



ADDIS ABABA UNIVERSITY
COLLEGE OF NATURAL SCIENCES
SCHOOL OF PLANETARY AND EARTH SCIENCES

PERMEATION GROUTABILITY ASSESSMENT
ON VOLCANI-CLASTIC -SEDIMENTS
(Particularly on Un welded Tuff and Volcanic Breccia);
A case of Kesem Dam Foundation Grouting,
In Afar regional state, Ethiopia

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ABSTRACT

Accurate prediction of groutability of a soil mass by permeation has always been complicated since the effective penetrability of grout in to voids is a function of several interrelated parameters contributed from the physical and mechanical properties of a medium to be grouted, the nature of grout used, efficiency of groutability assessment techniques adopted, and other factors related to grouting design and operation. Special concern on groutability sounds most relevant while dealing with grouting of medium with very fine fractures and/or porosity. Various researchers so far have developed relations or techniques by which the permeation groutability of a soil mass can be assessed. The present research has attempted application of those selected techniques for preliminary permeation groutability assessment on volcano-clastic sediments (particularly on unwelded tuff and volcanic-breccia deposits).

The study was conducted as a case study at Kesem dam and irrigation project located in Afar regional state of Ethiopia. The proposed dam site abutments geology comprises of thin to thickly bedded, pervious and very weak un-welded tuff and volcanic breccia deposits which occur sandwiched in between competent rock units. Though intensive permeation grouting was implemented at both the abutments, it did not bring satisfactory result on those layers. The present research therefore has dealt about permeation groutability evaluation of un-welded tuff and volcanic breccia deposits composing abutments of the Kesem dam project.

The research has followed two basic approaches for evaluation of permeation groutability: one is applying soil permeation groutability assessment techniques, and second is based on insitu condition manifestations. For the former approach, grain size distribution, permeability and porosity of media were studied and used in the analysis. In the later case, the actual foundation permeation grouting response was examined on the basis of evaluation of project records kept for previous actual and test grouting that were conducted in the study area. Besides, indications of an in-situ chemical permeation grouting test (attempted during the present research using sodium silicate grout) were used.

The evaluations have led to identification of suitable grout types viable for effective permeation in to porespace in unwelded tuff and volcanic breccia deposits studied. Effective permeation range of the deposits has fallen within chemical grout zone, colloidal (silicate) solution or any less viscous solutions. Moreover, the dependability of the current research approach, for use on preliminary permeation groutability assessment on Volcanoclastic sediments, was verified since results from both approaches end up with good agreement.

Finally, on the basis of findings of the research, recommendations are forwarded to help on the on going efforts to look for solutions against encountered geotechnical problems of the site.

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TABLE OF CONTENTS

ABSTRACT	i	
AKNOWLEDGEMENT	ii	
TABLE OF CONTENTS	iii	
LIST OF FIGURES, PLATES AND TABLES	v	
LIST OF ANNEXURES	vii	
CHAPTER 1	INTRODUCTION	1
1.0 Overview		1
1.1 Problem Statement.....		2
1.2 Research Objectives		3
1.2.1 General		3
1.2.2 Specific.....		3
1.3 Significance of the Research		3
1.4 Methodology		4
1.5 Limitations and Scope of the Research		5
CHAPTER 2	THE STUDY AREA	6
2.0 Location.....		6
2.1 Previous Studies		7
2.2 Site Characteristics		7
2.2.1 Topography.....		7
2.2.2 Climate		8
2.2.3 Seismicity		9
2.3 Project Salient Features and Current Status		10
2.4 Dam Site and Reservoir Geology		11
2.4.1 Previous Studies		11
2.4.3 Geologic Structures (Faults and Joints).....		14
CHAPTER 3	LITERATURE REVIEW	18
3.0 Preamble.....		18
3.1 Grouting.....		18
3.1.1 Purpose		18
3.1.2 Type and Components of Grouts.....		19
3.1.3 Injection Techniques		21

3.1.4	Groutability.....	22
3.1.5	Closure Criteria	26
3.2	Volcano-clastic Sediments Composing Kesem Dam Site	27
CHAPTER 4		29
DAM SITE GEOTECHNICAL PROBLEMS AND FOUNDATION GROUTING.....		29
4.0	Preamble.....	29
4.1	Groundwater Condition and Related Problems.....	29
4.2	Problems Related to Incompetent Layers.....	30
4.3	Kesem Dam Foundation Grouting.....	33
4.3.1	Design Aspects	33
4.3.2	Current Status	34
CHAPTER 5		37
EVALUATION CONSIDERATIONS,TECHNIQUES AND DATA INPUTS		37
5.0.	Preamble.....	37
5.1.	Basic Consideration.....	37
5.2.	Evaluation Techniques	37
5.3.	Data Types and Sources	39
5.3.1.	Laboratory and Insitu Tests.....	39
5.3.2	Grain size Distribution of Suspension Grout Components.....	44
CHAPTER 6		46
PERMEATION GROUTABILITY EVALUATION.....		46
6.0.	Preamble.....	46
6.1	Groutability Evaluation Based on Permeation limits	46
6.1.1.	Groutability Evaluation Based on N.....	47
6.1.2	Groutability Evaluation Based on Karol (1985) and Littlejohn (1985) Permeation limits 48	
6.1.3.	Groutability Evaluation Based on Void size and Dmax in Grout	50
6.2.	Evaluation Based on Insitu Condition Reactions	51
6.2.1	Previous Normal Portland Cement Permeation Grouting Evaluation	52
6.2. 2.	Insitu Permeation Test Indications	57
6.3.	Overall Groutability Evaluation	59
CHAPTER 7		60
CONCLUSION AND RECOMMENDATIONS.....		60
7.0	Conclusion.....	60
7.1	Recommendations	62

REFERENCES66

LIST OF FIGURES, PLATES AND TABLES

Figures and Plates

Fig. 2.1 Location map of study area (Kesem Dam project area) 6

Figure 2.2 Surface topographic map of the study area (source: WWDSE, 2005a)..... 8

Fig. 2.3 Prominent head work structures built at the study area and their current status..... 10

Fig. 2.4 Main dam layout and location of investigation boreholes (Source: WWDSE, 2008) 12

Fig. 2.5 geological cross-section along the dam axis AB (m: WWDSE, 2008) 13

Fig. 2.6 Geological map of the dam site and the surrounding area (source: WWDSE, 2005a)..... 15

Figure 3.1 typical properties of grout types (adapted from Littlejohn, 1985)..... 20

Figure 3.2 soil and grout materials grain size curves (adapted from U.S. ACE, 1984)..... 23

Figure 3.3 (a&b) limits of groutability based on Karol, 1985 (source: Fell R. et al, 2005)..... 24

Figure 3.4 range for effective and feasibility of grout types and grouting methods (source: Keller, 2005) 25

Figure 3.5 Limits of injectability of grouts based on the permeability of sands and gravels (Littlejohn, 1985) .. 25

Figure 3.6 grain size range for permeation groutable soils by solution (Baker, 1982)..... 25

Figure 4.1 Kesem dam foundation grouting holes pattern..... 34

Figure 4.2 Curtain grouting layout of Kesem Dam foundation grouting (source: WWDSE 2005c)..... 35

Fig. 4.3 Temporary Claybackfill at the riverbed section for grouting before excavation..... 36

Fig. 5.1 Gradation summary of reddish brown and yellowish tuff samples 39

Fig. 5.2 Gradation summary of pummicious tuff 40

Fig. 5.3 Gradation summary of volcanic breccia..... 41

Fig. 5.4 Insitu permeation test model 43

Fig. 5.5 Grain size distribution of suspension grout components 44

Fig. 6.1 permeation evaluation based on N 47

Fig. 6.2 Permeation groutability evaluation based on Karol (1982) and Littlejohn (1985)..... 49

Fig. 6.3 Graphical interpretation of groutability for void 51

Fig. 6.4 Lugeon and grout take improvements for progressive sequences 53

Fig. 6.5 Location of test grouting at the right	54
Fig. 6.6 Geology of test zones at the right abutment	54
Fig. 6.7 Right abutment test grouting Lugeon achievement summary	55
Fig. 6.8 Pressure filtration evaluation.....	55
Fig. 6.9 Geology of test section (left abutment)	56
Fig. 6.10 Effect of high pressure permeation grouting on tuff and volcanic breccia.....	56

Plate 2.1 Lithology exposed at dam site (left abutment)	14
Plate 2.2. Major and minor faults in and around dam site and their interconnection for leakage path.....	16
Plate 4.1 Artesian groundwater at the dam site	29
Plate 4.2 Red bole at the river bed channel section of the dam foundation	31
Plate 4.3 Incompetent layers exposed on abutments (A= yellowish and reddish tuff, B= Pummicious tuff, C= Volcanic breccia and D= paleosol).....	33
Plate 5.1 Insitu density test on tuff	41
Plate 5.2 Insitu falling head permeability test on tuff.....	42
Plate 5.3 Insitu permeation test.....	43
Plate 6.1 Grouting induced hydro-fracturing on tuff (left abutment)	56
Plate 6.2 Showing status of recovered tuff samples after insitu permeation injection of sodium silicate:	58

Tables

Table 2. 0-1 Seismic record in and around the study area (Source: Kesem dam and irrigation project report) ..9	
Table 3.1 Curtain grouting Closure criteria (Houlsby, 1985).....	26
Table 3-2 Upper limit of grout absorption (Deer, 1982)	26
Table 3-3 Grain size and corresponding terminology for Volcanoclastic sediments.....	27
Table 5-1 Laboratory test result summary.....	42
Table 6-1 Groutability ratio (N) values as obtained for normal Portland cement, Microfine cement and	47
Table 6-2 Considerations for permeation grouting limit of grouts (adapted from Karol, 1982; combined	49
Table 6-3 Soil class, permeability and effective grain size of tuff and volcanic breccias	50
Table 6-4 The effective diameter of average pores in reddish brown, yellow and Pummicious tuff	51
Table 6-5 D_{100} of considered suspension grout components	51

Table 6-6 Summary of Grout hole sequence and spacing for each pattern along with corresponding average grout take, and maximum grouting pressure applied (Source: KDIPR 2008).....	52
Table 6-7 Lugeon statistics for left abutment pattern 2 gorge area after quinary sequence grouting (KDIPR, 2008).....	53
Table 6-8 Summary of insitu permeation grouting test record	58

LIST OF ANNEXURES

Annexure I summary of dry density, specific gravity and porosity of tested samples.....	70
Annexure II summary of grain size and permeability of tested samples	71
Annexure III - (A, B, C) Insitu falling head permeability test records	74
Annexure IV -Insitu density test record.....	75
Annexure V- Insitu chemical grouting test record.....	76
Annexure-VI Geological cross-section along the dam axis AB (A3 paper size).....	78

CHAPTER 1

INTRODUCTION

1.0 Overview

In recent years, the amount of land which is favorable for construction purposes have shown a steady decline, as evident by the number of construction projects broadened over the land and the sea, deepened in to the ground, and raised high in to the sky (IS-Tokyo, 1996). The need to improve unfavorable ground conditions to make it suitable for construction poses a new challenge to engineering geologists/ geo-technical engineers. On the other hand, civil engineers, with the increasing use of the advanced technology in construction engineering, are able to make detailed and sophisticated structural designs with intention to serve safely and effectively to the life time planned. In order to establish construction which can satisfy such conditions, the ground must first be suitable for a structure to stand on it. This is, therefore, emphasizes the fundamental role of foundation engineering in construction, and the degree of relevance of ground improvement. Nowadays, several ground improvement techniques are in use. Grouting, the injection under pressure of a liquid or suspension into the voids of a soil or rock mass or into contact voids between these materials and an existing structure (US ACE, 1984), is one of such ground improvement methods. It has been widely applied since more than a century to improve the engineering performance (increasing bearing capacity, reducing deformation and reducing permeability) of materials forming foundation of various engineering structures (dams, buildings, bridges). Besides, it is applicable in stabilizing soil and rock mass composing unstable slopes, mitigating liquefiable soils, facilitating underground excavations, and to fill the contact void between existing structures and ground, and voids formed in civil structures (cracking, pores or cavities in concrete). Curtain grouting, the most widespread type and application of grouting, is done to reduce the permeability of formations by forming thin barrier against water flow under a dam or in to area of excavation (US ACE, 1984).

Depending on the type and nature of material to be grouted, various modes of grout injection have been practiced worldwide. The intension of this research is specific to permeation grouting, the process of filling joints or fractures in rock or pore spaces in soil with grout without disturbing the formation. Accurate prediction of groutability of a medium by permeation has always been complicated as the penetrability of grout in to voids is a function of several interrelated parameters contributed from the medium to be grouted (size of voids, permeability and strength), the nature of the grout (type and rheology) and the grouting pressure used (IS-Tokyo, 1996).

Various researchers have generated different relations based on which proper preliminary assessment of groutability of a medium can be made. Accordingly, it is possible to find out whether a medium could be groutable or not by permeation. Besides, the type of grout which could be viable and feasible can be assessed from the relations available.

The present research work was carried out at Kesem Dam and irrigation project, located 225 km east of Addis Ababa and 40 Km NW of Metehara town. The project involves construction of a 95 meters high gravel-rock fill dam to impound half a billion cubic meter of water to irrigate 20,000 hectare of land for sugar cane plantation (WWDSE, 2005). Since the project is located in the southern end of Afar depression close to the main Ethiopian rift, the dam foundation geology is known for its complexity and intensive fracturing. Accordingly, the Dam foundation is not watertight, and unsatisfactory to support the overlying load in its natural state. Thus, to improve the engineering property of the foundation materials, reduce water seepage through the foundation, and to improve the foundation material bearing capacity, dam design has incorporated foundation treatments, one of which is normal Portland cement permeation grouting. So far, on both abutments, intensive curtain grouting (by permeation) have been carried out integrating very closely spaced rows of grout holes. Accordingly, considerable reduction in permeability was attained; however, the anticipated watertightness could not be fully satisfied (KDIPR, 2008).

1.1. Problem Statement

Several factors control effectiveness of a grouting project. The major ones are; (i) penetrability of the grout in to voids, (ii) the radius of influence of penetration and (iii) the quality of the grout in view of reopening and durability questions.

Penetrability and radius of influence are interrelated and are both reliant on compatibility between grout fineness and void sizes of pore spaces or fractures to be grouted, the amount of grouting pressure applied and rheological properties of the grout used (viscosity and cohesion). Reopening of grout-injected-voids would result if the grout is characterized by excessive percentage of bleeding or decantation (recommended not being more than 4%, Deere, 1982).

The durability of grout paste depends on the stability of the grout (Houlsby, 1977, 1978, 1985). Besides; the rate of dissolution of grout curtain is greatly accelerated by the presence of aggressive water. Conversely, if the presence of aggressive water is known or suspected, the use of a relatively inert cement-pozzolan grout would be expected to produce a grout curtain that is resistant to leaching by chemical attack; use of this type of grout should be mandatory (Houlsby, 1982).

As per the KDIPR (2008), at Kesem dam site the probable cause of problem of groutability on abutments is related mainly to difficulty of grouting inter-granular porosity of the incompetent units (unwelded tuff and volcanic breccia deposits) and very fine fractures on hard rock units. Therefore, on the basis of this problem, the current research aims at attempting permeation groutability evaluation on unwelded tuff and volcanic breccia deposits emphasizing to question of compatibility of grout fineness in relation to void sizes of medium to be grouted.

1.2. Research Objectives

1.2.1. General

- ✓ Assessing permeation groutability of volcano-clastic sediments particularly unwelded tuff and volcanic breccias deposits that compose Kesem dam abutments.
- ✓ Verifying the applicability of soil permeation groutability assessment techniques for use in preliminary permeation groutability assessment on volcano-clastic sediments
- ✓ Identification of viable grout type if permeation is positive.

1.2.2. Specific

- ✓ Analyzing the grain size and permeability of the tuff, and volcanic breccia groundmass with respect to groutability limits for a particular grout type by permeation
- ✓ Checking whether groutability ratio (N) is in the groutable range for particulate grout considering normal Portland cement, micro fine cement and bentonite grouts
- ✓ Estimating the effective diameter of the average pore in tuff and comparing to the maximum size of grains in grout
- ✓ Correlating the semi-analytical analysis outcome with the practical situation responded by the actual dam foundation on the basis of evaluation of records kept for actual and test grouting previously executed in the project, and based on evaluation of current research insitu permeation test indications
- ✓ On the basis of analysis results, suggesting possible means by which the engineering property of the incompetent layers could be improved.

1.3. Significance of the Research

In most of the volcanic terrains, it is common to find thin to moderately thick unwelded tuff layers and paleosol sandwiched between sound rock units. When such formations form a dam foundation, they can form potential leakage path and can also cause instability of the structure. On the other hand, they may not be groutable in ordinary standards thus requires special attentions.

The present research showed dependable methodology to make preliminary permeation groutability evaluation on such volcano-clastic debris. Understanding on groutability response of Volcanoclastic sediments in advance would have a meaningful input on giving basis for a test grouting program considerations; moreover on minimizing the risk of unnecessary wastage of time and economy while attempting trial and error groutability evaluation during the progress of the actual grouting work. Furthermore, the present study forwarded the relative feasible and practical solutions to improve the engineering performance of the incompetent layers composing Kesem dam abutments.

1.4. Methodology

The findings of the current research were born through an organized and systematic methodology followed. Broadly, four major activities briefed hereunder were addressed to fulfill the preset objectives of the research;

- ✓ Literature review
- ✓ Field work
- ✓ Laboratory tests on collected samples
- ✓ Data analysis and Interpretation of results

The literature survey has encompassed both published and unpublished reports of investigations, case studies, text books and Journals that were obtained from different sources for better understanding of aspects related to the current research topic such as: building the framework concepts on grouting, available standard specifications, integration of various adopted methodologies on previous studies and collection of secondary data.

The field work has involved four major activities: Collection of representative samples for desired laboratory tests, insitu tests for permeability and density, permeation groutability test on tuff using sodium silicate grout, and review and analysis of data obtained from the ongoing project grouting works. Hand augers were used while drilling holes for testing permeability and groutability, and the sand replacement method was used for insitu density testing.

Laboratory tests, on collected samples, for permeability, grain size and porosity have been carried out in the Water Works Design and Supervision Enterprise (WWDSE) Central laboratory service in Addis Ababa. The combined grain size analysis (sieve and hydrometer) was conducted for each sample. The British Standard (BS) and American Society for Testing and Material (ASTM) standards were used for the testing procedures and analysis.

The present research has made use of both secondary and primary data for analysis and interpretation. Existing grain size data for the currently targeted tuff that was made for use by WWDSE, grain size distribution for cements (OPC, PPC and MC-500) obtained from Ethiopian Mughher Cement Factory and from various literatures review, and the Kesem project previous grouting data records were among the major secondary data used for the analysis in this research. Besides, primary data generated from laboratory test of collected samples and from insitu tests during the current research were also used.

The analysis of data and interpretation of results has followed selected soil permeation groutability assessment techniques provided by previous researchers. Besides, correlation was made for results

from the adopted techniques themselves and with the actual project permeation grouting record and insitu model experimental result observations and indications.

Softwares such as AutoCAD, ArcGIS, ERDAS, Global MAPPER, Google Earth, PHOTOSCAPE and SPSS, Microsoft offices (word and excel) were used as an assisting tool in this research.

1.5. Limitations and Scope of the Research

All efforts were being made to carry out the present study in a very systematic and organized manner, well supported by the actual field data, laboratory tests and the secondary data obtained from various sources. However, these efforts were made under the limitations of time, resources and the financial constraints.

In the research, in one of the approaches, the research method utilized correlation for the effective average pore size of tuff with the maximum grain size (D_{max}) of grout grains where the pore size was determined empirically. However, it would have been much more reliable if direct examination of pore sizes with scanning electron microscope was made. This was not possible for the present research because of the non-availability of the device in the country.

Further, non availability of grain size data in grout form compelled to use grain size data of dry powder form of cements during the present study. The analysis and interpretation would be more realistic if the grain size of suspension grout was analyzed and used. However, according to Fell et al. (1992) tests with water: cement ratios ranging from 5:1 to 0.5:1 (by weight) showed that the different proportions didn't have a significant effect on the resulting gradation of the grout particles. Besides, it was noticed that there was no significant difference with respect to grain size coarser than the mean diameter (D_{50}) of the powder cement and D_{100} of the resulting grout. However, the finer cement particles aggregate together in water giving a coarser effective size (D_{15} or D_{10}) than the dry cement powder. The permeation analysis of current research has used grain size of grout coarser than the mean diameter. Thus, as frame to the research scope, further enrichment with additional input data and direct examinations would make the findings of this research more dependable and practicable.

CHAPTER 2

THE STUDY AREA

2.0. Location

The study area, Kesem Dam and Irrigation Project, is located at the southern end of the Afar depression (rift) in Afar regional state, 225km East of Addis Ababa and 40 Km NW of Metehara town (Fig.2.1).

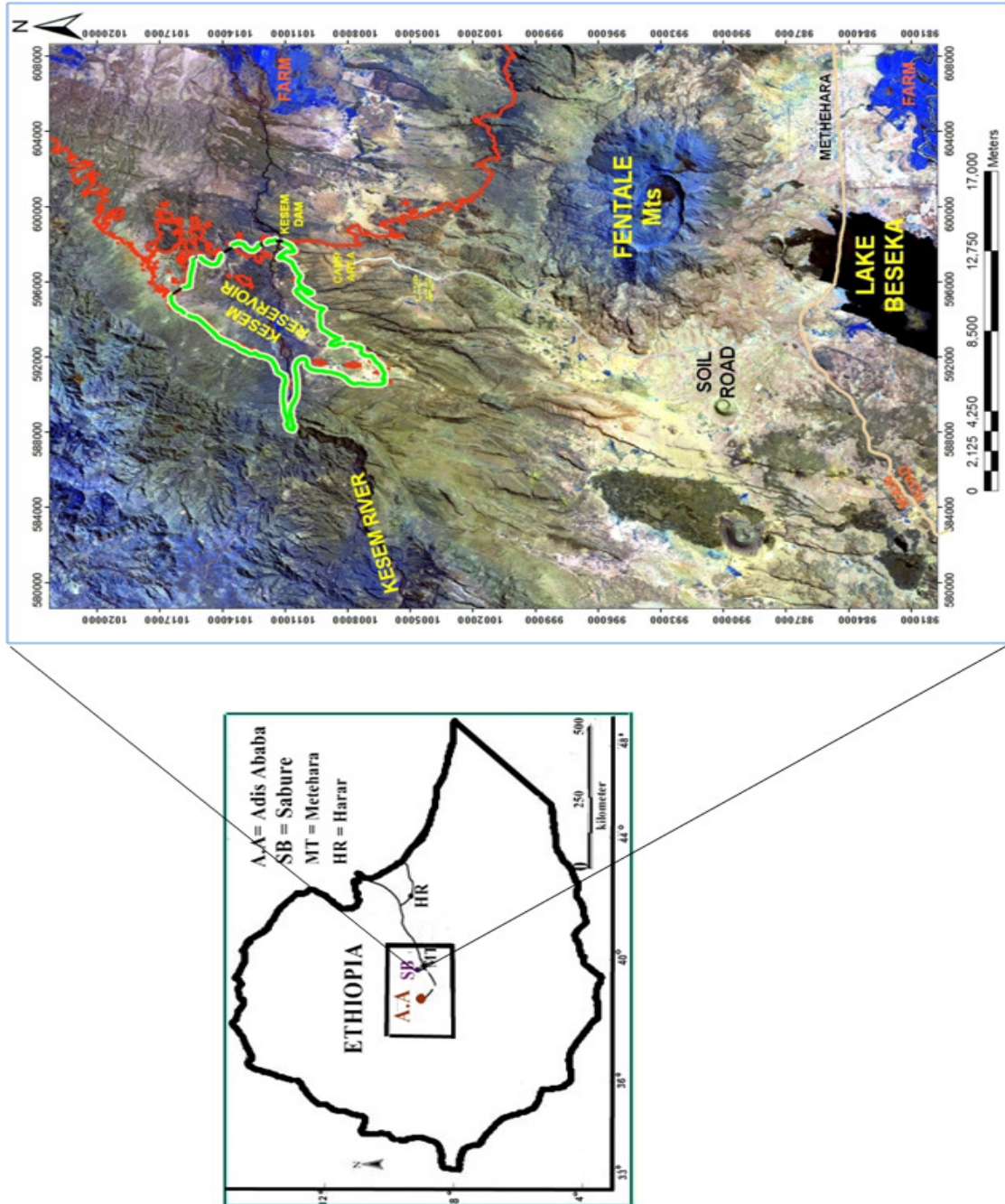


Fig. 2.1 Location map of study area (Kesem Dam project area)

It lies in between UTM 37 zone coordinates of 580000, 608000mE and 9810000-1020000mN covering 200 Km² of area in western part of Sabure sub-sheet (index name, 1:50,000 scale topographic map). The project involves construction of dam to impound half a billion cubic meters of water in order to irrigate 20,000 hectare of land for sugar cane plantation (WWDSE, 2005).

2.1. Previous Studies

With respect to the development of Awash basin many studies have been carried out by different agencies for about five decades. Some of these studies highlighted that Kesem-Kebena plain on left bank of Awash, in the middle Course, was among the areas identified for potential irrigation development.

These studies are listed below;

- Report by SOGREAH for FAO, 1965 “Survey of the Awash Basin”.
- Updated profile on Kesem Irrigation Project prepared internally by the WRDA/FAO team 1985.
- A Feasibility study for Kesem Irrigation Project by Sir Mac Donald & Partners Ltd (1987).
- Master Plan for the Development of Surface Water Resources in the Awash Basin by HALCROW 1989.
- Kesem Dam and Irrigation project by Water works Design and Supervision Enterprise in association with Power and Water Consulting service (2005).

Among aforementioned studies carried out in and around the study area, the feasibility study (MoWR,1987) and detailed geotechnical studies by WWDSE (2005) are considered more relevant to the present research which provided sufficient understanding on site characteristic, geological aspects and project features.

2.2. Site Characteristics

2.2.1 Topography

As cited in WWDSE (2005), Kesem River forms a part in the Awash River basin forming wide “Rift Valley” or “graben”, opening out in the form of a wide “V” towards Red sea. The prominent features of valley are steep slopes linking the plateau towards north, west and south ends with gorges and canyons in the central area (Fig.2.2).

The channels are narrow and highly sinuous at higher levels before cascading into narrow flat floored trenches. The dam site lies on a steep sloped valley. The topography of the site has been shaped by tectonic activity with subsequent erosion. The slopes on the valley walls range 70° to 85° forming

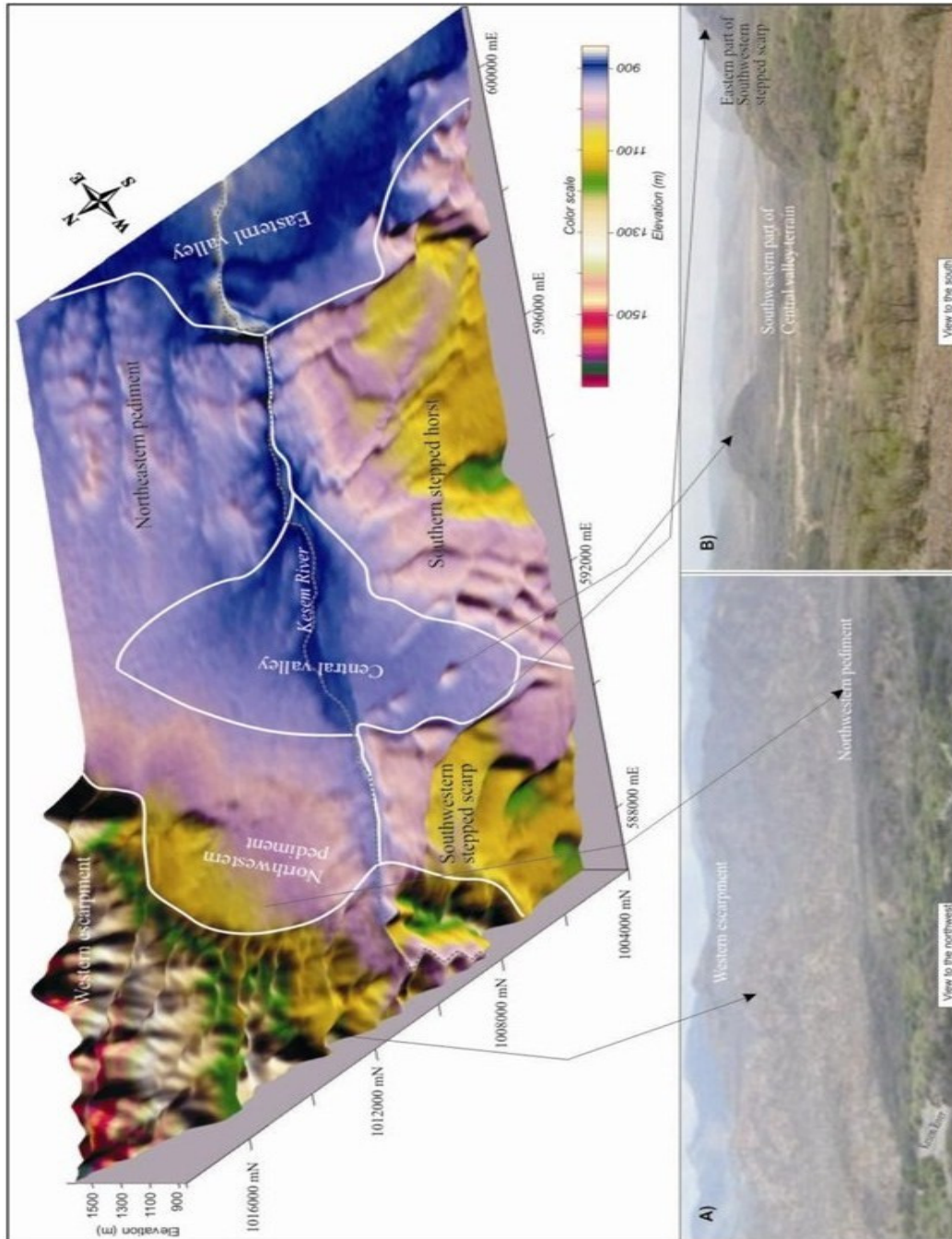


Figure 2.2 Surface topographic map of the study area (source: WWDSE, 2005a)

plateau surface at the top of the abutments. The study area has a wide variety of topographic features ranging from Moderate to high hills, faulted lava platform steps, dissected gorges and high volcanic pediments hills, an upland plateau, and escarpments, deep dissected gorges.

2.2.2 Climate

The Awash Basin is under the influence of Inter-tropical Convergence Zone (ITCZ), which produces a rainfall distribution characterized by two distinct wet seasons, Spring (February to May) and Summer (July to September), in the northern plains of the Basin (HALCROW, 1989).

As per WWDSE (2005) the rainfall within the basin increases three fold with the rise in altitudes from the Awash basin into the highland plateau. Though this increase is asymmetrical, the north of the basin being much wetter and the evapo-transpiration (ET) increases as the altitude decreases.

The climate of the Kesem dam site is hot and semi-arid to arid (mean maximum of 38°C and mean minimum of 15°C) climatic zone with very low rainfall (Mean annual rain fall of 899.3mm). The elevation at the dam site varies from 850m to 1040m above mean sea level. However, in the Upper Kesem watershed, the climate is cool and moist with elevations variation between 1,500 and 2,800m above mean sea level.

2.2.3 Seismicity

Due to its geographical location in the main Ethiopian Rift, the site is characterized by active faults and susceptible to considerable seismic activity. According to the seismic hazard evaluation carried out by Geophysical Observatory of Science Faculty, Addis Ababa University (2004), the area forms a part of the rift valley and is famous for its seismic and volcanic activities. The earthquake events are concentrated in the rift valley area, north and north east of Addis Ababa.

A perusal of list of earthquake occurrence within 200km of Kesem dam site indicates 17 events varying in magnitudes from 3.9 to 6.7 (Table 2.1). Two events of occurrence in 1989 fall close to Kesem dam site and magnitudes of these events were around 5. It is reported that in early part of 1981, some 1200 earthquake of different magnitude occurred in the immediate vicinity of the dam site (WWDSE, 2005).

Table 2. 0-1 Seismic record in and around the study area (Source: Kesem dam and irrigation project report)

S.No	Day of occurrence	Earthquake Epicenter Location		Magnitude
		Latitude	Longitude	
1.	1961/06/01	10.6	39.3000	6.7 mb
2.	1964/07/03	11.0500	39.7000	4.9 mb
3.	1971/11/13	11.0273	39.7133	5.1 mb
4.	1974/02/25	10.1927	39.8325	4.6 mb
5.	1975/08/23	10.5190	39.7544	4.9 mb
6.	1977/07/08	11.0798	39.6241	5.0 mb
7.	1958/08/14	8.2836	38.6986	4.8 mb
8.	1985/10/28	9.4663	39.6105	4.6 mb
9.	1989/05/11	9.0114	39.9661	5.0 mb
10.	1991/07/26	7.5587	38.2829	4.2 mb
11.	1993/02/13	8.3067	39.3160	4.9 mb
12.	1995/01/20	7.1506	38.4272	5.0 mb
13.	1999/06/17	10.6990	41.3000	4.2 mb
14.	2000/05/10	10.0880	41.0410	3.9 mb
15.	2000/05/10	10.0230	41.1490	4.0 mb

2.3. Project Salient Features and Current Status

The prominent head work features of construction in the study area (Kesem project) are the main dam, saddle dam, diversion tunnel, spillway, intake shaft and intake tower, and two Bunds. The main Dam is a 95m high gravel- rock fill dam with central clay core having the maximum plane width of 50m and the maximum excavated slot of 35m wide at its bottom in the key trench (WWDSE, 2008).

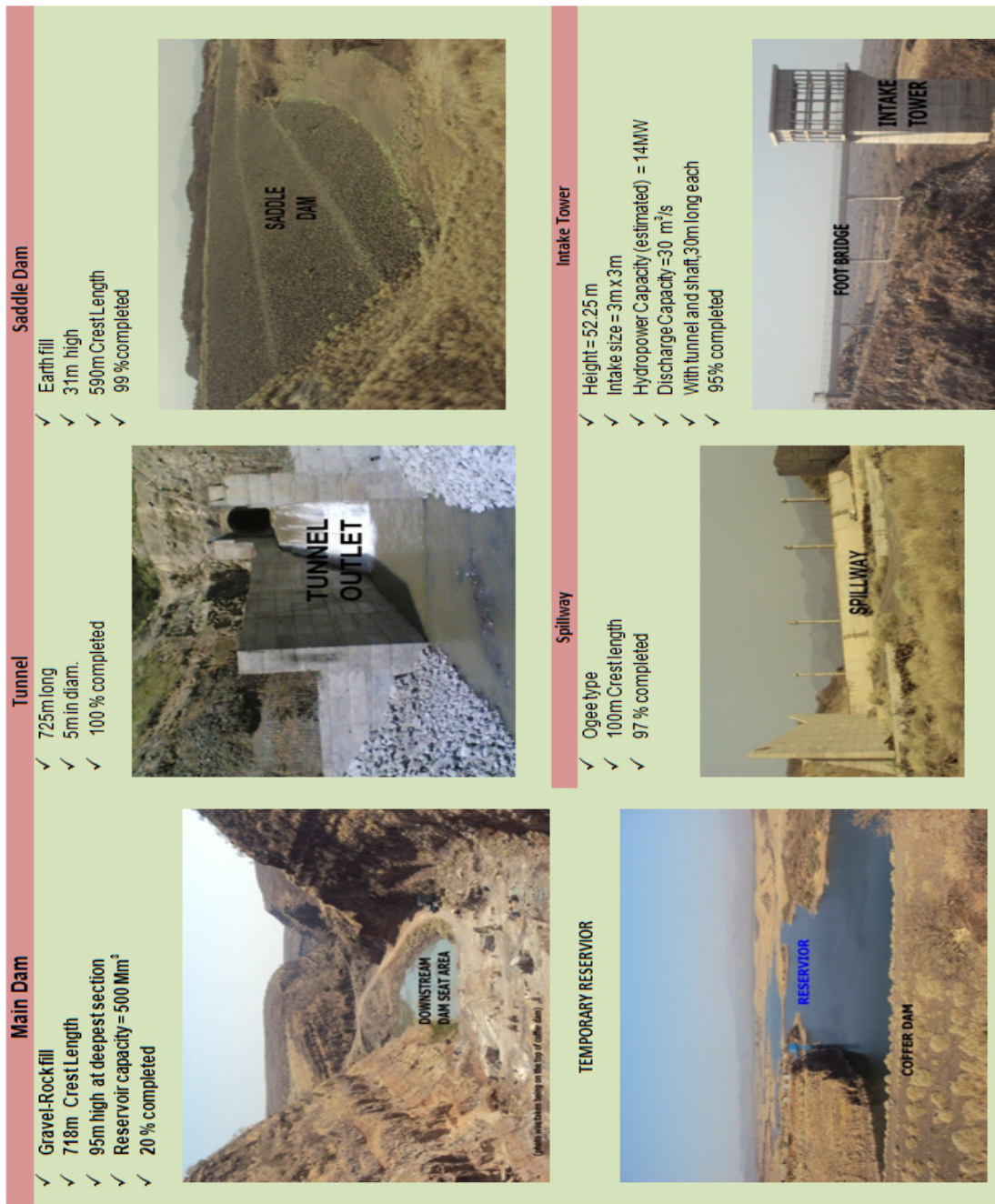


Fig. 2.3 Prominent head work structures built at the study area and their current status

A short summary of the project salient features is presented in Fig.2.3. Construction activity in the project has been underway since 2005. Currently construction of diversion tunnel and coffer dam is completed, whereas about 90% construction of saddle dam and intake tower is completed. On the other hand construction of the main dam is yet to be started as the foundation grouting and excavation works are in progress.

2.4. Dam Site and Reservoir Geology

2.4.1. Previous Studies

The geology of Kesem dam area has been previously studied by Sir M. Macdonald & partners Consulting Engineers in 1978, as part of the Kesem irrigation project feasibility study, under the water resource Development Authority with financial support from the United Nation Development Program. This work provides basic background on the type and distribution of lithological units, major boundary faults and geological section logs of 12 bore holes drilled to depth in the range of 13 to 101 m. In addition to the feasibility study, the detailed geotechnical investigation was undertaken by WWDSE (2005), work sublet to Construction Design Share Company. The investigation has involved drilling of total 14 bore at the Main Dam and saddle Dam sites (Fig. 2.4) Additional geological studies, more specific to understanding the geologic structure of the area and its implication on watertightness, were carried out by WWDSE (2005).

Lithology

The studies conducted by Sir M. Macdonald and partners (MoWR, 1978) and detailed geotechnical investigation by Water Works Design and Supervision Enterprise (WWDSE, 2005a, 2005b) indicated that the project dam site and reservoir area are comprised of Volcanic rocks of the Nazareth Group; consisting basalts, ignimbrites, tuff deposits of rift valley margins, forming a rugged massif along the Western and South Western boundary of the reservoir area. Layered ignimbrites are widely distributed rock unit in the reservoir and the dam site areas.

Basalt occurs along the Eastern margin of the reservoir area. Alluvial recent deposits occupy a large portion of the Western reservoir area and consists of coarse gravel, cobbles, sand and silt and clay in order of dominance. The main dam area is located in the deepest gorge of Kesem River where thick exposure of lithologies comprises the older rift volcanic formation, Kesem paleo channel fluvial deposits and recent Kesem river deposit (WWDSE, 2005a).

The older rift volcanic formation is well exposed on both sides of the Kesem gorge (dam abutments) and it has a maximum thickness of 90m (Plate 2.1, Fig. 2.5). Further, bore hole log indicated the occurrence of another 60m of the formation below the surface. Based on the rock type and ages, the

formations have been divided into; lower Basalt, Lower pyroclastic rocks, upper basalt and upper pyroclastic rocks (WWDSE, 2005a).

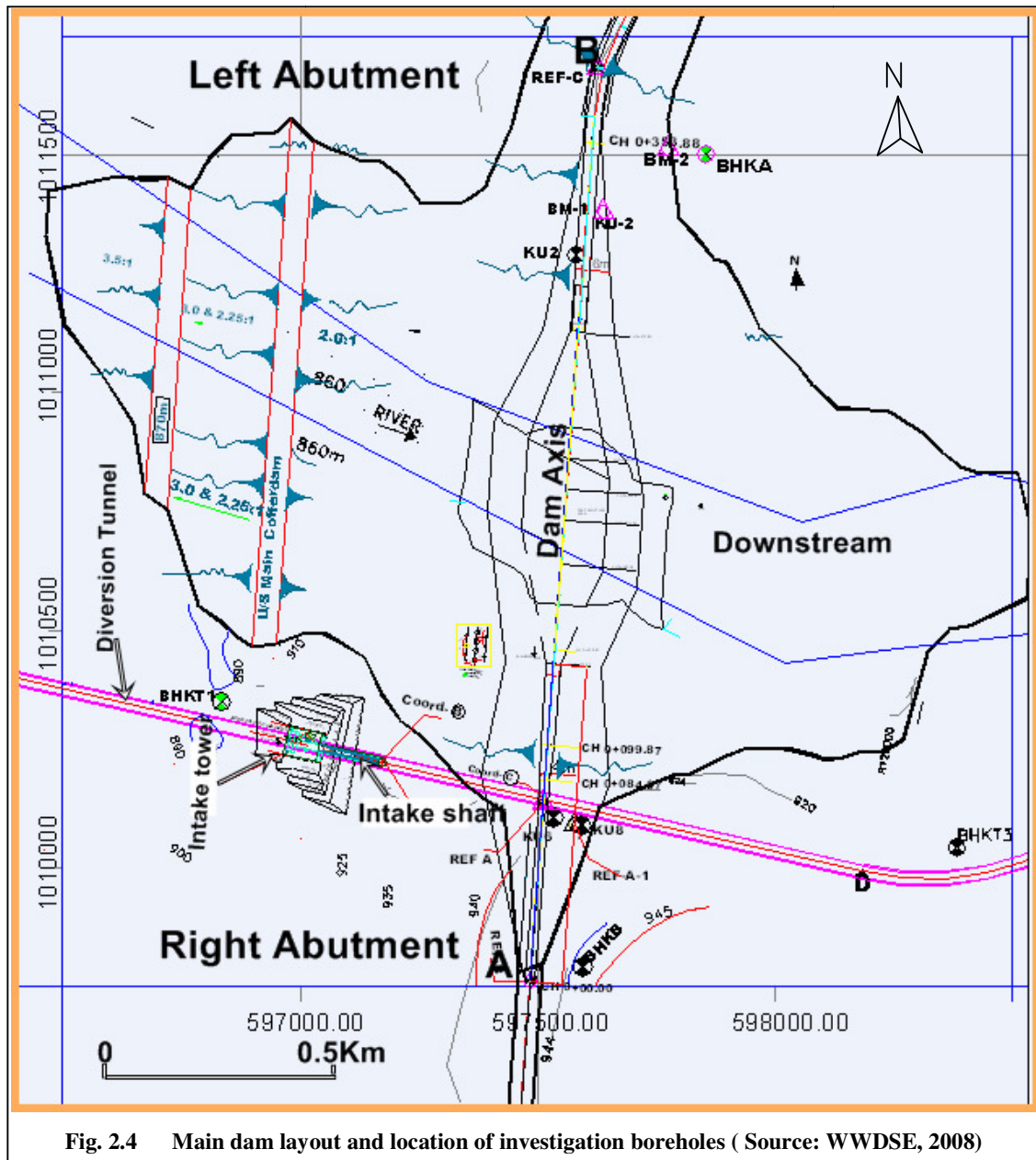


Fig. 2.4 Main dam layout and location of investigation boreholes (Source: WWDSE, 2008)

The lower basalt is more than 50m thick. However, its total thickness is not known since the bore hole was terminated before it reaches the base. It consists of thin (0.8-0.4m) to thick (10-17m) highly to slightly vesicular layers of basalt, aphanitic basalt and scoraceous basalt layers. It appears in various shades of grey and show variable degree of weathering. Vesicles are commonly filled by calcite and the fractures being coated with travertine material.

The lower pyroclastic formation refers to the volcanic formation between the lower basalt and upper basalt, which composes the lower part of the gorge and go deep under the river bed up to 20m.

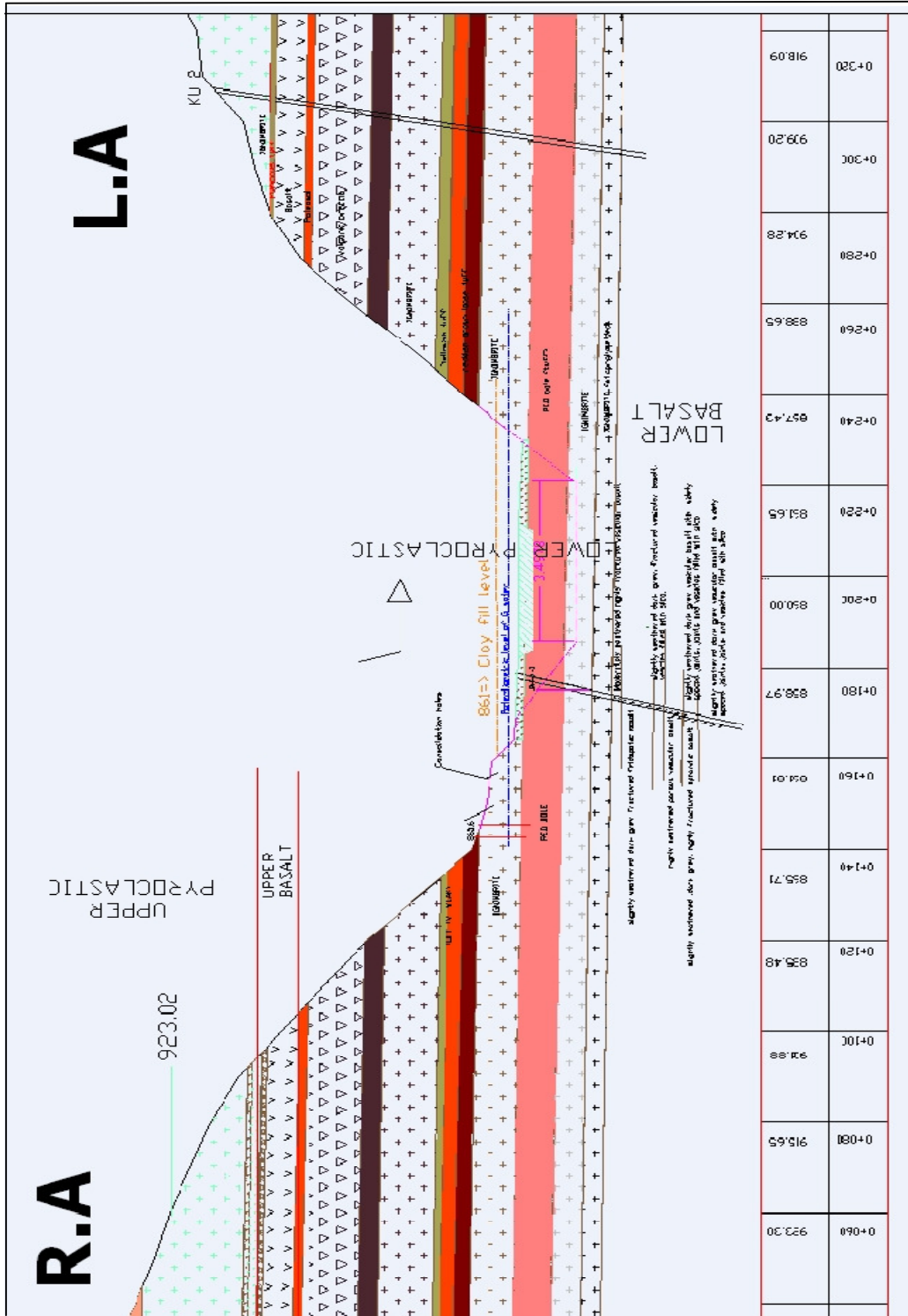


Fig. 2..5 geological cross-section along the dam axis AB (m: WWDSE, 2008)

N.B. For clear view, A3 paper size print is attached as annexure-VI

The unit is formed of thickly layered sequence of highly VARIGATED IGNIMBRITE (which appears in different shades of brown, gray and green), VARIGATED TUFF layers (reddish, brownish, yellowish and greenish) and VOLCANIC BRECCIA units. The ignimbrite units are characterized by

medium to poor degree of welding, moderate to slight weathering and widely spaced jointing. The tuff layers are friable, very weak in strength and dispersive in nature. The reddish tuff “RED BOLE” at the river bed is impervious except where it is fractured, but the other tuff units are highly porous and pervious. The volcanic breccia unit comprises of Lapilli to bombs of basalt (aphanitic and scoracious) with brownish tuff ground mass and it is loose in occurrence.

The upper basalt composes the middle part of the gorge. It consists of 20-25m thick layered series of vesicular and aphanitic basalts and appears dark grey in color. The basalt is characterized with intensive fracturing and has high to moderate degree of weathering. Besides, a very thin band of red to purple scoracious basalt /agglomerate is inter-layered at the middle of the series.

The upper pyroclastic unit covers the upper portion of the Kesem gorge and abutment plateau areas. It has an average thickness of 30m. It mainly comprises of strongly welded ignimbrites (grayish and greenish) containing coarser pumicious vesicles. Besides, the green ignimbrite is present which forms medium thick beds and characterized by columnar jointing. Between the ignimbrite beds very thin (0.4-0.5m) layer of loose yellowish green tuff comprise the contact. Brownish poorly welded (agglomeratic tuff) is sandwiched between the grayish and greenish ignimbrite layers. The upper pyroclastic rocks are unconformably overlain by the Kesem paleo fluvial deposits.

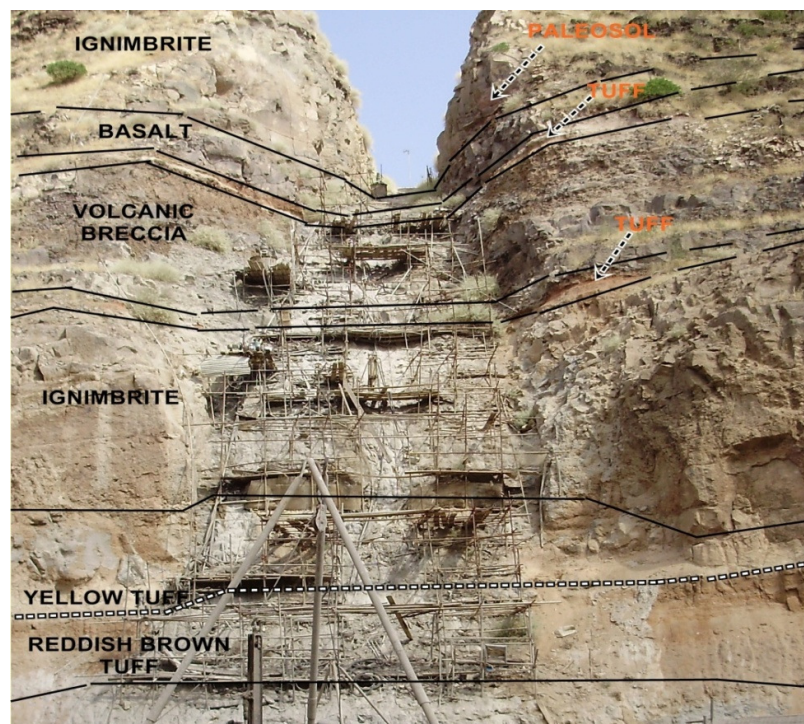


Plate 2.1 Lithology exposed at dam site (left abutment)

2.4.3 Geologic Structures (Faults and Joints)

The bedding planes of the formations generally possess 4°-5° tilting in the North West direction. The direction of major tilting can also be visualized from a distance having a look of the land form standing at a particular location. As the area is located in the main Ethiopian rift zone, faulting and fault enhanced joints are structurally prominent features in the area (Fig. 2.6).

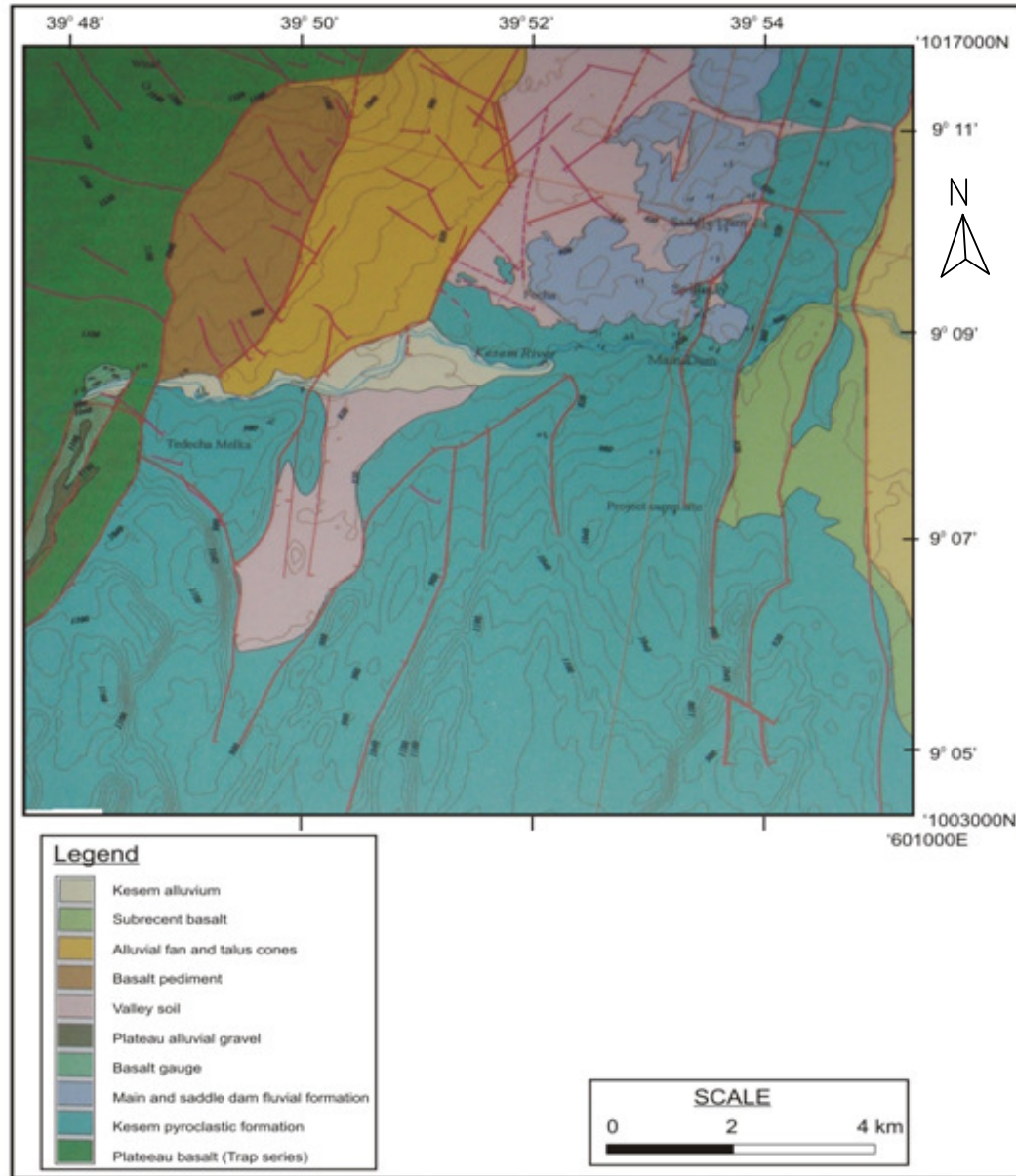


Fig. 2.6 Geological map of the dam site and the surrounding area (source: WWDSE, 2005a)

Four major joint sets are identified in the area. These are N20°W, N60°E, N20°E and N70°W, respectively in order of decreasing frequency. Joints are open and wide except in the case of the top ignimbrite unit of the upper pyroclastic formation for which joints are filled with amorphous calcite and brown amorphous limonitic material. Joints dip nearly vertical. The dam site is dissected by several faults; these faults are mainly non-planar and normal, having a finite length with the

displacement decreasing towards their end. Thorough examination of those faults indicates that the major, longest normal faults are composed of many overstepping small fault segments propagating laterally, increasing in size and amount of displacement. The fault segments are commonly associated with complex deformation zones with remarkable sinus geometry. In most cases relay zones are separated by small transverse normal-faults/fracture discontinuities at oblique angle to the main direction of normal-fault relays. The inter-site fault, a little east of the Main dam axis can be sighted as the best examples of normal-fault relay. The inter-site fault is a prominent, generally NNE trending, easterly dipping normal-fault relay which begins south of the Main Dam area (Plate 2.2), from a small NE trending transverse fault, and ends in the dry stream valley east of the saddle Dam area, along another E-W striking transverse structure.

The southern and central segments meet at the river valley with an increased vertical displacement of 40m, and an associated E-W striking transverse fault developed across the Dam axis. The central and northern segments intersect north of the river with an increased vertical displacement of 80m. Similarly accommodative faults and fractures occur at an angle to the main normal-fault relay to the east and west.



Plate 2.2. Major and minor faults in and around dam site and their interconnection for leakage path
At the dam site, several faults were traced crossing the dam axis nearly perpendicular and having extension in to the reservoir. These faults are threats for stability and excessive water loss. To amend

instability problem suspected from the faults presence, the dam design has incorporated satisfactory dental treatment designs.

The trace of the faults revealed their interconnection thus may form potential leakage path. Faults at the river section and left abutment are within the longitudinal extent of the grout curtain, where as faults at the right abutment lie away from the grout curtain extension. Besides, except for the river bed fault, no special consideration for grout hole depth and arrangement was designed at fault zones on abutments. These faults could form potential leakage path after the reservoir impoundment unless examined in detail and treated to the required degree.

CHAPTER 3

LITERATURE REVIEW

3.0 Preamble

The aim of this section is to provide a conceptual framework for the present research problem. A thorough and extensive literature review was undertaken to gain the necessary knowledge about the present research topic encompassing both published and unpublished reports of investigations, case studies, text books and Journals which are found related to the research topic from different sources. This literature review mainly deals with the types and purpose of grouting, controlling factors for groutability of a medium, and various techniques known and applied so far for groutability assessment on soils and rocks. Besides, a brief review on Volcanoclastic sediments is also form part of this compilation. An attempt was also made to understand, what methodologies were adopted by the previous researchers and what the findings were. This was mainly done to evolve the systematic methodology for the present study. A summary of literature review relevant to the present study is presented in the following paragraphs.

3.1 Grouting

3.1.1 Purpose

A number of ground improvement techniques have been developed and are known to be used in civil engineering projects. Grouting, *the injection under pressure of a liquid or suspension into the voids of a soil or rock mass or into contact voids between these materials and an existing structure (US ACE, 1984)*, is one of such methods well known for its application in ground improvement such as improving the engineering performance of materials forming foundation of various engineering structures (dams, buildings, bridges), stabilizing soil and rock mass composing unstable slopes, mitigating liquefiable soils, facilitating underground excavations and forming barrier against water flow under a dam or into area of excavation. Besides, it is widely applied to fill the contact void between existing structures and the ground, and voids formed in civil structures (cracking, pores or cavities). Accordingly, the grouting treatment types are categorized as curtain grouting (grouting to cutoff or reduce seepage under a dam or in to area of excavation), consolidation grouting (grouting to increase strength of foundation materials, to mitigate liquefaction or to stabilize materials on a slope), contact grouting (grouting to fill contact void between structures or between engineered structure and the ground), cavity filling (grouting intended to fill cavity within ground material or within structure) and other specialized applications.

Grouting for dam foundation takes two forms: Curtain and consolidation grouting. Curtain grouting refers to the construction of a curtain or a thin barrier in the dam foundation with the view of reducing water seepage through fractures or interconnected pores. It is constructed by drilling and grouting a

linear sequence of grout holes arranged along a single line or on multiple lines depending on the geologic condition and intended degree of foundation watertightness. Its purpose is to reduce permeability. The location of the curtain primarily depends on the dam type. For an embankment dam, the grout curtain is constructed under the impervious core below the cutoff trench. Although not dependable, various rules of thumb are known to determine the depth of the grout curtain, all being a function of the height of the dam. Some of such rules of thumb which are given by; Ewart (1985) and Houlsby (1977, 1978) are: Depth in meter (D) = $1/3H+C$, $D= H$ or $D=1/3$ to $2/3H$ (where, C= 8 to 25 depending on type of foundation, H= height of dam). The lateral extent of the curtain, parallel to the dam axis, could cover the whole stretch of the dam (valley floor and abutments) and even can extend to adjoining structures (like side channel spillway) if geologic condition is prevailing high permeability. Consolidation grouting on the other hand is required where the foundation material is of low bearing capacity, and could bring excessive settlement. The depth of consolidation grouting is pre-determined and is relatively shallower ranging in the order of 5-15m.

3.1.2 Type and Components of Grouts

The suspension or solution that is forced to inject in to the material to be grouted is referred as Grout. Grouts can be divided into three groups based on the type of major ingredients and rheological properties (Fig 3.1): **particulate grout**, **colloidal solutions** and **pure solutions**. The later two are chemical grout types.

Particulate grouts are suspension/slurry grouts with Bingham fluid behavior, a fluid that acts as a rigid body at low shear stress and flows like a viscous fluid at high shear. These types of grouts are prepared by thorough mixing of water with cement or clay (or a mixture of these two) using selected proportions of water and cement and/or clay components on the basis of the quality (stability, pumpability, strength, setting properties) of resulting grout mix. The type of cement could be OPC or PPC or special purpose cements (micro fine cement, sulphate resistant cement, thermal resistant cement) depending on governing factors such as the size of voids to be grouted (fracture apertures and pore sizes) and consideration to chemical and thermal effects on cement grout from the surrounding where a grout would be injected. Besides, fillers such as fine sand, wood chips, and admixtures (any material added to the grout other than cement, water and fillers such as fluidizers, accelerators and super plasticizer) could be used in the mix with various intensions such as reducing cement consumption in cases of very high grout takes or cavity filling, improving pumpability and stability of grout by reducing percentage of decanting water from mix, to facilitate penetrability by lowering Cohesion and viscosity of grout, and when fast setting of grout is desired under circumstances of grouting under water or to limit unnecessary spread or wastage, (U.S. ACE, 1984). The properties of cement-based grouts can be classified into two categories;

- a) Fluid characteristics: cohesion, viscosity, bleed and density
- b) Set characteristics: initial and final set-time, unconfined compressive strength and durability

Viscosity, Cohesion and percent bleed: The cohesion, apparent viscosity and bleed percent of a grout are related but impact differently on the rheology of a fluid.

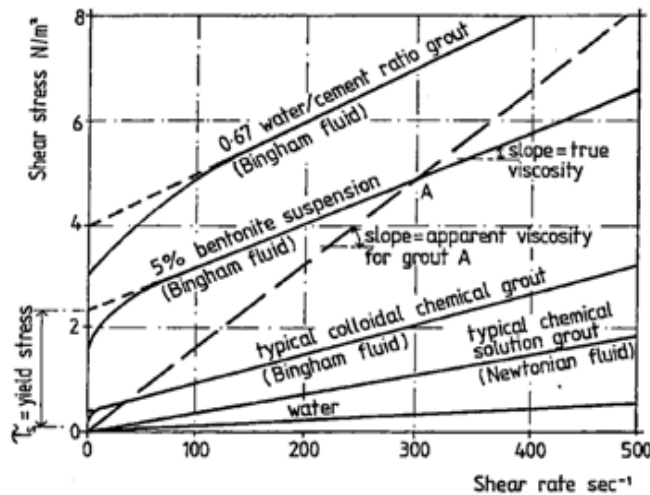


Figure 3.1 typical properties of grout types (adapted from Littlejohn, 1985)

or even with a Marsh cone. The latter two properties are closely related: the higher the cohesion, the higher the apparent viscosity. Ideally, admixtures only marginally impact on the cohesion or viscosity at moderate to high shear rates, but cause a major increase once the grout moves slowly or are at rest. These grouts are often referred to as stable suspension grouts with shear thinning rheology. Properties and characteristics of suspension grouts can be significantly altered by means of admixtures. Generally, to have a stable and durable suspension cement grout, the fluid characteristics are required to fall within acceptable ranges proposed by various Authors (Houlsby, 1986; Deere and Lombardy, 1993) based on their practical experience and Laboratory testing. As per their suggestion, water : cement ratios 3:1 (by weight) or thicker are preferred, bleed percent in 2hrs should not exceed 5%, the viscosity (measured in marsh cone funnel) should be 29-32 seconds and the relative cohesion (measured using Lombardi's plate cohesion meter) should be in the range of 0.08-0.15mm.

Colloidal solutions: These grouts are silicate based chemical grouts which have an evolutive Newtonian behavior characterized by an increasing viscosity over time. The silicate in the grout forms a gel which has properties that can seal fissures or pore spaces to ensure desired engineering property of grouted mass.

Pure solutions (resin): non-evolutive Newtonian solutions whose viscosity remains constant until the grout sets. This category of grout, which is made by solution of organic products in water, has the lowest viscosity of the three types.

For the reason of economy, need of advanced grouting technique and its likely negative environmental impact, the use of chemical grouts (colloidal and pure solutions) will be restricted to those cases where seepage is critical and not controlled by cement grout, or incases of remedial works particularly in (alluvial) soil foundations where cement grout is not applicable (Fell et.al, 2005).

3.1.3 Injection Techniques

Depending on the type of material to be grouted (soil or rock) and its nature (strength, void size), various modes of grout injection have been practiced. In different parts of the world, grouting of wide fractures in hard rock has been carried out relatively smoothly with a considerable lower cost than grouting of fine fractures and pore spaces in soil. Both the traditional and GIN grouting techniques have been under use efficiently for injecting cement grouts in rocks. The GIN method (Lombardi and Deere, 1993) uses only one stable moderately thick mix (chosen between 0.67 to 1:1 water cement ratio by weight) and generally uses higher limiting grouting pressure than the traditional method, which could be reduced as the volume take increases in accordance to the GIN limiting envelope and grouting path interception. On the other hand, the traditional method uses a series of different grout mixes keeping a constant limiting (maximum) grouting pressure as discussed in detail by Houlsby (1977, 1978), WRC (1981), Deere (1982), US ACE (1984), and Deere and Lombardi (1985).

The grouting technique gets more and more advanced and expensive for very fine fracture and soil (porosity) grouting for obvious reasons of using ultrafine cement grouts or chemical solutions to effect grouting by permeation or use of other soil improvement techniques such as compaction grouting, jet grouting or deep mixing (Kazemian and Haut, 2009).

Permeation grouting: This method describes the process of filling joints or fractures in rock or pore spaces in soil with a grout without disturbing the formation i.e, the grouting pressure being without exceeding the in situ horizontal stress. More specifically, permeation grouting refers to the replacement of water in voids between soil particles with a grout fluid at low injection pressure without inducing hydraulic fracturing (Manfred, 1990).

Jet grouting: The basic mechanism for jet grouting is especially high pressured grout, water and/or air jet acting under nozzles on the injection tube. The soil around the injection tube is excavated by the jet force and then mixed in place with a suitable stabilizer (Kamon, 1991).

Compaction grouting: In this method, grout mix is specifically designed so as not to permeate the soil voids or mix with the soil. Instead, it displaces the soil into which it is injected. In granular deposits not at their maximum density, the volume of voids are reduced and the deposit is locally densified (Reuben, 2003). In Compaction grouting a very stiff (say 25-mm slump) mortar is injected

into loose soils, forming grout bulbs which displace and densify the surrounding ground, without penetrating the soil pores (Manfred,1990).

Deep Mixing Methods (DMM): The Deep Mixing Method (DMM) is today accepted world-wide as a soil improvement method which is performed to improve the strength, deformation properties and permeability of the soil. It is based on mixing binders, such as cement, lime, fly ash and other additives, with the soil by the use of rotating mixing tools in order to form columns of a hardening material since pozzolanic reactions between the binder and the soil grains are developed.

3.1.4 Groutability

The safe construction and operation of many structures frequently demands the improvement of the mechanical properties and behavior of soils by permeation grouting using either suspensions or chemical solutions. The former have lower cost and are friendly to the environment but cannot be injected to soils with gradation finer than coarse sands; the latter can be injected in fine sands or coarse silts but are more expensive, and some of them, pose health and environmental hazard (Karol 1982, 1985).

An important component for effective groutability of a medium using particulate/suspension grout by permeation is the penetrability of the grout in to voids (fine fractures or pore spaces) with the application of allowable grouting pressure. However, accurate prediction of groutability of a medium by permeation has always been complicated as the penetrability of grout in voids is a function several interrelated parameters contributed from the medium to be grouted, the nature of the grout and the grouting pressure used. Related to the medium to be grouted, the size of voids, permeability and the strength has relevance. In case of rocks in addition to the fracture aperture, the presence and type of secondary infilling material should be considered. Grain size and the state of being looseness or compactness controls the Pore size in soils hence the penetrability (Akubulut and Saglamer, 2002).The property of the soil are fundamental to determining its amenability to improvement by grouting, as well as to selection of the best method to be used. The particulars of the soil also dictate the grouting parameters such as an injection rate, pressure and so forth. Sufficient exploration and testing to establish the existing condition are thus required (Warner, 2004).

Past studies regarding suspension grouts dealt a lot about permeation limits (King and Bush, 1963; Mitchell, 1981; Littlejohn, 1982; Karol, 1985; Baker, 1982), and it has been pointed out that permeation of grout depends on the size of voids in relation to the size of individual suspended particles or of their aggregations. Accordingly, to assist the preliminary assessment of groutability of a medium using particulate grout by permeation, based on experience, laboratory testing and empirical derivations, the aforementioned authors have forwarded their opinion on limits of

groutability correlating the size and number of fractures, the grain size and permeability of soil with the grain size distribution of grout material. As cited in Fell et al (1992, 2005), when considered in conjunction with the relation between Lugeon Value and fracture width along with the experience of Littlejohn (1982), it is found convincing to adopt the groutability limiting fracture width equals $3D_{100}$ where D_{100} is the sieve size for which all the grout particles are finer (Kennedy, 1958; Mitchell, 1970; Karol, 1985; Tjandrajana, 1989). Based on the relation between D_{15} or D_{10} of soil with D_{85} or D_{95} of grout respectively, “groutability Ratios” _____, has been suggested as a criteria to evaluate the injectability of suspension grout in soil (Mitchell 1981, USACE, 1984, Verfel 1989) permeation would be achieved.

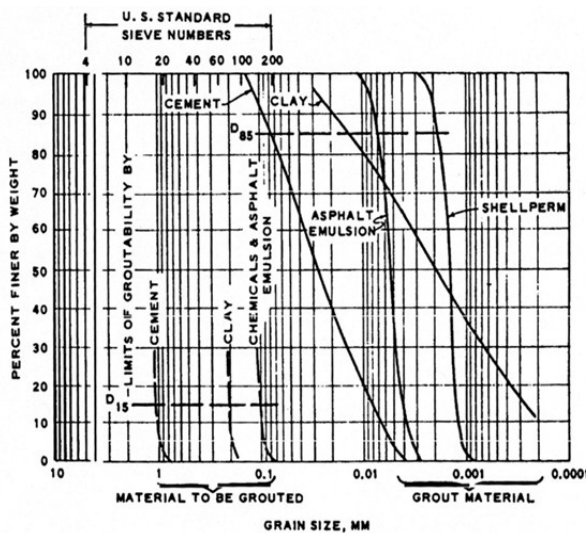


Figure 3.2 soil and grout materials grain size curves (adapted from U.S. ACE, 1984)

Generally, grouting is considered possible for $N > 25$ or $N_c > 11$ and not possible for $N < 11$ or $N_c < 6$. Figure 3.2 shows the D_{15} range required for a soil for effective permeation grouting considering various types of suspension grouts and minimum groutability ratio for permeation. the Reference to Fell et al (1992), tests with water: cement ratios ranging from 5:1 to 0.5:1 (by weight) showed that the different proportions didn't have a significant effect on the resulting gradation of the grout particles.

Besides, it was noticed that there is no significant difference brought regarding the D_{100} of the powder cement the D_{100} of the powder cement and D_{100} of the resulting grout. However, the finer cement particles aggregate together in water giving a coarser effective size (D_{15} or D_{10}) than the dry cement powder.

While evaluating permeation groutability effectiveness of pulverized fly ash grout in to sands, as cited in IS-Tokyo'96, MarKou I.N. and Atmatzidis D.K., have used the groutability ratio (N) as a preliminary permeation groutability assessment technique before conducting experimental injection on four sand specimens prepared with varying gradation and relative density. The indications of the groutability ratio evaluation have shown a good agreement to the permeation evaluation manifested from the experimental injection conducted.

Fell et al (1992, 2005) has adapted limits of effectiveness of a particular type of grout for permeation grouting of a soil from Karol (1985, Fig.3.3). The same concept has been modified by Haut (2009) with some additions such as the Jumbo eco pile system grouting technique (a type of jet grouting), mortar grouting and indicating the feasible range for application of ultrafine and chemical grouting (Fig. 3.4).

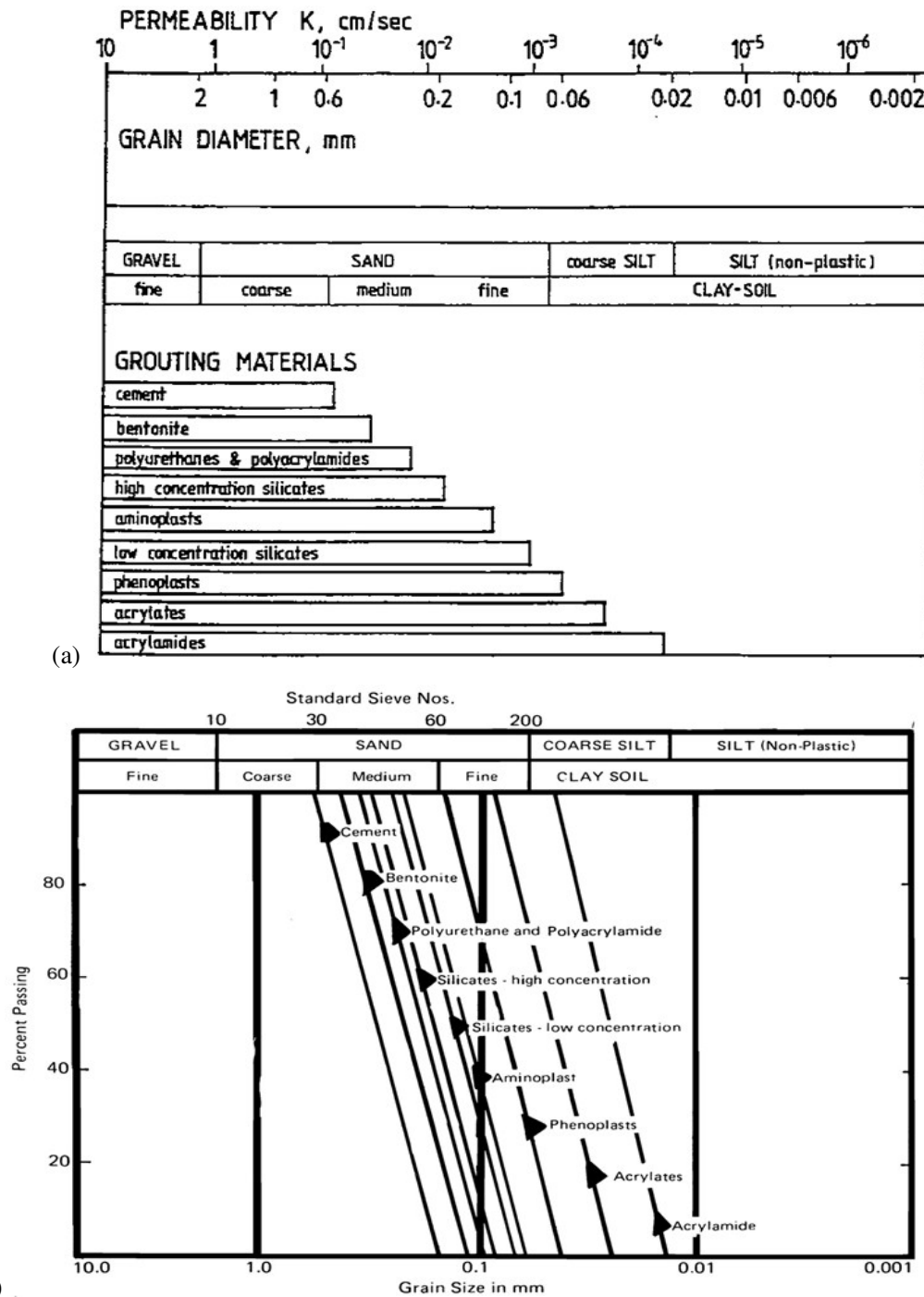


Figure 3.3 (a&b) limits of groutability based on Karol, 1985 (source: Fell R. et al, 2005)

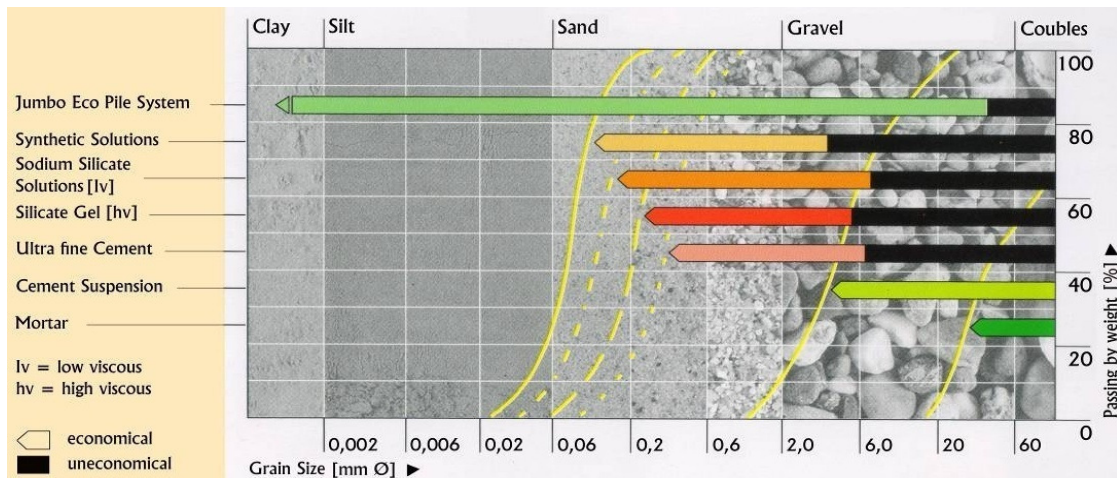


Figure 3.4 range for effective and feasibility of grout types and grouting methods (source: Keller, 2005)

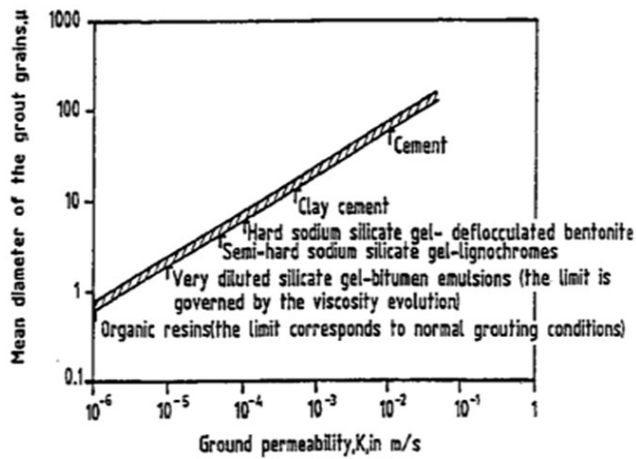


Figure 3.5 Limits of injectability of grouts based on the permeability of sands and gravels (Littlejohn, 1985)

Littlejohn (1985) has shown graphically the limits of injectability of grouts by correlating the mean diameter of grout and permeability of soil (Fig. 3.5). Besides, he has indicated that penetrability of Bingham suspensions is not warranted where D_{10} of soil is less than 0.5mm, and permeability coefficient $K < 10^{-3} \text{ m/s}$ (these agree with the opinion of Caron, 1982 and Karol, 1985 and Baker, 1982).

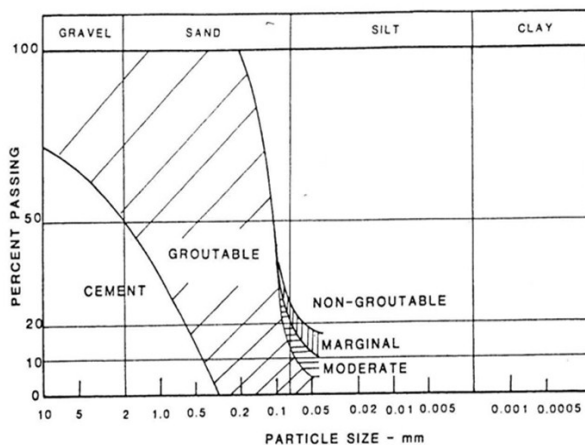


Figure 3.6 grain size range for permeation groutable soils by solution (Baker, 1982)

Baker (1982) has shown graphical summary (Fig.3.6) of the range of gradation of soils which could be permeation grouted using suspensions and chemical solutions. He has come up with Four major zones: zone groutable with suspensions, groutable with solutions, moderately groutable with solutions and non groutable at all. According to him, in agreement to Littlejohn (1985), permeation grouting using suspensions is practical for soils with $D_{10} > 0.4 \text{ mm}$.

3.1.5 Closure Criteria

There are different opinions on how to define acceptable closure criteria for grout curtains, and on how to select the limits. This is critical to the question to as to whether sufficient grouting has been carried out. Houlsby (1977, 1978, 1982) and WRC (1981) proposed the use of the Lugeon water pressure test values in the grout holes prior to grouting as basis for decision making.

According to Lugeon (1933), grouting should continue until the pre grouting Lugeon value becomes less than 1 for dams over 30m high, and less than 3 for dams less than 30m high. In later paper, Houlsby (1985) gives revised criteria which is marginally less conservative and took into account the type of dam, the foundation erodability, the number of rows of grout holes and embankment dam core width (Table 3.1). Deere (1982) suggests a closure criteria based on grout takes, the amount of grout injected in to the rock per meter of grout holes (Table 3.2). However, this criterion could not be practicable in cases where a coarser grout suspension is used while porosity and fine fractures exist in foundation materials which have ungroutable size compared to the suspension size, but appear pervious to water flow.

Table 3.1 Curtain grouting Closure criteria (Houlsby, 1985)

Parameters Considered	Suggested curtain Standard (Lugeons)
Concrete Dams (Gravity, arch, buttress) <ul style="list-style-type: none"> • Single row curtain • Multiple row curtain 	3-5 5-7
Embankment dams <ul style="list-style-type: none"> • Narrow core earth/rock fill • Wide core earth/rockfill, and membrane faced <ul style="list-style-type: none"> ✓ Single row curtain ✓ Multiple row curtain 	3-5 5-10 7-15
All types of dam if foundation contains material able to be removed by seepage <ul style="list-style-type: none"> • Single row curtain • Multiple row curtain 	3 4
All types of dams if water lost by seepage is sufficiently valuable to warrant considerable expenditure to stop it, or environmentally hazardous (single or multiple row curtains)	1-3

Table 3-2 Upper limit of grout absorption (Deer, 1982)

Depth interval (m)	Grout absorption (kg/m of hole)
0-10	25
10-20	35
20-30	50
>30	100

3.2 Volcano-clastic Sediments Composing Kesem Dam Site

The term volcano-clastic has been used to encompass the entire spectrum of fragmental volcanic rocks by any mechanism or origin, emplaced in any physiographic environment, or mixed with non-volcanic fragment types in any proportion (Fisher, 1961, 1966a). As stated in Pettijohn F.J. (1984), the term well describes pyroclastic debris deposits which are truly pyroclastic-ejectamenta from a volcanic vent- they are collectively designated tephra (Thorarinsson, 1954). Textural aspects of volcano-clastic sediments was first addressed by Wentworth and Williams (1932), and later was modified by Blyth (1940), Fisher (1958, 1961, 1966a). The grain size limits and corresponding terminologies for pyroclastic debris are given in Table 3.3.

Table 3-3 Grain size and corresponding terminology for Volcanoclastic sediments

Volcanoclastic sediments		Engineering classification of grain size for soils	
Grain Size, mm	Terminology	Grain size, mm	Terminology
>64	Breccia=Blocks and bombs	>75	Boulders
64-4	Lapilli tuff	75-4.75	Gravel
4-0.06	Coarse ash	4-0.075	Sand
< 0.06	Fine ash	<0.075	Fines (silts & clays)

The Kesem dam site pyroclastic debris (tuff and volcanic breccia), studied for the current research, belongs to Volcanoclastic sediment category. Laboratory tests for collected representative samples were carried out in accordance to BS test 7. The fragmental or loose nature of the deposit as well as the consistency in grain size class between the Volcanoclastic sediment and soil mass in engineering supported the possibility of adopting soil mechanics laboratory test procedure for the volcano-clastic sediments too.

Volcanoclastic deposits form incompetent foundation to the Kesem dam because of their dispersivity, compressibility and pervious nature. To amend these constraints of the deposit, the dam design has incorporated permeation grouting; however, the required watertightness has not been fully satisfied even though intensive grouting with very closely spaced grout holes was made (KDIPR, 2008). The report also stated that the main probable cause to the incompetence of the grouting could be inconsistency between the sizes of cement grains used in the grout compared to the size of inter-granular pore spaces (voids) in the deposit. Therefore, this research has made consideration to the similarity of the nature of the tuff and volcanic breccia (Volcanoclastic sediments) with soil for grouting perspective. Besides, on the basis of extensive literature review made, practiced and efficient soil permeation grouting effectiveness evaluation techniques are used. Hence, it is believed that the preliminary permeation grouting effectiveness of the Volcanoclastic sediments could be well addressed by the soil permeation groutability assessment techniques discussed under section (3.1.4). Moreover, the findings of the study would have a significant contribution for preliminary permeation

groutability assessment on Volcanoclastic sediments so that it would be helpful to identify whether a particular Volcanoclastic deposit under consideration could be permeation groutable or not. Besides, if found groutable, the possible grout type could also be worked out by the same analysis the current research has followed. These would have a meaningful input on giving basis for a test grouting program considerations; moreover on minimizing the risk of unnecessary wastage of time and economy while attempting trial and error groutability evaluation during the progress of the actual grouting work.

CHAPTER 4 DAM SITE GEOTECHNICAL PROBLEMS AND FOUNDATION GROUTING

4.0 Preamble

The Kesem dam site, in addition to active seismicity, is also known for its complexity in geology, and high artesian thermal discharge of groundwater which contribute to the obstruction of dam construction progress and consequent dalliance. Failure to satisfy grouting closure criteria on incompetent layers and excavation obstruction, due to groundwater, are the bottleneck problems challenging the dam construction. This chapter presents the geotechnical problems of the site emphasizing to the role of the groundwater and incompetent layers in conjunction with excavation and foundation grouting difficulties experienced during the construction phase of the dam project. Besides, design aspects and current status of dam foundation grouting are also overviewed in the present chapter.

4.1 Groundwater Condition and Related Problems

The groundwater at Kesem dam site possesses an artesian condition (Plate 4.1). Both springs and seepages are commonly found on the lowest level and both banks of the Kesem river valley. Those springs discharge hot water with temperatures in the range of 35° to 53 °C, signifying the existence



Plate 4.1 Artesian groundwater at the dam site

of shallow thermal anomaly in the area. Besides, the surface manifestation of groundwater leakages, artesian thermal springs have been recorded at shallow depths (5-33 m) during core drilling in the river channel (MoWR, 1987; WWDSE, 2005b). The artesian hot springs show water pressures in the range of 1.5 to 2.55 m head above the ground level and temperatures more than 40° C. In most cases high yielding spring sites are linked with continuous seepage fronts from the bank down to the riverbed. The dimension of seepage fronts varies from one to another discharge site depending upon gradient of the bank slope and proximity (distance) to the river base level. This kind of groundwater discharge often takes place along bedding planes of pyroclastic rocks comprising loosely comp-

-acted scoraceous and pumiceous tuffs. In general isolated springs emerge from traces of fault planes and fractures of the underlying bedrock and flow into the river through much recent alluvial gravels.

The water bearing formation is basalt rocks mainly scoraceous basalt where as the tuff, ignimbrites and massive aphanitic basalt act as an aquiclude and the groundwater in the scoraceous basaltic aquifer is confined except for fault zones (Dewatering scheme by WWDSE, 2008). The existence of deep faults with possible interconnection to surface joints could lead to the recharge of the groundwater with the future reservoir water.

The fumaroles, which also include steaming grounds and isolated boiling mud pools, are located on the western flank of the central valley area south of Tedecha Melka 5km upstream of the dam site. These hydrothermal manifestations record different temperatures in the range of 60°C to 97 °C and appear to be influenced by the NNE trending faults affecting the rift margin silicic and mafic volcanic rocks (MOWR, 1987).

The groundwater at the dam site has brought challenges to the dam project, at least by affecting excavation, grouting work and stability of the superstructure. At the river section of the dam, the foundation is required to be excavated to a maximum depth of 15 meters (up to an elevation of 845 masl) from the current ground level (at an elevation of 855 masl). On the other hand, the thermal artesian groundwater has shown gradual rise in potentiometric level (static water level) from 858 m to an elevation of 866 masl following coffer dam's temporary reservoir water impoundment. With a consequence that the 15 m key trench foundation excavation submerged under water, it now requires a well designed and efficient dewatering technique and or advanced means of excavation. Thus, this problem has contributed in delay of overall progress and subsequent rise in the cost of the project.

Furthermore, the thermal nature, enrichment in sulphate content and high pressure of the groundwater has forced special considerations for grouting design and operation. Accordingly, special cement and accelerators are being used for grouting works required under the thermal water zone in order to overcome sulphate attack on resulting curtain, and to prevent unnecessary spread of grout being diluted by further mixing with flowing groundwater. These considerations were must and have incurred significant increment in the cost of the grouting project. Besides, it is also anticipated that the groundwater under these conditions may create significant uplift pressure unless provided with appropriate drainage system for pressure relief.

4.2 Problems Related to Incompetent Layers

Three incompetent layers are known composing Kesem Dam site geology: Various types of **Tuff**, **Volcanic breccia** and **paleosol** layers. Within the lower and upper pyroclastics formations, unwelded

to poorly welded tuff layers having 0.5m to 8m thickness with various shades of reddish brown, yellowish and greenish colors are found sandwiched in between the hard rock units.

Based on their physical and engineering properties, the tuff layers are identified as: “Red bole”, reddish brown tuff and Pummicious tuff. Previous studies have put more emphasis on tuff layers composing the lower part of the valley gorge and river bed area. The “Red Bole” has a red color, and it is the thickest possessing better engineering performance comparatively than other tuff layers exposed at the dam site (Plate 4.2). It covers the lower part of the abutments, the center and downstream part of the main dam foundation area. It occurs slightly welded in saturated condition, but easily friable under dry state. Laboratory test results obtained from WWDSE revealed that the red bole possess high liquid limit (64%), plasticity index (27), free swell (50%), PH Value (9.11) and activity (4.1). Moreover, according to these data, its average bulk and dry densities are 1.5 and 1.1, respectively; direct shear strength test revealed average value of Cohesion (C) equal to 157.4 Kpa and angle of shearing resistance (ϕ) equals to $28^\circ - 35.24^\circ$.

Besides, referring to KDIPR (2008) high value for insitu permeability (>100 Lu) was obtained where as laboratory test permeability data obtained from WWDSE indicates impervious drainage condition (3.7×10^{-8} cm/sec) for block sample of the red bole. This difference is justified for the presence of columnar jointing and dissecting fault enhanced joints that increase the water intake during insitu testing. The red bole forms confinement to the thermal groundwater except where it is fractured. According to Ali (2008), the red bole mineralogy comprises of montmorillonite and alkali feldspar (Na, K). As noticed from the ongoing grouting operation, the red bole couldn't withstand more than 1.5MPa grouting pressure with the existence of additional of 7m confining clay backfill overlying the layer.

According to WWDSE (2008), the dispersivity, relatively low shear strength and excessive settlement potential of the red bole is mentioned as its main geotechnical problems. The report, as a counter measure, has suggested the complete removal of the red bole in the river channel section of the core



Plate 4.2 Red bole at the river bed channel section of the dam foundation

trench to the depth of sound stratum. Whereas for the general dam seat portion, the report suggested deep excavation to be conducted in the vicinity of dam foot, at overburden (red bole) through which unfavorable sliding plane crossed, or consideration of other methods such as consolidation grouting over overburden or local concrete plugging has been suggested. The best option recommended by the designer (WWDSE, 2008) is excavation of the red bole at places wherever it is encountered in the river channel section of the dam foundation. On the other hand, the red bole layer is as thick as eight meters in the key trench area; besides forms a confining layer to the thermal groundwater which can raise more than six meters above the ground surface. For these reasons, excavation of the red bole at the river bed has become one of the bottleneck problems to the project progress.

The reddish brown tuff, yellowish tuff, Pummicious tuff and volcanic breccia (Volcanoclastic deposits, Pettijohn., 1984), and paleosol layers constitute the bottom and middle portion of dam abutment (Plate 4.3). Based on laboratory test results, obtained from WWDSE, the tuff layers are characterized by sandy silt grain size distribution and dispersivity. Besides these are loosely deposited and sandwiched in between competent rock units which are thin to moderate in thickness. The loose, 6-8m thick volcanic breccia deposit, underlies the upper basalt formation. The deposit comprises ranges of grain sizes of volcanic products; mainly coarse ash and Lapilli size volcanic products (scoracious basalt) with bombs and blocks of aphanitic and vesicular basalt inclusions (Plate 4.3c). A reddish brown tuff (fine ash) forms the ground mass of the volcanic breccia deposit. Based on in situ permeability test results reported (MoWR, 1987; WWDSE, 2005b), all the incompetent layers exposed on the dam abutments form pervious strata within the dam foundation. The yellowish tuff has been manifested for its rich sulfate composition which needs special consideration for selection of grouting material.

The incompetent layers on abutments, as aforementioned, are reported in different reports by WWDSE (WWDSE, 2005, 2005b, 2008; KDIPR, 2008) giving special attention and examination of geotechnical problems associated with those layers. Accordingly, based on the existing situation and forecasted adverse conditions, the possible geotechnical problems pointed out hereunder were studied and reported;

- ✓ Dispersivity
- ✓ Settlement Potential
- ✓ High permeability
- ✓ Difficulty on grouting

The combined effect of dispersive and pervious nature of the incompetent layers triggers the erosion of fine particles from the deposit and could progressively lead to piping failure. To prevent the migration of fine particles, the dam design satisfactorily provided sand and geotextile filter at the downstream portion of the abutments and dam foundation where the dispersive layers are exposed. Intensive curtain grouting has been implemented at both the abutments; however, design closure

requirement (< 3 Lugeon) has not been fully satisfied (KDIPR, 2008). The main reason mentioned in the report is the difficulty of grouting inter-granular pore spaces on incompetent layers probably due to permeation difficulty and hydro-fracturing with small grouting pressures (< 1 bar/meter). Moreover, the loose nature of the deposit could bring excessive settlement when loaded and may lead to differential settlement with subsequent cracking on the dam clay core.

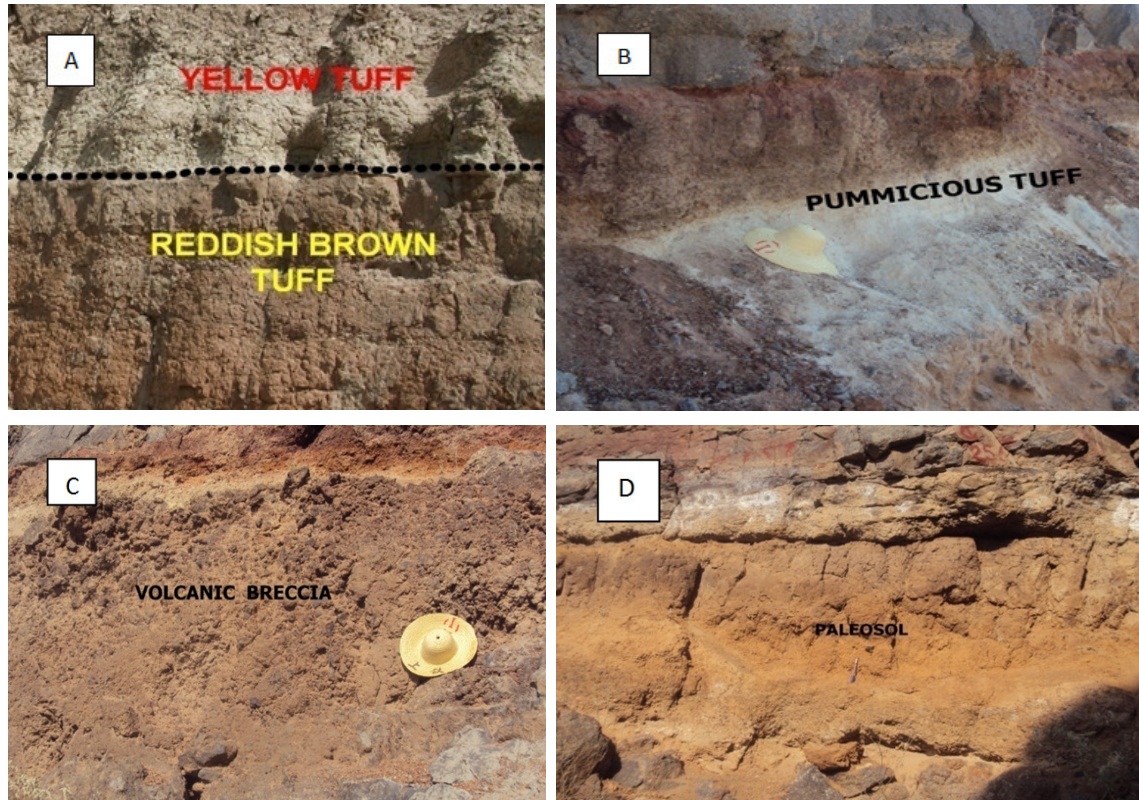


Plate 4.3 Incompetent layers exposed on abutments (A= yellowish and reddish tuff, B= Pummicious tuff, C= Volcanic breccia and D= paleosol)

4.3 Kesem Dam Foundation Grouting

The natural state of Kesem dam foundation material, because of intensive fracturing, weathering and porous nature, are not watertight, and unsatisfactory to support the overlying load. Thus, to improve the engineering property of the foundation materials, for sustainability to the planned life time of the project, dam design has incorporated foundation treatments, one of which is permeation grouting.

4.3.1 Design Aspects

The grouting design for Kesem Dam foundation involves both curtain and foundation grouting works. The Kesem Dam foundation curtain grouting covers the whole stretch (718m) of the dam axis categorized into three patterns with varying width and depth according to the future reservoir water head distribution. In shallow head zones, grout holes depth ranges 10-15m, in the intermediate

pressure zones 20-25m and in high pressure areas 35 to 50m deep with single, double and triple line grout holes patterns, respectively (Fig. 4.1& 4.2). Curtain grout holes are oriented inclined 15-20 degrees from the vertical dipping in to the abutments.

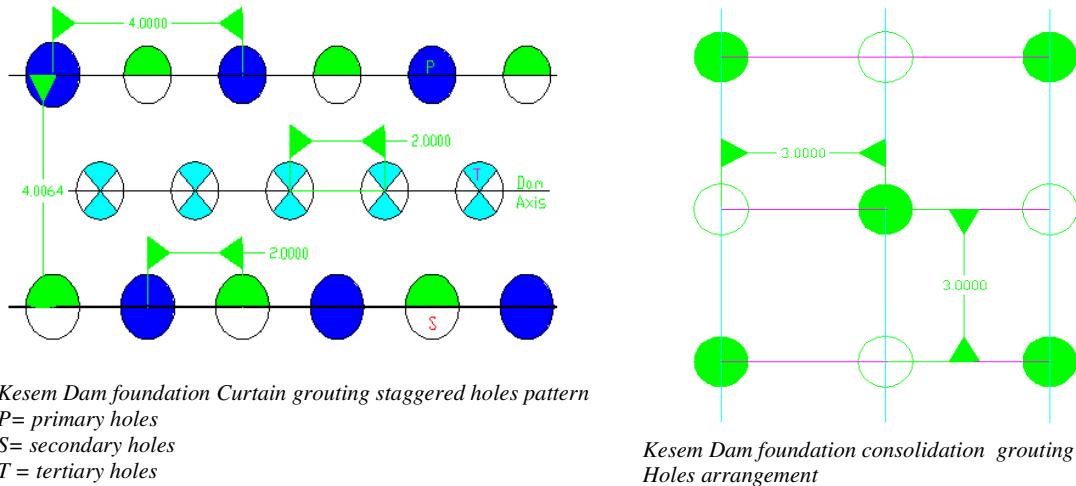


Figure 4.1 Kesem dam foundation grouting holes pattern

According to grouting technical specification (WWDSE, 2005c), the closure requirement is targeted to attain not more than three Lugeon units. The conventional grouting method (use of series of mix adjustments) was used with permeation grouting injection of cement suspension grout. Two types of cements have been used as a major component of the particulate grout: local Portland cements (Ordinary Portland Cement, OPC, & Portland Pozzolana Cement, PPC) were used for the elevated part of the abutment, above the thermal groundwater zone; whereas the portion within the thermal groundwater zone has been grouted using special cement imported abroad (Oil Well cement, OWC). The later is because of safety against the aggressive chemical attack present in the groundwater. Besides, fine sand (up to 30 % by weight of cement) and bentonite (2-3%) were added in the grout whenever it was necessary. The adopted grout rheology was specified to meet certain requirements: bleeding less than 4%, viscosity (marsh funnel) between 35" to 45", Relative cohesion between 0.08 to 1.15mm and Compressive strength at least 5Mpa after 28 days. Progressively increased Grouting pressures were used ranging from 1 bar to 35 bar depending on the geology and confinement condition.

According to KDIPR (2008) the consolidation grouting is limited to the key trench area of the river bed portion of the dam foundation. The grout holes pattern constitute 3x3 square grid arrangement, each extending 10m in depth from the design foundation level and oriented vertical. Similar technical specification to curtain grouting has been used.

4.3.2 Current Status

The Kesem dam foundation grouting work has been started in November 2005 on the right abutment top plateau area.

It was progressing from the top of the abutment towards the river since the diversion of the river flow was not completed at the time.

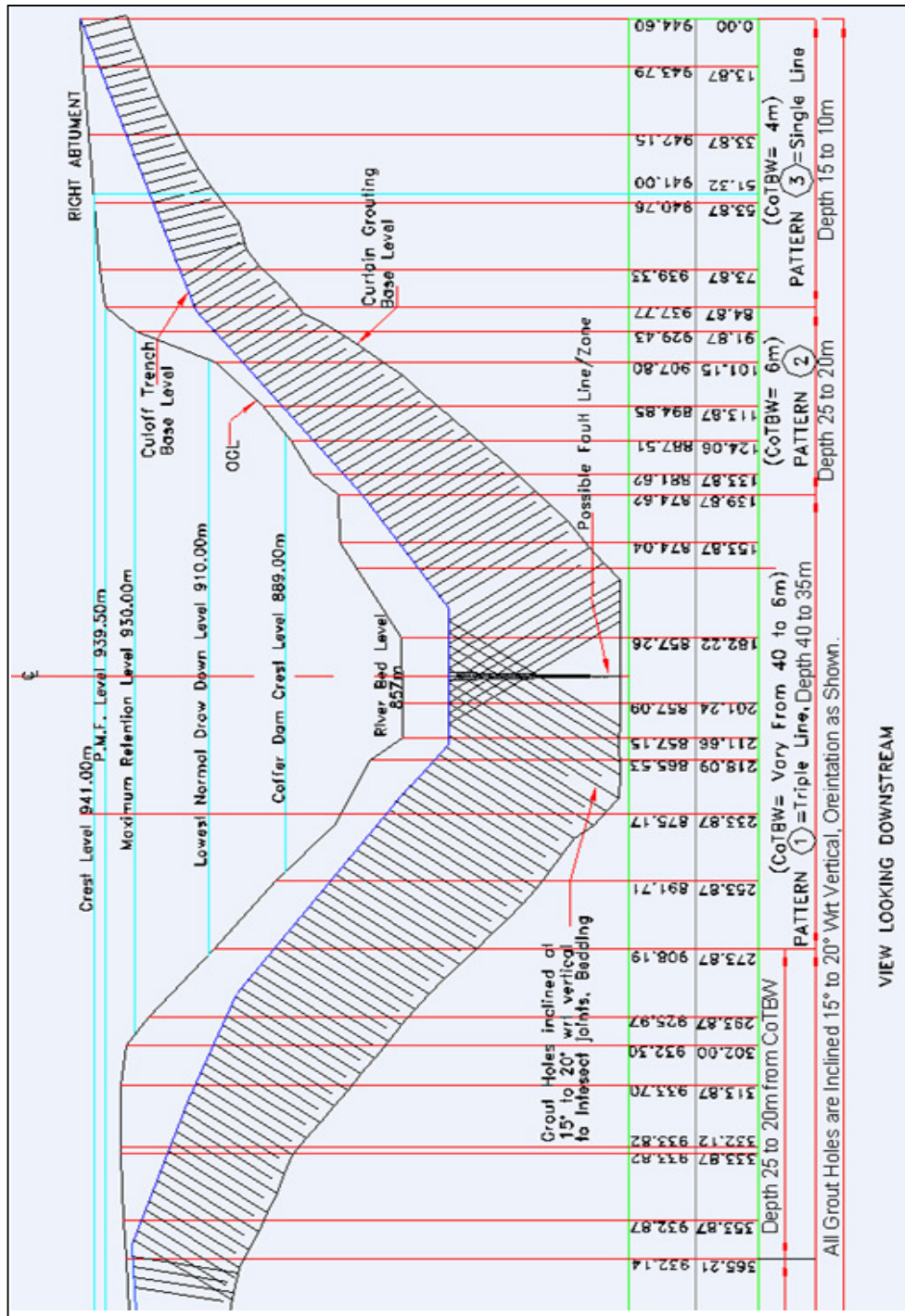


Figure 4.2 Curtaining grouting layout of Kesem Dam foundation grouting (source: WWDSE 2005c)

The grouting operation at the project has not been executed smoothly mainly due to the site condition, complex geology and pressurized thermal artesian groundwater existence. Currently, curtain and

consolidation grouting operations are in progress at the river channel section of the dam foundation. The presence of artesian groundwater and limitation on dewatering conveniences has long been the constraint to the dam foundation excavation and grouting work delay (KDIPR, 2008). Since 2009, river bed grouting has been continuing after partial excavation to elevation of 855m and placing temporary compacted clay back filling up to elevation of 861m (Fig.4.3). The consolidation grouting is planned to serve a double objective; firstly for foundation strength improvement, secondly temporary objective is to confine the groundwater so that excavation to design foundation level would be possible (KDIPR, 2009). Thus, the method incurs extra cost for additional drilling and relatively wider zone adopted for consolidation grouting.

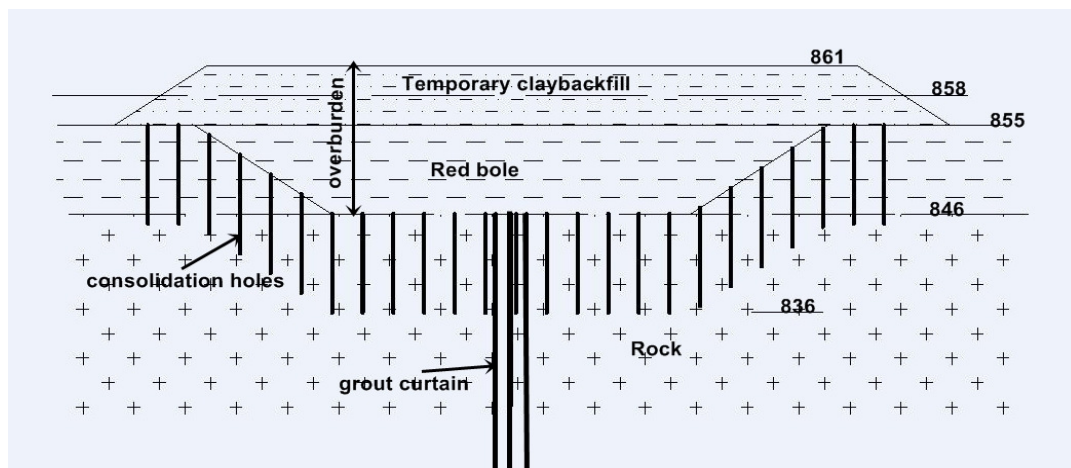


Fig. 4.3 Temporary Claybackfill at the riverbed section for grouting before excavation
(modified from WWDSE, 2008)

Intensive grouting on abutments was carried out integrating very closely spaced rows of grout holes split spaced to 0.4 and 1.5 m on the single and multiple line patterns, respectively. Compared to the initial situation of the foundation watertightness in all grouted zones, significant reduction was attained; however, the anticipated watertightness ($< 3Lu$) could not be satisfied (KDIPR, 2008). The report stated that the major cause to the problem of groutability on abutments is probably related to difficulty of grouting inter granular porosity of the incompetent layers and very fine fractures on hard rock units. Thus, the present research was focused on permeation groutability evaluation of the incompetent layers.

CHAPTER 5 EVALUATION CONSIDERATIONS, TECHNIQUES AND DATA INPUTS

5.0. Preamble

The permeation assessment in the present research is more specific to evaluation of permeation compatibility of void spaces in tuff and volcanic breccia for various particulate and chemical grout types. The analysis is based on primary and secondary data obtained for the medium and grout components and applying selected soil permeation groutability assessment techniques in conjunction with cross-validation of actual conditions. This chapter forwards details of the basic considerations, techniques, input data types and sources utilized in the evaluation of groutability.

5.1. Basic Consideration

The present permeation groutability assessment research has made its basis on considering the unwelded tuff and volcanic breccia deposits (volcano-clastic sediments) as a soil mass so that the basic analysis and interpretation techniques followed for the assessment are well known for permeation groutability assessment application on soils.

Engineering behavior similarity and existing constraints for groutability are taken while correlating the volcano-clastic sediments with engineering soil class. Some of the supporting arguments on this regard are listed here under:

- ✓ The insitu state of the pyroclastics debris on abutments (tuff and volcanic breccia) is fragmented
- ✓ The studied deposits fulfill the basic definition of engineering soil mass. According to Pettijohn (1984), pyroclastic debris deposits such as unwelded tuff and volcanic breccia are considered as “volcano-clastic sediments” composed of grain sizes that could range from Bombs ($\geq 64\text{mm}$) to fine ash ($\leq 0.25\text{mm}$).
- ✓ Possibility to generate required data for analysis (gradation, porosity, permeability) following same procedure as characterization for engineering soil
- ✓ Most of the tuff deposits in the study area are highly dispersive
- ✓ Easily Friable; washout during water pressure testing of sections
- ✓ Very weak in strength; fail to withstand even low grouting pressure or (1 bar/ meter)
- ✓ Grouting of intergranular porosity in both cases.
- ✓ Pressure-Filtration of grout was observed from previous grouting on those zones

5.2. Evaluation Techniques

The evaluation approach followed during the present research can be broadly classified into two categories;

- (i) Analysis of data and interpretation of results applying integrated soil permeation groutability assessment techniques.
- (ii) Cross-validation of findings of approach- (i) with actual condition indications; obtained from previous grouting records and insitu permeation grouting attempted during the present research.

Three well known and efficient preliminary soil permeation groutability assessment techniques were adopted for the present research:

- a) The “groutability Ratios” $N = \frac{D_{15} \text{ of soil}}{D_{85} \text{ of grout}}$ or $N_c = \frac{D_{10} \text{ of soil}}{D_{95} \text{ of grout}}$, suggested as a criteria to evaluate the injectability of suspension grout in soil (Mitchell, 1981; U.S. ACE, 1984; Verfel, 1989) is adopted. Generally, grouting is considered possible for $N > 25$ or $N_c > 11$ and not possible for $N < 11$ or $N_c < 6$.
- b) The average grain size range of the volcano-clastic sediments is analyzed and the corresponding permeation groutability limits for various grout types are interpreted as per the charts provided by Karol (1985) and Littlejohn (1982).
- c) Comparing the maximum grain size of suspension grouts with average effective diameter of voids of medium to be grouted. As per Kennedy (1958), Mitchell (1970), Karol (1985) and Tjandrajana (1989) groutability limiting void width equals $3D_{100}$, where D_{100} is the sieve size for which all the grout particles are finer. For the present research, the effective diameter of the average void in vitric tuff and ground mass of volcanic breccia was estimated based on Littlejohn (1985) relation; $d = 2\sqrt{8\mu k / \delta gn}$ where, d = effective diameter of the average pore, μ = grout viscosity in centipoises, k = permeability of soil, n = porosity of soil, g = acceleration due to gravity and δ = density of water.

The groutability assessment techniques as stated above (a & b) are applied particularly when dealing with suspension grout penetrability. For the present research, three suspension grout types are considered; normal Portland cement grouts (Ethiopian Mughher Ordinary Portland Cement (OPC), Mughher Portland Pozzolana Cement (PPC), ultrafine cement (MC-500) grout and bentonite grout.

For permeation groutability by chemical grouts, the dominant grain size and corresponding average permeability of the Volcanoclastic sediment deposit was analyzed in accordance to permeation limits of various grout types as suggested by Karol (1985) and Littlejohn (1985).

In the case of approach (ii), existing records of previously executed foundation grouting, and insitu permeation chemical test (attempted during current research) were analyzed and interpreted. Existing records are thoroughly examined for the type of grout used, occurrence of ‘grout filtration, applied grouting pressure sufficiency and any related ‘hydro-fracturing. Analysis and interpretation of insitu

permeation test results emphasized on radius of influence grout penetration, improvement in engineering performance of the material, and the magnitude of injection pressure.

5.3. Data Types and Sources

Required and relevant data characterizing the grouting medium and grout component considered in analysis were obtained from laboratory and insitu tests, and reviewing literatures or other secondary sources. Grain size, porosity and Permeability values characterizing the grouting medium are among the basic input data used for the analysis in the present research. Besides, the grain size distribution of the major components of three suspension grout types (normal Portland cement, ultra fine cement, and bentonite/clay) was adopted. Moreover, the review of records of previously executed foundation grouting in the study area has contributed valuable information which was later used for the fortification of the analysis and interpretation for the present research.

5.3.1. Laboratory and Insitu Tests

Grain size distribution, permeability, density and porosity properties of tuff and volcanic breccia deposits were determined from insitu and laboratory tests conducted during the present research.

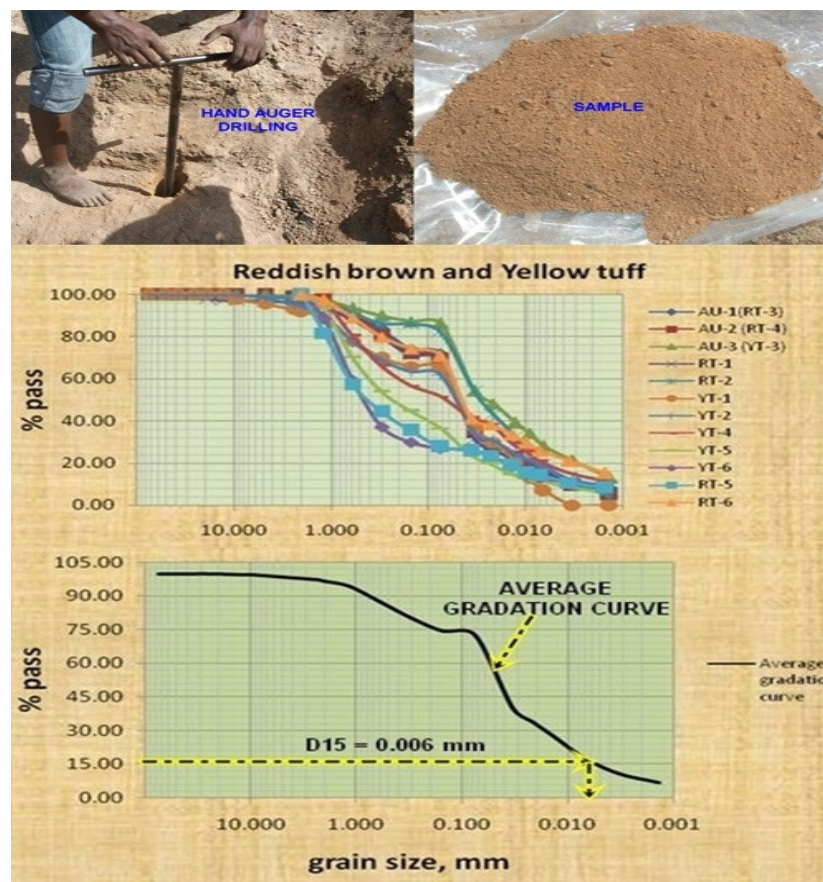


Fig. 5.1 Gradation summary of reddish brown and yellowish tuff samples

Representative samples, collected from tuff and volcanic breccia deposits composing the dam abutments, were analyzed in the laboratory for specific gravity, dry density, permeability and grain size distribution. The Laboratory testing was carried out in Water Works Design and Supervision Enterprise Soil mechanics Central laboratory in Addis Ababa.

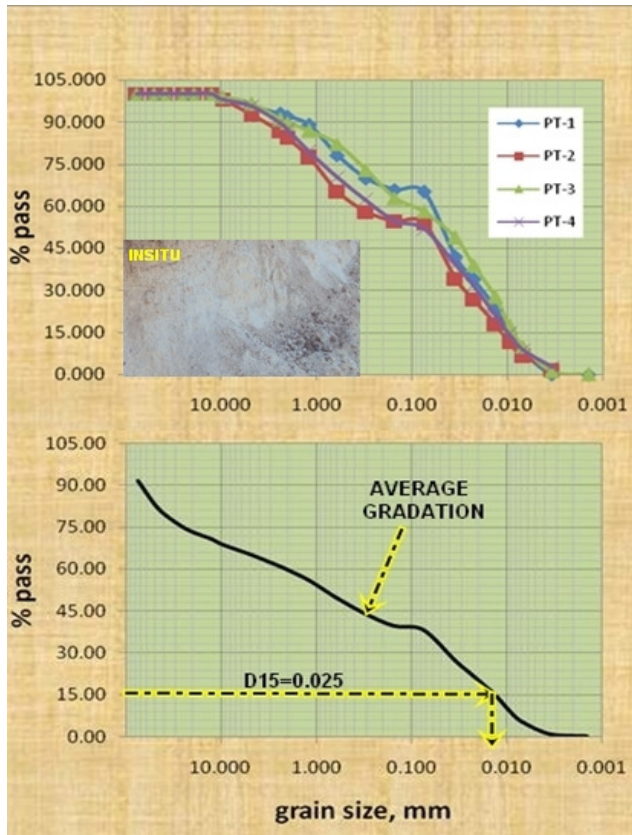


Fig. 5.2 Gradation summary of pummicious tuff

the deposit finer than 4.75mm so that grain sizes above 4.75mm are discarded in accordance to US ACE (1994). Average grain size distribution for each material type was summarized and averaged out as shown in Fig. 5.1, 5.2 and 5.3.

Data on permeability of samples was needed for the determination of effective diameter of pore sizes. Besides, it is correlated with groutability limit in support of the corresponding soil grain sizes as per the charts provided by Karol (1985), Baker (1982) and Littlejohn (1985).

The laboratory permeability test was performed only for tuff samples and was excluded for volcanic breccia because of its high permeability, as manifested from the field observation and its coarser grain size distribution. Sample preparation for laboratory permeability test accounted the insitu density of

Values for porosity were generated indirectly from the corresponding void ratio which in turn was obtained by determining the dry density and specific gravity of the deposits. As indicated in the assessment techniques, the D_{15} or D_{10} sizes of the grouting medium is requisite for the permeation evaluation analysis. Accordingly, combined grain size analysis (sieve analysis and hydrometer) was carried out for each sample.

The permeation groutability problem using normal Portland cement, in view of grain size, is uncertain for sand or finer grain size deposits. For this reason, the permeation analysis on volcanic breccia was preferably emphasized for grain size percent of

the deposit. Insitu density and insitu falling head permeability tests were conducted for the lower yellow and reddish brown tuff layers.

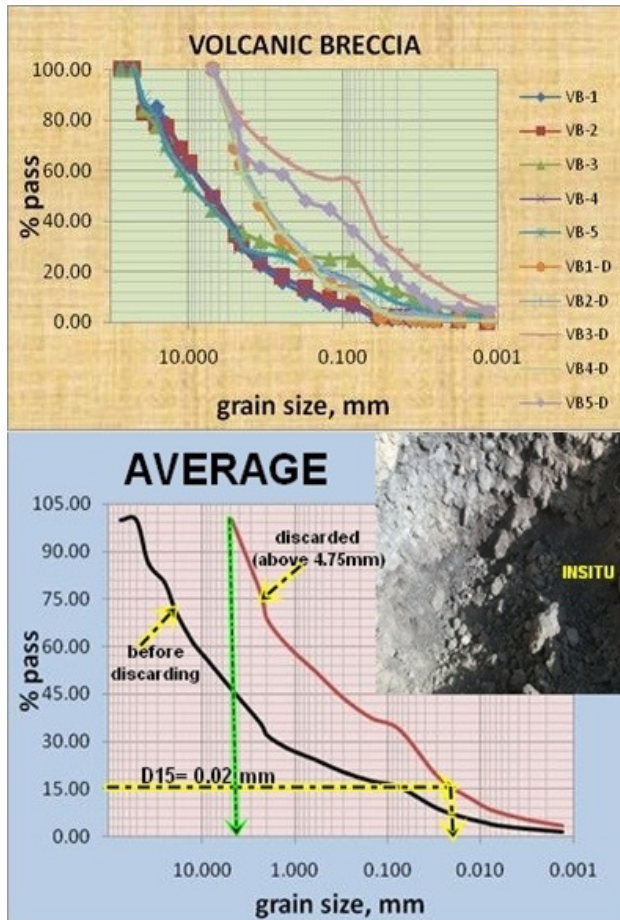


Fig. 5.3 Gradation summary of volcanic breccia

The density tests were conducted at three selected locations using sand replacement method. Similarly, the insitu falling head permeability was performed in three auger holes each having 60mm diameter, and depth drilled to 100cm of the test section length.

A minimum of 24 hrs of continuous saturation, and one meter water head was provided using an extended anchored plastic casing placed just above the test section (Plate 5.2). The raw data records for laboratory and insitu tests are presented in Annexure 1 to 8 with appropriate recording sheet formats. Tabulated summary of laboratory and insitu tests (porosity, permeability and density) and the corresponding results are Summarized and presented in Tables 5.1 and 5.2, respectively.



Plate 5.1 Insitu density test on tuff

An insitu permeation grouting test, using sodium silicate grout, was attempted on a block sample prepared with dimensions of 60cm depth x 60cm width x 50 cm Length. Environs of the block were backfilled with granular material with the view to monitor any possible leakage (Plate-5.3). A 50 cm deep and 60 mm wide test section was bored using a hand operated auger at the center of the above

mentioned test block. The test initially was planned and modeled to be done with the application of safe grouting pressure. However, during the actual test the chemical grout was observed to permeate by gravity only and no expected resistance was offered by the test section, thus no external pump pressure was applied.

The model experimental arrangement during the present study was adopted to minimize the cost and optimum utilization of chemical. For this model experimental pressure application was planned through water circulation system. An expandable plastic tube was used between the two medium, separating the chemical from the water with in the injection pipe. Besides, at the same time this plastic tube also served as a pressure transferring medium to the chemical fluid. The arrangement allowed gradual lowering of the tube level with corresponding permeation, and thus the tube can expands down and push the plastic tube in response to pressure applied by the incoming water.

If it had not been for the constraint of cost and shortage of the chemical grout, a full circulation of chemical grouting with pressure application would have been possible (Fig. 5.4).



Plate 5.2 In situ falling head permeability test on tuff

Table 5-1 Laboratory test result summary

Laboratory test	Reddish brown	Yellow tuff	Pummicious tuff	Volcanic breccia	Remark
Specific gravity, Gs	2.75	2.71	2.42	1.01	Average
Grain size (%)					Minimum and maximum values.
Clay	7.0-9	10-14	0.0-1.0	0.00-3.00	
Silt	60-70	51-53	53-65	6.0-21	
sand	27-29	30-39	27-30	10-23	
gravel	1.0-4.0	7-15	8.0-16	64-70	
Permeability, k (cm/ sec)	7.66 x 10 ⁻³	2.04 x 10 ⁻²	5 x 10 ⁻³	Highly pervious	Average
Porosity, n (%)	68	68	70	---	Average, from void ratio
Dry density, g/cc	0.98	0.89	0.78	1.33	Average

Table 5.2 Field test result summary

Insitu Tests	Tests on reddish and yellow tuff			Remark
	Test-1	Test -2	Test-3	
Dry density (ρ_d , g/cc)	1.16	0.93	1.11	Sand replacement, $\rho_d = \rho_b / (1+w)$
Falling head Permeability (k, cm/sec)	3.8×10^{-4}	1.52×10^{-3}	1.09×10^{-2}	—
Average Porosity (derived from void ratio)	60%			$e = ((G_s \rho_w) / \rho_d) - 1$, Gs = 2.65, from lab. Test

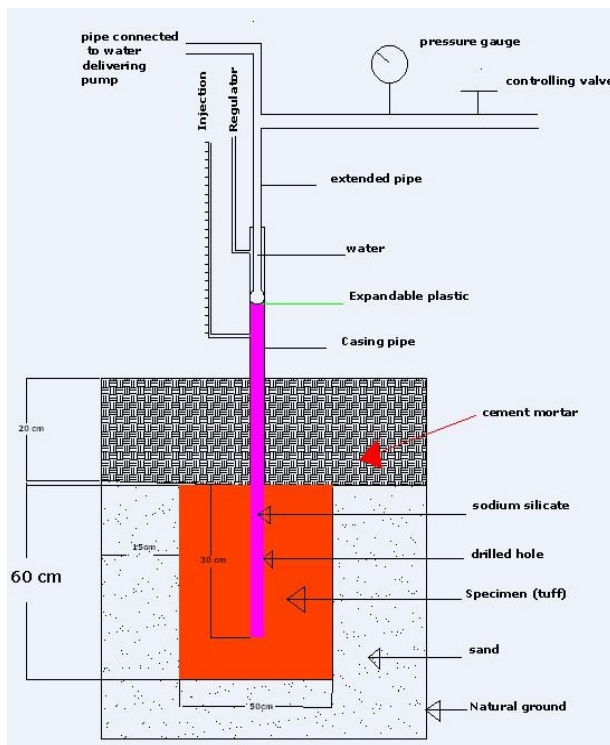


Fig. 5.4 Insitu permeation test model



Plate 5.3 Insitu permeation test

Six observations, each for 5 minutes run, were recorded by falling head technique after reasonable time delay of 10, 20 and 30 minutes. Injection was continuous for 2 hrs and 20 minutes. Later, 3 days after the injection, sample was recovered and observed for the radius of influence of penetration, improvement in strength and for leakage conditions if any.

5.3.2 Grain size Distribution of Suspension Grout Components

The present research has considered three types of particulate/suspension grouts (normal Portland cement, Microfine cement and bentonite grouts) while analyzing permeation groutability of the tuff and volcanic breccia.

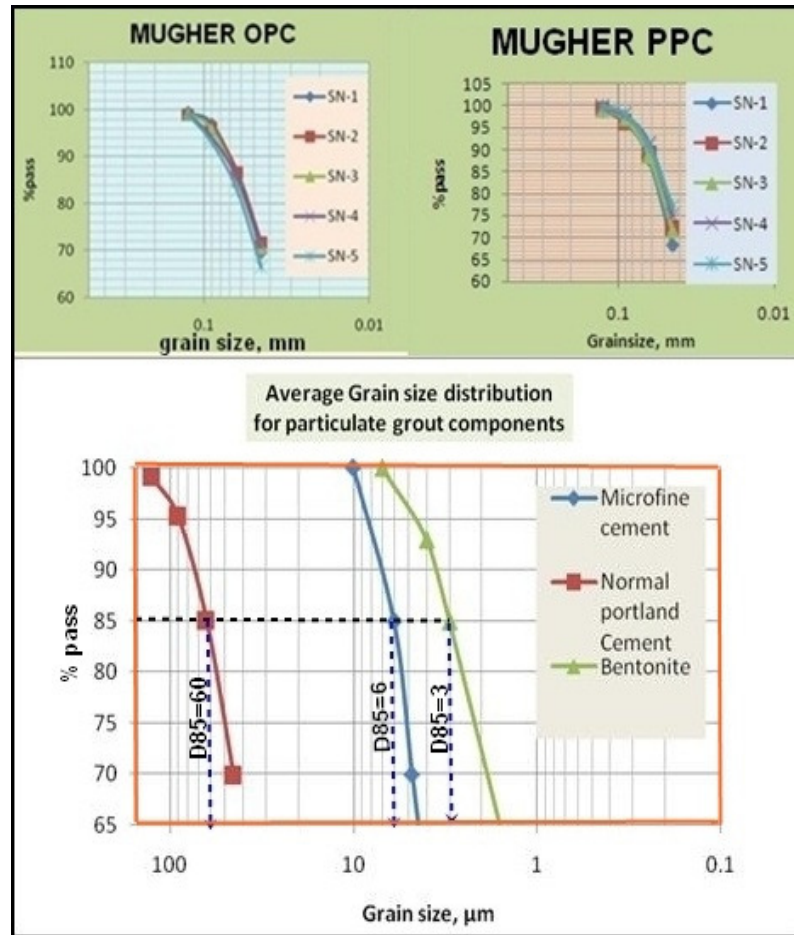


Fig. 5.5 Grain size distribution of suspension grout components

The present technique followed for the permeation groutability analysis inquires the grain size distribution of each particulate grout considered. Accordingly, representative gradation data for each major grout component were procured from available sources. The normal Portland cements (OPC and PPC), grain size data was obtained from local cement factory, Ethiopian Muger Cement Factory Enterprise, and ten independent batch representative grain size analysis records were plotted (Fig. 5.5) and their average curve have been considered for the purpose of analysis. The average grain size distribution for Microfine cement and bentonite were adapted from IS-Tokyo'96. The 85% finer (D_{85}) and the maximum grain size diameter (D_{100}) of each type of grout component aforementioned were utilized while analyzing permeation groutability of the tuff and volcanic breccia on the basis of the

groutability ratio (N), and comparing void size with maximum grain of grout particle in suspension, respectively.

Furthermore, to help on verifying dependability of the semi-analytical approach made using soil permeation groutability evaluation techniques, review and analysis was made for previously executed actual foundation grouting and test grouting records. For more than two years, intensive curtain grouting work was executed on both the abutments of Kesem dam. Moreover, since the grouting has encountered penetrability problems, test grouting programs were carried out at selected places with adjustments to injection controlling parameters such as grouting pressure magnitude, dilution of mix and cement grain size. On the basis of records kept by the project, the grout take and Lugeon value improvements were analyzed along with consideration to grout hole spacing and geology type.

In general, maximum possible efforts were made to use representative values in analysis. For tuff and volcanic breccia samples considered, average values were worked out and used for each engineering parameter of interest. Besides, cross validation between insitu and laboratory test results has shown agreement revealing pervious and loose nature of the deposits except for slightly greater density at insitu condition. Thus, both laboratory and field tests representative values were used during the groutability evaluation. Moreover, the actual condition response for suspension grout and chemical grout injection by permeation was addressed on the basis of analysis of data kept for previous actual foundation grouting, and from indications of insitu sodium silicate injection attempted during current research.

CHAPTER 6

PERMEATION GROUTABILITY EVALUATION

6.0. Preamble

Accurate assessment of groutability of a medium by permeation has always been complicated. The penetrability of grout in to voids is a function of several interrelated parameters contributed from the physical and mechanical property of a medium to be grouted, the nature of grout, the efficiency of groutability assessment techniques used, and other factors related to grouting design and operation. The compatibility among the size of voids in soil or rock, the maximum grain size of suspension particles in grout, the grout rheology (viscosity and cohesion) and maximum allowable grouting pressure applied controls the permeation groutability effectiveness for a particular soil or rock mass. Permeation groutability evaluation for the present study on tuff and volcanic breccia, as explained under section (5.2), was performed primarily by using integrated soil permeation groutability assessment techniques or permeation limits. Besides, it was further supplemented by cross-validation of the actual permeation conditions inferred from thorough analysis of the previous records of grouting, and an insitu experimental study which was attempted during the present study. In each of the analysis technique, the representative value of interest for an engineering parameter was worked out by taking the average value of anticipated worst conditions for permeation. The present chapter in two main portions aforementioned covers the permeation groutability evaluation made in this research.

6.1 Groutability Evaluation Based on Permeation limits

Almost all of the soil permeation groutability evaluation techniques derived so far, directly or indirectly, are a function of the grain size distribution, permeability and grain size category of a given soil (Karol, 1985; Littlejohn, 1985; Baker, 1982). The present research, in place of epiclastic sediments, considered and dealt with permeation groutability of the Volcanoclastic sediments (particularly unwelded tuff and volcanic breccia) based on their average laboratory test results of grain size, permeability and porosity.

For the present study three basic permeation groutability evaluation techniques were adopted:

- ✚ Evaluation based on Groutability ratio ($N = \frac{D_{15} \text{ of soil}}{D_{85} \text{ of grout}}$)
- ✚ Limits of permeation based on soil class, effective grain size (D10) and permeability of grouting medium.
- ✚ Evaluation by comparing the effective diameter of average pore ($d = 2\sqrt{8\mu k/\delta gn}$) with the maximum grain size of suspension in grout (D100).

6.1.1. Groutability Evaluation Based on N

According to Mitchell (1981), U.S. ACE (1984), and Verfel (1989), permeation grouting is generally considered possible for $N > 25$. For each grouting medium studied during the present study (reddish brown and yellow tuff, Pummicious tuff and volcanic breccia), grain size distributions of three types of suspension grout components (normal Portland cement, Microfine cement and bentonite) were adopted for the analysis of groutability by permeation. The 15 percent finer grain size of the medium to be grouted (D_{15}), and 85 percent finer size of the grout components (D_{85}) with corresponding calculated value of groutability ratio (N) are tabulated in Table 6.1. Results, thus obtained revealed the relative increase in value of groutability ratio with grain size reduction of the grouting material for each medium; however, in all the cases, the value of 'N' is much smaller than the lower limit of permeation groutability ($N=25$). Hence, none of the volcano-clastic deposits considered in the present research are permeation groutable by Bingham suspension grouts.

Table 6-1 Groutability ratio (N) values as obtained for normal Portland cement, Microfine cement and bentonite grouts for tuff and volcanic breccia.

Grout component		RT & YT		PT		VB		Remark
Type	D85 (mm)	D15 (mm)	N	D15 (mm)	N	D15 (mm)	N	
normal portland	0.06	0.01	0.10	0.03	0.42	0.02	0.33	N=D15/D85
microfine cement	0.01		1.00		4.17		3.33	
Clay/bentonite	0.003		2.00		8.33		6.67	

N = groutability ratio, D85= the 85 percent finer grain size in suspension grout, D15= the 15 percent finer grain size for grouting medium.

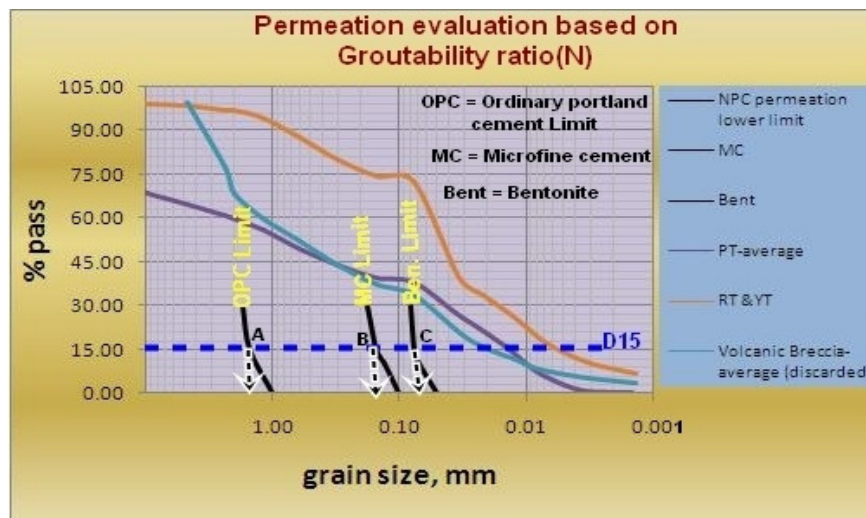


Fig. 6.1 permeation evaluation based on N

A perusal of Fig. 6.1 shows the groutability evaluation on the basis of groutability ratio. Points A, B and C (as indicated in Fig.6.1) corresponds to predetermined or minimum expected D_{15} values of the grouting medium (tuff or volcanic breccia) in order to be permeation groutable by normal Portland

cement, Microfine cement and bentonite, respectively. The value of D_{15} corresponding to each point was calculated considering $N=25$ which is the lower limit of permeation grouting effectiveness for a medium. A medium groutable by normal cement need to have D_{15} not more than 2mm; 0.2mm for the Microfine cement and 0.07 for the bentonite grout. However, as can be seen from the figure 6.1, none of the grain size distribution of the tuff and volcanic breccia satisfies the minimum condition of groutability ratio. Therefore, permeation grouting using suspension grouts could not be effective. Thus, the likely means of getting improvement on desired engineering properties of tuff and volcanic breccia deposits under interest cannot be achieved by suspension grouts.

6.1.2 Groutability Evaluation Based on Karol (1985) and Littlejohn (1985) Permeation limits

During the present study permeation limits derived by Karol (1985) and Littlejohn (1985) were combined and used for the permeation assessment. This combination is based on the grain size class, the effective diameter of the medium (D_{10}) and soil permeability.

Both Bingham suspension and chemical grouts permeation limits were considered in view of the aforementioned properties of the grouting medium. Karol (1985) showed the permeation limits of various grout types from normal Portland cement to resins. In the present analysis these limits are summarized to the major grout category as Bingham suspensions (BG), colloidal solutions (COS) and pure solutions (PS).

Bingham suspensions or particulate grouts are grouts having Binghamian performance, such fluids acts as a rigid body at lower shears stress and they flows like viscous fluid at higher shear stress. The mix comprises of water and one of several particulate solids such as; cement (normal Portland to ultrafine cement), clay/ bentonite, flyash etc. as major components with or without additives (Bruce et al., 1998). Particulate grout components considered in the present case of groutability evaluation are normal Portland cement, Microfine cement and bentonite.

Colloidal and pure solutions are categorized under chemical grout types. The former comprises a mixture of sodium silicate and reagent solutions, and are characterized by an increase in viscosity with time. Pure solutions, on the other hand, offer the advantage of better permeation possessing constant viscosity until setting with adjustable period. Acrylic, Phenol, Aminoplasts and Polyurethane are some common examples of resins / pure solution grout types (Bruce et al, 1998).

According to Karol (1985) and Littlejohn (1985), Bingham suspensions could effectively permeate a grouting medium characterized by grain size as fine, as coarse sand and permeability not more than 10^{-1} cm/ sec. Similarly, the effective permeation limits for chemical grout goes up to fine silts having permeability of 10^{-5} cm/sec. Moreover, Baker (1982) and Littlejohn (1985) specified the effective grain size (D_{10}) of grouting medium to help on better permeation evaluation of groutability.

Accordingly, permeation in voids is effective where D_{10} of soil is not more than 0.5 mm for Bingham suspensions, between 0.5 and 0.02 mm for colloidal solutions and could be even less than 0.02 for pure solutions with requirement of non-plastic nature of the soil.

Table 6-2 Considerations for permeation grouting limit of grouts (adapted from Karol, 1982; combined with Littlejohn, 1985)

Consideration	Bingham Suspension	Colloidal solutions	Pure solution
Soil class	Coarse sands	Fine silt	Fine silts /non-plastic clay
The effective grain size (D_{10})	Greater than 0.5mm	Between 0.5 and 0.02 mm	Less than 0.02
Permeability (cm/sec)	$> 10^{-1}$	10^{-1} & 10^{-4}	10^{-4} to 10^{-5}

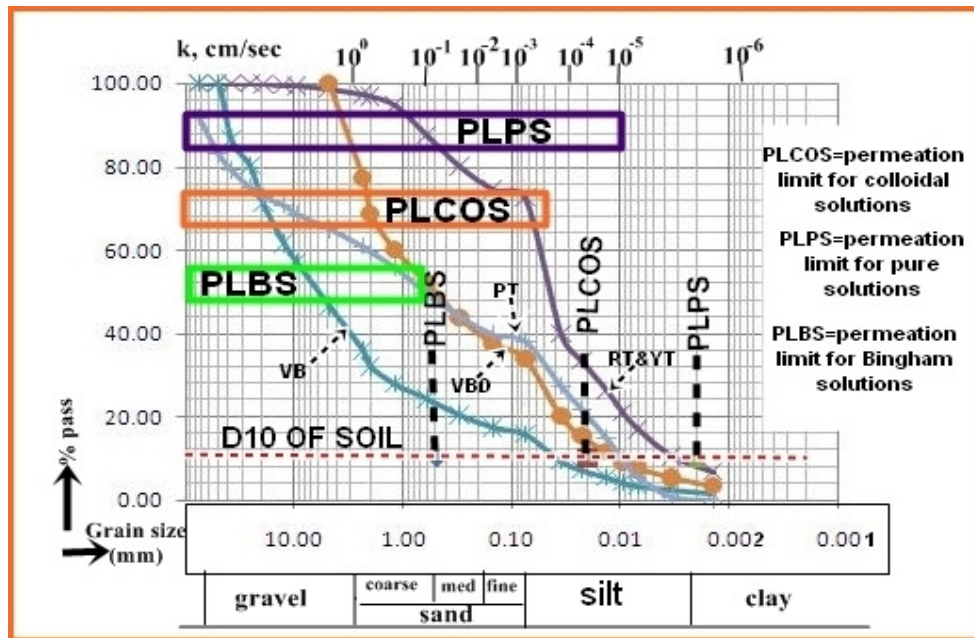


Fig. 6.2 Permeation groutability evaluation based on Karol (1982) and Littlejohn (1985)

The permeation groutability evaluation in the present research has combined all considerations aforementioned as presented in graphical form in Fig. 6.2. The average representative gradation curves of studied tuff and volcanic breccia samples were plotted over the combined permeation evaluation graph. The volcanic breccia was analyzed for two cases; firstly, considering full gradation of the sample (VB), and secondly only for the sand and fines portion discarding the grain size above 4.75mm (VBD). As it can be inferred from the graph (Fig. 6.2), the coarser grain size portion of the volcanic breccia comprising of 16 % coarse grained sand and 55 % gravel, when analyzed for full gradation (VB), it lies within the suspension grout permeation zone. However, the main controlling factor regarding permeation of grout to the deposit is the presence of tuff ground mass. For this reason, permeation analysis was made exclusively for the ground mass of the deposit, sand and fines

portion (VBD). In the later case, the gradation curve mainly fell outside the suspension groutable zone, moreover it is within the effective permeation zone for colloidal solutions. Therefore, the volcanic breccia deposit in the study area could partly accept suspension grouts by permeation; however, the success of the overall grouting could not be satisfied due to non-penetrability of grout grains in to voids of the ground mass.

Table 6-3 Soil class, permeability and effective grain size of tuff and volcanic breccias

Sample	Classification	Permeability	Effective grain size
Reddish brown and yellow tuff	Sandy silt (3% gravel, 27% Sand, 65.5% Silt and 4.5% clay)	3.4×10^{-3}	0.003
Pummicious tuff	Sandy Gravelly silt (34% gravel, 28% sand and 38% silt)	5×10^{-3}	0.015
Volcanic Breccia (No discarding)	Sandy gravel (55% gravel, 39 % sand and 6% silt)	Highly pervious	0.03
Volcanic breccia (discarded)	Silty Sand (65% sand, 32% silt, 3 % clay)	--	0.02

The groutability analysis on tuff samples was also made in a similar manner. It was observed that the samples are completely non-groutable using Bingham suspension grouts, however the samples are permeation groutable with chemical grouts. Since the dominant portion of the sample grain size falls within the range of medium grained sand to coarse silt, the colloidal solutions or silicates satisfactorily permeate the voids.

6.1.3. Groutability Evaluation Based on Void size and Dmax in Grout

The effective diameter of the average pore in the grouting medium (d) was estimated for tuff samples in accordance to Littlejohn (1985) relation $d = 2\sqrt{8\mu k/\delta gn}$ where, d= effective diameter of the average pore, μ = grout viscosity in centipoises, k= permeability of soil, n= porosity of soil, g = acceleration due to gravity and δ = density of water.

Permeation groutability evaluation was made on the basis of permissible limits, as suggested by Kennedy (1958), Mitchell (1970), Karol (1985) and Tjandrajana (1989). For effective permeation groutability of a medium, the void size should be greater than three times of the maximum grain size in grout ($d > 3D_{100}$).

For the present study the field and laboratory test results of permeability and porosity were used for the determination of the effective diameter of the average pore (d) size. The effective grout viscosity (μ) was obtained by considering a moderately thick grout (on average of density 1.35g/cc and funnel viscosity of 32 seconds), and converted into centipoises adopting the relation $\mu = \delta_g (t-25)$, where δ_g = density of grout in cm/sec and t = funnel viscosity in seconds (Encyclopedia, 2009).

Table 6-4 The effective diameter of average pores in reddish brown, yellow and Pummicious tuff

parameters	Yellow tuff			Reddish tuff			pummicious tuff	
	YT-1	YT-2	AU-3 (YT-3)	RT-1	Au-1	AU-2	PT1	PT-2
K	1.00E-04	1.00E-04	1.00E-03	1.00E-04	1.00E-04	1.00E-04	1.00E-04	7.57E-05
μ	10	10	10	10	10	10	10	10
n	0.648	0.78	0.66	0.73	0.7	0.63	0.67	0.72
g	980	980	980	980	980	980	980	980
δ	1	1	1	1	1	1	1	1
d (cm)	0.000709863	0.000647	0.00222428	0.000668807	0.000682988	0.000719932	0.0006981	0.00058593
average, d (mm)	0.007			0.007			0.006	

Table 6-5 D₁₀₀ of considered suspension grout components

Average, mm	D100	3D100
Normal portland cement	0.125	0.375
Microfine cement	0.01	0.03
Bentonite	0.008	0.024

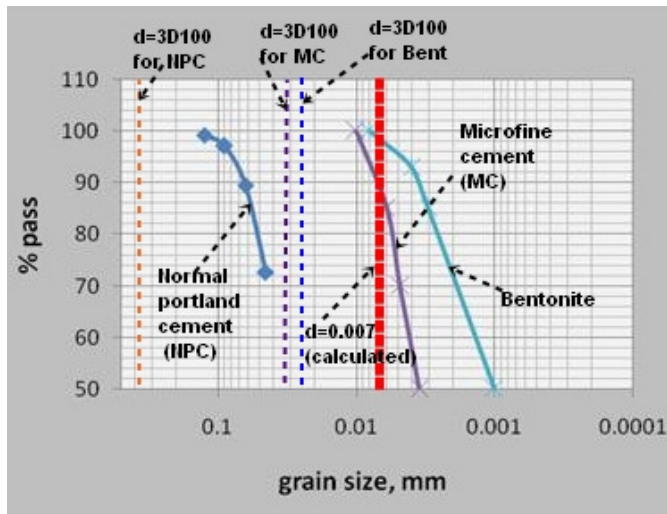


Fig. 6.3 Graphical interpretation of groutability for void size and grain size compatibility

The estimated effective diameter of average pore was found to be smaller than three times of the maximum grain size of particulate grout components considered for the present study (Table 6.4 and 6.5). Fig. 6.3 presents the graphical interpretation of the groutability evaluation. In order to be permeation groutable effectively, the effective diameter of the average pore should fall to the left of corresponding lines marked for normal Portland cement (3D₁₀₀ NPC), Microfine cement (3D₁₀₀ MC) and bentonite (3D₁₀₀ bentonite). However, in each case, the estimated effective diameter of the

average pores fell to the right of the boundary lines of groutability. Therefore, the analysis result revealed that the tuff deposits considered are not permeation groutable by particulate grouts.

6.2. Evaluation Based on Insitu Condition Reactions

For the evaluation of groutability based on insitu condition reactions, mainly secondary data records, as obtained from previous grouting activity, executed by the project, were utilized. Besides, the observations and results from insitu permeation test, as conducted during the present study was also utilized to make interpretations. This has primarily helped to understand the real response of the dam foundation condition for permeation grouting adopted during grouting program. Further, it also enabled the cross-validation of analysis results as obtained from section (6.1) with the actual condition

response. Thus, it made possible verification of the dependability of the preliminary groutability assessment approach followed during the present research.

6.2.1 Previous Normal Portland Cement Permeation Grouting Evaluation

Effectiveness of the permeation grouting was mainly evaluated considering grout take magnitude and the resulting watertightness improvement (% of Lu <3) achieved. Besides, adjacent hole spacing for final grout hole sequence, the geology type, grout mix ratio, grouting pressure magnitude and occurrence of grout filtration were also assessed.

Table 6-6 Summary of Grout hole sequence and spacing for each pattern along with corresponding average grout take, and maximum grouting pressure applied (Source: KDIPR 2008).

Location	Pattern	Hole sequence	Spacing b/n adjacent		Kg/m	Grouting
			holes (m)	lines (m)		
Left Abutment	3	Primary (P)	6	single/ center line	215	0.2
		Secondary (S)	3	single/ center line	79.63	0.3
		Tertiary (T)	1.5	single/ center line	107.4	0.3
		Quaternary (Q)	0.75	single/ center line	28.7	0.3
	2	Primary (P)	6	4.4	472.8	0.2
		Secondary (S)	3	4.4	281.8	0.3
		Tertiary (T)	3	2.2	132.2	0.3
		Quaternary (Q)	1.5	2.2	45.6	0.3
		Quaternary (F)	3	1.1	34.4	0.4
	1	Quinary (v)	1.5	0.55	25.3	1
		Primary (P)	6	3	187.4	0.2
		Secondary (S)	3	3	149.5	0.3
		Tertiary (T)	3	1.5	65	0.3
		Quaternary (Q)	1.5	1.5	56.8	0.3
		Quinary (F)	1.5	0.75	33.7	0.4
Right Abutment	3	Sextary (v)	0.75	0.75	50.1	1
		Primary (P)	6	single/ center line	479.2	0.2
		Secondary (S)	3	single/ center line	303.5	0.3
		Tertiary (T)	1.5	single/ center line	222.9	0.3
		Quaternary (Q)	0.75	single/ center line	108.4	0.3
	2	Quinary (F)	0.37	single/ center line	29.97	0.3
		Primary (P)	6	4.4	173.3	0.2
		Secondary (S)	3	4.4	105.3	0.3
		Tertiary (T)	3	2.2	35.8	0.3
		Quaternary (Q)	1.5	2.2	29.1	0.3
		Quaternary (F)	3	1.1	29.2	0.3
		Quinary (v)	1.5	0.55	27.4	0.4
	1	Sextary (G)	1.5	0.55	14.4	1
		Primary (P)	6	3	159.8	0.2
		Secondary (S)	3	3	63.2	0.3
Tertiary (T)		3	1.5	31.7	0.4	
		Quaternary (Q)	1.5	1.5	22.6	0.4

Intensive grouting work have been carried out on both abutments of Kesem dam foundation deploying very closely spaced grout holes split spaced to quinary sequence. For the gorge area, double line pattern was designed and each line was placed 2.2 meter upstream and down stream of the center line of core keytrench. The initial hole spacing between adjacent primary holes on same line was six meter (KDIPR, 2008).

Record evaluation of previous grouting program revealed that most of the primary and secondary holes encountered alarmingly high grout take (> 350 kg/m, Table 3.1) even at low grouting pressure of 0.2-0.3 bar per meter, (Table 6.4). The initial stage of the grouting operation has involved several intermittent grouting and use of fillers (fine sand). Check hole grouting test results evaluation made

after completion of primary and secondary sequence of holes revealed satisfactory improvement in watertightness and grout take conditions of the foundation (Fig 6.4). However, grouting on the tertiary and further sequences has encountered reverse condition of grout take compared to previous sequences, although water intake was high (up to 146 lit/ min), quick refusal of adjusted grout mixes was a common phenomenon. Consequently, more closely spaced holes were adopted for the grouting, to compensate the limited radius of influence of penetration. The grouting was beyond the design (onwards from tertiary) continued up to quinary sequence. This happened where the adjacent hole spacing and line spacing reached upto 0.75m with a little improvement on watertightness in each case. However, even with this arrangement the anticipated watertightness requirement were not fully attained. This indicates that grouting of primary and secondary holes was successful in filling wide fractures and cavities in the foundation, however fine fractures on rocks and porosities on incompetent layers were left un-grouted in that stage. Thus, further grouting with additional sequence of grout holes was necessary; however this also did not provided required results.

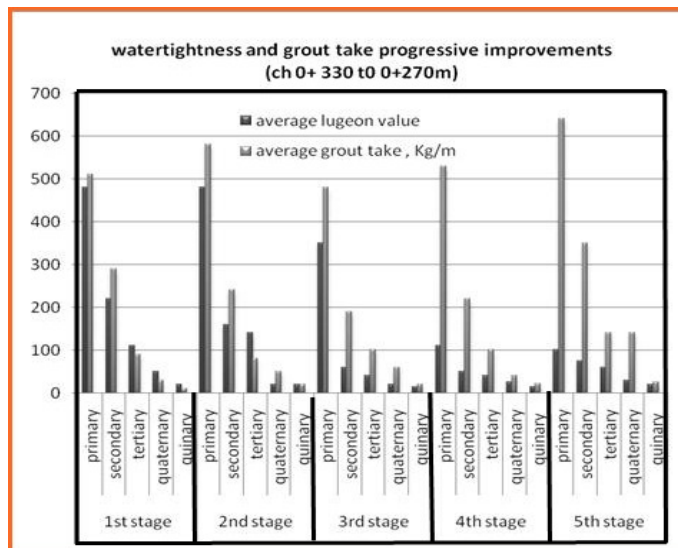


Fig. 6.4 Lugeon and grout take improvements for progressive sequences

Table 6-7 Lugeon statistics for left abutment pattern 2 gorge area after quinary sequence grouting (KDIPR, 2008)

Grouting stages	5 th	4 th	3 rd	2 nd	1 st
% Lu _{≤3}	20	0	11	13	11
% Lu _{<10}	60	57	100	63	66
% Lu _{≤15}	80	56	100	75	100
Average kg/m	16	27	16	66	10

According to KDIPR (2008), the probable causes of the encountered groutability problem is due to penetrability of grout mix into; inter-granular pore spaces of incompetent layers (tuff and volcanic breccia), very fine fractures and secondary infillings in fractures.

The present research has also confirmed this based on the evaluation of grouting record in comparison to the geology.

To investigate for an amendment for better penetration of grout into voids, two major test grouting programs, one on each abutment, were implemented by the project. For this, modification to certain

parameters was made; these include Dilution of mix, increasing grouting pressure and Wet mill grinding of Portland cement grout.

Very thin cement grouts, diluted to 10:1 water cement ratio by volume along with progressive increment of grouting pressure (up to 35 bars) were tested. Besides, wet mill grinding of the cement grout was made in order to reduce the size of cement grain size in grout. However, the grinder used for this purpose did not have appropriate specification record to indicate the achieved amount of size reduction. According to Lombardi and Deer (1993), dilution does not reduce the grain size of cement in grout. It reduces the viscosity of grout to favor penetrability; conversely however, increases rate of sedimentation which would result into reopening of filled joints. Correspondingly, dilution of mix did not bring satisfactory results for the groutability problem encountered at Kesem dam project site.

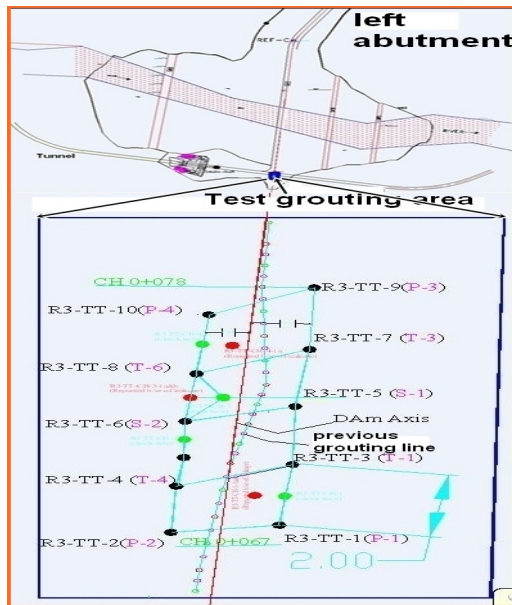


Fig. 6.5 Location of test grouting at the right Abutment

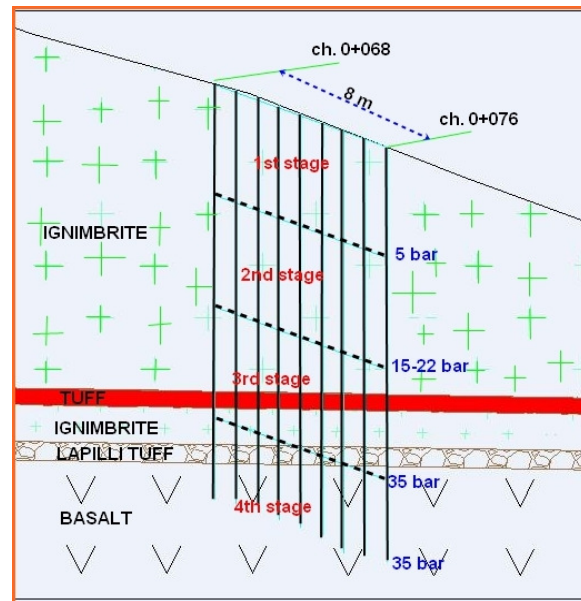


Fig. 6.6 Geology of test zones at the right abutment

Right abutment test grouting was located between chainage 0+068 and 0+076, covering eight meter stretch. Double rows of grout holes at 1m and 0.8 m distance downstream and upstream of center line, respectively were arranged. The primary hole spacing of 8 m, and later split spaced to 2 and 1 m after secondary and tertiary grout holes insertion were provided (Fig. 6.5).

A total of 11 holes were grouted, followed by insertion of 4 check holes. Later, water pressure tests were performed to assess the effectiveness of the test grouting. The test has involved certain adjustments/ modifications over the previous grouting operation. Grouting pressure was applied up to 35 bars, thin to moderately thick grout mix ratios were used (W:C= 3:1 by weight or thicker), and two wet mill grinders were placed after the agitator for further grain size reduction of cement grains in grout. First and Second stage (test section from 0-5m and 5-10m, respectively) laid fully on slightly

weathered fractured ignimbrite rock where as the third and fourth stages (10-15m and 15-20m, respectively) comprised a thin layer of reddish and Lapilli tuff ($\approx 1.5\text{m}$) sandwiched between ignimbrite and basalt rock units (Fig. 6.6). First stage grouting was incomplete due to foundation disturbance in response to high pressure applied.

Figure 6.7 presents the Lugeon achievement reached by the previous grouting (after quinary sequence) and test grouting (after tertiary) conducted at the right abutment. As can be inferred from the graph, previous grouting did not fully satisfy the project watertightness requirement ($\text{Lu} < 3 \geq 90\%$ and remaining $10\% < 5\text{Lu}$, WWDSE , 2500c) to each stage. On the other hand, the test grouting has shown better results comparatively; however, the groutability problem still remained. For the test grouting, Lugeon requirement was satisfied only for 2nd stage. Whereas, 3rd and 4th stages, though showed improvement compared to previous grouting, still did not satisfy the watertightness requirement of the project.

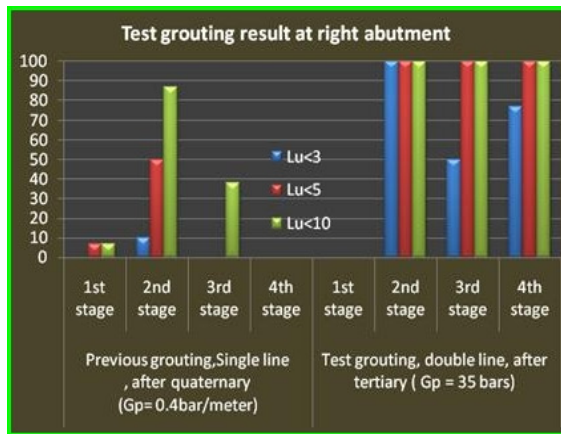


Fig. 6.7 Right abutment test grouting Lugeon achievement summary

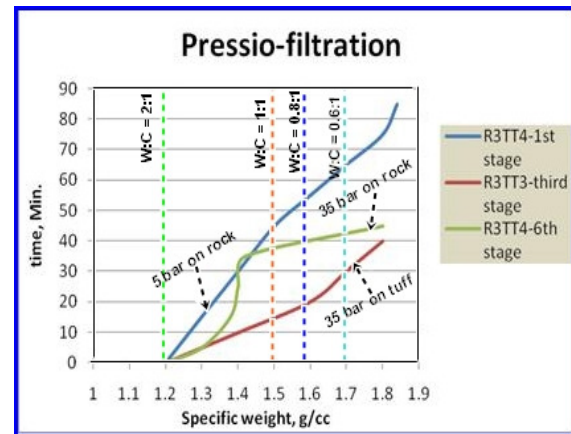


Fig. 6.8 Pressure filtration evaluation

The geology played the main controlling factor for the difference in the Lugeon achievement in the test zones aforementioned. The presence of very thin tuff layers with in grouted stages (3rd & 4th) controlled the effectiveness of permeation grouting. On the other hand it implied the possibility of filling of fine fractures on rock by the test grouting procedure. Grout injection in to fine fractures on rocks has been improved with increment of pressure.

The improvement however was not by the effective permeation as there was considerable grout filtration, formation of filter cake of coarser particles around the finer voids which then results in reduction of cement content of the grout mix actually penetrating in to the voids (Fell et. al, 1992). On the other hand, the improvement could partly come from filling of some of the fine fractures by hydro-jacking which could occur in response to high applied grouting pressure.

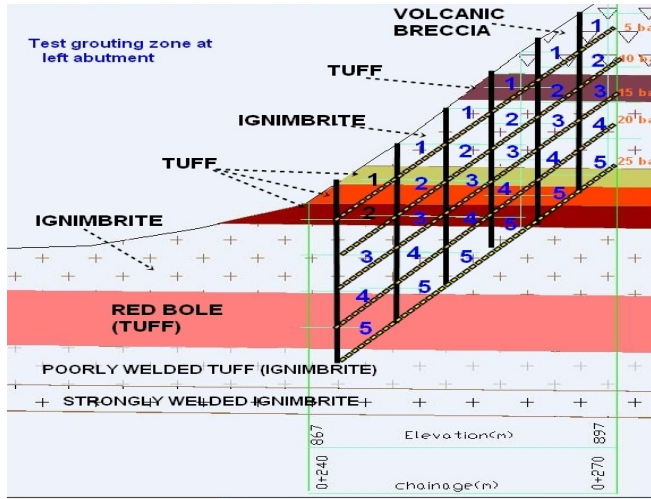


Fig. 6.9 Geology of test section (left abutment)

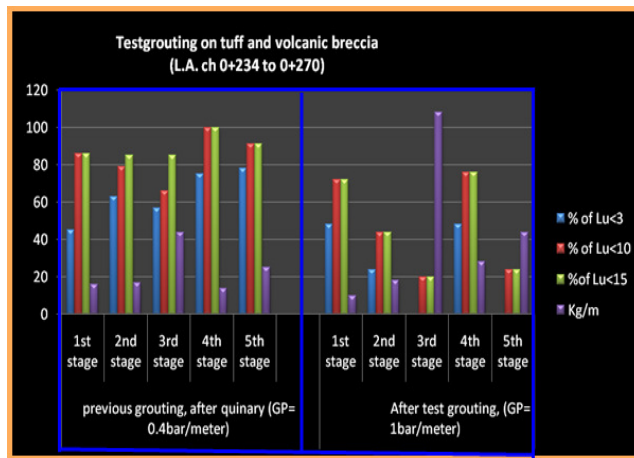


Fig. 6.10 Effect of high pressure permeation grouting on tuff and volcanic breccia



Plate 6.1 Grouting induced hydro-fracturing on tuff (left abutment)

The occurrence of pressure filtration was evident from thickening of return circulating grout compared to the grout initially injected in to hole based on comparison of specific weight measurements made for the grout before injection and for the returning (circulating) grout from hole. As shown in Fig. 6.8, 3:1 or 2:1 water: cement ratio was used as initial grout mix; however, on the course of grouting due to pressure filtration, without any mix adjustment, returning mix was observed and confirmed to thickened shortly to viscous grout ratios.

Additional test grouting was also conducted by the project at the dominant portion of the tuff and volcanic breccias on the left abutment (Fig 6.9). The test has accounted use of increased grouting pressure (5 and 15 bars for 1st and 2nd stages, and 20 bars for 3rd and 4th stages, respectively), and diluted/thin grout (up to 4:1 W: C by weight). Analyzed Lugeon and grout take achievements before and after the test are presented in Fig. 6.10. The analysis has revealed washout of previous grouting and hydro-fracturing occurrence on tuff layers instead of improvement in engineering performance. The percentage of Lugeon value obtained by previous grouting became lower after increased pressure test grouting for same test sections/stages considered.

As remarked on data records, during the test grouting operation, surface leakage and foundation uplift was encountered several times almost everywhere on the slope across the yellow and reddish brown tuff layers. For this reason, grouting was left incomplete on 1st and 2nd stages where the tuff zones were intercepted. With the intention of inspecting hydro-fracturing, after 10 days of completion of test grouting, a test pit was excavated on these tuff layers. Plate 6.1 shows grouting induced hydro-fracturing on the wall and collected tuff samples of excavated pit in the test area (tuff). The grout take recorded for corresponding stages appeared higher for the increased pressure test grouting result. Conversely however, the percent of Lu <3 units became lower than that was attained by the previous low pressure grouting. This is also in concurrence to the occurrence of hydro-fracturing and wastage of grout through instead of penetrating in to voids of the incompetent layers deposit. Moreover, the ease of friability and loose nature of the tuff on either sides of the cement paste filling grouting induced hydro-fracture enlightens that the grout suspension particles were bigger than the permeation limiting size to penetrate in to the pore spaces.

Generally, evaluation of existing records made for previous actual grouting and test grouting that was conducted at Kesem dam abutments revealed that the permeation limitation of grout in to fine fractures on rocks, and pore spaces of incompetent layers was mainly controlled by the coarseness of the adopted normal cement particles in grout suspension compared to the practicable permeation void size on medium to be grouted.

6.2. 2. Insitu Permeation Test Indications

To get signal about the reaction of the actual foundation condition to permeation grouting by chemical grout, insitu permeation grouting test was attempted on the lower yellow tuff portion. The test was conducted at the left abutment dam core keytrench area where a better accessible tuff exposure was found. Sodium silicate grout (colloidal solution; US. ACE, 1995) was used for the test grouting without any additive. The grout type was selected for two main reasons: first and for most sodium silicate grouts are the most popular chemical grouts because of their safety and environmental compatibility (U.S. ACE, 1995). Second reason was the availability of sodium silicate chemical at the Kesem project site. As summarized under section (5.3.1), a continuous injection of sodium silicate was made for 140 minutes with the action of gravitational force only. A total of 13 liters of sodium silicate grout was injected in to the rectangular tuff block insitu sample of volume 0.18m³. Injection observation was monitored for the grout intake rate and total volume of each falling head measurements were also recorded over a scaled time intervals (Table 6.6).

At the early stage of the test, the grout intake was on average at the rate of 247 ml per minutes. After 2hrs and 20 minutes injection, the average rate was decreased to 57.5 ml per minute. However, due to economic constraint the test was stopped before actual complete refusal.

Table 6-8 Summary of insitu permeation grouting test record

Permeation by falling head (Gravity)		
Observation	Duration	Average Rate (ml/min)
1	3:00 pm – 3:05 pm	247
2	3:15 pm – 3:20 pm	215
3	3:40 pm – 3:45 pm	112
4	4:05 pm – 4:10 pm	105
5	4:40 pm – 4:45 pm	85
6	5:15 pm – 5:20 pm	57.5
Grout Type		Na ₂ SiO ₄
Density of Grout		1.28 g/cc
Initial head for each observation		100 cm
Total	Time of continuous injection	140 min.
	Volume injected	13 lit.

Sample was recovered from the test section on the fourth day since test grouting, and visual observation was made for the radius of influence of grout penetration and improvement in engineering performance of grouted tuff. The test block (tuff) was obtained in two conditions: for the zone penetrated by the grout, the grains were found cemented and hardened to siltstone and appeared brownish in color compared to the soft and yellow portion where penetration did not reach (Plate 6.2).

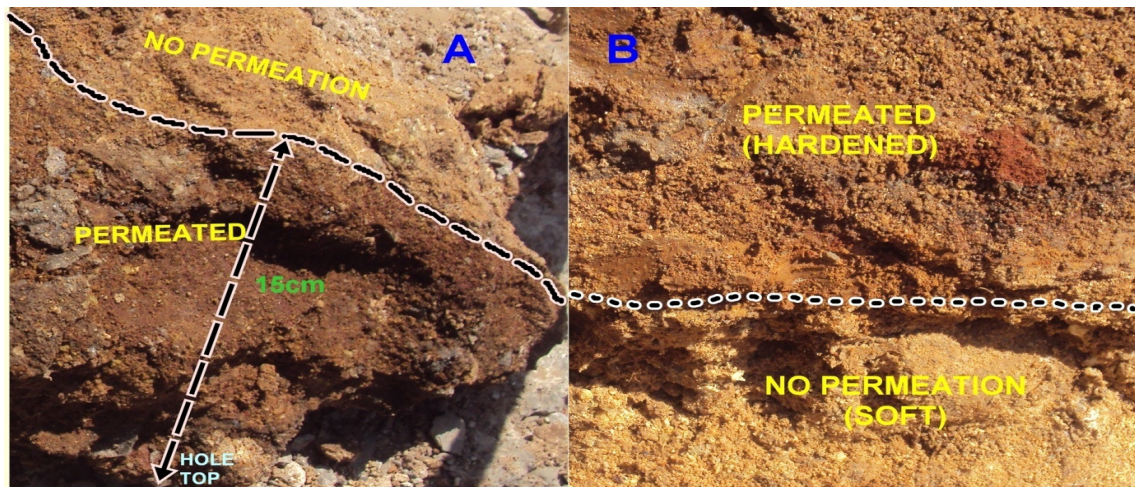


Plate 6.2 Showing status of recovered tuff samples after insitu permeation injection of sodium silicate:
a) Permeation influence at the top of test section;
b) Boundary between permeated and non permeated zone 10 cm below bottom of test hole

Moreover, it was observed that the influence of the grout penetration was radial and it increased with depth. At the top portion of the block, a maximum of 15 cm radius of influence of penetration was measured (Plate 6.2a), and at 50 cm depth (bottom of test section hole), entire section of block was found permeated. Besides, at 60 cm depth (10 centimeter below bottom of test section), a clear boundary between permeated and non permeated zones was observed (Plate 6.2b). As explained under section (5.3.1), pressure was not applied to force the injection. Hence, better radius of permeation influence would result if the sodium silicate is injected with application of even small grouting

pressure. Besides, even for the existing insitu test if the injection was allowed to be continued for more time, better result would have been achieved. Generally, the insitu permeation test has revealed the effective permeation groutability of the tuff with chemical grout which is in good agreement to the evaluation result obtained under section (6.1.2).

6.3. Overall Groutability Evaluation

Analysis results obtained have shown good agreement for both the approaches followed, semi-analytical soil permeation groutability assessment techniques and actual site manifestations. For the unwelded tuff deposits and volcanic breccia ground mass, the groutability ratio (N) obtained, considering suspension grouts of coarser to finest grain size components, fell in the range of 0.0-8.33. These values are much less than the lower limit of permeation groutability. Similarly, for each suspension grout types considered, the estimated effective average diameter of pores on tuff was found narrower than the minimum void size limit required for effective permeation. Moreover, evaluation based on the effective grain size, the permeability and dominant grain size category of tested samples were found in a good agreement with the results aforementioned for suspension grout. Besides, it has proven the effective permeation groutability of all the deposits for chemical grouts. Furthermore, in conjunction with insitu permeation test, it was found out that the colloidal solution or silicate solution grouts could effectively permeate and influence wider area applying low grouting pressure.

The volcanic breccia deposit is comprised of wide range of grain sizes from gravel (Lapilli) to silt size (fine ash) grains plus inclusions of blocks or bombs (breccia). The groutability difficulty of the deposit is mainly controlled by its ground mass (ash). Separate analysis for finer portion (ground mass) and entire composition indicated the partial groutability of the deposit with Bingham suspension grouts (even for normal Portland cement grout) reasonably for the dominant coarse sand and gravel grains in the deposit. However, the finer ground mass requires supplementary chemical grouting for effective permeation satisfying project requirement.

Evaluation of results obtained from analysis of previous actual foundation grouting and test grouting records (kept by Kesem project) revealed the ineffectiveness of cement permeation grouting on incompetent layers and fine fractures on rocks. The occurrence of considerable pressio-filtration of injected grout for both competent and incompetent units, besides hydro-fracturing on incompetent units are the evidences for ineffectiveness of permeation grouting for normal Portland cement grout adopted. The Grout filtration was pronounced for incompetent units, and with increased grouting pressure. The subsequent effect of grout filtration affects the durability of resulting curtain. Therefore, overall evaluation reveals that the pyroclastic-debris composing Kesem dam abutments could effectively be groutable by permeation only by chemical grout.

CHAPTER 7 CONCLUSION AND RECOMMENDATIONS

7.0 Conclusion

Kesem Dam abutments are comprised of thinly to thickly bedded volcano-clastic sediments. These formations are sandwiched in between the competent rock units, thus leads them to form incompetent foundation for the dam. The incompetent units may be associated with three basic geotechnical problems; (i) foundation erosion potential due to their dispersivity and soft/ loose nature which can leads to subsequent piping failure; (ii) clay core cracking possibility for differential settlement potential due to the competence difference between the rock units and volcano-clastic sediments (incompetent units) in the core key trench area at abutment; and (iii) they may form potential leakage path under the dam because of their pervious nature.

One of the remedy designed to improve the engineering performance of the tuff and volcanic breccia was permeation grouting. However, it did not bring satisfactory results. The present research therefore was focused on Permeation groutability effectiveness evaluation on volcano-clastic sediments, particularly on un-welded tuff and volcanic breccia deposits occurig at the dam foundation area. The present research has followed organized and systematic methodology to meet the research objectives framed possessing rigorous literature review, analysis of data collected from secondary sources, in-situ and laboratory tests conducted during present research work. Later, the analyzed results were interpreted to evaluate the effectiveness of permeation grouting of the said incompetent formations.

For grouting perspective, the volcano-clastic sediments were considered similar to engineering soil mass. On the basis of this consideration, the permeation groutability evaluation of un-welded tuff and volcanic breccia was approached. This was primarily done by adopting selected soil permeation groutability assessment techniques: (i) Evaluation based on groutability ratio, N, (ii) Evaluation based on the grain size class, the effective grains size (D_{10}) and permeability of medium, and (iii) Comparing estimated effective diameter of the average pore with the minimum limiting void size for permeation considering the maximum grain size in grout.

Moreover, the evaluation was supplemented by cross-validation of the in-situ condition reaction inferred from scrupulous analysis of records kept for the actual and test grouting that were executed previously at the dam foundation. For the later approach, evaluation was based on improvements achieved on grout take magnitude and the watertightness (% of Lu <3). Besides, adjacent hole spacing for final grout hole sequence, the geology type, grout mix ratio, grouting pressure magnitude and occurrence of grout filtration were also considered.

Furthermore, the cross-validation has involved indications of an insitu permeation grouting test, attempted during the present research. The radius of influence of permeation, magnitude of injection pressure, and resulting improvement in engineering property of medium were analyzed based on records kept during the test, and observation and measurements made for block sample recovered after the test.

The analysis has enabled the identification of suitable grout type for grouting by permeation, for the tuff and volcanic breccia deposits. Accordingly, effective permeation range of the Volcano-clastic deposit has fallen within chemical grout zone, starting from colloidal (silicate) solution onwards (less viscous solutions). Moreover, verification of the dependability of the current research approach for use on preliminary permeation groutability assessment on Volcanoclastic sediments was made possible.

Generally, the analysis and interpretations made by the present research has led to the fore mentioned findings:

1. During the present study various assessment techniques were adopted to evaluate groutability permeation for Volcanoclastic sediments (unwelded tuff and volcanic breccia). All these techniques have ultimately provided similar results for permeation groutability. Besides, the semi-theoretical analysis results were also found to be in concurrence to the actual foundation reaction to permeation groutability. The actual foundation reaction was addressed following evaluation of the records for the actual and the test foundation grouting, previously executed in the project, and from an in-situ permeation test conducted during the present research. Altogether, these have verified the dependability of the soil permeation groutability assessment techniques for its use in preliminary permeation groutability evaluation on Volcanoclastic sediments.
2. Except for partial injectability in to coarser fraction of volcanic breccia, the various tuff deposits and ground mass of the volcanic breccia are proven to be non-groutable with Bingham suspension grouts of normal Portland cement, Microfine cement (MC-500) or bentonite for incompatibility of grain size of suspension grout with the size of voids in the media.
3. Evaluation combining grain size, permeability and effective grain size of grouting media has shown that the ground mass of volcanic breccia deposit (ash) and the rest un-welded tuff layers fell in the chemical grout zone for effective permeation. Moreover, supplementary evaluation from in situ permeation grouting (using sodium silicate grout) considering radius of influence of permeation, injection pressure magnitude and obtained improvement in engineering performance of the media has revealed satisfactory permeation condition for colloidal solutions (silicate solutions).

4. Analysis of results during the present research study revealed that the ineffectiveness of permeation grouting at Kesem dam abutments was mainly controlled by the incompatibility of the grain size of the adopted normal Portland cement suspension grout, and that of void size in the grouting medium (pore spaces on tuff and ground mass of volcanic breccia and very fine or filled aperture fractures on rocks) for permeation.
5. Grouting of fine fractures in competent rock, using the normal Portland cement and applying increased grouting pressure, has satisfied watertightness requirements, as specified in design for the project. The occurrence of significant pressio-filtration of pumped grout; however, revealed the ineffectiveness of the grouting by permeation. The improvement might partly have come from filling of some of the fine fractures by hydro-jacking. Otherwise, the durability of the resulting curtain left questionable for the occurrence of pressio-filtration of pumped grout. This has considerably reduced the cement content of the grout, which actually has penetrated in to voids. On consequences, the resulting cement paste might have formed very weak strength curtain which may not be guaranteed for possible erosion by the seeping water from the reservoir under hydraulic head.
6. As revealed by the analysis result of normal Portland cement test grouting records previously done in the project, the pressio filtration of grout was much higher and has induced hydro-fracturing on incompetent layers in response to injection applying increased grouting pressure. These, in addition to cement grain and void size incompatibility, has indicated that the incompetent layers are too weak to withstand grouting pressure more than 5 bars at surface and 10 bars within confinement. Furthermore, the result disclosed the control of grain size of grout component over grouting pressure magnitude for the groutability problem encountered on incompetent layers composing Kesem dam abutments.

7.1 Recommendations

All efforts were made to conduct the present study in a systematic manner, well supported by actual test results and scientific observations made at the site; however; all these efforts were carried out with the constraint of resource, time and finance. Therefore, it is recommended to make further analysis using more number of samples and in-situ tests for a more dependable and practicable result to come. Besides, the result can be enhanced if direct examination for pore spaces is used in place of empirical approaches, for instance examining the tuff samples using scanning electron microscope for estimation of the effective diameter of the average pore may be carried out. Furthermore, it should be noticed that the permeation groutability evaluation adopted in current research is preferably applicable to conduct preliminary permeation groutability assessment. Therefore, planning a corresponding supplementary test grouting program is advisable before detailed designing or start of operation of an

actual grouting work.

As discussed earlier, three geotechnical problems might be associate with the incompetent layers composing the Kesem dam site; unsatisfied watertightness condition for grouting difficulty, potential for foundation erosion and potential for causing differential settlement on clay core. On the basis of indications of the present research findings and scientific background, the fore mentioned recommendations are forwarded to help on amending those problems or looking for alternative options.

- ✓ The current research has shown the effective permeation of chemical grout in to voids of incompetent layers. Accordingly, as observed from recovered samples after insitu permeation test, the grains were found cemented and hardened owing to filling of voids by the chemical grout which proved the improvement in engineering performance of the deposit. However, applying chemical grouting for Kesem dam site should be viewed as an ultimate option for the reason of economic consideration and environmental safety factor. Chemical grouting is expensive for chemical cost, unavailability in local market and specialized operational requirements. Besides, most chemicals are not environmentally friendly. Furthermore, the presence of groundwater at the dam site can aggravate impact on environment. Otherwise, permeation chemical grouting surely brings satisfactory results for improvement in watertightness and strength of the incompetent units.
- ✓ As revealed from previous test grouting evaluation, the ease of inducing hydro-fracturing on incompetent layers could be taken as clue for possibility of grouting by fracturing technique (CLAQUAGE). This can be done using the normal Portland cement grout and applying high grouting pressure with counter balanced by sufficient concrete grout cap placed at surface against foundation uplift. The technique results in constructing a thin grout curtain through the incompetent mass by pumped grout induced hydro-fracturing and filling. A test grouting program can be laid down in advance to assess the arrangement of holes, grouting pressure magnitude and grouting equipment compatibility for implementation of the technique.
- ✓ According to Houlsby (1985), an embankment dam project with (i) wide core design, (ii) little or no risk to foundation erodibility, (iii) multiple row curtain design, and (iv) less sensitivity to water loss can be designed for watertightness requirement in the range of 7-15 Lugeon units. The closure requirement Lugeon specified for Kesem Dam project foundation grouting (less than three Lugeon units) can be relaxed to the aforementioned range. The fore mentioned justifications were under taken to evaluate the Kesem dam site and design conditions agreement with Houlsby's considerations.

- I. The embankment dam clay core has a width wider than 50% of the dam height. Thus considered as a wide clay core.
- II. Though the foundation and abutments comprised of erodible materials, a dependable prevention measure are provided in the design. Layers of graded filter and geotextile are made part of the design where in competent layers are exposed. Thus, it is rational to abandon foundation erodibility risk.
- III. Water seeping through the dam abutments foundation can be safely guided to join the river channel downstream which is already considered as main canal for bearing the irrigation water for about 11 km until joins the diversion weir. Besides, if there is any possibility of excessive seepage, it can be counterbalanced by regulations from the irrigation intake. Thus, there would not be significant control of the sensitivity factor for water loss.
- IV. Except the shallow pressure hydraulic zones (pattern three, plateau portion of abutments), multiple row curtain was designed and constructed.
- V. Deep keytrench excavation was made at the abutments

Therefore, It is an alternative and justified way to relax the watertightness requirement of the project considering the points stated above (from I-V) along with closure Lugeon requirement considerations provided by the famous Author who contributed a lot to the development of the Art, A. C. Houlby-grouting engineer. This therefore, would significantly minimize the remaining grouting volume of work at the abutments.

- ✓ In case of preference to the Lugeon requirement relaxing option, the toe drain level should be re-examined if any tail water level change could be imposed.
- ✓ Grouting of fine fractures on rocks using the normal Portland cement was evident for grout filtration and hence could not be guaranteed for non-erodibility in the future. If it is found necessary to apply additional grouting on the rock, effective result would be obtained for a single line holes provided that finer cement than normal Portland is used. In case of limitation to adopt finer cement grout, use of dispersants in grout along with three or more series of appropriate wet mill grinders can give better results on the basis of test grouting results and referring effect of dispersants as discussed in literatures cited in Fell, et.al 1992.
- ✓ At the dam site, several faults were traced crossing the dam axis and having extension in to the reservoir. These faults are threats to stability and excessive water loss. To amend instability problem suspected from the faults presence, the dam design has incorporated satisfactory dental treatment designs. The trace of the faults revealed their interconnection thus may form potential leakage path. Faults at the river section and left abutment are with in the longitudinal extent of the grout curtain, where as faults at the right abutment lied away from the grout curtain extension. Besides, except for the river bed fault, no special consideration for grout hole depth and

arrangement was designed at fault zones on abutments. Therefore, it is advisable to extend the grout curtain at the right abutment to include the fault zones. Moreover, detail assessment of the extent and depth of the faults at both the abutments should be examined and review of design for grout hole depth and arrangements should be made.

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Kesem Dam and Irrigation Project
 Summary of Test Results
 Date: 09/12/2010

Sample No	Dry Density(gm/cc)	Specific gravity	Porosity (%) ($e = [(G_s \gamma_w / \gamma_d) - 1]$),	Porosity(%) $\eta = (e/(e+1))$
PT-1	0.82	2.56	2.12	67.99
PT-2	0.71	2.54	2.58	72.08
YT-1	0.88	2.51	1.85	64.81
YT-2	0.75	2.78	2.71	72.94
AU-1	0.81	2.72	2.36	70.37
AU-2	0.91	2.51	1.76	63.82
AU-3	0.87	2.63	2.02	66.96
RT-1	0.74	2.75	2.72	73.26
VB-1	1.28	2.72	1.13	52.80
VB-2	1.46	2.73	0.87	46.40
VB-3	1.32	2.80	1.12	52.94



[Handwritten signature]

Annexure I summary of dry density, specific gravity and porosity of tested samples

Kesem Dam and Irrigation Project
Summary of Test Results
Date: 09/12/2010

Parameters	AU-3	RT-1	VB-1	VB-2	VB-3
Grain Size Analysis					
Clay %	17.00	8.50	0.22	0.00	3.00
Silt %	69.41	60.32	5.97	7.42	21.79
Sand %	12.75	27.00	23.68	23.77	10.60
Gravel %	0.84	4.18	70.13	68.81	64.61
Permeability (cm/sec)	1.25×10^{-1}	5.28×10^{-6}	-	-	-
Porosity (%)	66.96	73.26	-	-	-

Parameters	PT-1	PT-2	YT-1	YT-2	AU-1	AU-2
Grain Size Analysis						
Clay %	0.00	1.00	14.50	10.00	8.00	7.00
Silt %	65.14	52.96	38.80	51.94	63.74	63.55
Sand %	26.93	30.54	39.03	32.50	27.20	28.84
Gravel %	7.93	15.50	7.67	5.56	1.06	0.81
Permeability (cm/sec)	2.54×10^{-4}	7.57×10^{-4}	1.27×10^{-4}	8.22×10^{-5}	1.09×10^{-1}	1.21×10^{-1}
Porosity (%)	67.99	72.08	64.81	72.94	70.37	63.82

Checked By 

Approved By 



Annexure II summary of grain size and permeability of tested samples

FALLING HEAD PERMEABILITY TEST RECORD			
LOCATION	left abutment	SATURATION Hrs (Min)	24
ELEVATION (m)		SATURATION (lit)	135
HOLE No.	AU-1	CONSTANT HEAD (cm)	100
HOLE DEPTH (cm).	100	TESTING DATE	12/10/2010
HOLE DIAM. (mm)	100	TESTED BY	DAWIT F.
MATERIAL TYPE	reddish brown loose tuff (Sandy silt)		

TESTING TIME				WATER HEAD LEVEL (cm)		DRAW DAWN (cm)
START (HH:MM)	FINISH (HH:MM)	ELAPSED (Min)	CUMMU. (Min)	STARTING	DYNAMIC	
3:00 PM	3:01	1	1	100	98.1	1.9
3:01	3:02	1	2	100	97.3	2.7
3:02	3:03	1	3	100	96	4
3:03	3:04	1	4	100	94.9	5.1
3:04	3:06	2	6	100	93.1	6.9
3:06	3:08	2	8	100	91.9	8.1
3:08	3:10	2	10	100	90	10
3:10	3:15	5	15	100	86.4	13.6
3:05	3:20	5	20	100	83.5	16.5
3:10	3:25	5	25	100	80	20
3:15	3:30	5	30	100	77.1	22.9
3:20	3:35	5	35	100	74	26
3:25	3:40	5	40	100	71.6	28.4
3:30	3:45	5	45	100	69.3	30.7
3:35	3:50	5	50	100	66.5	33.5
3:40	3:55	5	55	100	64.6	35.4
3:45	3:60	5	60	100	61.9	38.1

$$k = \frac{\ln\left(\frac{h_1}{h_2}\right) \Pi r}{5.5(t_2 - t_1)} \dots \dots \text{for insitu falling head (adapted from V.N.S.Murthy, 1989)}$$

$$k = 3.8 \times 10^{-4} \text{ cm/sec}$$

(A)

CONSTANT HEAD PERMEABILITY TEST RECORDING SHEET			
LOCATION	left abutment	SATURATION Hrs (Min)	24
ELEVATION (m)		SATURATION (lit)	150
HOLE No.	AU-2	CONSTANT HEAD (cm)	100
HOLE DEPTH (cm).	100	TESTING DATE	12/10/2010
HOLE DIAM. (mm)	100	TESTED BY	DAWIT F.
MATERIAL TYPE	reddish brown loose tuff (Sandy silt)		

TESTING TIME				water intake (q)	
START (HH:MM)	FINISH (HH:MM)	ELAPSED (Min)	CUMMU. (Min)	lit/min	cummulative (lit)
4:00 PM	4:05	5	5	0.5	2.5
4:05	4:10	5	10	0.3	4
4:10	4:15	5	15	0.32	5.6
4:15	4:20	5	20	0.2	6.6
4:20	4:25	5	25	0.2	7.8
4:25	4:30	5	30	0.2	8.5
4:30	4:35	5	35	0.2	9.5
4:35	4:40	5	40	0.2	10.3
4:40	4:45	5	45	0.2	11.4

$$k = \frac{0.18q}{rh} \dots\dots\dots \text{for insitu constant head (adapted from V.N.S.Murthy, 1989)}$$

Where k= permeability in cm/sec

q= flow rate in cubic centimeter second

r= radius of hole

h= constant head of testing

$$k=1.52 \times 10^{-3} \text{ cm/sec}$$

(B)

FALLING HEAD PERMEABILITY TEST RECORD			
LOCATION	left abutment	SATURATION Hrs (Min)	24
ELEVATION (m)		SATURATION (lit)	205
HOLE No.	AU-3	CONSTANT HEAD (cm)	100
HOLE DEPTH (cm).	80	TESTING DATE	12/10/2010
HOLE DIAM. (mm)	100	TESTED BY	DAWIT F.
MATERIAL TYPE	YELLOW tuff (Sandy silt)		

TESTING TIME				WATER HEAD LEVEL (cm)		DRAW DAWN (cm)
START (HH:MM)	FINISH (HH:MM)	ELAPSED (Min)	CUMMU. (Min)	STARTING	DYNAMIC	
5:00 PM	5:01	1	1	100	62	38
3:01	5:02	1	2	100	51	49
3:02	5:03	1	3	100	43	57
3:03	5:04	1	4	100	37	63
3:04	5:06	2	6	100	27	73
3:06	5:08	2	8	100	20	80
3:08	5:10	2	10	100	10	90

$$k = \frac{\ln\left(\frac{h_1}{h_2}\right) \Pi r}{5.5(t_2 - t_1)} \dots \dots \text{for insitu falling head (adapted from V.N.S.Murthy, 1989)}$$

$$k = 1.09 \times 10^{-2} \text{ cm/sec}$$

(C)

Annexure III - (A, B, C) Insitu falling head permeability test records

Insitu Density Test Record

Location	Kesem Dam, left Abutment	Testing method	Sand Replacement
Hole depth	15cm	Unit weight of sand	1.3 g/cc
Tested By	Dawit Fikru		

Material type	Reddish tuff		Yellowish tuff
Wt. of wet soil from hole (W, g)	4174	4052	4190
Wt. of sand + jar before pour (w1, g)	10,000	10000	10000
Wt. of sand + jar after pour (w1, g)	4989	4711	5156
Wt. of sand in cone (w3)	1454	1454	1454
Wt. Of sand in hole (w1-w2-w3)	3557	3835	3390
Bulk density (δ_b)	1.53	1.37	1.61
Moisture content determination			
Wt. of wet soil (g)	4174	4052	4190
Wt. of dry soil (g)	3180	2770	2900
Wt. of moisture (g)	994	1284	1290
Moisture content (m, %)	31.3	46.3	44.5
Dry Density = Bulk Density/(100+m)*100	1.16	0.936	1.11

Annexure IV -Insitu density test record

Determination of porosity from dry density and specific gravity

Dry density determined in the field by sand replacement method

$$\delta_d = \delta_b / (1+w)$$

Where, δ_b is the bulk density, w= the moisture content

$$\text{Average value of } \delta_d = 1.06 \text{ g/cc}$$

The porosity is computed from the void ratio (e):

$$n = e / (1+e)$$

From soil weight-volume relation:

$$e = ((G_s \delta_w) / \delta_d) - 1 \text{ where, } \delta_w \text{ is the density of water}$$

From laboratory test specific gravity of solids (Gs) is taken to be = 2.65

Average value of porosity of undisturbed state (insitu) of tuff = 60%

Permeation grouting (insitu test) record

Location	Left abutment with in keytrench	Elevation	870 m
Tested medium	Yellow tuff	Grout type	Sodium silicate
Grouting pressure	Head of 1 m (by gravity)	Density of Na ₂ SiO ₄	1.28 g/cc
Tested by	Dawit Fikru	Date of testing	25/11/2010

observation-1				
testing time			Injected volume	
start (HH:MM)	finish (HH:MM)	Interval (cumm; Min)	draw dawn (cm)	ml
3:00pm	3:01	1	25.5	318.75
3:01	3:02	2	51	637.5
3:02	3:03	3	72.5	906.25
3:03	3:04	4	82	1025
3:04	3:05	5	99	1237.5
average rate = 247 ml/ min				

Observation-2				
testing time			Injected volume	
start (HH:MM)	finish (HH:MM)	Interval (cumm; Min)	draw dawn (cm)	ml
3:15	3:16	1	17	212.5
3:16	3:17	2	34	425
3:17	3:18	3	46	575
3:18	3:19	4	63	787.5
3:19	3:20	5	86	1075
average rate = 215 ml/ min				

Observation-3				
testing time			volume	
start (HH:MM)	finish (HH:MM)	Interval (cumm; Min)	cm	ml
3:40	3:41	1	8	100
3:41	3:42	2	18.5	231.25
3:42	3:43	3	29	362.5
3:43	3:44	4	37	462.5
3:44	3:45	5	44.8	560
average rate = 112 ml/ min				

Observation-4				
testing time			volume	
start (HH:MM)	finish (HH:MM)	Interval (cumm; Min)	cm	ml
4:05	4:06	1	8	100
4:06	4:06	2	17	212.5
4:07	4:07	3	27	337.5
4:08	4:08	4	34.5	431.25
4:09	4:10	5	42	525
average rate = 112 ml/ min				

observation-5				
testing time			volume	
start (HH:MM)	finish (HH:MM)	Interval (cumm; Min)	cm	ml
4:40	4:41	1	5	62.5
4:41	4:42	2	10	125
4:42	4:43	3	18	225
4:43	4:44	4	26	325
4:44	4:45	5	34	425
average rate = 85 ml/ min				

observation-6				
testing time			volume	
start (HH:MM)	finish (HH:MM)	Interval (cumm; Min)	cm	ml
5:15	5:16	1	4	50
5:16	5:17	2	7	87.5
5:17	5:18	3	13	162.5
5:18	5:19	4	18	225
5:19	5:20	5	23	287.5
average rate = 57.5 ml/ min				

N.B. Total volume injected= 13 liters with in a period of 140 minutes

Annexure V- Insitu chemical grouting test record