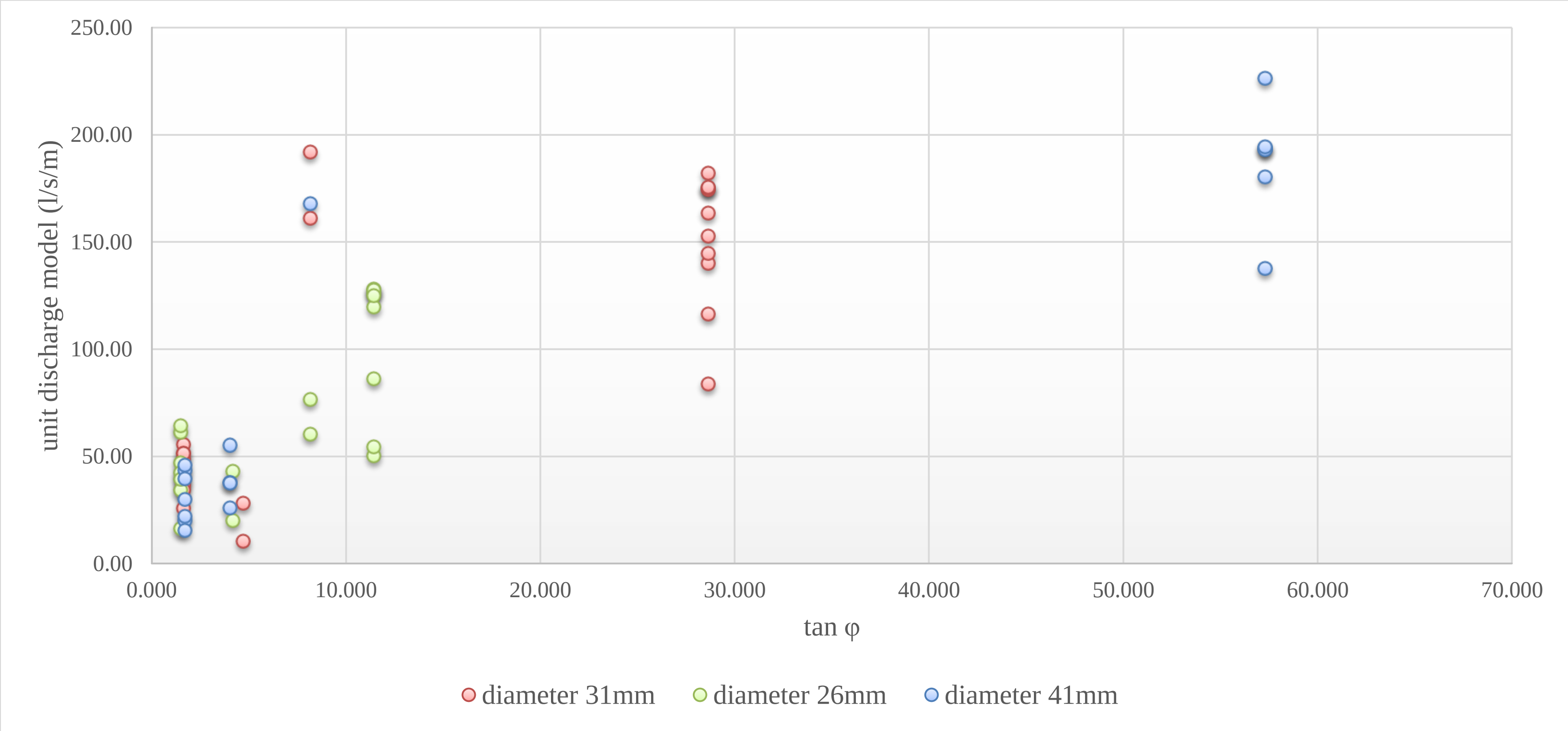
Graph 7.12 Packing factor vs. failure unit discharge for random orientation



Graph 7.13 Packing factor vs. failure discharge for 71 degree orientation

7.4.3 Results from the quasi angle of repose

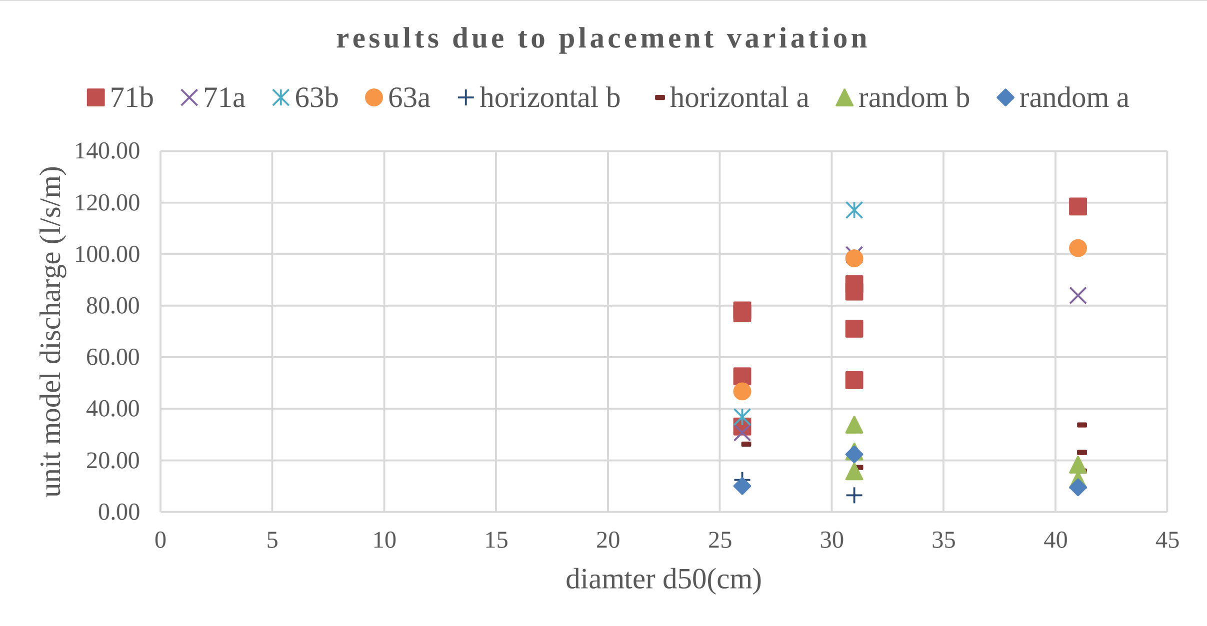
To study the performance of the rip rap in the failure discharge, the quasi angle of repose representing the approximate angle of repose is studied which is already discussed in the methodology. The tangent of the quasi angle of the repose is plotted against the failure unit discharge for different diameter of stones and the height of the dam. The plot is shown in Graph 7.14 Variation of the unit failure discharge with the tangent of quasi angle of repose. There is no clear trend shown for this parameter. However, there is rising of the failure unit discharge for the high tangent value of the quasi angle of repose. The tangent value is higher for the 71 degree and 63 degree orientation of the longest axis of the rip rap whereas the value is lesser for the horizontal orientation and the least for the random orientation. The change in the orientation changes the angle of repose and ultimately the change in failure discharge. More study is to be done in this parameter with accurate way of measurement.



Graph 7.14 Variation of the unit failure discharge with the tangent of quasi angle of repose

7.5 Difference in the work placement

The same placement of rip rap was made by the different persons with all the same parameters. Graph 7.15 shows that the results vary a lot when the person making placement differs for the same orientation. In the following Graph 7.15 , 'a' indicates the placement made by another student and 'b' indicates the placement by me. For the different orientations, placement of one person gives good or poor results than the other person. It shows that the same person can result very good placement result for one orientation and bad for other orientation.



Graph 7.15 Variation due to placement variation

7.5.1 Failure mechanism

The placement of the rip rap with the dam slope influences the variation in the failure discharge. Besides that, it also shows the variation in the mode of the failure. The modes of failure can be studied in detail from the videos recorded during the experiment. There are two distinct initiation of failure mechanisms that were observed in the experiments.

The condition of unraveling failure, mostly in all the cases either starts from the toe stones by the dislodging of the toe stones or by the erosion of the top stones. For the rip rap placed 63 degree or 71 degrees, most of the movement starts from the top by erosion of the stones causing the crest to be eroded ultimately causing the failure of the rip rap. None of the failure in that orientation, started from the toe. In such conditions, dam almost fails by the overtopping flow than the throughflow.

For the random placement of rip rap, the movement is initiated by the dislodging of the toe stones. During the flow, the toe stones get exposed and get dislodged from the slope and then the failure occurs by the sliding of the mass due to the instability caused by the movement of the toe stones. In the random placement of the rip rap, the failure is caused by both combined overflow and the throughflow where the throughflow mostly causes the initiation of the movement.

The observation of each experiment can be seen in detail in the following tables Table 7.10 Failure modes in the slope of 1:1.8for the slope of 1:1.8 and Table 7.10 for the slope of

1.5. In the table, failure modes in each experiment of the general and additional tests are shown below for each orientation carried out. In the table, the mass sliding failure as a result of geotechnical instability is represented as FS and the erosional failure is represented by E. The results of the horizontal and the 63 degrees are done together with Ole Kristian and data is take from him.

Table 7.10 Failure modes in the slope of 1:1.8

SN	Diameter mm	Date	Orientation	slope	Q start l/s	failure mode	Q collapse	ht cm
RY1b	26	07.05.2013	Random	1.8	23.96	few toe stones moved down and whole dam collapsed down from the top	FS	23.96
RY2b	26	07.05. 2013	Random	1.8	37.39	first the top stones came out the top stones removed, the toe stones dislodged and whole dam collapsed when the bottom toe stones removed and collapsed	FS	37.39
RY3b	26	07.05. 2013	Random	1.8	39.25	one toe stone got moved, top stones moved down and then toe stones moved and finally all the stones slumped down the slope	FS	39.25
RR1b	41	13.06. 2013	Random	1.8	26.56	The exposed stone went down and the dam failed from started from the bottom took out the whole dam.	FS	26.56
RR2b	41	13.06. 2013	Random	1.8	28.10	dam moved down from bottom taken away the exposed stone moved and took away whole dam	FS	28.10
RR3b	41	13.06. 2013	Random	1.8	24.14	from thebottom the toe stones got removed and the whole dam slided down	FS	24.14
RR4b	41	13.05. 2013	Random	1.8	13.46	movement of the whole dam fail by sliding of the dam toe	FS	13.46
RB1b	31	10.05. 2013	Random	1.8	20.93	from the bottom it started and moved to the top and part of it collapsed	E	20.93
RB2b	31	10.05. 2013	Random	1.8	29.77	from the bottom it started and moved to the top and whole dam collapsed	FS	29.77
RB3b	31	10.05. 2013	Random	1.8	25.90	dam failed starting from bottom to the top due to dislodged stone at the toe	FS	25.90
RB4b	31	10.05. 2013	Random	1.8	31.35	From the top few stones moved and the failure occurred by sliding from top to all the stones due to the lost stones at the toes	FS	31.35

RB4b	31	10.05.2013	Random	1.8	31.35	From the top few stones moved and the failure occurred by sliding from top to all the stones due to the lost stones at the toes	FS	31.35
71B1b	31	17.04. 2013	71 degree	1.8	111.10	stones stones fall down from the top of the dam the stones got dislodged	ET	
71B2b	31	17.04. 2013	71 degree	1.8	106.46	some stones slided down from top moved down	ET	
71B3b	31	17.04. 2013	71 degree	1.8	106.85	top stones moved down	ET	
71B4b	31	16.04. 2013	71 degree	1.8	93.20	sudden movement and whole dam collapse at a sudden from the top	FS	
71B5b	31	16.04. 2013	71 degree	1.8	107.11	The top stones eroded and slided down and got removed by the flow	Et	
71R1b	41	12.04.2013	71 degree	1.8	138.10	some stones slided down from top moved down	E	
71R2b	41	14.04. 2013	71 degree	1.8	110.05	some stones slided down from top moved down	E	
71R3b	41	10.04. 2013	71 degree	1.8	118.10	some stones slided down from top moved down	E	
71R4b	41	22.03. 2013	71 degree	1.8	117.88	some stones slided down from top moved down	E	
71Y1b	26	06.05. 2013	71 degrees	1.8	77.91	The failure occur from the left side of the top and few stones got removed and finnally all got removed upto half	Et	77.91
71Y2b	26	06.05. 2013	71 degrees	1.8	73.07	the top stones dislodged and then the toe stones dislodged and from the top the dam moved foreward and slided away	ET	73.07
71Y3b	26	06.05. 2013	71 degrees	1.8	76.23	some of the stones from the right side moved down	FS	93.33

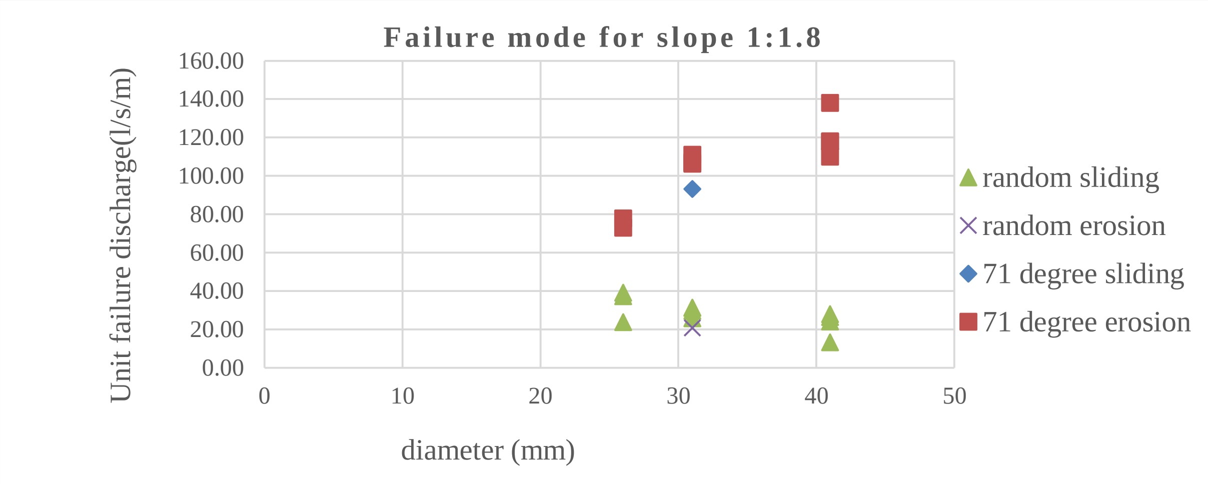
Table 7.11 Failure modes in the slope of 1:1.5

SN	Diameter mm	Date	Orientation	slope	Q start l/s	failure mode	Q collapse	ht cm
RY1a	26	08.05. 2013	Random	1.50	28.77	firstly the toe stones got moved and by the continual removal of the top stones, top few stones came down	E	47.42
RY2a	26	08.05. 2013	Random	1.50	25.87	from top few stones moved and from bottom the toe stone got dislodged and whole dam almost failed	FS	25.87
RY1G	26	13.03. 2013	Random	1.50	10.00		FS	10.00
RY2G	26	13.03. 2013	Random	1.50	21.00		FS	21.00
RB1a	31	09.05. 2013	Random	1.50	33.86	toe stone moved and whole dam collapsed	FS	33.86
RB2a	31	09.05. 2013	Random	1.50	23.45	the dam collapsed started from the bottom to the top and pump stopped	FS	23.45
RB1G	31	07.03. 2013	Random	1.50	22.26	first few stones disappeared , moved down in the left and during movement try to stabilize the position by longest axis and then complete failure occurred	FS	22.26
RB2G	31	07.03. 2013	Random	1.50	15.75	the movement started from the bottom from where the stones got washed away and travelled to the top and failed whole dam	FS	15.75
RR1G	41	01.03. 2013	random	1.5	9.50	Stones gradually got moved from the top on each sides from the first movement was registered, the riprap itself did not fail.	E	14.20
HR2G	41	01.03. 2013	Horizontal	1.50	23.10	Erosion from top, quick slide, stability problem of the top.	E	23.10
RR1a	41	13.05. 2013	Random	1.50	12.36	the failure started by the movement of the toe and whole dam collapse by sliding down	FS	12.36
RR2a	41	13.05. 2013	Random	1.50	18.35	dam fails started from the removing of the stones from the toe and dam fails from the exposed toe stones that carry away the whole dam by sliding	FS	12.36

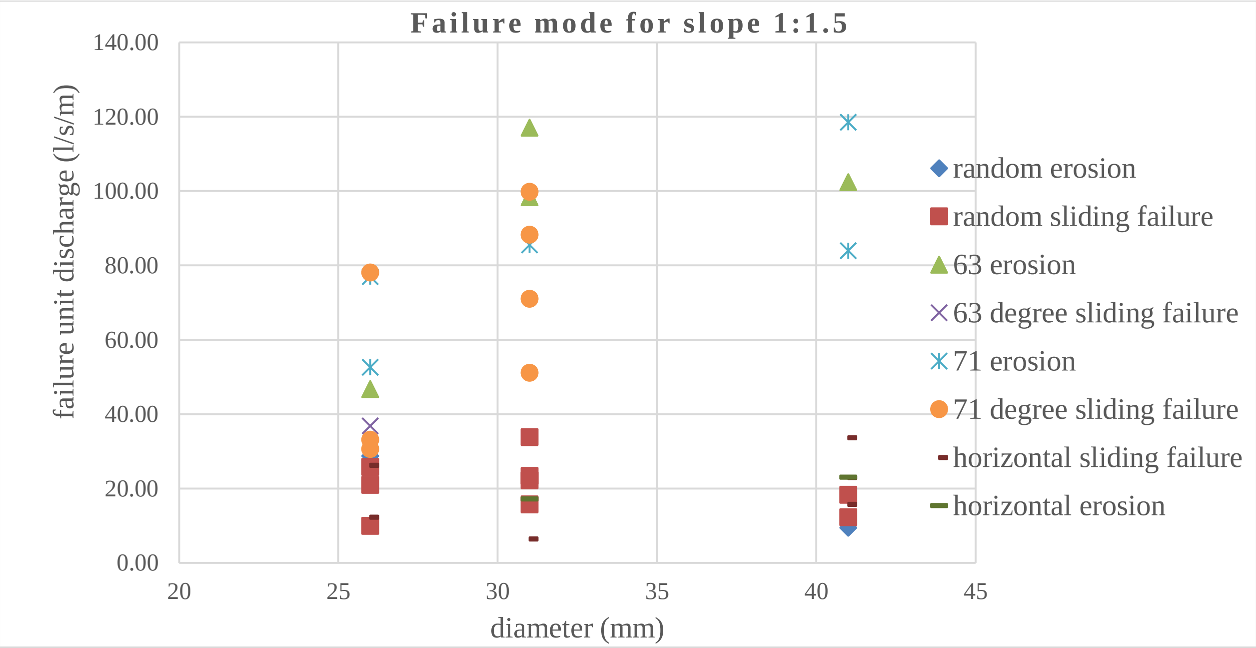
HR3G	41		Horizontal	1.50	23.00	Stability problem because the whole top got lifted outwards	FS	23.00
HR5G	41		Horizontal	1.50	15.80	Moved 2-3 stones from the lower part of the dam before an area at the bottom right part broke and the whole riprap failed.	FS	15.80
HR4G	41	21.02. 2013	Horizontal	1.50	33.70	Stability problem, the whole top got lifted forwards/ outwards	FS	33.70
HB1G	31	04.03. 2013	Horizontal	1.50	17.20			40.90
HB2G	31	04.03. 2013	Horizontal	1.50	6.47	From the top few stone stones moved and then carried away the whole dam with the increment in the discharge	FS	6.47
HY1G	26	08.03. 2013	Horizontal	1.50	26.30	the sliding occurred starting from the top and then pushed away the dam and whole dam failed from top	FS mix	26.30
HY2G	26	08.03. 2013	Horizontal	1.50	12.35	failure complete first by dislodging of one toe stone then whole dam failed from the top	FS	12.35
63R1G	41	25.02. 2013	63 degree	1.50	102.40	Top of dam failed, made a v-shaped failure area but the rest of the dam did not fail.	E	106.00
63B1G	31	05.03. 2013	63 degree	1.50	98.40	Top of dam failed and the riprap and the supporting fill got removed from the upper part of the dam. The lower half of the dam was still intact.	ET	98.40
63B2G	31	07.03. 2013	63 degree	1.5	117.05	erosion of the top stones failed	ET	exceed ed the capacit y
63Y1G	26	09.03. 2013	63 degree	1.50	46.80	Erosion of riprap and support filling from the top of the dam, going fast.	E	52.60
63Y2G	26	09.03. 2013	63 degree	1.50	36.85	starting the top left the whole dam failed by sliding	FS	36.85
71R1a	41	13.05. 2013	71 degree	1.50	118.52	top stones got removed	E	118.52
71R1G	41	21.02. 2013	71 degree	1.5	84.00	Stones gradually got moved from the top on each sides from the first movement was registered, the riprap itself did not fail.	E	120.70

71B1a	31	09.05.2013	71 degrees	1.50	51.14	several stones moved from the top and whole dam failed from top	FS	51.14
71B2a	31	09.05.2013	71 degrees	1.50	71.03	From the top few stones got removed and whole rip rap collapse along with the dam	FS	71.03
71B1G	31	06.03.2013	71 degrees	1.50	99.80	Stability problem, the whole top got lifted forwards/ outwards	FS	99.80
71B2G	31	06.03.2013	71 degrees	1.50	88.32	movement of the stones form the top most dam perhaps due to the placement of the stones and failure from the top of the erosion and then complete failure	FS	88.32
71B3a	31	09.05.2013	71 degrees	1.50	85.53	top layer of the stones got washed away and rip rap failed	E	85.53
71Y1a	26	08.05.2013	71 degrees	1.50	52.63	From the top left portion certain stone moved and then carried out whole portion of the left part and failed down	E	52.63
71Y2a	26	08.05.2013	71 degrees	1.50	76.98	The top stone came out and the sliding of certain mass of the rip rap and failure	E	76.98
71Y3a	26	08.05.2013	71 degrees	1.50	78.09	the top stones slide down first and whole dam collapsed finally	FS	78.09
71Y1G	26	12.03.2013	71 degrees	1.50	30.70	Fast Erosion of riprap and support filling from the top of the dam	FS	48.30
71Y2G	26	12.03.2013	71 degrees	1.50	33.16	The top stone moved down and the whole dam slide down	FS	33.16

The mode of the failure is also found different for different slope of 1.5 and slope of 1.8 which can be shown in the Graph 7.16 and Graph 7.17. The erosion seems more frequent in the proper rip rap placement whereas the sliding is more frequent in random placement. But for the slope of 1:1.5 and 1:1.8, Graph 7.16 and Graph 7.17 shows the combined process of erosion and sliding failure in the test results:



Graph 7.16 Failure modes for slope 1:1.8



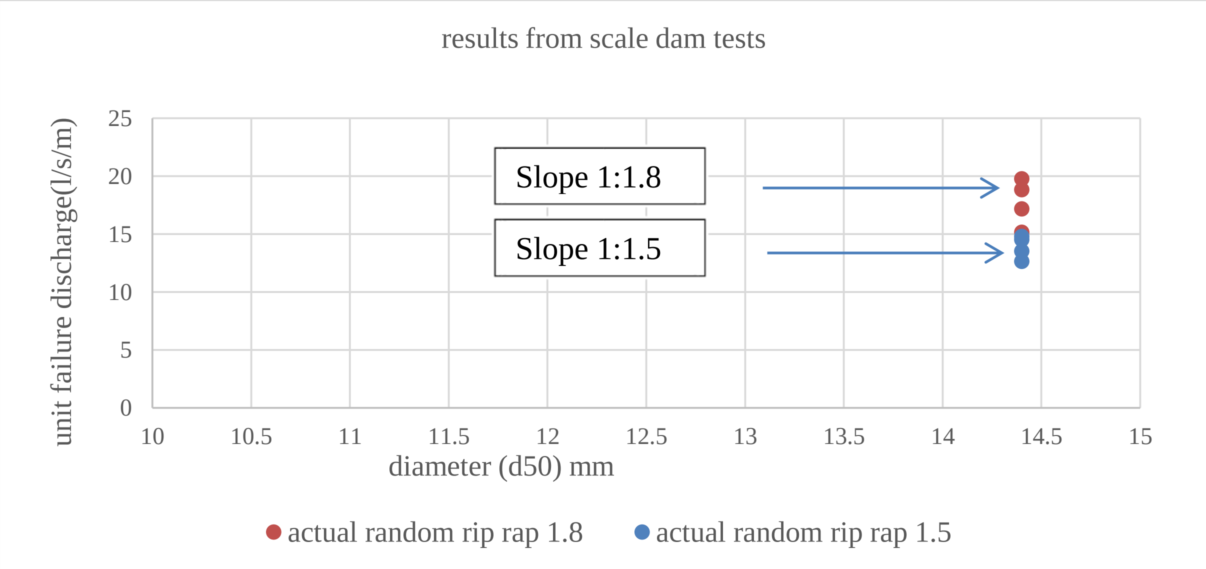
Graph 7.17 Failure modes for slope 1:1.5

7.6 Discussion and analysis from the experiments

7.6.1 Failure mode at steeper and the flatter slope

Two different slopes are used 1:1.8 for real prototype and 1:1.5 for general rip rap tests. As mentioned in the literature before too, the inclination of the major axis of the rip rap with the dam slope plays the important role in the stability of the rip rap and the resistance to the failure. From the tests, it can be observed that most of the slope failure occur at the less discharge in the steep slope of 1:1.5 than the flatter slope 1:1.8. It can be shown in the Graph 7.18

Similarly in the scale dam tests also the unit discharge failure is found to be low for the 1.5 slope than 1.8 slope of the prototype which is presented in the given Graph 7.18. From the several tests with the several orientations, it can be seen that the failure discharge for 1.8 slope is about 1.27 to 1.30 times greater than for the slope of 1.5. It is not possible to use different scales for the scale dam tests of the prototype as it has large variation in the size of the particles.



Graph 7.18 Results from the scale dam test for two different slopes

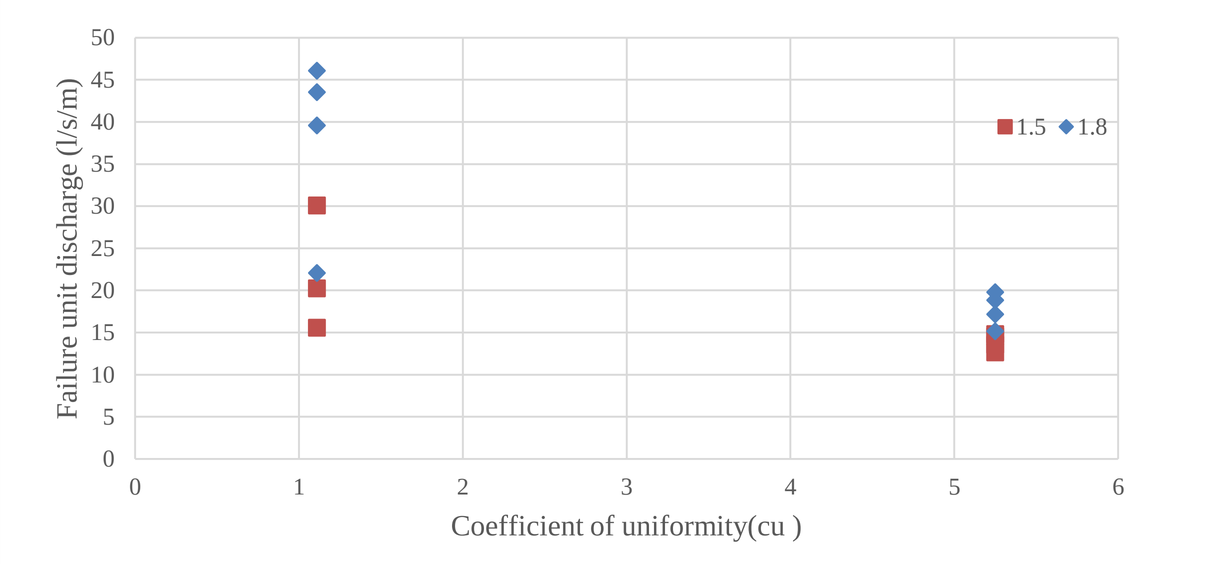
Also when the scale dam tests were carried out for the two different slopes in the random rip rap for the given rock fill, pattern of the slope failure is found to be different. The slope failure at the flatter slope is observed to be slightly different than the slope failure at the steep slope. But it cannot give clear interpretation of the failure.

Interpretation of failure in the real prototype

The Kulekhani rockfill dam is quite different from the normal randomly placed rip rap rockfill due to the variation in the grading of the stones used in the exposed area of the downstream toe. The physical modeling of this rockfill type is carried out and the failure mode and drainage capacity is checked.

In the real prototype, Kulekhani dam the various rockfill grading. The coefficient of uniformity is very high unlike the normal rip rap sizing placed in the rockfill protection given recommended**.** The several bigger stones are interlocked with the smaller stones and the voids are filled up by the group of the small stones. When the rockfill dam is overtopped by the PMF flood, then the flow overflows over the embankment. The flow at the exit depth increases as it travels down and the high flow occurs which is already calculated as the methodology and appendix. The flow erodes away the small stones in between the bigger rock boulders and the finally the bigger boulders slide down due to the lack of the interlocking and the weight of the rock itself. This can be the probable cause of the failure in the randomized rockfill with the various grading of the stones.

For the safe side, the normally and randomly placed rip rap is also modeled for the existing type of the dam with the small coefficient of uniformity. It shows that the randomly placed rip rap is capable of taking more discharge than the randomly placed rockfill of various grading. The plot from the same rockfill but the different coefficient of uniformity is shown in the Graph 7.19 which shows that the variation of the coefficient of uniformity also varies the result.



Graph 7.19 Coefficient of uniformity vs failure unit discharge model (l/s/m)

7.6.2 Initiation of the movement

Generally when the flow overtops the flow is carried downstream down the toe through the downstream slope. During the flow, the rock sets down and adjusts itself to resist the forces implied on it. It is considered as the threshold flow However, at a point when the flow is increased, a point is reached where the rock begins to vibrate and displaces from its original position. During the movement, it starts to change the position from flat placed to edge placed in most of the cases which is observed in the case of the random arrangement of the rip rap. The vibration of the rock occurs at that transition to rock entrainment and displacement in most of the cases in the experiment. When the individual rock gets displaced, it gets carried to the certain distance across the toe and then gets lodged into other rocks in the basin.

In the low discharge condition, it is possible to observe the rock vibration. It is also possible to see its movement and the displacement mode occurring at the initial stage. However, it is not easy to know which rock moves first place and to what extent or point the stone is carried away is also not easily evaluated in the case of high discharge.

7.6.3 Channelization

The channelization of the flow occurs between the threshold and the collapse flow during the overtopping. When the small stones are washed away and the threshold condition exceeded, the flow gets concentrated in the channels so formed and takes in the high discharge before the whole dam collapse.

When the test was done for the poorly graded stones at the initial stage with the stone fillings only, the flow is small. The channels of flow are formed when the flow increases to certain time. When the flow increases, the resistance to the small stones being less than the resistance for the bigger stones, the small stones are moved. Between the moved stones, the gap between the stones increases. The gap is created between the bigger and the smaller stones. Among the gap, the flow is concentrated through this and the channelization occurs. The flow channelization occurred after stone movement and immediately prior to collapse of the riprap layer.

7.6.4 Rip rap thickness

Throughout the experiment, the rip rap is placed in a single layer. Most of the cases, the single placement of the rip rap is done. The thickness of the layer of rip rap is not taken into the consideration.

7.6.5 Packing factor

The results of the discharge at which the rock movement begins are found to be different. This is mainly due to the tractive theory. This takes into the consideration of the diameter of the stones, the angle of inclination, angle of the repose of the stones and packing factor. The difference in the observation in the theory and the observation is mainly governed by the constant P, which depends upon the shape and the packing of the stones. The manual arrangement of the stones and the machine arrangement of the stones give substantial effect on the threshold flow. The packing factor in the experiment conducted cannot be judged as the main parameter for the threshold and failure discharge as shown in the Graph 7.12 and Graph 7.13

7.6.6 The effect of roundness

The angular stones are used for the test in the laboratory. The rounded stones are more likely to the slide and roll in the steeper slope than the angular stones. The roundness and the sphericity also influence the tests. But in this experiment, the effect of roundness is not taken into the consideration.

7.6.7 Shape factor of the rip rap

The shape factor plays an important role in the performance of the rip rap. The shape factor which is given by the dimension of the stones can influence the performance. The criteria for the rip rap which is given by

$$\frac{3}{2} < \frac{a}{b} < \frac{3}{1}$$

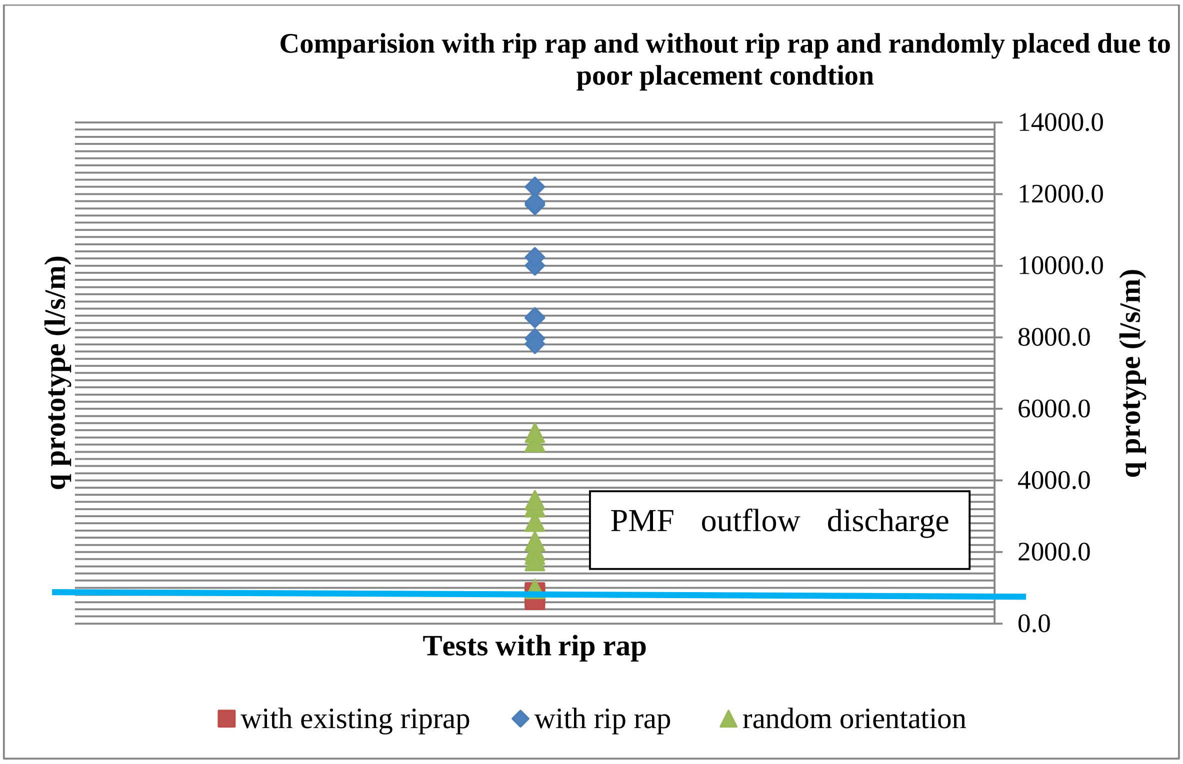
$$\frac{b}{c} < \frac{3}{2}$$

$$\frac{a}{c} < 3$$

This is checked for the existing stones taking the sample of about 30 stones each available in the lab. It shows that the ratio of b/c is satisfied in most of the stones but the ratio b/c and a/b is not satisfied in most of the stones. Most of the blue stones are not satisfied with these criteria. This factor also influences the test results nevertheless these stones are used during the tests. The flaky and elongated stones are though avoided during the tests. The other criteria are not measured for each stone but it was visually inspected for each stones before placement.

7.7 Sensitivity analysis

The most uncertainty parameter here involved is the work labour during the placement of the rip rap and the diameter of the stones chosen. The change in the orientation of the rip rap placement can greatly affect the drainage capacity and the strength of the rip rap. Therefore, the tests are performed with the random placement of the stones which is the extreme case of the improper placement. From the results of the random orientation, it shows that if the same size or bigger stones are used only randomly also, it might be able to take the PMF during the closing of the gates. But if the rip rap is properly placed, the strength is 12 times than the existing one. The experiment performed in the laboratory involves the d50 stones but the site condition, the bigger boulders exist. If the bigger boulders are used, the failure unit discharge will increase as mentioned previously in the results. With such changes also the proposing the rip rap in the Kulekhani can be useful. Following Graph 7.20 gives the existing condition of rip rap represented by red colour, the rip rap strength if placed properly with 71 degree orientation as proposed represented by blue color and the strength of the rip rap when the work placement becomes poor condition as dumped placed due to the poor working skill and condition represented by green lines.



Graph 7.20 Graph for sensitivity check

Chapter 8

8 Numerical Approach

This part of numerical approach mainly deals with the study of the available numerical models, the complication and the general possible modeling of the rockfill dam. The rockfill dam is modeled and tried to see how it works during the modelling in this part. This part uses KRATOS multiphysics for the simulation of the rockfill dam during the overtopping conditions.

8.1 Introduction

In the rock fill the gravitational force is the most important force to be considered that influences the stability of the rock fill dams. The dam mainly fails due to the hydrodynamic forces that act on the rock fill face in the downstream face. The dam fails either by the dragging of the rock fill materials with the some erosive effects or by the slipping of the rock mass downstream region losing the stability of that part. The whole process can be represented in either the numerical models or the physical models. Physical modeling can easily interpret the whole process but for the numerical approach is quite complicated and it needs to use the discontinuous models.[51]Two stages can be observed in the failure of the rockfill; initiation stage or head cutting initiation stage and the breach formation stage.[52]Head cut initiation can be modeled using the tractive stress based approaches.

For the flow through and over the embankments, two types of the flow occur. The seepage flow occurs through the rock fill and the overflow occurs through the main channel above it, which can be described by the Figure 8.1.

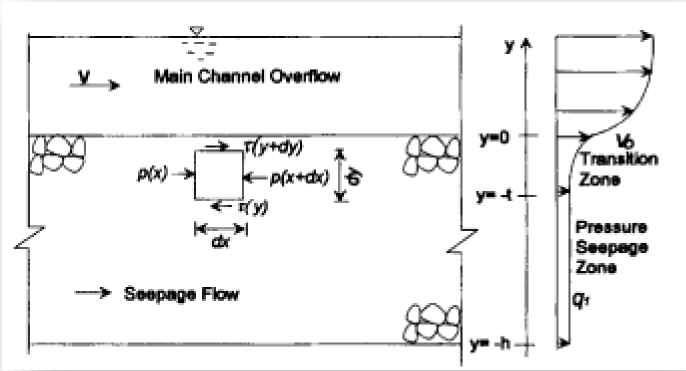


Figure 8.1 Sketch for flow over and through gravel bed[19].

The Figure 8.1 depicts the simplified section along the flow direction. The normal seepage pressure exists in the domain at the lower part of the rock fill and the upper part of the domain characterizes the overtopped flow. During the overtopping phenomenon, the hydraulic open channel flow is dominated rather than the seepage flow. The velocity of the flow at the bottom where the open channel flow begins is not zero but it is the slip velocity v0.Below that the normal seepage flow condition occurs with the seepage velocity of q1.The shear stress above the rock fill is influenced by the overtopping flow and the seepage flow conditions.

8.2 Numerical approach and problems

Numerical approach of the overtopped dam must be represented by the three phases (solid liquid and gas) and in three dimensions. This system consists of the solid with the non-linear material properties with the complex boundary conditions, the complex flow within the rock fill dam body. During the overtopping condition, the solid materials are eroded and collapsed by the flowing water which demands the need of some movable boundaries. In addition the movements are governed by the dynamic interactions, which might not easily fit in the normal finite element methods, which uses the continuum approach. During the overtopping the water flows inside as well as along the downstream face of the dam, which can be difficult to interpret the seepage line. Mass slip failure occurs when the pore pressure increases. Further breaching of the dam due to the combinations of various mechanisms occurs. This adds the complexity in the modeling of the failure mechanism.

Many numerical modeling methods have been presented for the overtopped embankment system. However, the main problem in the overtopping embankment consists of number of the separate phenomena acting together. The problem mostly involves more unsteady flow and the movable boundary. The unsteady flow including the rapidly varying and the gradually varying flow has to be dealt separately.

8.3 Methodology used in the numerical analysis

Different types of numerical models use different way of analysis for the simulation of the rockfill materials. CIMNE (International Centre for Numerical Method of Engineering) developed the numerical model, which characterizes the failure process by combination of the Lagrangian Particle finite element methods, and Eulerian level set technique. This numerical model is used for the study in this thesis. These are used for optimizing the coupling between the dynamic effect of water that flows inside and outside the dam with the structural deformation and collapse that occurs on the downstream slope. The two different models dam model for the structural response to the dynamic action and the fluid model to study the water behavior and the seepage line variation are used and they are coupled in order to consider the influence of the both two media. The water and the dam model are studied using the continuum mechanics approach. First the fluid model is analyzed, results are presented and then structural model is used for the failure mechanism.

Evolution of the seepage

The flow through the large granular rock fill is turbulent, which might not be the amenity of the classical Darcy law. Linear variation of the velocity with the hydraulic gradient is not validable to the flow through the rock fill. In many models the flow in the porous media is studied using the Biot theory but the drawback is that it follows the linear Darcy's law, which is not applicable. Therefore, the Brinkmann's and the Forchheimer's modification Darcy's law are necessary which consider the non-linear relation between the water velocity and the hydraulic gradient[53]. However, they fail to consider the porosity of the material and they do not give the free fluid flow when the porosity is equal to 1.In the numerical model later, the classical flow is modified giving the effect of the porosity using non-linear relation given by Ergun modification to Darcy law which is discussed in the previous chapters.

In the continuum mechanics, the movement can generally describe in two ways

- Eulerian approach
- Lagrangian approach

Eulerian approach gives the fixed mesh and the various variables like velocity and the pressure is calculated at each mesh point. In the lagrangian model, the motion of the particle is studied and the mesh also follows the movement of the particles. The level set method is used to determine the free surface of the fluid.[54]

Particle finite element method (PFEM)

During the overtopping phenomena of the rock fill, the hydrodynamic forces brings into the erosion and the failure of the dam. For the modeling of such process, the Particle finite element method is used. In PFEM, the domain is modeled using the updated Lagrangian formulation. The finite element method is used to solve the continuum equation in a mesh that is built up from the underlying nodes. The mesh is generated then to solve the governing equation. Then each node is viewed as particle and using the Lagrangian model, the motion is tracked. This is important part of modeling as it is able to model the particles being detached from the bed surface and subsequent motion as individual particles.

Programs and languages used

Mainly GiD programming is used to run the numerical model. The main code that is used for the simulation of the process is the module of KRATOS multiphysics. The geometry, material properties and the boundary conditions are defined in the GID and the whole model is discretized into the suitable mesh that is also done in GID preprocessing.

The calculation and the simulation are done automatically by KRATOS multiphysics using the problem type named edge level based algorithm along with the 4^{th} order Range Kutta integration scheme. Kratos uses the python programming language from which the parameters can also be changed manually. The main process involved can be represented in the form of flowchart diagram.

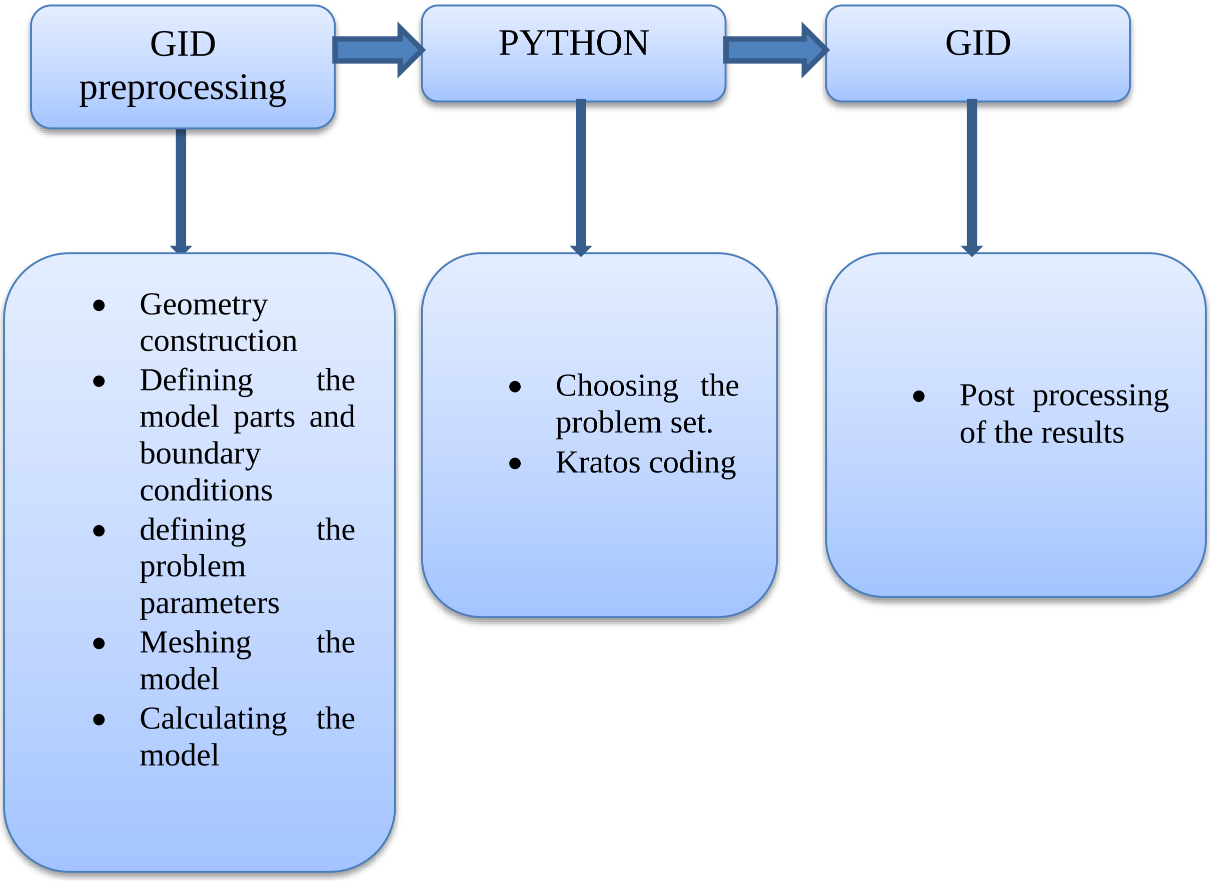


Figure 8.2 Schematic of the modelling

GID preprocess

The pre-processing of the GID includes the construction of the model, defining the boundary condition and the properties of the elements involved in the system. Then the whole model is discretized into the elements by the help of meshing. Two types of the model are constructed as the main model.

- *Geometry*

The first model 1 includes the whole rockfill dam and other part 2 includes only the rockfill zone to see the phenomenon how it works in the numerical model. The geometry can be seen in the Figure 8.3 and Figure 8.4.

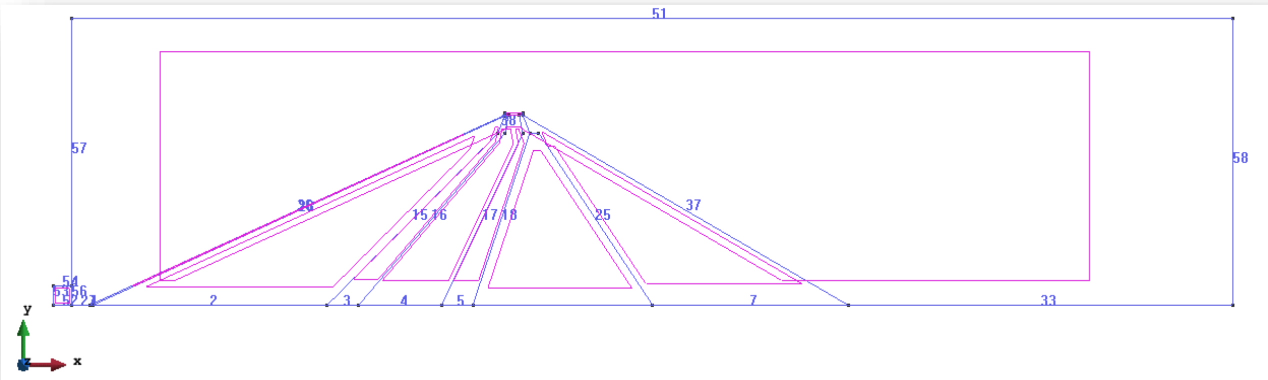


Figure 8.3 Geometry of dam 1 in GID

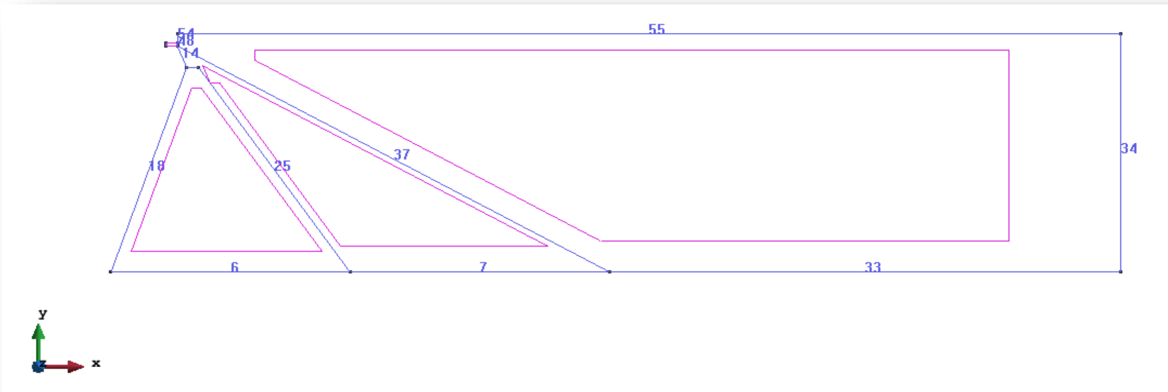


Figure 8.4 Geometry of dam 2 in GID

The geometry is drawn from the design report of Kulekhani dam.

- *Boundary condition*

The boundary condition defines the limit of the models. Every boundary faces is classified and done through the model parts. The inlet has the fixed velocity model parts, the walls are applied with the slip condition and the outlet with the fixed pressure model part. The slip condition defines the constant speed law and the no slip condition defines the solid with no speed in the contour: This is important boundary condition used here.

- *Problem parameters*

In the problem parameters, the properties of the materials that form the whole mesh are defined. Another important parameter which is the porosity is calculated by using the Ergun resistance law which is already discussed in the previous chapters.

- *Meshing*

Another important part of the modelling and simulation is the discretization of the whole body into the several small elements. The meshes used in the model are formed by the triangular nodes. It is tried to use the finer mesh for calculation. The number of grids increases the simulation time too so needs to be optimized. Figure 8.5 and Figure 8.6 also shows the grid mesh used in the final model.

- *Input value*

The velocity in the inlet of the rockfill dam calculated from the overtopping flow calculation is used as the inflow rate for the given rockfill dam. This is important parameter to be given as the input as it influences the calculation. The values used in both model are different. The value of the flow rate is obtained by the flow divided with the area of the entry in the inlet. Following Figure 8.5 and Figure 8.6 shows the area of the inlet. The arrow shows the velocity vectors resulted by the input flow rate. The overtopping discharge outflow from the calculation is 2.514m3/s/m which is converted into the inflow at the inlet section and feed into the model.

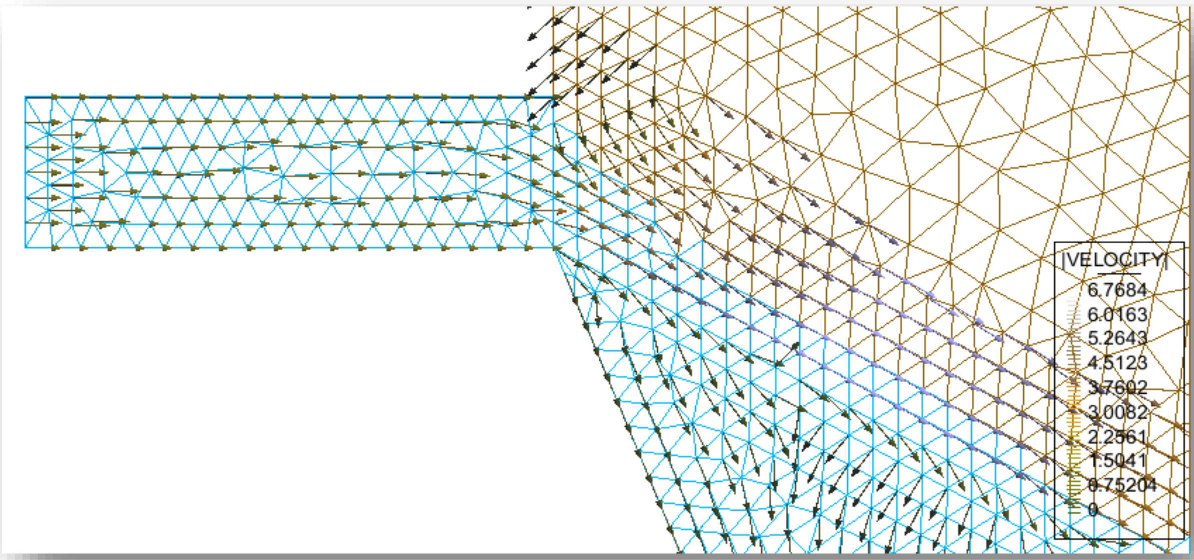


Figure 8.5 Initial velocity vector at the entrance of the model

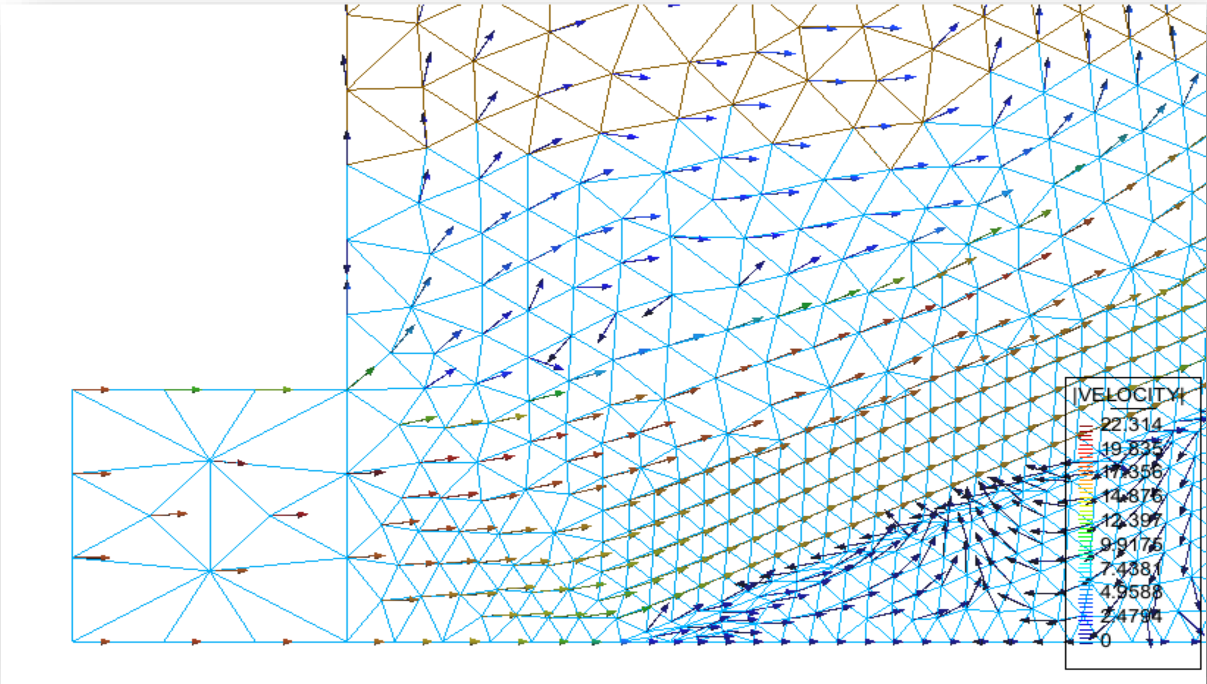


Figure 8.6 Initial velocity vectors at the entrance of the model

- *Calculation*

All the calculation is done by KRATOS programming. It is the open source C++ programming developed for solving the problems of multiphysics. The edged based level set algorithm is used for the calculation. It does not need any additional scripting in the python.

- *Post process*

For the post processing, the GID post processing is used. The simulation is done until the stable stage is acquired. In the post processing, the various plot of the graphs, result surfaces, contour lines, isolines can be done for the calculated variables for each time step. Following graphs show the results of the post processing for the given final step.

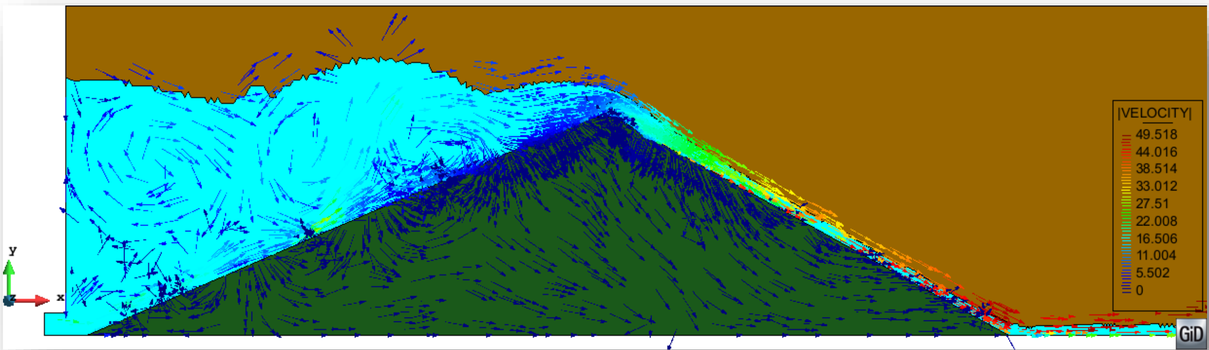


Figure 8.7 Velocity vectors after the simulation of the overtopping.1

Figure 8.8 Result surface of the simulated distributed velocities in the model 1

K

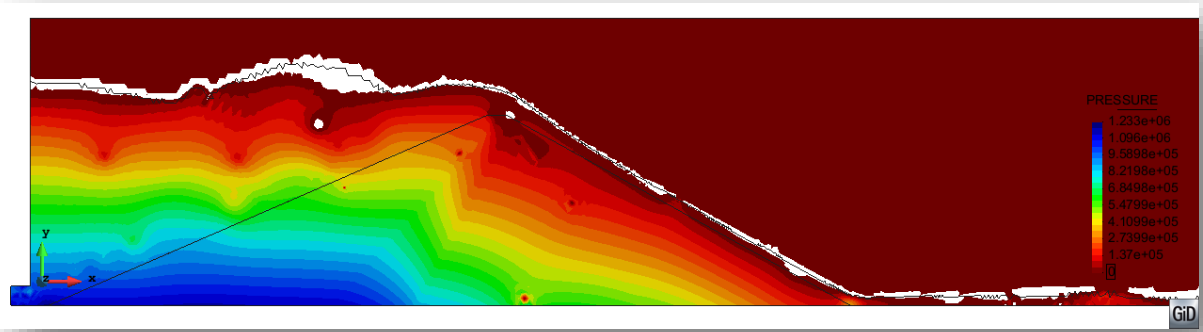


Figure 8.9 Pressure distribution in the numerical model 1

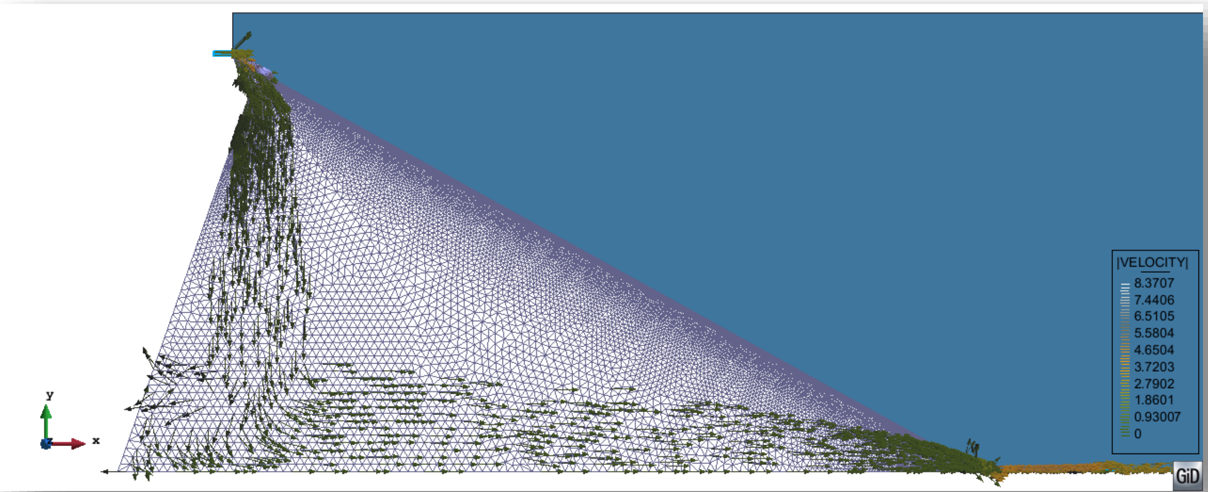


Figure 8.10 Velocity vectors after the simulation of the overtopping flow in model 2

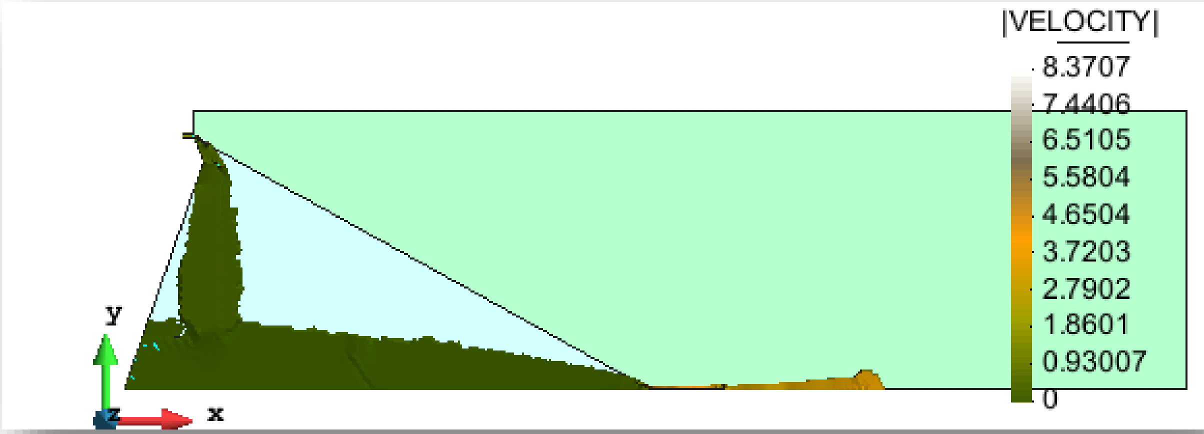


Figure 8.11 Result surface of the simulated distributed velocities in the model 2

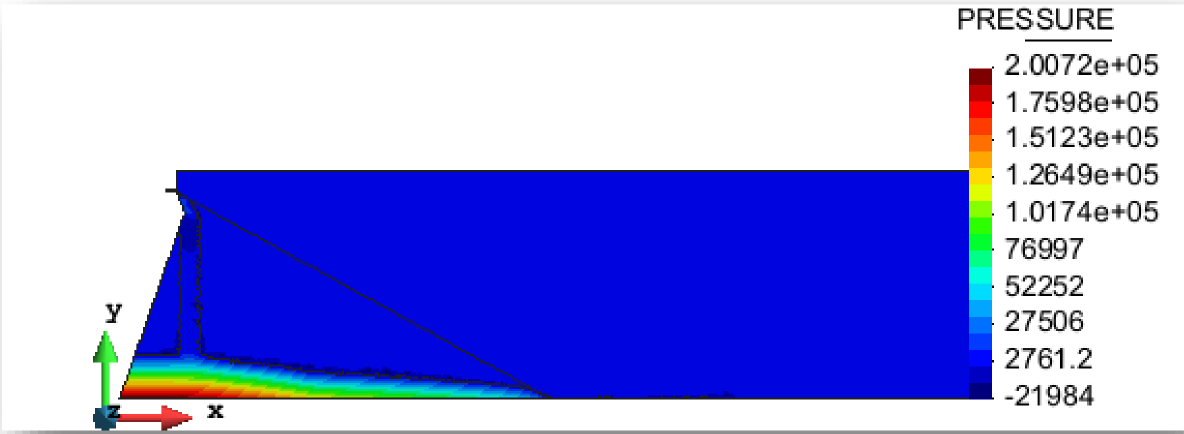


Figure 8.12 Pressure distribution in the numerical model 2

Figures above show the difference in the patterns of the flows in the numerical model. The input flow rate is at different position for the same overtopping discharge. In the model with the complete rockfill dam, the flow rate is fed as input into the inlet opening of 10m. The opening is made to this amount because the velocity will be relatively very low in the case of the small opening. From the model Figure 8.7, it can be seen that the flow rate is very high in the whole dam model 1 and during the overtopping mode, all the water runs downstream. During the overflow, the part of the water overflows downstream and some flows through the rockfill which is shown by the Figure 8.7 . For the second model 2, the inflow rate is fed through 1m of the inlet opening through the top of the dam. From the simulation, it can be seen that the all the water goes through the rockfill and exits at the certain height above the downstream as assumed in the calculation.

Calculation of the exit depth

From the calculation shown in Table 4.2, the height of the water that emerges above the bottom of the dam is calculated as 9.383m above the bottom of the dam. In the first numerical model

1, it is difficult to see the emergent depth above the bottom of dam in Figure 8.13 and it can be estimated to be around 6.924m

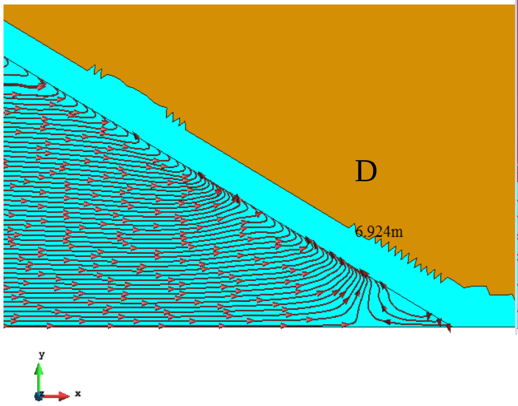


Figure 8.13 Flow vectors at the downstream toe.

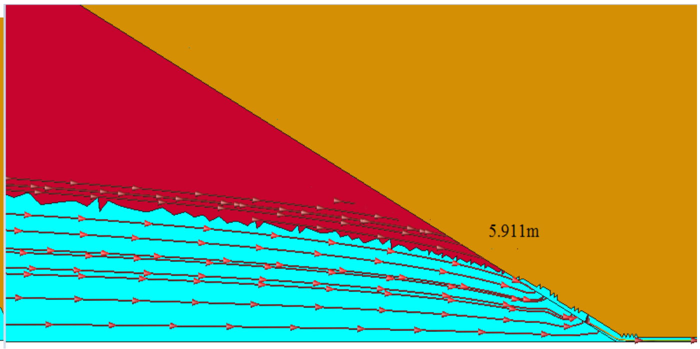


Figure 8.14 Flow vectors at the downstream toe.

For the second model, the depth of the water emerging at certain depth above the bottom of the dam is found to be 5.911m as shown in Figure 8.14. The calculated and the numerical model show the difference in values in the emergent depth. The results might be varied by various parameters. The main cause may be the dimension of the model. The formulae uses the three dimensional flow where as the model being used is two dimensional. Therefore, the study needs to be done in this numerical model.

Validation of the model

To validate the hydrostatic distribution, in the model, two different sections are taken in the model in Figure 8.15 A and B. The hydrostatic pressure is calculated for theses points and the model values are also noted from the simulated value and they are checked with calculated values. The model 2 with partial dam gave quite satisfied result but for the model dam 1, the pressure calculated and simulated is not observed to be the same as the simulated model. The validation of model in terms of velocity is not done but the depth of the emergence of the flow is checked.

Position	node position in model	pressure value in model Pa	depth m	presssure calculated Pa	Variation
A	219.27,0,0	2.0072E+05	20.735	2.03E+05	1,123%
B	412.75,0,0	3.56E+04	3.8	3.73E+04	4,55%
D	407.08	1.26E+00	11.464	1.12E+05	12,5%

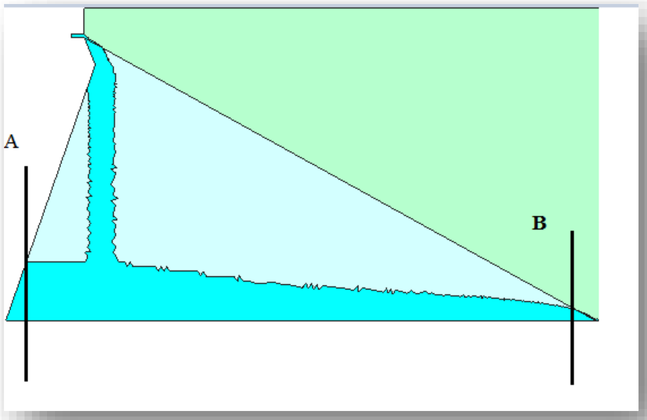


Figure 8.15 Section of the dam

8.3.1 Out comings from the current uses

Some of the researchers have been using this kratos multiphysics and it is one of the innovative approach for the simulation of the rock fill dams. When the fluid module is used, the good correspondence is found between the experimental and numerical pressure heads in both 2D and 3D[55]. The challenging aspect is in the case of the small discharges where it fails to deform. In such cases, the mesh needs to be very fine close to the bottom.

In the case of coupled module, the B parameter, which quantifies the length of the failure of the dam, is less reliable. It is not able to correctly represent the failure when the shear stress decreases under the yield stress threshold. In such condition, the viscosity increases due to the low discharge and the simulation stops.

8.4 Other Computational model for estimating the flow through the rock fill dams

The numerical method, artificial method neutral network (NNGA) and neutral network genetic algorithm (NNGA) were used for the predicting the flow through trapezoidal and rectangular rock fill dams. They were obtained based on the experimental data. The flow through the rock fill does not obey the linear Darcy law due to the large size of pores in dams making it more turbulent. The flow hydraulics is solved by using the numerical methods such as finite volume and finite element. ANN is a set of highly interconnected mathematical processing elements that are capable of modeling and forecasting[44]

The model is based on the 2D continuity equation. It uses the exponential relationship between Reynolds number and Darcy Weichbach friction factor and the spatially varied overtopped flow is combined and used as part of the boundary conditions to the 2D seepage flow model.

$$f = \frac{a}{Re} + \mathrm{b}$$

Eq. 8-1

$$f = a'Re^{b'}$$

The finite volume method is used for the 2D flow seepage equation. Many numerical models are developed and some The NNGA and 2D models are similar and they are superior to the ANN method. They give closer value to the experimental data. Some of the present study is also studying the potential of artificial intelligence for the prediction of flow through rockfill dams.

8.5 Star+ ccm-

The star+ccm- is also used for the numerical modeling of the rockfill dam. However due to many complexities in the flow inside the rockfill dam, it is also not able to simulate correctly the realistic behavior of the rockfill dam.

8.6 Conclusions

From the available use of the numerical model for the throughflow and the overtopping embankment, it can be concluded that those model must be founded more on a sound theoretical base and should be improved for modeling actual system behavior. The full confidence in the numerical approach solution to the problem cannot be fully achieved though it is the most economical and desirable for the analysis and the use. The numerical models work well for the homogeneous and the monolithic structures rather than the structures with the various porosity and the material characteristics. Thus its detailed study should be done. In the Kratos, it is also possible to do the modelling of the rip rap and other various factors. Therefore, Kratos needs to be studied in more detail. Besides that, the numerical analysis can judge the scale effects as it doesn't give the scale effect as the experimental study. In addition, many numerical models have been developed to study and investigate the scaling relationships.[56] Although various studies have provided most of the significant insights into the performance of the rock fill, there is always a need of the quantitative physical modeling of the hydraulic properties and the flow conditions in order to validate the assumptions made during the numerical model.

Chapter 9

9 Discussion and Conclusion

The experimental tests carried by the Norwegian researchers show that the rock fill dam can be prevented by preventing the rock fill toes. This can be the effective and useful protection for the rock fill dams against the failure due to overtopping. The model studies of the riprap on the downstream slope are also carried out in flumes of the NTNU and UPM at Spain. It shows that the various parameters influence the effectiveness of the protection and the limit flows that it can withstand. Several conclusions and the discussions can be drawn into attention with the experiments conducted so far regarding

- Orientation of the longest axis of the stones with the slope of the dam
- Diameter of the stones
- Downstream slope
- The gradation of the rockfill.

- During the overtopping of the dam, the hydraulic loads causes the reaction of the slope protection layer and accordingly of their single element. The possible scenario during the overtopping is
  - erosion of the single stones,
  - sliding of the protection layer
  - disruption of the protection layer caused by lifting forces

  These cause ultimate failure of the protection layer. The possible scenario is the washing out of the fine materials underneath the protection layer, which is not discussed in this part. This is due to the insufficient filter criteria of the dam but the filter criteria is checked for the existing dam and it seems to meet the criteria.

- The proper orientation increases the strength of the rip rap and prevents the disruption of the velocity by reducing the shear stress and the velocity of the flow. The sliding of the protection which is also another scenario during the overtopping, it can also be prevented by designing the toe stones and interlocking the toe stones. Generally in the rockfill dam, the failure starts from the toe so the protection and the interlocking of the toe stones eventually help in the prevention from failure.

- Another major observation is the increase in the strength of the rip rap with the use of the bigger diameter stones. From the results obtained during the tests show that the unit failure discharge increases with the sizing of the stones. The use of the bigger stones gives the better strength for the rip rap.
- Another important discussion is the role of the gradation of the rockfill in the downstream. In the random placement of the dam, the tests are conducted with two different placement of the rip rap; one is with the existing Kulekhani dam which has high coefficient of the uniformity and other with the random placement of the dam for the same downstream slope with low coefficient of uniformity. The failure discharge at the one with high coefficient of uniformity is found to be less in relative to the one with the low coefficient of the uniformity. This shows that the gradation of the rip rap affects the stability of the downstream slope.
- The scale dam tests were done in which the downstream slope is only varied and all other properties of the rockfill being same. From the results discussed above, the failure discharge for the flatter slope is 1.27 times greater than the failure discharge of the steeper slope. Thus, the inclination of the downstream slope can also be considered as the major parameter for the downstream stability.
- The packing factor though can be considered as the parameter influencing the hydraulics of the rockfill but it has no direct influence in the failure discharge and the strength of the riprap as obtained from the laboratory tests.
- The angle of the repose which is another parameter is also studied. There is no clear trend showing the influence of the angle of repose with the failure discharge but the failure discharge increases for the high tangent of the angle of repose. For the establishment of the relation, more detail study needs to be done in this regard. This can be interesting parameter to look at.
- Regarding the drainage capacity of the Kulekhani rockfill dam, the calculations are made based on the existing literature and the texts. When the same rockfill are properly oriented with the proper rip rap orientation, the drainage capacity of the given dam can be increased by more than 12 times the initial drainage capacity.
- Another interesting thing to see is the variation of the placement by different persons. The placement technique methods used by the people and the experience in the placement vary a lot in the rip rap placement in the dam. It can also be inferred that the same person can give the best results and sometimes the bad results.
- The density of the rockfill and the compaction of the rockfill are not considered here in the experiment. Nevertheless they are also another parameter for the variation in the results due to the change in the porosity and permeability.
- Another important part which is the boundary effect is also not considered in the thesis though it has remarkable effect in the porosity in the contact between the rock fill material and the wall of the flume. Some of the dams fail due to the boundary effect. It is also necessary to study in detail the boundary effect and impact on the failure discharge.

Thus, the through flow capacity of the downstream slope of the Kulekhani rockfill dam is studied. From tests, it can be concluded that the existing Kulekhani dam is not able to take the overtopping load in any conditions either it be from the mechanical failure of the gates or the settlement of the dam causing the overtopping. This is mainly due to the lack of downstream slope protection and the poor arrangement of the rip rap. Thus it is necessary to have the proper oriented rip rap in order to have the safe dam. The drainage capacity of the dam can be increased to 12 times from the original capacity only by changing the orientation of the rip rap only. This is very economical and necessary option to be considered. The failure of the Kulekhani dam could be the failure of the hydropower projects in Nepal as it is the only storage hydropower project built so far with such enormous capacity. Therefore, this method of protection is the necessity for the existing condition in Nepal

In addition to it, different model tests are carried out for the different orientation of the rip rap and different person. Thus, it can be concluded that the proper orientation and the proper workmanship is quite necessary and it greatly affects the strength of the rip rap. It is also suggested to have a detail study on the angle of the repose as it affects the existing drainage capacity of the dam.

The throughflow and overflow modeling condition in the experiments give the better condition of modelling. In the case of the random placement of the rip rap, throughflow causes the initiation of the failure at threshold whereas in the case of properly placed riprap, the toe never fails. In the case of the proper orientation of the riprap, the threshold failure does not start from the toe, the failure always occurs from the top. The failure mostly occurs due to the overtopping flow than the throughflow. But in the case of the random rip rap; the failure is initiated by the throughflow indicated by the movement of the toe stones which is enhanced by the combination of the throughflow and the overtopping flow. Therefore the toe stones are greatly protected by the proper rip rap placement in the dam. In the case of the dam, the placement and condition of the rip rap and the angle of the dam slope also plays a vital role in the failure mechanism of dam.

9.1 Recommendations

The present report is mainly based in the experimental approach and site investigations. There are many uncertainties in the theoretical approach to deal with, which can be overcome by the physical modeling. However the scale effects cannot be taken care of in the physical modeling. For this numerical methods can be good solution to see the scale effects. Besides this, it is recommended for the more experiments to be done by changing the different parameters like height of the dam, angle of repose, coefficient of uniformity. The main recommendation would be the rehabilitation of the Kulekhani dam with the proper orientation of the rip rap and more detail study on it. The present condition of the Kulekhani is acceptable unless it is hit by the mechanical failures during probable maximum flood but in any case the overtopping might be very vulnerable leading to the failure.

For the numerical simulation, the Kratos can be studied in more detail to know its application and uses. It is user friendly but it takes lots of computational cost for the simulation. The kratos might also simulate the rip rap if the computational efforts are made by changing the angle of repose in the programming script it uses.

For the laboratory test programs, it would be nice if there are more instrumentation which can measure the piezometric level or the failure surface by means of the laser surface which will help in the better study of the downstream slope and stability analysis. The use of the existing laser techniques can be more interesting to study in detail about the downstream slope and modes of failure and the factors influencing the failure. Besides that most of the stones which are used as the rip rap in the lab does not meet the specified condition necessary in terms of the dimension and the ratio $3/2<a/b<3$, $b/c<3/2$ and $a/c<3$. Therefore, it could be changed to give the better results from the model.

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10 APPENDIX1

Table 10.1 Properties of different zonings of dam

Zonings of dam		dry density ϒd	density of water	wet density ϒwt	sat. density ϒsat	sub. Density ϒsub	internal frictional angle φ	cohesion c	specific gravity
		ϒd gm/cm3	**ϒw gm/cm3**	**ϒ gm/cm3**	**ϒsat gm/cm3**	**ϒsub gm/cm3**	**Φ degrees**	**c t/m2**	**Gs**
Main dam	Random	1.62	1	1.85	2	1	35	1 to 4	2.6129
	rock Quarry zone	1.62	1	1.99	2.21	1.21	41	0.5 to 3	3.951
	Rip rap	1.62	1	1.99	2.21	1.21	41	0	3.951
	Impervious core	1.62	1	1.98	2.06	1.06	30	3 to 6.2	2.893
	filter	1.62	1	1.93	2.08	1.08	33	1 to 4.8	3

Table 10.2 Field data observed in Kulekhani

SN	b	c	a	angle of orientation	d	SF
1	790	500	1160		61.7	0.522309
2	1420	590	1920		93.7	0.35732
3	690	350	1170		52.5	0.389539
4	520	500	800		47.4	0.775217
5	600	430	1710		60.9	0.424517
6	180	100	230		12.8	0.491473
7	1080	750	1740		89.7	0.54711
8	600	390	870		47.1	0.539796
9	690	970	750	69	63.6	1.348393
10	700	1700	680	70	74.5	2.464027
11	390	1050	760	50	54.2	1.928636
12	790	1520	1210	75	90.6	1.554667
13	680	590	770	45	54.1	0.815365
14	1550	1020	2220	60	121.6	0.549867
15	1150	600	1350	68	78.1	0.481543
16	960	520	1170	75	66.9	0.490653
17	650	560	1950	54	71.4	0.497409
19	1030	840	1150	20	79.9	0.771812
20	420	230	510	23	29.3	0.496956
21	650	500	950	40	54.1	0.636285
22	610	210	800	35	37.4	0.300614
23	730	490	1460	50	64.4	0.474633
24	1150	600	1800	90	86.0	0.417029
25	530	420	830	45	45.6	0.633246
26	340	150	470	50	23.1	0.375235
27	650	330	2070	55	61.0	0.284493
28	320	90	150		13.0	0.410792
29	930	520	1320	73	68.9	0.469326
30	430	450	650	60	40.1	0.85118
31	500	1140	1190	65	70.3	1.477904
32	160	90	270	85	12.6	0.433013
33	430	300	800	43	37.5	0.511496
34	270	160	850	27	26.6	0.333986
35	260	350	230	70	22.0	1.431256
36	200	120	220	65	13.9	0.572078
37	330	530	450	35	34.3	1.375348
38	150	90	130	small	9.6	0.644503
39	250	230	44	23	10.9	2.192964
40	560	320	660	60	39.3	0.526361
41	1140	90	1350	50	41.4	0.072548
42	180	500	780	72	33.0	1.334401
43	80	50	340	40	8.9	0.30317

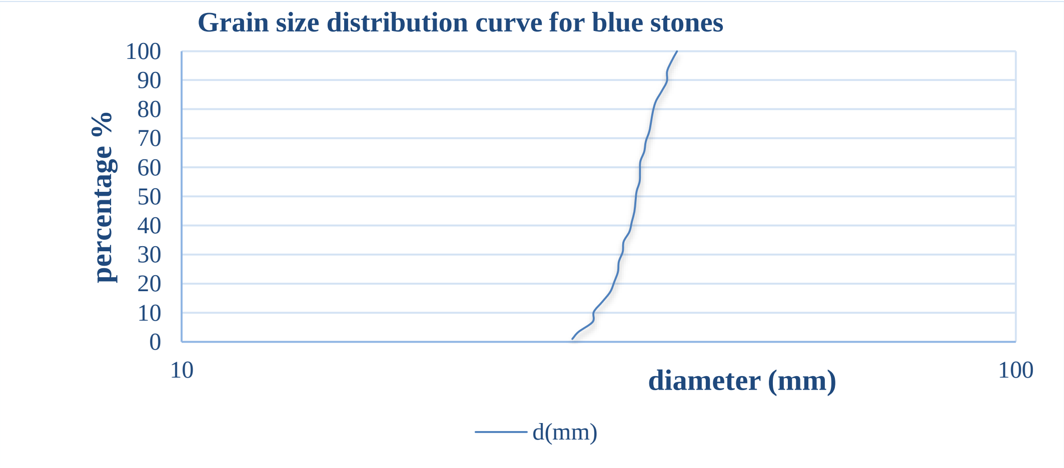
44	220	200	390	65	20.6	0.682789
45	650	420	720	80	46.5	0.613941
46	380	220	390		25.6	0.571477
47	150	135	350	49	15.4	0.589188
48	370	230	450	89	27.0	0.563665
49	535	210	700	25	34.3	0.343157
50	280	150	340	60	19.4	0.486153
51	150	120	150	44	11.1	0.8
52	500	160	720	26	30.9	0.266667
53	180	100	290	15	13.9	0.437688
54	220	100	400	12	16.5	0.3371

Table 10.3 Discharge at different heads during the both gate closing condition during PMF

Head above crest (m)	Water level (m)	Q1gate (m3/s)	Q side (m3/s)	Q dam(m3/s)	Qtotal (m3/s)
0	1530	0.00	0.00		0.00
1	1531	32.03	100.00		132.03
2	1532	89.32	282.84		372.16
3	1533	161.76	519.62		681.38
4	1534	227.41	800.00	0.00	1027.41
5	1535	277.92	1118.03	608.79	2004.74
5.1	1535.1	282.55	1151.74	705.03	2139.33
5.2	1535.2	287.12	1185.78	806.38	2279.28
5.3	1535.3	291.64	1220.15	912.69	2424.48
5.4	1535.4	296.10	1254.85	1023.84	2574.79
5.5	1535.5	300.51	1289.86	1139.74	2730.11
5.6	1535.6	304.86	1325.20	1260.29	2890.36
5.7	1535.7	309.17	1360.86	1385.41	3055.44
5.8	1535.8	313.44	1396.82	1515.04	3225.31
5.9	1535.9	317.66	1433.11	1649.11	3399.87
6	1536	321.83	1469.69	1787.57	3579.10
6.10	1536.10	325.97	1506.59	1930.36	3762.92
5.38	**1535.38**	**295.08**	**1246.89**	**998.02**	**2540.00**

Table 10.4 Grain size distribution for blue stones

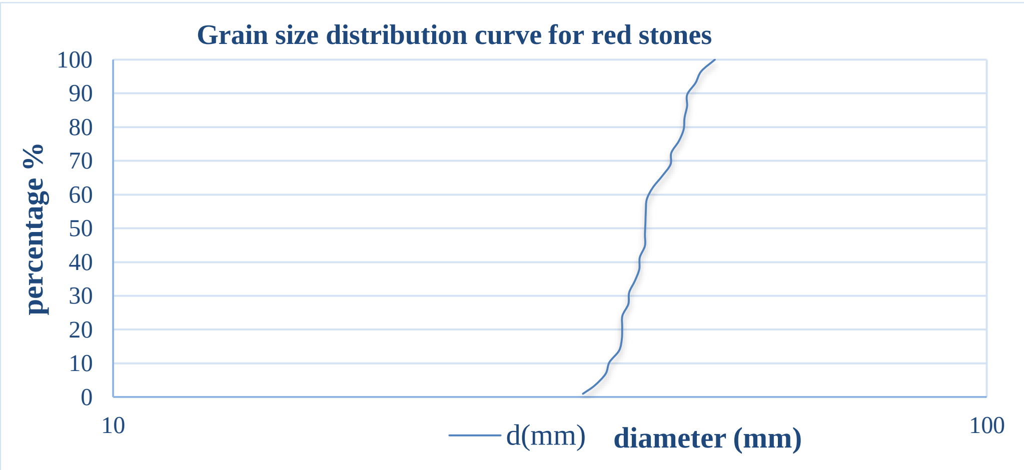
SN	b (mm)	c	a	wt (gm)	d (mm)	%	d(mm)
1	22.8	19.3	57.7	31.4	29.39156	1	29.39156
2	37	24.4	59	52.6	37.62541	3	29.90247
3	40.5	21.7	42.2	45.6	33.34846	7	31.09527
4	37.8	20.3	56.4	48.9	35.10931	10	31.19438
5	34.7	26.1	53.1	42.8	36.36538	14	31.9343
6	30.3	28.1	65.5	51.4	38.20587	17	32.64795
7	35.9	25.1	55	54.9	36.73192	21	33.01333
8	28.4	19.9	53.2	36.1	31.09527	24	33.34846
9	33	20	70.8	51.3	36.01851	28	33.42841
10	37.3	29.7	52.2	46.4	38.67039	31	33.79176
11	32.8	21.2	55.9	42.2	33.87457	34	33.87457
12	35.4	33.8	46.5	55.1	38.17605	38	34.42236
13	38.5	22.3	49.5	45.4	34.89717	41	34.65447
14	31.7	27.2	58.9	48.1	37.03234	45	34.89717
15	28.2	27.2	48.7	45.9	33.42841	48	35.00243
16	36.1	21.4	59.7	57.1	35.86177	52	35.10931
17	35.5	21.4	58.6	45.4	35.44159	55	35.40582
18	31.8	25.1	51.1	42.5	34.42236	59	35.44159
19	39.4	28.5	43.5	49	36.55472	62	35.47788
20	26.4	25.2	58	44	33.79176	66	35.86177
21	35.4	21.8	42.2	38.3	31.9343	69	36.01851
22	34.3	19.7	51.5	42.3	32.64795	72	36.36538
23	38.8	20.2	53.1	53.2	34.65447	76	36.55472
24	41.9	25.3	57.1	59.3	39.26361	79	36.73192
25	34.3	16.8	46.4	40.8	29.90247	83	37.03234
26	35.4	22.9	52.9	54.4	35.00243	86	37.62541
27	33.6	18.9	47.8	32	31.19438	90	38.17605
28	34.2	23.9	54.3	48.1	35.40582	93	38.20587
29	31.5	24.2	47.2	43.1	33.01333	97	38.67039
30	33.3	29.8	45	51.8	35.47788	100	39.26361



Graph 10.1 Grain size distribution curve for blue stones

Table 10.5 Grain size distribution for red stones

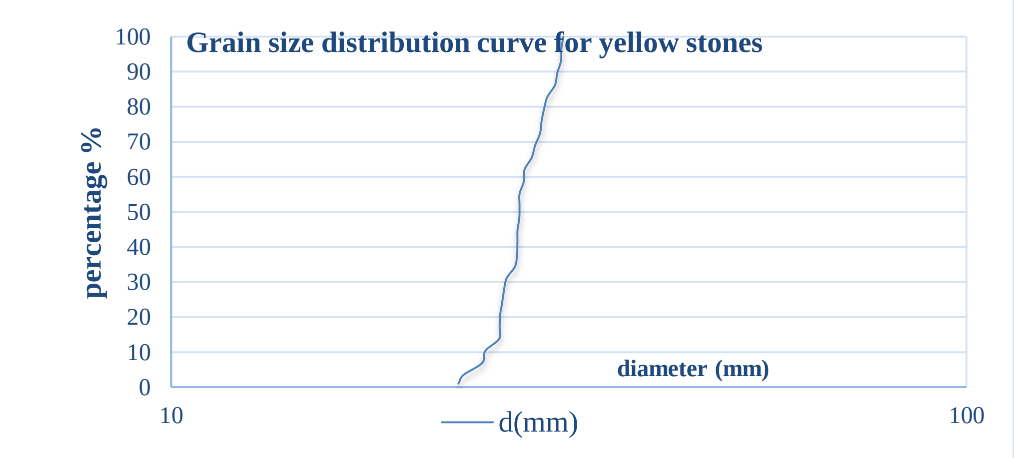
SN	b (mm)	c	a	wt (gm)	d (mm)	%	d(mm)
1	32.3	28.2	64.5	66.1	38.875	1	34.48848
2	43.6	33.6	63.8	88.7	45.38185	3	35.57445
3	37.6	35.1	70.9	87.8	45.39908	7	36.61018
4	33.8	21.8	61.1	64.6	35.57445	10	36.98637
5	37.6	33.8	52.7	64.5	40.6105	14	37.93651
6	32.8	27.2	55	64.4	36.61018	17	38.21368
7	40.7	36.6	61.5	77.8	45.07996	21	38.24893
8	37.9	25.5	57.9	60.8	38.24893	24	38.27271
9	40.2	31.6	53.1	69.1	40.70701	28	38.875
10	39.7	26	62.3	85.2	40.06366	31	38.96096
11	30.5	25	53.8	73.6	34.48848	34	39.55575
12	30.5	26.8	61.9	63.1	36.98637	38	40.01396
13	37.4	35.4	68.7	84.9	44.97217	41	40.06366
14	31.8	29.5	58.2	63.1	37.93651	45	40.6105
15	38.2	31.4	68.8	89.4	43.5372	48	40.61091
16	38.2	26.8	66.4	85.3	40.81208	52	40.66749
17	38.7	32.3	65.6	84.6	43.44493	55	40.70701
18	33.2	33.1	58.3	89.6	40.01396	59	40.81208
19	39.2	21.4	70.5	80.9	38.96096	62	41.4571
20	37.3	30	50.1	73.4	38.27271	66	42.50288
21	33.6	32	51.9	60.9	38.21368	69	43.44493
22	45.2	23.9	62	90.1	40.61091	72	43.5372
23	32.8	32.6	62.9	61.4	40.66749	76	44.40802
24	37.8	32.5	62.5	76.6	42.50288	79	44.97217
25	40.7	39.7	54.2	76.8	44.40802	83	45.07996
26	39.8	34.1	52.5	74.9	41.4571	86	45.38185
27	50	29.4	79.2	130.4	48.82934	90	45.39908
28	43.8	33.5	68.1	89.5	46.40399	93	46.40399
29	36	30.7	56	75.4	39.55575	97	47.11178
30	44.4	36.4	64.7	76.9	47.11178	100	48.82934



Graph 10.2 Grain size distribution curve for red stones

Table 10.6 Grain size distribution for yellow stones

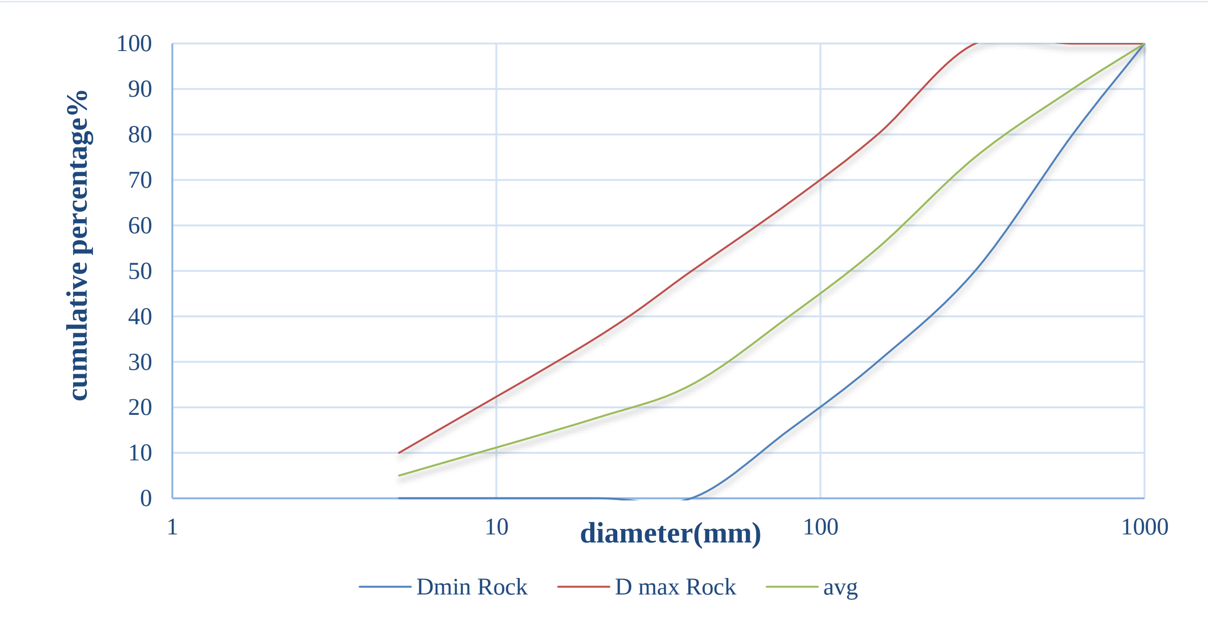
SN	b (mm)	c	a	wt (gm)	d (mm)	%	d(mm)
1	25.9	16.75	41.5	22	26.20923	1	22.97088
2	26	19.1	49.6	27.9	29.09574	3	23.30715
3	27.1	19.9	39.65	27.3	27.75589	7	24.61014
4	25.4	19.3	53.5	26.6	29.71084	10	24.83387
5	27.2	21.9	42.8	25.9	29.43194	14	25.85286
6	30.1	24.2	40.7	28.9	30.94986	17	25.88234
7	26.3	14.3	54.7	27	27.40058	21	25.92423
8	23.3	18.5	46	25	27.06628	24	26.07557
9	31	20.2	45.7	24.1	30.58744	28	26.20923
10	25.2	22.4	41.8	22.5	28.68192	31	26.41002
11	23	21.1	41.7	20.6	27.25098	34	27.06628
12	29.4	25.1	38	23	30.38096	38	27.22279
13	27.7	22.3	37.1	26.8	28.40444	41	27.25098
14	27	18.1	30.5	17.2	24.61014	45	27.25949
15	23.3	18	41.2	17.9	25.85286	48	27.40058
16	25.6	13.7	36.1	17.8	23.30715	52	27.41828
17	27.6	16.7	37.8	20.1	25.92423	55	27.42058
18	32.6	23.6	39.2	24.8	31.12709	59	27.75589
19	19.9	18.3	56.6	23.1	27.41828	62	27.81808
20	28.5	21.2	48.9	27.5	30.91457	66	28.40444
21	22.6	17.2	39.4	18	24.83387	69	28.68192
22	25.2	15.6	45.1	26.3	26.07557	72	29.09574
23	23.5	15.4	50.9	23.4	26.41002	76	29.22326
24	28.8	14.3	42.1	21.1	25.88234	79	29.43194
25	24.9	18	46	19.8	27.42058	83	29.71084
26	29.6	18.6	39.1	23.2	27.81808	86	30.38096
27	26.8	15.3	49.4	28.4	27.25949	90	30.58744
28	30.8	18.8	43.1	22.9	29.22326	93	30.91457
29	24.5	18.8	43.8	19.9	27.22279	97	30.94986
30	24.3	14.5	34.4	17.7	22.97088	100	31.12709



Graph 10.3 Grain size distribution curve for yellow stones

Table 10.7 Particle size distribution of the rockfill

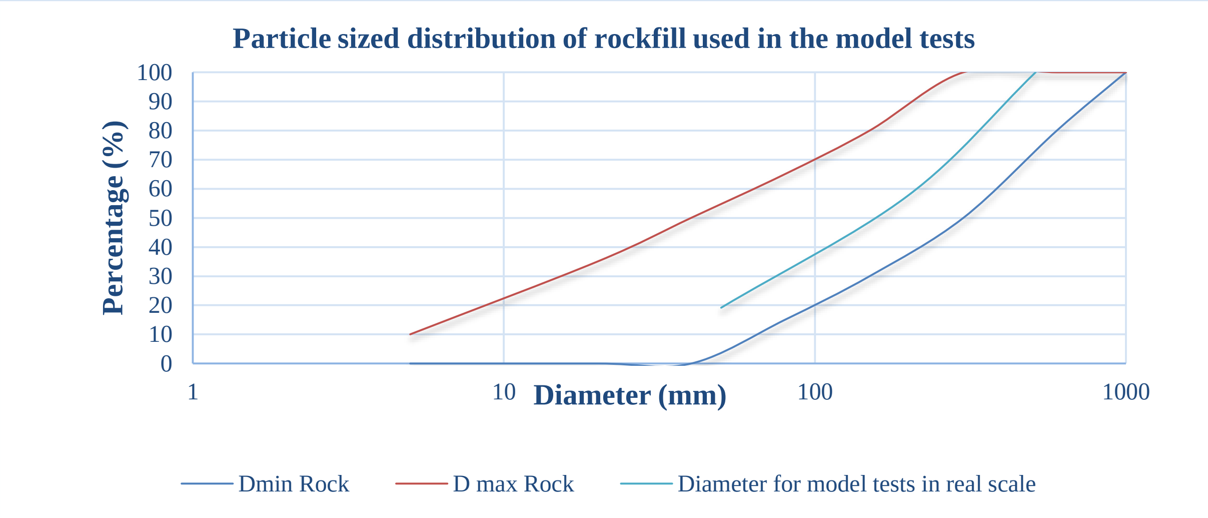
Rock fill				
Material size (mm)	**Summation percentage (%)**	**D_{min}Rock%**	**D $_{max}$ Rock %**	**D_{avg}rockfill**
1000	100	100	100	100
600	80-100	80	100	90
300	50-100	50	100	75
150	30-80	30	80	55
80	15-65	15	65	40
40	0-50	0	50	25
20	0-35	0	35	17.5
5	0-10	0	10	5
D60	190			
D10	9			
Cu	21.11111	>15 so well graded		



Graph 10.4 Particle sized distribution curve of rockfill of Kulekhani dam

Table 10.8 Particle sized distribution for the rockfill used in the laboratory tests.

scale	12.5			
Scaled diameter (mm)	**Diameter (mm)**	**weight kg**	**% by weight**	**cumulative percentage**
512.5	41	10	42.55319	100
198.125	15.85	9	38.29787	57.44680851
50	4	4.5	19.14894	19.14893617
		23.5		



Graph 10.5 Particle sized distribution curve of rockfill used in the model tests

List of excel files

- *drainage capacity computation.xls*
- *field data and zingg diagram.xls*
- *number of tests on 71 and random.xls*
- *readings taken in test.xls*
- *readings taken in test.xls*
- *tests compiles for report.xls*
- *measurement of the density and angle of repose.xls*