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Seepage Assessment of Concrete Face Rockfill Dhap Dam

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Abstract

Dhap dam is the first CFRD type dam under construction in the Gokarneshwor Municipality in Bagmati Province, Nepal. The dam is constructed in the gneiss rock foundation underlain by the colluvium deposit. This paper presents the seepage assessment through the dam with associated geotechnical investigations undertaken during the course of this study. As a part of this study, an ongoing geotechnical investigation organized by the Bagmati River Basin Irrigation Project was supervised and necessary data relevant to this study was collected. The geotechnical investigation consisted of drilling boreholes, Lugeon tests, Lefranc tests and groutings undertaken around the plinth region of the upstream face of the dam. A geotechnical model was proposed based on the information collated from borehole logs. The hydraulic conductivity of the foundation materials was interpreted from the Lugeon tests. Two-dimensional Seep/W numerical models were prepared considering dam sections and foundation materials to estimate the seepage through the dam. The necessary data required in the models including foundation materials and corresponding hydraulic conductivity values are employed from the data collected during geotechnical investigations. Numerical models were analyzed and seepage at the downstream toe of the dam was recorded. Additionally, numerical models were analyzed by incorporating grouting in the models to evaluate the influence of grouting on the seepage performance of the dam. The study revealed that the inclusion of grouting in the plinth region of the CFRD dam reduced the seepage by approximately 6 times than that constructed without grouting. Although there are uncertainties regarding the foundation profile and hydraulic conductivities because of varied foundation material, this study presented an insight into the potential seepage characteristics of the CFRD dam built on the gneiss rock foundation.

Keywords

Seep/W, CFRD, Seepage Assessment, Permeability, Lugeon test, Lefranc test

1. Introduction

A dam is used to impound water for several reasons viz. flood control, water supply for human or livestock, irrigation, energy generation, recreation, or pollution control. Among the various types, the dams that are based on construction material are: Concrete, Embankment, Steel, Wooden and Masonry. Embankment dam can be earthen, rockfill or composite. Among which the dam that is composed of more than one type of material and has a thin impervious core is called zoned type diaphragm dam.

Dhap dam is a diaphragm type zoned dam with a reasonably impervious layer of concrete on the upstream face of the dam so called Concrete face rockfill dam (CFRD). This dam is aimed to impound the water head upstream and raise the existing Chisapani lake to store about 850,000 m^3 of water. The project was initiated to build water impounding reservoir by the construction of a dam at the toe region of the Dhap area within Shivapuri-Nagarjun national park. It is aimed to collect the monsoon rain and discharge the outflow to maintain the flow in Bagmati river during dry seasons [1]. Dhap dam is 24 m high in the deepest section and 172.7 m long.Though the dam is of medium head, ICOLD [2] classified this as a high dam.

A cut-off wall and grout curtain are usually adopted as the foundation anti-seepage measures in water retention dams such as Dhap dam [3]. However, the seepage control system of high CFRD may have anti-seepage deficiencies during both construction and operation. The incomplete and defective nature of the dam face slab during construction and operation phases may lead to seepage through the CFRD [4]. Based on an assessment of more than 30 CFRDs with height ranging from 25 m to 185.5 m, G. Hunter [5] demonstrated that the maximum leakage rate at first filling observed was from 5 l/s to more than 3000 l/s, which reduced during operation.

The proposed study site is located in the eastern part of the Gokarneshwor Municipality in Bagmati Province and almost at the top of Shivapuri Hills. The site lies near the border of Kathmandu district with Sindhupalchowk and Nuwakot districts. The dam site can be accessed by either Chisapani Nagarkot Road or Chisapani-Sundarijal Road and is located at approximately 18 km road distance from Sundarijal. The footprint of the Dhap dam is located at the intersection of the longitudes from 27º48’36”N to 27º48’50”N with the latitudes from 85º27’18”E to 85º27’30”E at approximately 2090 m above sea level. The location of the dam is shown in Figure 1.

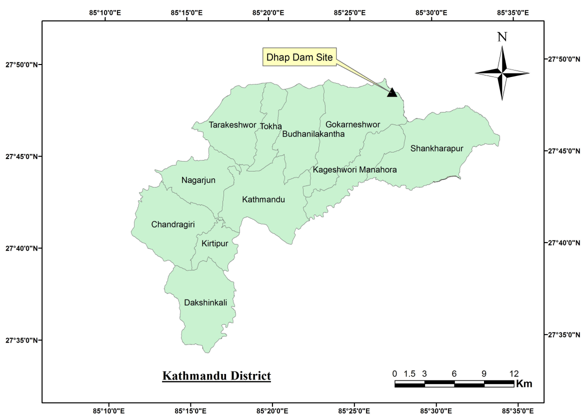

Figure 1: Location of Dhap Dam in Kathmandu district

The topography of the site is relatively rolling around a small lake (the existing Chisapani Lake) which is surrounded by small hilltops reaching up to 2090 m asl, covered by slightly dense forest. Colluvium soil underneath the organic soil is found at the shallow depth. Sub-surface geology comprised of the presence of completely weathered to moderately weathered gneiss rock with some schist bands and granitic intrusions. Lacustrine soil deposit is found on the existing Chisapani lake on the upstream side of the Dhap dam.

1.1 Problem Statement

Dhap dam is the first CFRD in Nepal and almost finished its construction during the study. Because of the complex and irregular geological formation in the dam foundation, there is much curiosity in its performance from the start of the impoundment to the operation phase.

Some of the existing pond formation on the upstream area of the Dhap dam indicated the seepage through colluvium formation. Additionally, the preliminary assessment of the available geotechnical data appeared to be insufficient to evaluate the seepage characteristics of the dam. Thus the requirement to undertake additional geotechnical investigation was felt which would assist in the evaluation of seepage characteristics of the materials encountered in the dam foundation.

Seepage through the foundation and abutment is more prominent than through the main body of the CFRD and may be the main cause of dam failure. Thus, seepage characteristics of the CFRD built on gneiss rock formation need to be analyzed in advance to operate the dam safely throughout its design life without causing population at risk and economic loss in the event of dam. failure, if any.

1.2 Research Objectives

The main objective of this research is to estimate the seepage on the downstream of Dhap Dam. The supporting objective is Geotechnical characterization by site investigations including borehole drillings, Lugeon tests, Lefranc tests and grouting.

Although this study is limited to observe the permeability and seepage characteristics of the Dhap dam, the outcome of the research will form a basis for similar CFRD such as Nagmati Dam proposed to build on the downstream side of the Dhap dam. Based on the preliminary study, the Dhap dam and the proposed Nagmati Dam are sharing similar geology comprising gneiss rock. Thus, this study would provide an insight into the seepage characteristics of CFRD built on the gneiss rock foundation.

2. Literature Review

Concrete dams were ruling since earlier because of the lack of large earth moving equipment. Later in the 18th century construction of rockfill dams started with proper knowledge and technology. The first rockfill dam was constructed in the 1850's in California, America, followed by British which consists of a 24 m high rockfill dam [6].

2.1 Concrete Faced Rockfill Dam

The practice of CFRDs to retain water was commenced from 1920 to 1930 with their height ranging from 80 to 100 m. One of the problems of high embankment dams is cracking of the face slab and joint openings resulting in higher leakage. CFRDs built in the early days were reported to have higher leakage rates and other deformations problems. But CFRD built after late 1960 had a significant reduction in post-construction deformations due to low compressibility of compacted rockfill as well significant reduction in leakage rate. [7]

Reinforced CFRD is a widely used dam because of its several advantages over embankments. Its anti-seepage system comprises mainly body and foundation anti-seepage measures. The anti-seepage body includes a wave wall, a concrete face, and a toe slab and joint water stop for water retention. A cut-off wall and grout curtain are usually adopted as the foundation anti-seepage measures [6]. A typical section of the Dhap dam considered in this study is shown in Figure 2.

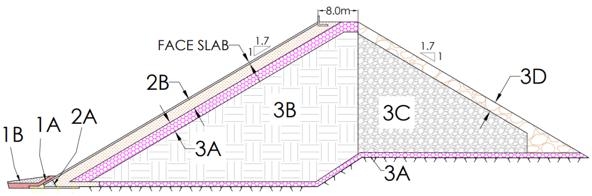

Figure 2: Typical cross-section of Dhap Dam

where,
1A = Cohesion less fine-grained soil
1B = Random Fill
2A = Perimeter zone filter
2B = Processed minus 75mm
3A = Selected rockfill – 0.4m layers
3B = Quarry run rockfill – 1m layers
3C = Quarry run rockfill – 2m layers
3D = Large rock dozed to face

2.2 Dam Failure

Failure of a dam can occur due to deficient geotechnical study before design and reliable seepage control measures adopted in the design. There are three main causes related to dam failure due to groundwater movement viz. piping, uplift and excessive seepage through the dam. Although the CFRD has many advantages, there have also been failure cases due to overtopping and seepage erosion [6]. However, routine monitoring and implementation of remedial measures may assure dam safety [8].

Incomplete seepage control can occur at two stages. The first stage is in construction when the concrete face is not complete. The second stage is during operation when concentrated seepage is induced by the defects in the concrete face and/or the joint water stop. The seepage control system of high CFRD may have anti-seepage deficiencies during both construction and operation.

3. Site Investigation and Data Collection

Site investigation consisted of series of activities including drillings boreholes, Lugeon tests, Lefranc tests and grouting in the plinth area of the foundation of the dam. Each field activity reported in this paper was supervised by the researchers to gain insight into the geological characteristics of the foundation and collect relevant data required for the subsequent numerical modeling.

3.1 Boreholes

Boreholes: A total to 14 boreholes were drilled to the depth ranging from 20 to 35 m using rotary drilling techniques. All the boreholes were logged following standard logging procedures. Each borehole log was prepared at the site and crucial information including core run, penetration rate, water loss, RQD and core recovery were recorded. The subsurface geology encountered in the borehole logs facilitated to produce the geotechnical model of the dam foundation. A typical borehole log for a borehole drilled at the deepest dam section including sub-surface ground profile, core recovery and RQD are presented in Figure 3.

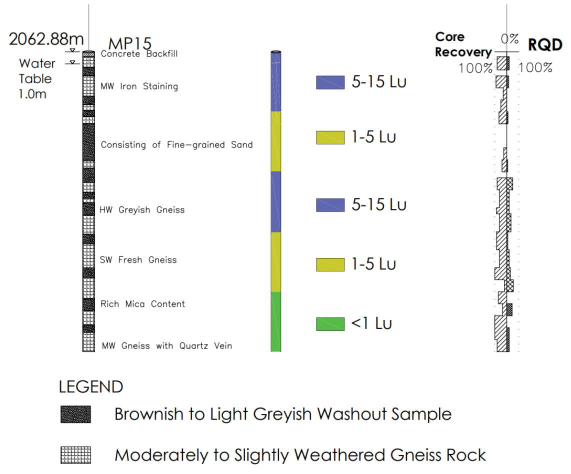

Figure 3: Geological profile and Lugeon test profile along the deepest embankment section

3.2 Hydraulic Conductivity

Two types of field tests were conducted to find the Hydraulic conductivity of foundation materials. Lefranc constant head tests were performed for the colluvium soils and the Lugeon tests were performed for the remaining section where moderately to slightly weathered gneiss rock was found along with the depth of the boreholes. It should be noted that either of the permeability tests were conducted at certain depth intervals along the boreholes.

Lefranc Test: Lefranc constant head test was carried out by measuring the quantity of water loss in the drill hole in a definite interval of time. Prior to the permeability testing, the drill hole was cleaned and flushed out the cuttings of the hole. Dirt-free water was pumped into the drill hole continuously keeping the water level stands at a constant predetermined height. The quantity of water loss in the hole was calculated by measuring the amount of water consumed at a definite interval of time until laminar flow was observed. The coefficient of permeability was calculated using the relation:

$$k = 0.37\frac{Q}{L.H_c}\log\frac{2L}{D} \tag{1}$$

where, k = coefficient of permeability in cm/s; Q = Rate of water consumed (cc/s); L = length of test section (cm); H_c= Average depth of test section (cm); D = Diameter of hole (cm)

Lugeon Test: Lugeon test also called Water Pressure Test was carried out by using a single or double packer in the drill hole. In this method, water flow is confined between the packer and the bottom of the test section. The adopted pressure was 0.5, 1.5, 3, 1.5, 0.5 bar for the tests conducted up to 5 m depth and 1, 2.5, 5, 2.5, 1 bar for the tests conducted below 5 m depth along the borehole. Lugeon value at each pressure was calculated using the relation:

$$L_u = \frac{Q*10}{P_e*L} \tag{2}$$

where, L_u is the Lugeon value (lt./min/m), Q is water amount given to the rock formation (lt./min), P_e is the corrected hydraulic head applied to the test zone (kg/cm^2).

In each Lugeon test, pattern was observed for 5 different pressures as mentioned above as per the pattern presented by Houlsvy, 1978 [9]. All patterns including Laminar, Turbulent, Dilation, Void Filling and Washout were observed, and their representative value was taken. A typical Lugeon profile along the borehole located at the deepest embankment section is presented in Figure 3. It can be observed that Lugeon values decreasing with increasing the depth of boreholes indicating permeability of the gneiss rock decreases with depth.

3.3 Grouting

Curtain grouting and consolidation grouting were undertaken in the toe region of the upstream face of the dam to reduce the seepage potential through the dam foundation. Consolidation grouting was limited to approximately 5 m depth and 2 m spacing under the plinth of the dam thus reducing the seepage potential through the shallow depth of the dam foundation. Similarly, curtain grouting was limited to approximately 15 m depth and spacing varied from 2 to 8 m from primary to tertiary holes.

4. Numerical Modeling

To estimate seepage through the dam, two-dimensional seepage analyses were carried out using the finite element program Seep/W [10] developed by Geo-Slope International. The software calculates phreatic surfaces, pore pressures and flux (seepage per linear meter of embankment length), given user-defined geometry, hydraulic conductivity and boundary conditions. Steady-state seepage analyses were undertaken to estimate the potential development of the phreatic surface through the embankment as a result of retained water on the upstream side.

A total of three dam cross-sections were modeled including one cross-section representing the left abutment region, one cross-section representing the central deepest section region, and one cross-section representing the right abutment region. It should be noted that the deepest cross-section represents a critical section as this section exhibited maximum head once the water surface in the dam is in the designed operational level.

The subsurface geological profile encountered in the nearest boreholes was adopted to create the foundation of the dam in the Seep/w models. Embankment geometry was created based on the actual size of the embankment zones and concrete slab. The hydraulic conductivity values for the foundation materials obtained from the Lefranc test and the Lugeon test were employed in the seepage models. Typical values of hydraulic conductivities were adopted in the Seep/W models for the embankment materials including concrete slab and Zones 3A to 3D. For the simplicity of the analyses, the hydraulic conductivities values were assumed to be the same in horizontal and vertical directions. The hydraulic conductivity values adopted in the Seep/W models are summarised in Table 1. Typical hydraulic conductivity functions produced in the Seep/W models based on the hydraulic conductivity of the materials are shown in Figure 4. The saturated water content values for the materials were adopted from the available database and are summarized in Table 1. Typical volumetric water content functions produced in the Seep/W models are shown in Figure 5.

Table 1: Summary of saturated water content and hydraulic conductivity of materials

Material	Saturated Water Content	Hydraulic Conductivity(m/s)
Zone 2A	0.38	1 x 10^{-5}
Zone 2B	0.39	1 x 10^{-4}
Zone 3A	0.25	1 x 10^{-3}
Zone 3B	0.25	1 x 10^{-2}
Zone 3C and 3D	0.25	1 x 10^{-1}
CW Gneiss	0.46	1.3 x 10^{-6}
MW Gneiss	0.35	6.5 x 10^{-7}
SW Gneiss	0.2	1.3 x 10^{-7}
Concrete	0.5	1.3 x 10^{-12}
Grout		1.3 x 10^{-12}

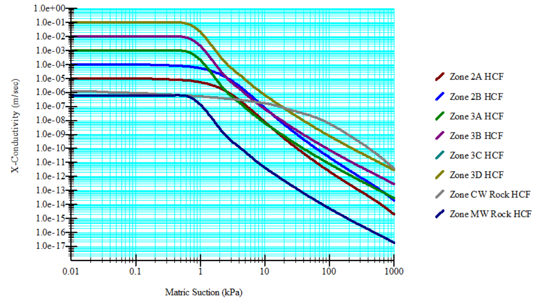

Figure 4: Hydraulic Conductivity function of materials

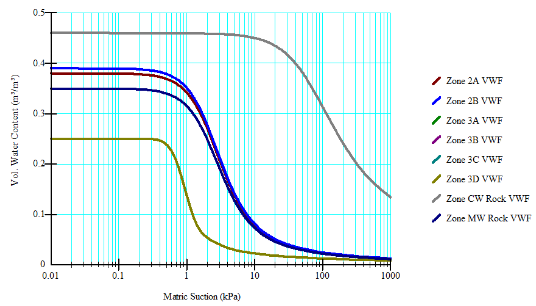

Figure 5: Volumetric Water Content function of materials

For the analyses, a finite element square mesh of 1 m size was adopted to discretize the embankment model. A hydraulic boundary condition representing the designed operational water level was applied on the upstream side of the dam. A potential seepage face boundary condition was applied at the toe region of the downstream face of the dam. Established models were run and seepage through the embankment dams was recorded at the toe in the downstream side of the dam. Total seepage through the dam is estimated by adding the product of seepage through a model and representative length of the dam for all three seepage models.

To account for the influence of grouting in the estimated seepage, models were also run by incorporating 15 m deep grouting in the upstream toe of the dam. Estimated seepage quantities and the observed phreatic lines are discussed in the following section.

5. Results and Discussion

Seepage models were undertaken at three locations of the dam without grouting and with grouting. Typical seepage model outputs along the deepest section of the dam before grouting and after grouting are shown in Figures 6 and 7 respectively. Both models showed that the phreatic surface passed through the base of the upstream concrete slab followed by through the foundation of the dam. The phreatic surface appeared to be in shallow depth below the downstream region of the toe of the dam.

The seepage models along the deepest section of the dam indicated that the estimated seepage before grouting is 1.96 x 10^{-5} m^3/s and after grouting is 3.60 x 10^{-6} m^3/s. The influence of grouting in the seepage behavior of the dam is apparent indicating grouting reduced the seepage through the foundation of the dam. It is noticed that the total water presure head in the foundation of the dam is different with the lower water pressure head observed in the dam model with grouting. In addition, there is a slightly head loss on the downstream side of the grouting

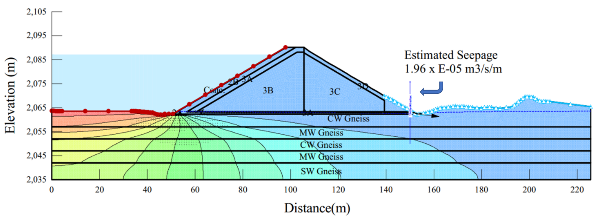

Figure 6: Seepage Model of Dhap dam without grouting

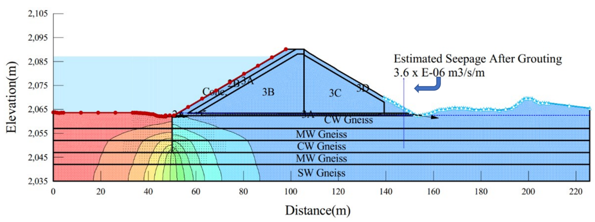

Figure 7: Seepage model of Dhap Dam with grouting

The total seepage through the dam is estimated from the models undertaken along three cross-sections of the dam. A summary of the estimated total seepage through the dam is presented in Table 2.

Table 2: Summary of estimated seepage on the dam

S.N.	Seepage Model	Estimated Seepage (lps)
1	Before Grouting	2.8
2	After Grouting	0.45

It can be observed in Table 2 that seepage is reduced by approximately 6 times when the dam foundation is grouted. This indicates that there is a significant reduction in seepage following the grouting of the dam foundation.

6. Conclusion

Dhap dam is built on the gneiss foundation underlain by the colluvium deposit. The geotechnical investigation undertaken during this study provided an insight into the geological profile of the dam foundation and offered the necessary data required for subsequent numerical modelings. The interpretation of the Lugeon test data indicated that the hydraulic conductivity of the gneiss rock decreases with increasing depth. Curtain groutings were constructed in the upstream plinth region of the foundation of the dam.

Seepage models were analyzed to estimate the seepage through the dam with grouting and without grouting. Based on the numerical models, the influence of grouting in the seepage behavior of the dam is apparent indicating grouting reduced seepage by approximately 6 times. Although there are uncertainties regarding the foundation profile and hydraulic conductivities because of varied foundation material, this study presented an insight into the potential seepage characteristics of the CFRD dam built on the gneiss rock foundation.

Acknowledgments

Bagmati River Basin Irrigation Project (BRBIP) provided an opportunity to collect samples for laboratory testing; supervise site investigation activities including drilling boreholes, Lugeon tests, Lefranc tests and grouting; and collect necessary data relevant to this study. Support and motivation offered by BRBIP personnel during this study are highly appreciated. The outcome from this study is the personal view of the authors and do not take any legal responsibility.

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