Foundation Assessment:
A case in Megech Dam Project, Northwestern Ethiopia

Abebe Mihret

A Thesis Submitted to

School of Earth Sciences

Presented in Partial Fullfillment of the requirements for the Degree of Masters of Science (Engineering Geology)

ADDIS ABABA UNIVERSITY

Addis Ababa, Ethiopia

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SIGNATURE PAGE

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School of Graduate Studies

This is to certify that the thesis prepared by **Abebe Mihret**, entitled: *Foundation Assessment: A case in Megech dam Project, North-western Ethiopia.* and submitted in partial fulfillment of the requirements for the Degree of Master of Science (Engineering Geology) complies with the regulations of the University and meets the accepted standards with respect to originality and quality.

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ABSTRACT

Foundation Assessment: A case in Megech dam Project, North-western Ethiopia.

Abebe Mihret

Addis Ababa University, 2020

Dams are civil engineering structures built to impound large volume of water for different purpose and are built in different sizes, shapes and types. Geotechnical and Engineering geological assessments play a great role in deciding size, shape, type, location and ensure safe performance of dams. The present study was carried out at Megech dam project which is an earth –rock fill embankment type dam with central clay core and is 76m high from the river bed. The study area is located in Amhara National Regional State, in Northwestern Ethiopia about 740 km from Addis Ababa. The main objective of the present study was to carry out geotechnical and engineering geological investigations to address Rock mass quality, permeability and deformability of the rock mass in the dam foundation. Besides, bearing capacity and settlement potential of piles under the intake and conduit outlet works of the dam foundation was also assessed. In addition, effect of groundwater chemistry on submerged piles was also studied.

For the present study systematic investigation was made to address the geological and geotechnical conditions prevailed in the dam foundation. Primary and secondary data was utilized to know the rock mass properties and to classify the rock mass through rock mass rating system. Further, the strength and deformability characteristics of the rock mass were estimated through empirical relations using rock mass rating system (RMR). Rock mass classification, deformability and strength results of the rock mass at dam foundation suggests low quality rock mass on left abutment as compared to the right abutment. In addition permeability of rock mass in dam foundation was assessed based on lugeon test results which indicated necessity of (improvement) grouting in the dam foundation with extensive grouting in the left abutment. The bearing capacity and settlement assessment for piles under conduit and intake structure suggests relatively high allowable bearing capacity for the right abutment as compared to the left abutment. Further, the study also suggests that the piles submerged in water under the intake and conduit outlet works will safely serve their design period without any deterioration resulted from the ground water aggressiveness.

Key words: Rock Mass Rating, Foundation, Bearing capacity, Permeability, Settlement

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LIST OF ABBREVIATIONS

ASTM	American Society for Testing and Materials
a.s.l	Above sea level
BH	Borehole
C	Cohesion (Kpa)
Ed	Deformation modulus (Gpa)
ESA	External sulphate attack
F- max	maximum shaft friction capacity
FAO	Food and Agriculture Organization
GPS	Global positioning system
GSI	Geological strength index
GSE	Geological Survey of Ethiopia
CR	Corrosivity coefficient
ICOLD	International Commission on Large Dams
ISRM	International Society for Rock Mechanics
Jv	Joint volumetric count
Mb	Material Constant
MBHN	Megech Borehole New
MDC-BH	Megech dam conduit borehole
Mm	Millimeter
Mpa	Mega Pascal
Mp	Megech pit
MTP	Megech Test Pits
Q-b	Base bearing capacity
Q-max	Maximum end bearing capacity
Q-s	Skin friction
R	Hammer rebound number
RMR	Rock mass rating
RQD	Rock Quality Designation
SBC	Safe bearing capacity
TP	Test pit
UCS	Unconfined Compressive Strength
USAC	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation
UTM	Universal Transverse Mercator
WWDSE	Water Works Design and Supervision Enterprise
$\sigma 1$	Major Principal Stress
$\sigma 3$	Minor principal stress
σ ci	Unconfined Compressive Strength of Intact Rock
σ cm	Unconfined Compressive Strength of Rock Mass
φ	Angle of friction

CHAPTER -ONE

INTRODUCTION

1.1 Background

Dams are civil engineering structures build across a river valley to impound large volume of water for different purpose (Sissakian et al., 2019) and serve a community in various ways including; power generation, irrigation, flood control, ground water recharge and water diversion (Abrehet Mekonen,2017; Umoren et al., 2016; Tesfa Kefale, 2016; Agerie Genetu, 2007; Johansson, 1997).The safety of a dam can be estimated when the features of its foundation are measured accurately and the design of the dam is proportional to the features of the foundation estimation results (Rajabi, 2014).These feature of the dam foundation can be obtained through engineering geological and geotechnical studies of dam sites carried out in detail including surface discontinuity surveying, borehole data and in situ as well as laboratory testing (Nezhad et al ,2012).

In dam projects the main topics that attract research attention in recent years are geotechnical investigation, permeability, seepage control, grouting and treatment of foundations and in line with this many researches were conducted so far and many are going on (Barzegari, 2017) Ajalloeian and Azimian (2013) studied the geotechnical and geological conditions of Nargesi dam site in Iran giving emphasis on dam foundation evaluation from water pressure test and come up with grout type and composition to design grout curtain. Geological and engineering geological properties of Chesshmeh-Asheq dam site was evaluated by Nezhad (2012) basically of dam foundation groutability and end up with addressing the necessity of providing curtain grouting below the dam foundation. In the same way Rajabi (2014) carried out geological and geotechnical characterization of Tanguyeh dam site in Iran and determined the variation in joint concentration of right and left abutments due to different degree of weathering.

Gebremedhin Berhane et al. (2017) evaluated the leakage problem in two micro dam reservoirs in northern Ethiopia through geological and geophysical methods and concluded that foundations of the two micro dam reservoirs are pervious due to bedding, joint and weathered rock mass and faults. Megech dam foundation was investigated from both geotechnical and geophysical perspectives (GSE, 2007; WWDSE, 2007).

The embankment-type dam has several advantages, including high adaptability to different topographic and geologic conditions, wide availability of materials, and simple and fast construction (Zhang and Zhang, 2004). Designing foundations requires, calculating the loads to be transmitted by the foundation structure to the soils or rocks supporting it, determining the engineering performance of these soils and rocks and designing foundation structure (Bowless, 1997). Dams come in different sizes, shapes and types in which in all cases sufficient studies has to be carried out such that to decide the location, type and size of dams is possible (Sissakian et al.,2019).

Engineering geological investigation/assessment give a brief insight about geological conditions in concerned sites of interest like depth of foundation, type, cut-off depth , quantity and quality of available material, type of expected geological hazards as well as type of treatment to be taken (Sisskian,2019). Of the dam failures experienced so far all over the world the most adverse factors outlined are seepage, differential settlement and foundation defects (Barzegari, 2017). Therefore, investigations in the foundation of dam projects must be conducted with care and caution for the dam to serve its design life properly (Agerie Genetu, 2007). The major risk in the characterization of any dam foundation is the uncertainty involved in predicting ground conditions and behaviors. The accuracy of these predictions will be improved with increasing efforts devoted to the subsurface investigation (Fang and John, 2006; Solomon Astbaha, 2012). The complexity of design and construction of dams is due to the nature of the varying foundation conditions and hence geological study of the dam site is being looked up on as a matter of considerable importance, since a number of projects happened to fail due to the adverse geologic conditions (Agerie Genetu, 2007).

When designing and building dam geological factors play a great role and are the prime and most important ones among natural factors affecting the design and construction of dam. In line with this there are number of projects where the foundation conditions were not well understood leading to high cost of construction and treatment (Ajalloeian and Azimian, 2013).The general characteristics of a dam site are revealed in the geotechnical, hydrological and environmental studies of the site. However, the design and construction of a dam to ensure its structural adequacy (foundation safety and slope stability) considers mostly the geotechnical and hydrological studies (Umoren et al., 2016).

According to Woodward (2005) and Sissakian (2019) the critical sites requiring engineering geological investigation/assessment are dam foundation, spillway, diversion tunnels, outlet works, reservoirs to be occupied, slopes in the abutment, reservoir, and hydrogeology of the dam site as well as source of construction materials. This can be done through rock mass classification from surface exposures and drill core samples applying common rock mass classification systems, drilling operation with associated in-situ and laboratory tests (Geotechnical), satellite image analysis, engineering geological mapping, seismicity studies and concerned geophysical methods (Barzegari, 2017).

In the light of this rock mass classification to characterize dam foundation involves discontinuity surveying to address the engineering performance of rock mass, rock strength measurement and other geological conditions to obtain rock mass classification parameters (Ajalloeian and Azimian,2012). Therefore, an adequate assessment of site geologic and geotechnical conditions is significant aspect of dam safety evaluation (Sissakian, 2019). Megech dam project for which this research has been conducted is currently under construction from the left abutment (embankment) and partly under investigation for outlet works in the right abutment of the dam till the beginning of 2020. Further investigations are in progress for the suitable construction material for the dam. Piles under the outlet works foundation were given emphasis to transit the embankment work from left abutment to the right abutment along with the slope cut design on the right abutment for this work. The investigation work has progressed from site identification through pre-feasibility-feasibility and now in final design investigation phases in general sense (WWDSE, 2007; 2010; Mitiku Eshetu, 2015).

1.2 Problem statement

Even though dams are valuable structures, they become more expensive to repair and a minor problem may lead to reconstruction or complete failure of dam projects if problems not solved on time (USACE, 2006). Likewise embankment dam failures get attention when preceded by geodynamic events inducing increased rate of deformation, cracking, leakage and pore pressure build-up (Abrehet, Mekonen, 2017). Dam failure initiated through conditions associated with dam foundation defects has been a commonly recorded feature (DSC, 2010, as cited in Gebremedhin Berhane, 2010). Ethiopia is building many earth and rock fill dams (Gebremedhin Brehane et al., 2017; Abrehet Mekonen, 2017). Despite the state of the art in embankment dam construction in Ethiopia has been developed rapidly, failures are still occurring in various ways (Gebremedhin Berhane et al., 2017) and several dam functional defects are identified as outlined by various MSc and PhD thesis.

Foundation settlement at Gidabo and seepage through the dam body, foundation and abutment with slope failure for Gomit embankment dams (WWDSE, 2008) were among those defects. The performance of a given dam is the combined effort of the performance of various sections adopted during design and thus, it is necessary to include main dam, outlet work, intake tower, and diversion tunnel structures foundation in the design investigation. This is because failure of one section may the pioneer for failure of others leading to an integrated failure (Boles, 1997). Since every foundation investigation represents at least partly a venture into the unknown (Boles, 1997) design and construction of an efficient foundation depends on proper characterization of the supporting soil and rock (Daniel Gebremichael, 2017).

Megech dam project is facing adverse geologic conditions during excavation to place the foundation contributed from high degree of weathering and jointing in the rock exposures along the proposed foundation departing from design investigation. In line with this a transit from shallow to deep (pile) foundation for outlet and intake tower structures (WWDSE, 2010) was made without performing any further investigations. Besides, these piles will be submerged in water and no consideration was given to the nature of the groundwater for possible sulfate attack and corrosion. Similarly paleosoil intercalated between basaltic successions at megech dam foundation was not investigated for typical engineering properties beyond personal judgment based on visible thickness at some localities before excavation.

Looking in to the design fluctuation and adverse geologic conditions faced during excavation with missed investigations with respect to paleosoil and hydrochemistry of ground water for submerged piles under conduit and intake structures the present research work was initiated and was aimed to characterize the rock mass including hydro chemical study on submerged piles and insight in to paleosoil. Besides bearing capacity and settlement potential for piles resting on rock foundation has been addressed in the present study.

1.3 Previous studies

1.3.1 Studies by USBR (1963); WAPCOS (1990); BCEOM (1999)

Prior to feasibility and detailed design study by Water Works Design and Supervision Enterprise of Ethiopia (WWDSE) with TAHAL Group, Israel (2006-2010), Megech dam site was assessed by various organizations at different times. It was first identified by USBR in 1963, as a part of Megech Irrigation Project and proposed an irrigation area of 6,940 ha under the Blue Nile River Basin Ethio-US cooperation programme. The dam type assumed was an earth-rock fill Dam with crest elevation of 1952m, height of 78m, crest length of 940m; spillway length of 60m and capacity of 1,587m^3/s.

Megech dam site identification by USBR (1963) was followed by a study which recommended different project size with reservoir level at 1916.5m; reservoir capacity of 164.6Mm^3 and dam height of 57.4m (Mitiku Eshetu, 2015; WWDSE, 2007). Further, in 1999, Megech Dam site was studied by BCEOM in association with ISL and BRGM which is a reconnaissance level study for irrigating a total area of 7311ha, commanded by a weir located downstream of the main dam. Likewise, an earth-fill dam comprising of central core; free-flow spillway; a bottom outlet and intake structure was proposed. The dam peak was outlined at an elevation of 1945m, with 2m free board and a crest width of 8m and an upstream slope of 3.5H/1V and a downstream slope of 3H/1V. The spillway was designed to discharge a peak flow of 626m^3 and located on the right bank for reasons of topography (Mitiku Eshetu, 2015; WWDSE, 2007).

Later, Megech dam project comes in to implementation and construction after consecutive reconnaissance, preliminary and detailed investigations conducted by Water Works Design and Supervision Enterprise (WWDSE, 2010) in association with Tahal consulting group. These studies was the route for the initiation of this research and includes investigations through borehole drilling along the main dam axis, intake tower structure, conduit outlet work and spillway locations with subordinate geophysical investigations. The relevant studies conducted so far to appraise Megech dam project site are outlined in Table 1.1.

1.4 Research questions

- ❖ What type of seepage conditions prevail in the dam foundation?
- ❖ What is the geological sequence along the dam foundation?
- ❖ What are the index and engineering geological characteristics of materials in the dam foundation and abutments?
- ❖ What are the foundation conditions around and below the piles under the intake and conduit structures?
- ❖ What would be the suitable remedial measures for foundation condition improvement?

Table: 1.1 previous studies conducted for appraisal of Megech dam project (WWDSE, 2007; Mitiku Eshetu, 2015)

Year	Previous studies
1998	Abay river basin integrated development master plane project phase-2,BCEOM in coordination with ISL,BRGM and MWR Addis Ababa

1999	Abay river basin integrated development master plane project phase-3,vol.IV prefeasibility studies-part-5-irrigation and drainage for Ribb, BCEOM in coordination with ISL,BRGM and MWR Addis Ababa
1998	Abay river basin integrated development master plane project phase-2-Data collection-Site Investigation survey and analysis. section–II-Sectorial Studies: volume-III-water resources development, part-3 dam and reservoir site investigation by BCEOM in coordination with ISL,BRGM and MWR Addis Ababa
1998	Abay river basin integrated development master plane project phase-2-Data collection-Site Investigation survey and analysis. Section –II-Sectorial Studies: volume-VI-water resources development, part-3 dam and reservoir site investigation by BCEOM in coordination with ISL, BRGM and MWR in Addis.
1999	Abay river basin integrated development master plane project phase-3,vol IV prefeasibility studies-part-I-irrigation and drainage for Gilgel Abay, BCEOM in coordination with ISL,BRGM and MWR Addis Ababa
1997	Abay river basin integrated development master plane project phase-2-Data collection-Site Investigation, survey and analysis. section –II-Sectorial Studies: volume-IIIb-water resources development, part-3 dam and reservoir site investigations (for Megech) by BCEOM in coordination with ISL,BRGM and MWR Addis Ababa
2010	Megech Dam project final Detail geological investigation and geotechnical investigations report, Volume-VI-by WWDSE in association with Tahal
2010	Megech Dam project Final Detail design investigation Report Volume-V- By water works design and supervision enterprise in association with Tahal

1.5 Objective

1.5.1 General objectives

Assessment of foundation conditions of Megech dam project.

1.5.2 Specific objectives

The typical specific objectives of the proposed research included:

- To characterize rock mass in the foundation through rock mass classification(RMR) and to assess properties of soil
- To generate geological cross-section along the main dam and outlet structures
- To determine seepage potential conditions in the dam foundation
- To estimate bearing capacity and settlement potential of piles under intake and conduit outlet structures
- hydro chemical analysis to determine nature of ground water and its interaction with piles for sulfate attack and corrosion
- To forward methods of treatment for foundation based on the assessment result

1.6 Significance of the research

Engineering geological investigation for engineering projects involves assessment of the ground condition at and below the surface in the respective areas that are assumed to rest the structure. It plays a very important role in proper site selection, designing, construction planning and ensuring long life of a project as well as economic design of a dam project

(Sissakian et al, 2019). In line with this the present study has the following importance. Engineering geological investigation for foundation assessment specifically in dam projects is vital for the researcher to develop scientific knowledge and awareness related to this research area of interest and be involved in solving such type of problems in future. In addition, this study adds knowledge and data about the dam site that future researchers can use as an input on same or related title of interest.

It provides information and possible remedies on some anticipated worst conditions expected during or after completion of the dam project. It gives over all insight on the role of geological and engineering geological investigation for dam foundation assessment and also used to elucidate the consequences when the investigation is inadequate and misinterpreted. It also provides additional knowledge about project that can be utilized as input for any defects encounter during or after the completion of the dam (Megech) project.

1.7 General Methodology

The outcome of this research has been through organized and systematic methodology that was followed. Mainly the following activities, as a part of methodology were undertaken;

- ✓ To fulfill the aim of the present study data/information was collected and reviewed from the relevant secondary sources such as; published articles, books, technical reports, published and unpublished MSc and PHD Thesis pertaining to dam foundation assessment. Basically, it is intended to address the nature of pile foundation relating to bearing capacity, settlement and durability including foundation rock mass characterization through rock mass classification using data obtained from field investigation to have conceptual frame work about this study.
- ✓ Collecting existing geotechnical and engineering geological data from concerned authorities, project site offices as well as individuals were part of this work.
- ✓ In addition rock and soil descriptions, digging of test pits for in situ observation and sampling for laboratory test, exposure description, and discontinuity survey were addressed in the entire field work termed collectively primary data collection (Gebremedhin Berhane, 2010).
- ✓ After completing the above basic activities (data gathering) the final step was integrating and compilation of these data from the above sources for retrieving relevant information through discussion, correlation, interpretation to meet out the objectives of the present study.

1.8 Scope and Limitations

Generally this study is focused on foundation aspects of Megech dam project and concerns about rock mass and soil characterization for dam foundation suitability. Besides, groundwater conditions and its possible effect on piles are also studied. Thus, the present study addresses the suitability of foundation materials and its likely adverse effect on the safe performance of the proposed dam structure. Mainly this study was constrained from limited number of samples for laboratory testing and analysis. Lack of sufficient surface outcrops at dam foundation to collect RMR data and getting published literatures related to the title were also parts of constraints for this research.

1.9 Thesis structure

The present research is organized into following chapters;

Chapter one: Introduction encompassing background of the research, problem statement, objectives, methodology, research questions, significance of the research and scope and limitations encountered during the present research.

Chapter two: Deals with overview of the study area, location and accessibility, features of Megech dam project, Physiography of dam site, climatic conditions, hydrology and geological setup of the study area.

Chapter three: Literature review and methodology

Chapter four: Data collection, preparation and analysis

Chapter five: Results and discussion

Chapter six: Conclusions and recommendations

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CHAPTER -TWO

GENERAL OVERVIEW OF THE STUDY AREA

2.1 Location and accessibility

Megech dam is one of the dam projects in Tana sub-basin, located in Amhara National Regional State, in Northwestern Ethiopia. The other projects are Gumara, Rib and Gilgel abay. The proposed dam site is situated southeast of Azezo and Tewodros Air Port about 11km south of Gondar town (Fig. 2.1). The dam site is accessed through Addis Ababa - Bahir dar–Gondar main asphalt road that connects central Ethiopia with north western and northern Ethiopia, some 740 km from Addis Ababa (Ibrahim Temam, 2018; Nigussie Ayehu and Mesenbet Sebhat, 2016; Mitiku Eshetu, 2015).The dam is constructed across Megech River which originates from Semain Mountains and terminates into Lake Tana with other tributaries. The dam axis is bounded in between the geographic grid references of UTM zone 37, 329493.02 - 335433.14E and1380769.59 -1385923.33N. The location of the river bed at the center of the dam axis is in between two volcanic hilltops on the right and left abutments with coordinate 332646E m and 1382648 m N and elevation of 1877 m (WWDSE, 2007).

2.2 Megech dam project - Salient features

Megech dam is designed across Megech River southwest of Gonder town and south of semen shield with dual purposes of irrigation and water supply for Gondar. Megech Dam is an earth-rock fill type Dam with impermeable central clay core and 940m long with structural and hydraulic heights of 78 and 76m from the river bed, respectively. It has 10m crest width, 2m free board, a side channel uncontrolled spillway and chute, required for the safe passage of a 2- day ½ probable maximum flood discharge of 662 m^3/s with dam crest elevation at 1952m. Megech dam rests on rock foundation that is highly weathered and jointed casing adopting different types of foundation along the intake tower and conduit outlet works from the main dam foundation. The capacity of the reservoir is $182m^3/s$. The dam is designed with intake, conduit outlet and spillway structures sited in the right abutment and upstream and downstream coffer dams (WWDSE, 2007). Appurtenant structures of Megech dam rest on rock foundation and all are situated on the right abutment of the dam due to less weathered rock mass as compared to the left abutment. The conduit outlet and intake tower structures rests on pile foundation with 15 m submergence in ground water (WWDSE, 2010a). The main features of the dam are depicted in Table 2.1.

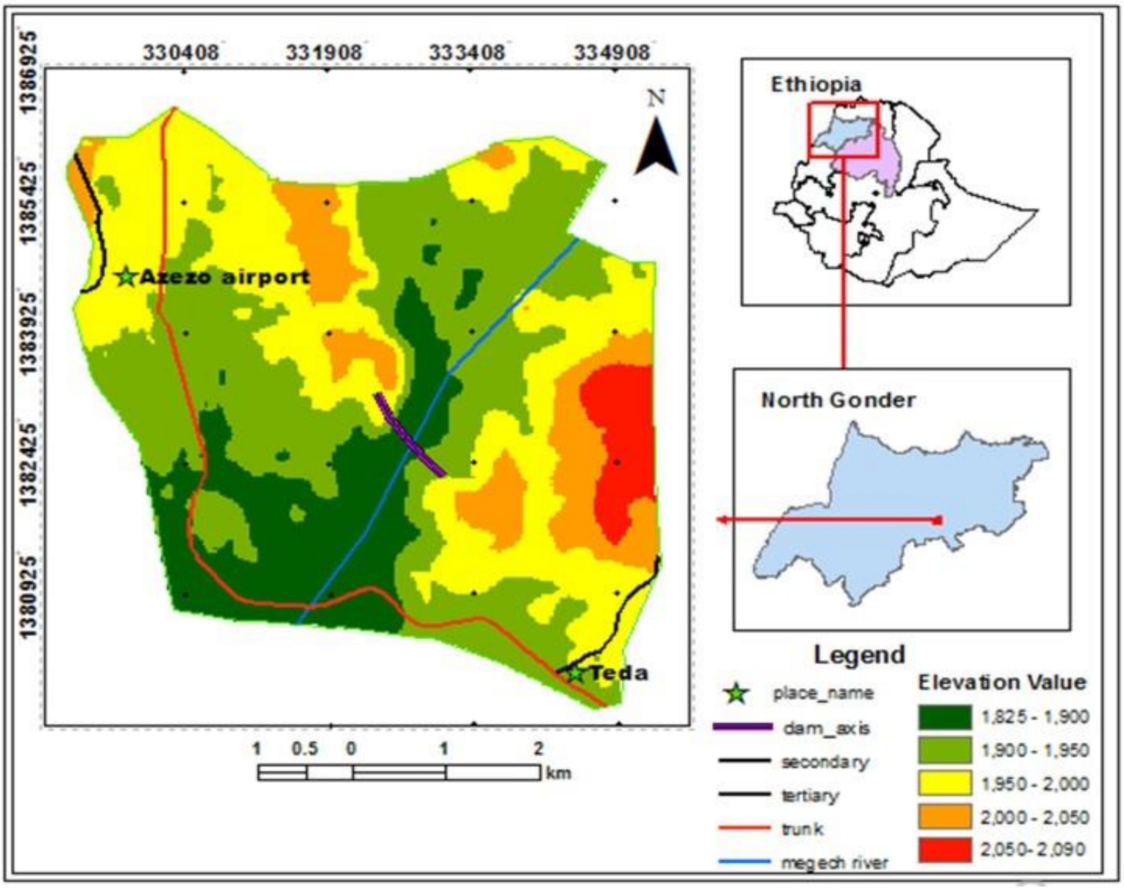


Fig. 2.1 Location map of the study area

Table 2.1 Salient features of Megech dam project (WWDSE, 2007 and Mitiku Eshetu, 2015)

Dam features	**Description of features**
Dam type	Earth – rock fill embankment type
Dam structural height (m)	78
Dam hydraulic height (m)	76
Dam crest length (m)	940
Dam crest elevation (m)	1952
Crest width (m)	10
Spillway type	Uncontrolled side channel spillway
Inflow design flood	1587 m^3
Intake structure	Rest on submerged pile foundation on the right abutment
Conduit outlet	Rest on pile foundation on the right abutment
Coffer dams	(2) 1 upstream + 1 downstream

2.3 Physiography of the dam site

As far as the physiography of the dam site is concerned it is characterized by broad and flat flood plains, old bench forming terrace and low to high relief basaltic hills with steep to moderately steep slopes. The catchment in general is mountainous rugged south facing topography with oval shape dendritic drainage pattern. Elevation difference with in the study area ranges from 1877m, a s l at the river bed to 2991 at hill tops (Fig.2.2). Four major tributaries join the Megech River: two from the right bank and two from the left bank which can be characterized as relatively dense drainage pattern. The dam site is characterized by densely populated short thorny bushes and few scattered remnants of broad leaved tress on old cultivated lands.The downstream areas of dam site and the reservoir area is extensively used for farming, settlements being denser in the upper reservoir slopes and top of the hills.

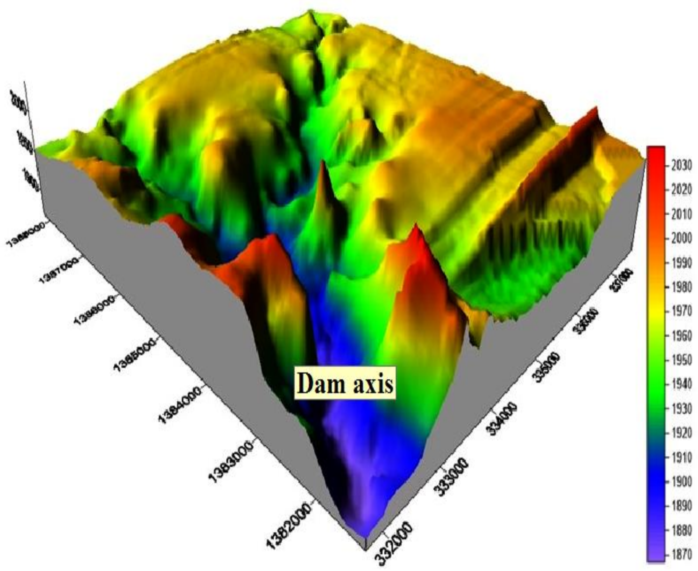


Fig 2.2 Physiographic map of the study area (Reservoir and dam foundation area)

2.4 Climate

According to Nigussie Ayehu and Mesenbet Sebhat (2016) the climatic conditions of Megech catchment, where the dam site is located, can be categorized in to two broad seasons; the dry season (winter) which covers the period from October to May and wet season (summer) that extends from June to September, with slight rainfalls during autumn and spring.

2.4.1 Rain fall

Rainfall over the Megech watershed is mono-modal with nearly 79% of the annual rainfall occurring in the period June – September. Dependable rainfall (85%) varies from less than 1.2 mm during the dry season to 88–225 mm/month during the period of June to July/August, to 55–75% of the average values (Ibrahim Temam, 2018). Like other parts of Ethiopia the rain fall of the catchment in which the dam is found is erratic and falls in moist weyina Dega zone as per Agro-climate zone classification. The dam site is located within the highlands of Ethiopia.The long duration records of rainfall from Gonder airport meteorological station for the years 1981 to 2018 shows total maximum and minimum rainfall to be 711.8 and 1382.2

mm, respectively while at Teda meteorological station for the years 2009 to 2018 the total maximum and minimum rainfall are 923.2 and 1538.3 mm, respectively (Table 2.2).

2.4.2 Temperature

Temperature variation throughout the year in the watershed where the dam is sited is minor.The annually mean maximum and minimum temperatures are 30 and 12°C, respectively. Maximum temperatures vary from 23 ºC in July to 30 ºC in March and April, whereas minimum temperatures range from 11.5 ºC in January to 15.6 ºC in April and May .There is a diurnal difference in temperature however the temperature is comparatively uniform throughout the year with a mean annual maximum and minimum temperatures of 27 and 14^{o}C, respectively, at representative station Gonder (1981-2018) (Table 2.2).

Table 2.2 Rainfall and Temperature data of Megech dam site represented by Gonder airport station (1981-2018) and Teda station rainfall (2009-2018) fully recorded years only (Source: Ethiopian Metrological Agency)

Particulars	Months											
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jun**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
R-max-Gonder	14.6	32.6	60.8	77.1	210.6	244.2	381.5	372	192.5	200.3	105.3	32.9
R-min-Gonder	0	0	0	0	1.4	59.4	214.2	195.2	33.1	1.4	0	0
R-mean-Gonder	2.7	5.1	17	32	88	146.6	294	274	117	56	22.7	7.7
R-max-Teda	14.9	28.3	63.7	175.8	211.7	268.4	467.1	430.6	194.4	71.5	91.3	21.8
R-min-Teda	0	0	0.1	0	13.2	119.4	197.2	263.5	67.4	7.1	2.5	0
R-mean-Teda	3.68	5.42	16.26	38.21	116.3	179.3	321.73	347.71	98.05	32.5	27.6	4.23
T-max-Gonder	27.8	28.8	29.47	29.37	28.4	25.7	23.21	23.38	25.36	26.57	27.32	27.6
T-min-Gonder	11.4	13	14.85	15.63	15.26	14.16	13.56	13.41	13.09	12.82	12.42	11.9
R - Rainfall and **T** - Temperature												

2.4.3 Evaporation and Humidity

The maximum relative humidity in Megech watershed corresponds to rainy seasons and the minimum to dry seasons (Nigussie Ayehu and Mesenbet Sebhat, 2016) (Table 2.3). The mean annual relative humidity perceived from data (1994-2004) at Bahir Dar and Gonder meteorological stations shows 58% and 52.7% respectively (Abe you Wale, 2008). The wind speed reaches its maximum in the dry season and minimum in rainy season (Fig.2.4, Table 2.3). Relative humidity is high in vegetated areas as compared to bare lands and increase towards the lake shore from massive hilltops from the north where the soil is thick as well. The main tributaries of Megech River include: Angereb, Keha, Shinta, Dimaza, Gilgel Megech and Wizaba have cut deep trenches that divide the watershed in to sub-catchments (Nigussie Ayehu and Mesenbet Sebhat, 2016).

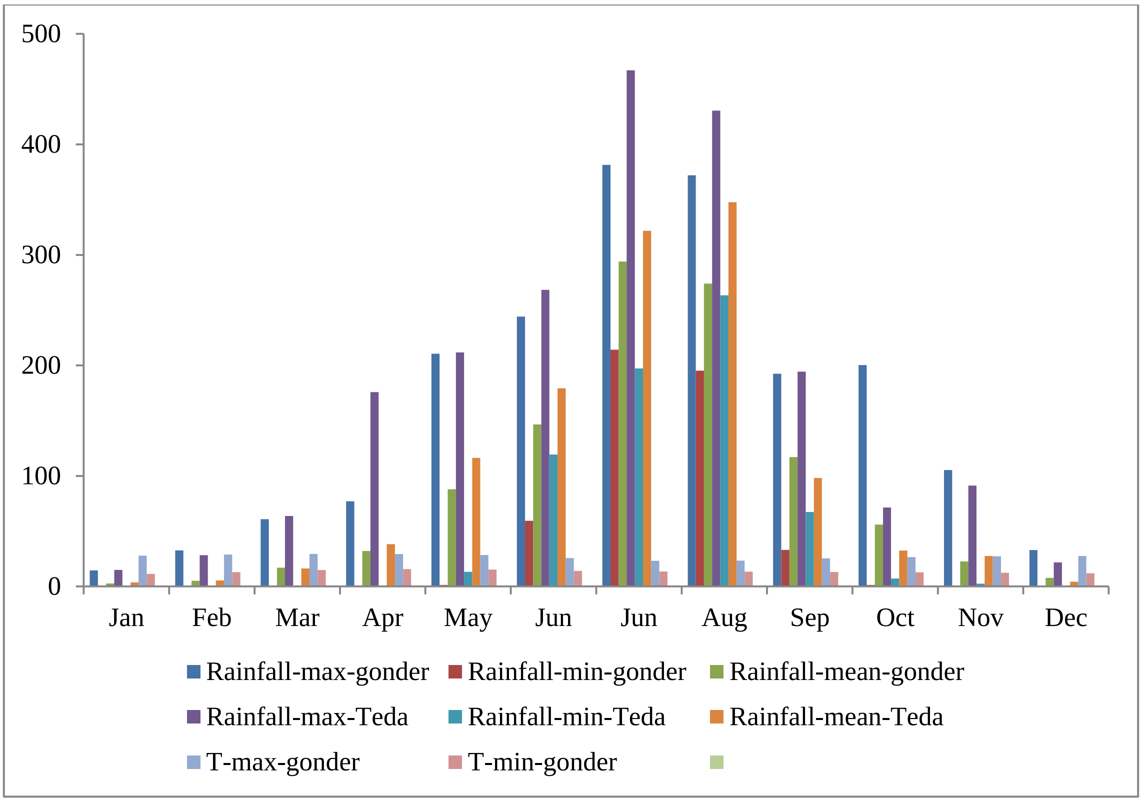


Fig.2.3 Temperature and rainfall characteristics of Megech dam site represented by Gonder airport and Teda meteorological station based on data as shown in Table 2.2.

Table 2.3 Evaporation and humidity data over Megech reservoir (Source: WWDSE, 2007)

Particulars	**Months**											
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
Max. Temp (^{o}C)	28	28.9	29.9	29.4	28.5	25.2	22.8	25.9	25.3	26.3	27.1	27.2
Min. Temp (^{o}C)	11.5	13	14.7	15.6	15.6	14	13.2	13.3	12.8	12..6	12.5	11.9
Humidity (%)	43.3	39.2	38.7	40.2	51.4	66	78.6	79.3	72.1	59.3	50.2	45.9
Wind (m/s)	1.6	1.8	1.9	1.8	1.9	1.9	1.4	1.3	1.4	1.4	1.5	1.5
Evaporation (mm)	152	147	179	169	176	146	121	126	147	154	145	142

2.5 Land use and land cover

Land use land cover of Megech River catchment in which the dam site and its reservoir is located includes: forest land, grassland, cultivated, water body and built up areas (Abeyou Wale, 2008; Nigussie Ayehu and Mesenbet Sebhat, 2016; Ibrahim Temam, 2018). Deforestation cause changes in soil properties and infiltration rates which in turn affects soil erosion process and hydrology of the catchment which is intense in mountainous areas being high energy environments where sediment transfer from hill slopes to the channels, greatly facilitated (Abeyou Wale, 2008). Based on the land use land cover classification carried out for whole Megech catchment by Ibrahim Temam (2018) and field observation in the present study all of the land use land cover types as mentioned above are present at the dam site and in the reservoir area.

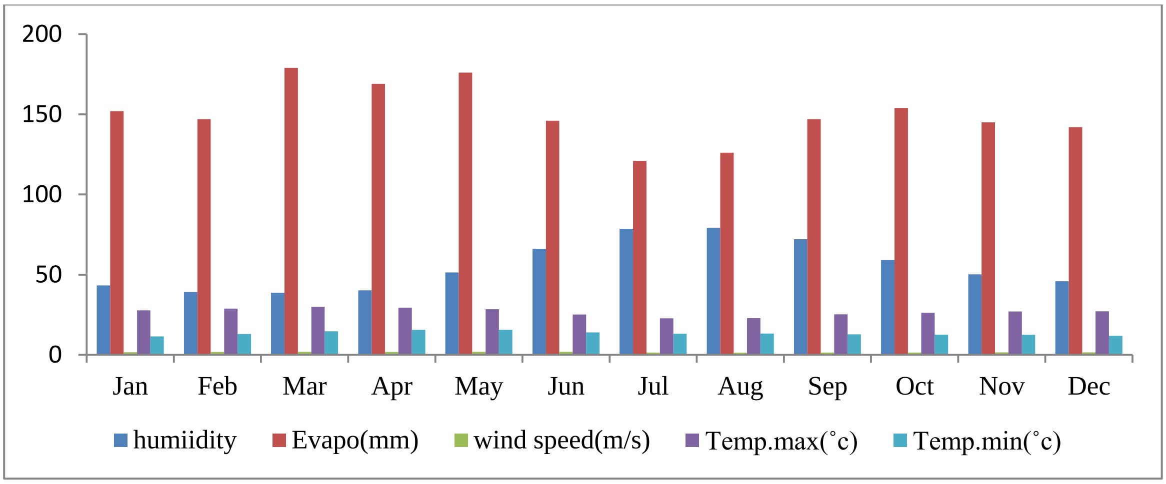


Fig: 2.4 Evaporation and humidity characteristics of the study area at a given temperature and wind Speed conditions (Source: WWDSE, 2007)

The dominant land use land cover type at the dam site and its vicinity is cultivated land which is true for the entire catchment also. Studies on land use land cover dynamics for Megech catchment (Ibrahim Temam, 2018) depicts that grass lands and forest covers are rapidly changing in to cultivated and built up type land cover types; thus cultivated and built up areas are increasing accordingly. According to Ibrahim Temam (2018) the Megech watershed land use land cover has been classified in to the land use land cover types as shown in Table 2.4.

2.6 Soil cover and Vegetation

According to Ibrahim Temam (2018) at Megech watershed including dam site and its vicinity the most common type of soils are shallow lepto-soils underlain by unconsolidated medium sized gravels with loose joints which in turn is underline by water tight rocky layers. The dominant soil textures as identified in this catchment are silt clay loam and silt clay which partly confirms with investigation on soils at Megech dam site located in the reservoir and overburden soils at the dam axis by WWDSE, (2007-2008). At dam foundation a recent alluvial soil has been deposited along the river's flood plain and thick soil is developed over the rocks in the reservoir basin.

Table 2.4 Land use land cover types of Megech watershed including dam site and reservoir area Ibrahim Temam (2018)

Land use land cover	Categories under land use land cover
Forest land	Trees with high density, ever green forest land, and mixed forest land and plantation forests that mainly are eucalyptus, junipers and conifers.
Cultivated land	Areas serving both annual and perennial crop production and large size cultivated fields.
Gras land	Areas covered with small pieces of trees like shrubs mixed with grasses used for grazing
Water body	Typically includes open bodies, ponds, rivers with their tributaries

Built-up/settlement areas	Concerns settlement areas, portions of land covered with infrastructures ,like transportation

In the central portion of the dam axis, river deposits that are mainly comprised of overburden materials; alluvial and colluvials overlaid on volcanic rock. It includes very loose - medium to coarse gravels and very loose unconsolidated - fine to medium grained sand. The right abutment of the dam also consists of similar type of soil formations as that of the left abutment with less than half a meter in thickness. The dam site area is characterized by scars and scattered remnants of trees which are mainly eucalyptus and thorny bushes with few remnants of broad leaved trees that are confined along old cultivated terrace lands, stream and river banks. A few patches of trees are found scattered over the upper part of the watershed, which could not be considered as forest. The dam site area is generally devoid of vegetation cover such as trees.

2.7 Hydrology

Hydrology is a science dealing with the properties, distribution and circulation of the Earth's water along with their chemical, physical properties and interaction with the environment (Houghtalen et al., 1996). Understanding of hydrology of a given site is important for proper planning, design, construction and operation of hydraulic structures. Ground water reduces the strength of the rock mass, may change the mineral constituents through chemical alteration and solution, change the bulk density of the rock, may generate pore water pressure (Raghuvanshi, 2019) and may cause erosion (Negash Anteneh ,2007). Megech Dam site study also outlines a hydrological assessment indicating Megech River being the only river on the north side of Lake Tana with sufficiently large watershed to supply surface water for the project (WWDSE, 2010a). In terms of geology the prevailing volcanic rock units consisting crevices formed along cracks and weathering has enhanced development of open water bearing layers that are open with interconnected porous spaces. Thus pour spaces can be potential means for infiltration and water bearing layers at the dam site typically in the foundation and the dam abutments (WWDSE, 2007).

2.8 Geological setup

2.8.1 Regional geology

The plateau within Ethiopia is not a simple un-deformed structural block rather it contains Tana basin a faulted depression located between erosional escarpments looking Sudan plains to the west and to the east as well as tectonic escarpment of the plateau margin looking the Afar depression in the east of northwestern Ethiopia (Chorowicz et al., 1998). The Cenozoic

Ethiopian continental flood basalt province sited at the locus of three rifts mainly crops out in Ethiopia while the rest in Yemen. The geological frame work of Lake Tana basin comprises a basement of Precambrian bed rock, overlain by Mesozoic Sediments, Tertiary Volcanic and Quaternary Volcanic with cover of recent alluvial sediments. The distribution of rock formations with in the basin and associated configuration of the basin has been controlled by ongoing tectonic activities. Although mostly covered by extensive tertiary flood basalt Mesozoic and Tertiary sediments are exposed in few locations surrounding Lake Tana area covered by 1-2km thick Eocene and Oligocene flood basalt (Hautot et al., 2006).

Perceived from magneto-telluric study (Hautot et al., 2006) conducted to establish the deep structure of Tana basin the estimated thickness of sediments overlying the Precambrian basement averages1.5-2 km. Four episodes of volcanism in the north western Ethiopia portion of Afro-Arabian large igneous province (Lake Tana) was defined through new geological and Geochronological data which are believed to happen at different time intervals with different composition. These episodes include Flood basalt, thick and extensive ignimbrite and rhyolites, mafic volcanism and scoraceouse basalt (Prave et al., 2016; Pik et al., 1997; Nigussie Ayehu and Mesenbet Sibhat, 2016).Following the bulk volcanism of the plateau through fissural activity a co-magmatic shield volcanism was there producing shield volcanoes on the surface of volcanic plateau (Sileshi Mamo, 2015). In general, the northwestern Ethiopian plateau is characterized by volcanic successions emplaced on sub horizontal Mesozoic sedimentary strata with distinct volcanic centers (Kieffer et al., 2004). The volcanic units include Oligocene flood volcanic Trap series, Miocene-Pliocene shield volcanoes, volcanic plugs and domes and Quaternary volcanic (Negash Anteneh, 2007).

2.8.2 Geologic structures and tectonic setting

Tana sub-basin is manifested through complex lithological and tectonic structures and located west of Afar margin and tectonically active main Ethiopian rift. Data from digital elevation modeling and satellite imagery analysis confirmed that the location of the Tana basin was at the intersection of three grabens. Thus being: Dengel- ber (buried), Gonder exposed through erosion and Debre-tabor reactivated (Chorowicz et al., 1998). It is thought that Lake Tana was formed by combination of a lava barrier blocking the Nile to the south and epirogenetic subsidence (Poppe et al., 2013). Further, the subsidence was resulted from convergence of three grabens and the basin comes in to this form through damming quaternary basalt flow to fill Nile exit to a depth of 100m (Chorowicz et al., 1998). The first subsidence was in Mid-Tertiary while the subsequent subsidence was due to reactivation of graben faults in Late

Miocene and Quaternary. This fault reactivation was accompanied locally by basaltic volcanism resulting series of volcanic rocks (Chorowicz et al., 1998; Mitiku Eshetu, 2015; Sileshi Mamo, 2015).

Tana and its vicinity are characterized by many dike and pipe feeders responsible to mid tertiary flood basalt flows erupted from fissures. In the latter stage of volcanism the magmatic ingress focused and built-up on shield volcanoes overlaying flood basalts (Chorowicz et al., 1998), depicted by Morphological setting of the basin (Poppe et al., 2013). Fig 2.5 presents Digital elevation model of the Tana basin and adjacent areas with Abay river canyon. As the lithospheric mantle has impregnated the lithosphere which has resulted from plume activity in the Tana region and induce thermal uplift responsible for formation of three grabens and results in ward-dipping fault blocks (Chorowicz et al., 1998). Megech dam site is located more than 200km from the main Ethiopian rift. Major faults in the vicinity of the dam site are marked over the regional geological maps. This indicates that there are no evidences of active faulting or crustal movements in the vicinity (WWDSE, 2010).

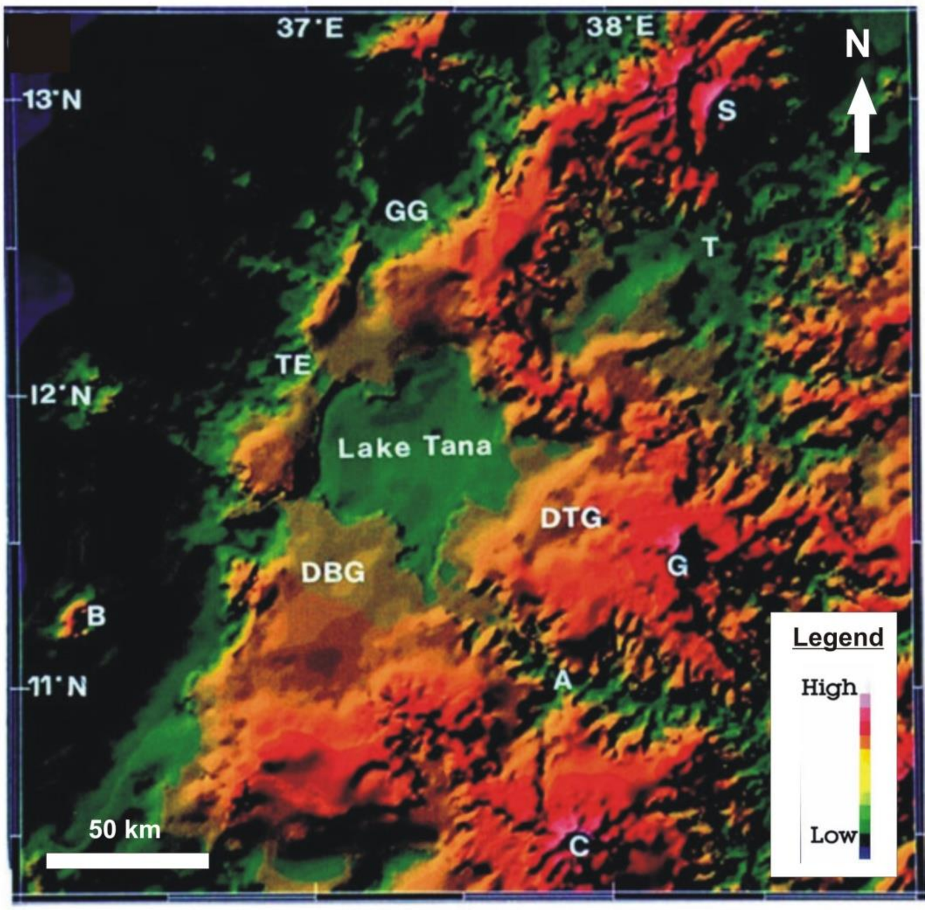


Fig2.5 Digital elevation model of the Tana basin and adjacent areas with A, Abay river canyon; DBG-Dengel -Ber graben; DTG - Debre Tabor graben; GG - Gondar graben; S - Semen Mountain; TE - West Tana escarpment (Chorowicz et al., 1998).

2.8.3 Seismicity of the study area

Megech dam project is located within a radius of 100 km from lake Tana and is not considered to fall in a very active zone, however it remain under the influence of rift valley seismicity (Attalay Ayele, 2006 therein WWDSE,2010). Figure (2.7) depicted the seismic hazard zonation map of Ethiopia such that Megech dam site and its reservoir area fall in zone zero zonation map of Ethiopia (0). The respective earthquake magnitudes as per Atalay Ayele, 2006 are 4.5 and 6 Richter and mercelai scales respectively. A study in the same geological province of megech (Gumara Dam Project) revealed that the hazard in the zone has a probability of expedience of 0.0033 with 300 years return period. Thus, the area is relatively in a less dangerous seismic location of the country. Perceived from (WWDSE, 2010) the peck horizontal ground acceleration value 0.05 -0.06 derived from its intensity values (6) and 6.5 with return period of 300 years. It was based on peak ground acceleration (0.06) and respective factor of safety megech dam has been designed with estimated epicenter location within 238km from megech dam site. The ground peak vertical acceleration coefficient is assumed to be half of the horizontal peak acceleration and given $0.06/2 = 0.03$ g (WWDSE, 2010).

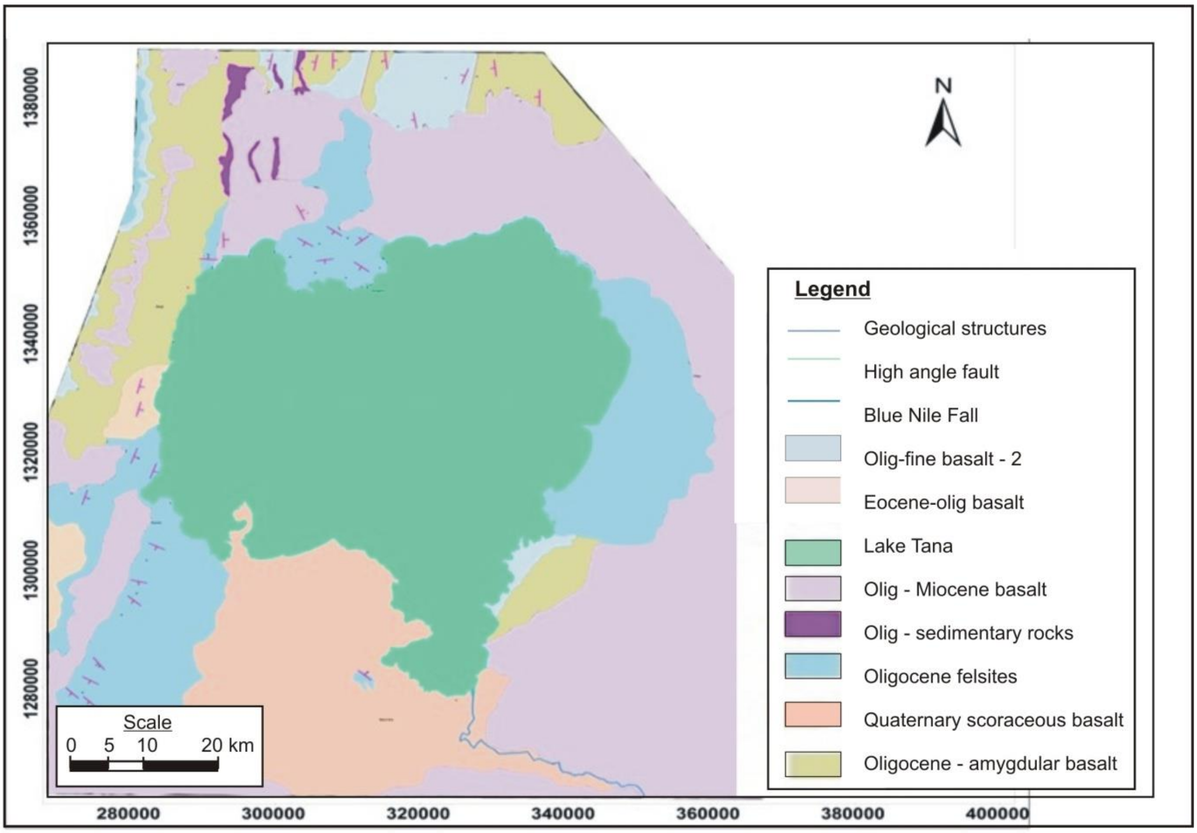


Fig2.6 Tana sub-basin: Geological map of Tana region including Megech dam site and its vicinity (Source: after Prave et al., 2016)

2.9 Local geology

The Lake Tana sub- basin including Megech catchment where the dam site is located consists of the following typical geologic units.

- Trap series (Oligocene-Miocene) volcanic with Ashangi, Aiba, Alaje and Tarmaber formations and common in the highlands and escarpments.
- Quaternary basalts, found in lowlands of the basin and can be old or recent quaternary
- Quaternary sediments that are found in the flood plains of major rivers like Megech and shores of the lake (Fenta Nigate et al., 2020; Mengesha Tierra et al., 1996; Abbate Ernesto et al., 2015; Rooney, 2017).

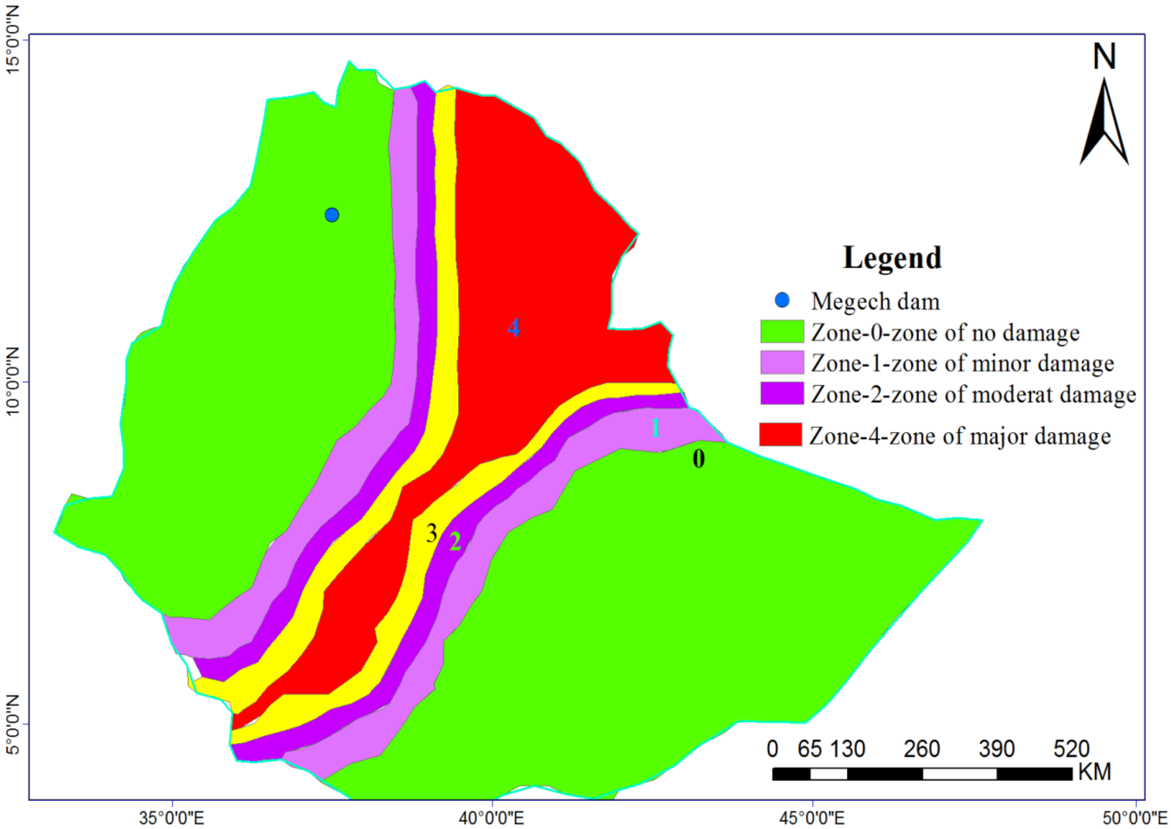


Fig2.7 Seismic hazard zonation map of Ethiopia (EBCS 8, 1995) with circle in blue indicating dam site

2.9.1 Trap series volcanic

As outlined above the main components of Trap series volcanic are Ashangi, Aiba, Alajae basalt/rhyolite and Tarmaber formations (Fenta Nigate et al., 2020). Ashangi formation represents the oldest (being the lowest) fissural flood basalt volcanism lying on the Mesozoic sediments (Mengesha Tefera et al., 1996; Sileshi Mamo, 2015; Rooney, 2017). It is characterized by deeply weathered/ altered with reduced horizontal extension and located below the pre-Oligocene unconformity (Abbate et al., 2015; Mengesha Tefera et al., 1996). Compositionally it varies from transitional to tholetic olivine basalts with alternating subordinate tuffs (Abbate et al., 2015) and interbeded pyroclastic with rare rhyolites injected by dolerite sills and dykes (Mengesha Tefera, 1996). Age description of Ashangi formation remains uncertain while the general consensus deciphers in between Eocene-Oligocene (Mengesha Tefera et al., 1996).

Aiba basaltic formation took the second phase of fissural basaltic volcanism in the basin as well (Mengesha Tefera et al., 1996). It overlies the tilted Ashangi basaltic unit erupting in to Ashangi pen plain (Rooney, 2017) and has well-developed columnar massive transitional flood basalt flows locally accompanied by intervening agglomerate units (Mengesha Tefera et al., 1996; Abbate et al., 2015; Rooney et al., 2017; Sileshi, 2015). Aiba formation sequence contains minor amount of felsic products with general basaltic flows that are extensive and layered (Sileshi, 2015). Compositionally it has distinctive tholeitic nature with transitions to mildly alkaline varieties and age falling in the range of Oligocene (Mengesha et al., 1996).

Alajae formation consist typical basalts, with rhyolites, ignimbrites and subordinate trachyte. It is the bulk volcanic succession in the Tana sub-basin (MengeshaTefera et al., 1996). This basalt is the final stage of widespread flood basalt sequence intercalated with rhyolite horizons (Rooney, 2017). Being located directly over the Aiba basalt with no major unconformity Alajae formation constitutes thick serious of felsic rocks and age ranges from late Oligocene - Miocene (Abbateet al., 2015). According to Mohr and Zanettin (1988 as cited in Rooney, 2017) Tarmaber formation is more localized basalt overlying Alajae units in place derived from large low-angle shields and are of fissure fed. It represents Oligocene – Miocene basaltic shield volcanism. (Mengesha Tefera et al., 1996). According to Abbate et al. (2015) Tarmaber basalts are composed of zeolitized, alkali basalt with large amount of tuffs, scoriaceous lava flows, pera-kaline rhyolites and typical red paleosoils. The age of shield volcanoes within the Tarmaber formation are early-middle Miocene (Abbate et al., 2015). As compared to underlying flood basalts Tarmaber basalt flows are thin, less continuous and heterogeneous with more porphyritic characteristics containing large phenocrysts of plagioclase, pyroxene and olivine (Sileshi, 2015).

2.9.2 Quaternary volcanic

Quaternary volcanic formations in Tana basin filling Tarmaber series (Negash Anteneh, 2007) are found relatively at higher stratigraphic locations (Ayenachew Alemayehu, 2018). Volcanic cones and corresponding scoraciouse basalt flows are well distributed around the Lake Tana grabens and are thought that Pleistocene in age (Negash Anteneh, 2007; Hussen Ayalew, 2010). The quaternary volcanic rocks found in Lake Tana are inferred as alkaline basalts (Merle et al., 1979 as cited in Husse Ayalew, 2010) with possible thickness of up to 1300m Mohr (1971, as cited in Negash Anteneh, 2007) and common in the south of the Lake. These basalts are composed of vesicular alkali basalts as well as cinder cones deciphering the original magma is volatile rich (Negash Anteneh, 2007). According to Chorowicz et al. (1998) quaternary basalts were associated with local rift structures typically north-south trending extensional faults and are partly covered by Quaternary Lacustrine sediments in and around the vicinity of the Lake Tana area. Perceived from field observation the outcropping quaternary volcanic in the basin was intensely weathered than tap series (Poppe et al., 2013).

2.9.3 Quaternary sediments

Quaternary sediments exist from lacustrine, fluvial and hill slope process. Thus, sediments are more of unconsolidated form and are situated north and east of the lake, where the lake is bordering flat to gently sloping ground (Poppe et al., 2013).Alluvial materials are present along the riverbeds and are derived from fluvial processes and even of expected below Quaternary volcanic deposits as inter-bedded paleo-soils (Hussen Ayalew, 2010; Poppe et al., 2013). Quaternary sediments are common at lower ages of rivers, mainly in the north and east of Lake Tana in the Megech, Ribb and Gumara flood plains. They range in lithology from clay to gravel of unknown thickness and are identified from high resolution seismic studies, supported by extensive analysis of cores drilled up to 9.5m and geophysics from a minimum of 9m across the entire lake bed to at least 40m at the northern end of the lake (Hussen Ayalew, 2010).

2.10 Geology of the dam site

2.10.1 Preamble

Megech dam site is located somewhere in between Semen massif and Lake Tana in the north-western Ethiopia and rests on volcanic rocks formed in different episodes of eruption time which is clearly depicted by existence of paleo soil in between such volcanic rocks. When seen from engineering point of view all of the formations at the dam site are affected by high degree of weathering and jointing from completely to moderately weathered in some locality.

2.10.2 Upper Aphanitic basalt

The upper aphanitic basalt is basaltic rock located immediately above the top paleo soil and has whitish colure (Plate 2.1(a)). With regard to weathering it varies both vertically and laterally from completely weathered to moderately weathered rock where the intensity increases upwards from upper paleo soil contact to the thick soil surface cover. Weathering is high immediately above the paleo soil and at the top surface. Upper aphanitic basalt is exposed only in the right abutment of the dam, exposed due to stripping done for the slope correction. The geological features associated with this formation are joints mainly dipping towards the river, favorably oriented from seepage point of view though they will affect significantly the engineering performance of the rocks. The joint in the upper aphanitic basalt are found open without secondary infill material. These open discontinuities may facilitate permeability or leakage along the dam axis.

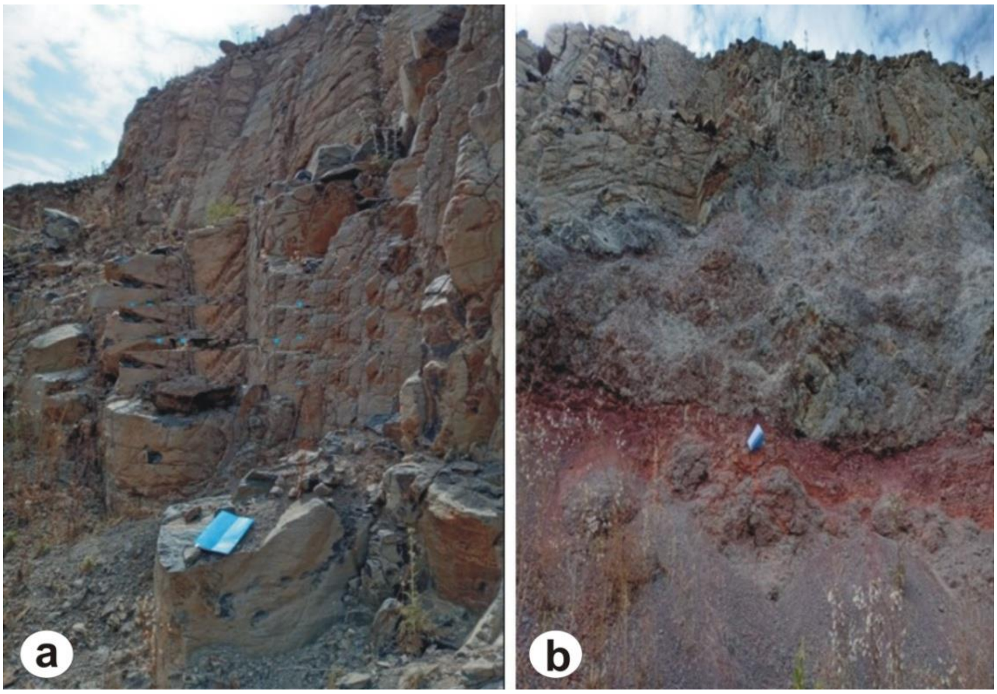


Plate 2.1 (a) Upper aphanitic basalt with joints and (b)Top paleo soil underlying upper aphanitic basalt Paleo soil

Paleo soil at Megech dam site is located between lower aphanitic basalt and porphyritic basalt (Plate 2.1(b)). The soil is exposed through stripping during construction preparation. This paleo soil has reddish colure. Though it is largely exposed locally at the dam site due to working excavation it has regional extent as confirmed from exposures in road cuts and some other excavations far away from the dam site. Being overtopped by porphyritic basalt the paleo soil repeats after the termination of the porphyritic basalt which shows other episodes of volcanism (third) possibly of Tarmaber formation. It varies in thickness though thick exposure exists in the right abutment.. Existence of this soil poses design change from preliminary investigation design (WWDSE, 2007).

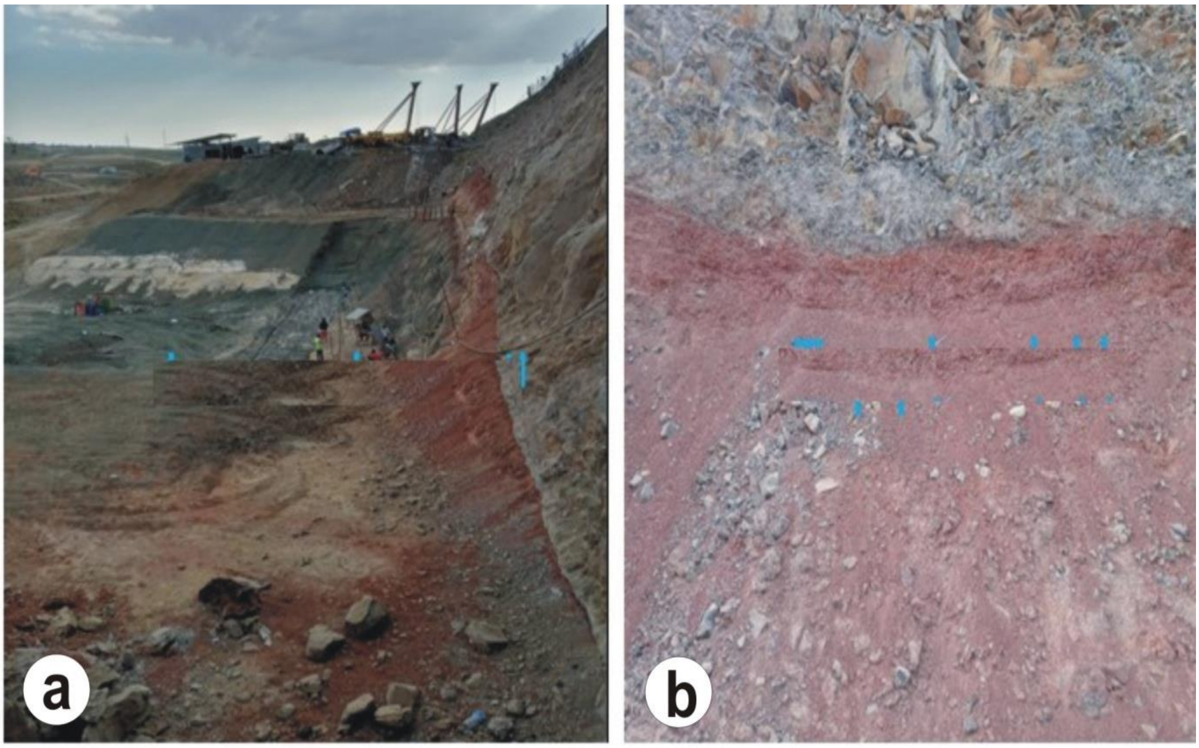


Plate 2.2 (a) Lower Paleo soil inter-bedded between lower aphanitic basalt and porphyritic basalt and (b) Upper Paleo soil inter-bedded between porphyritic basalt and upper aphanitic basalt

2.10.3 Porphyritic basalt

Porphyritic basalt in Megech Dam foundation site is located between the bottom and top paleo soils both in the right and left abutments. It consists of feldspar phenocrysts within mafic mineral matrix and medium to fine grained texture (WWDSE, 2007). Though it is defined by dominant lithology porphyritic basalt some random aphanitic and vesicular basalt exist within this section. In terms of weathering it varies both laterally and vertically from completely decomposed rock to slightly weathered rock in some limited localities in the dam site specifically downstream of the right abutment. The color variation reflected in porphyritic basalt is whitish immediately around paleo soil contact and reddish in vesicular basalt exist while dark grey is the common colure in the entire basalt. Geological features within porphyritic basalt are joints dipping towards the valley though some local anomaly. The joints in the porphyritic basalt are mostly unfilled but in sections the joints are filled with quartz perceived from visual observation in the field.

Plate 2.3 (a) Porphyritic basalt with thin paleo soil -different degree of weathering (b) Porphyritic basalt portraying almost similar degree of weathering

2.10.4 Lower aphanitic basalt

Lower aphanitic basalt is located immediately below the lower paleo-soil and buried by un-stripped alluvium near the river bed. It holds joints dipping towards the river. The geological features within the lower aphanitic basalt (joints) are relatively less consistent but less spacing leading to high degree of weathering for this formation. Texturally it is fine grained with intensity of weathering high at the contact to the lower paleo-soil and varies laterally and vertically as well. Lower aphanitic basalt is exposed due to stripping and the exposure ceases when the stripping terminates.

2.11 Structures at dam site

The common structures observed at the dam site are mainly joints and fractures with orientations of N-S, NNW-SSE, E-W and N-S.The joints found at Megech dam axis and abutments vary in persistency, close to widely spaced, mainly of three sets that are roughly perpendicular to each other and sub vertical dipping. Fractures were observed in all of the lithologic units and most of the joints were unfiled rarely in some locations in both of the abutments by calcite, silica and zeolite.

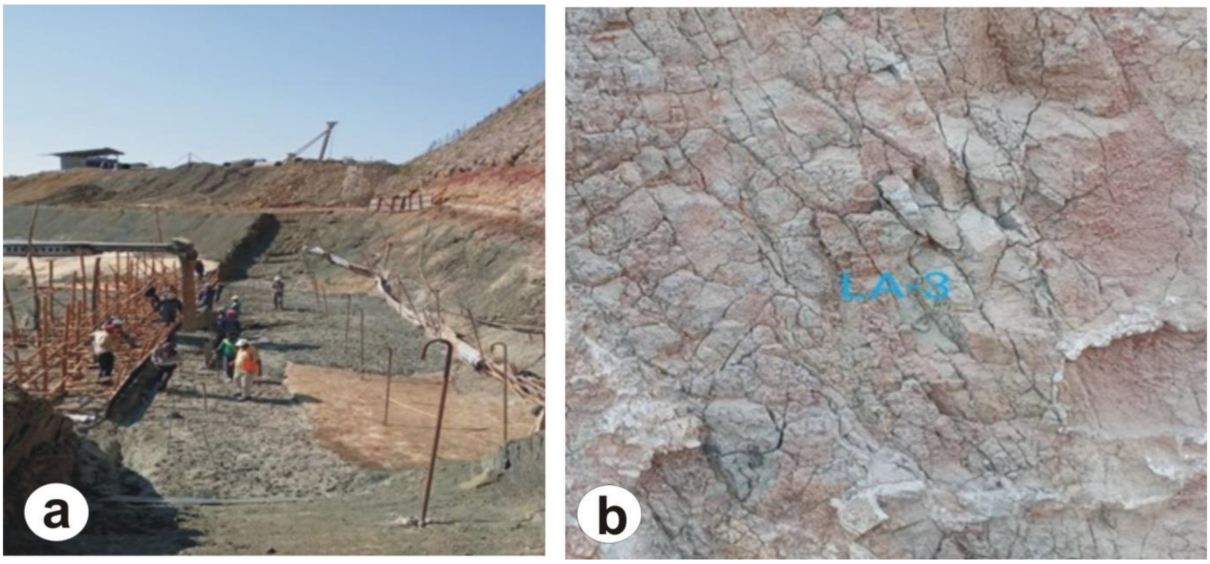
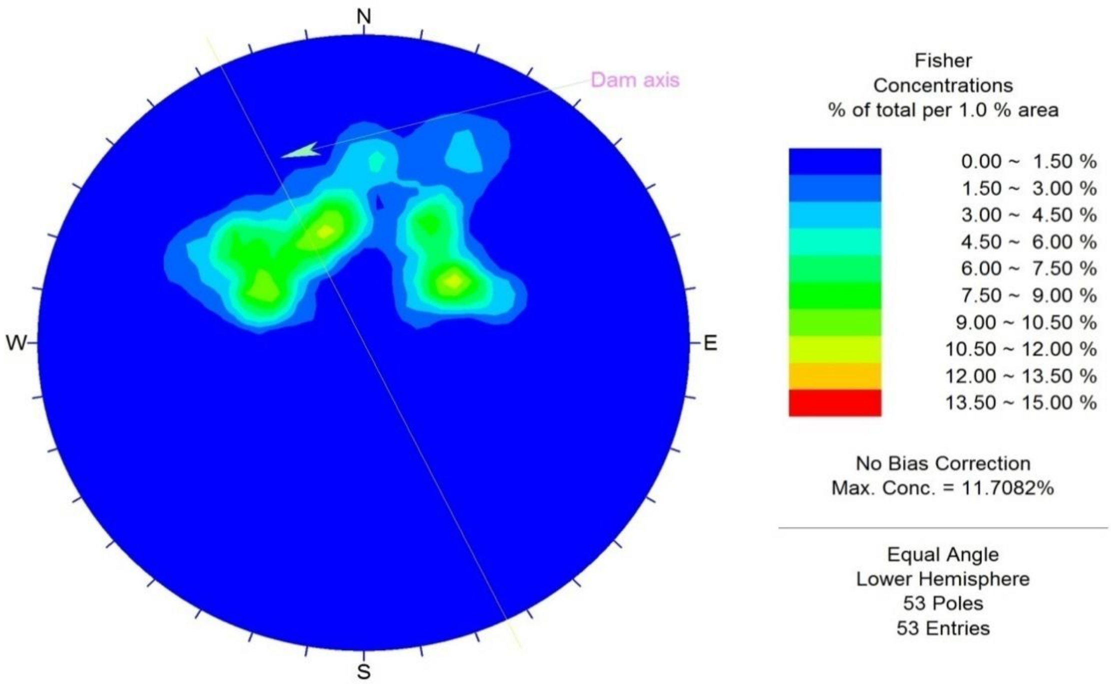
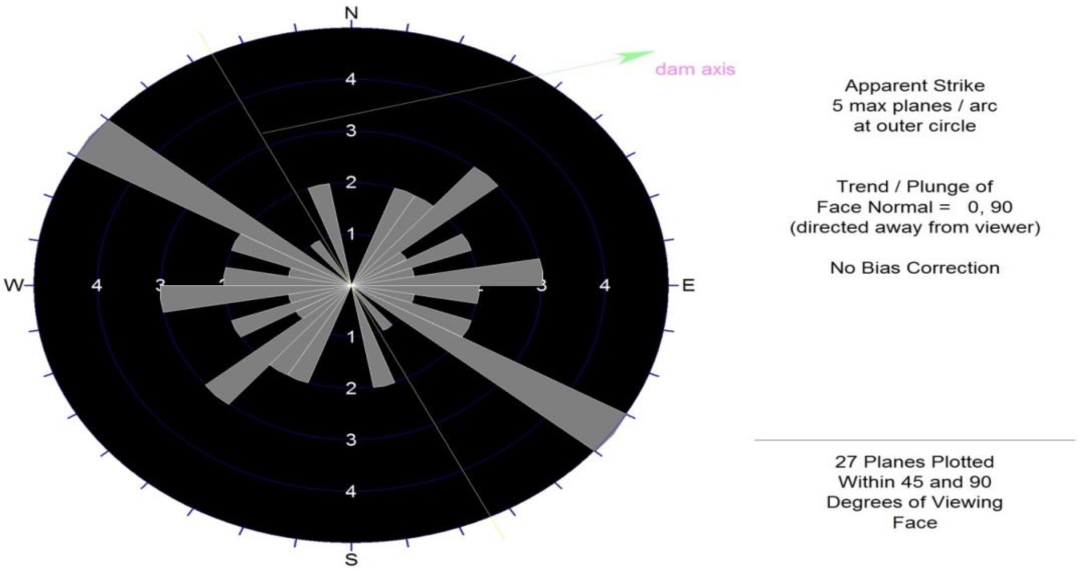


Plate 2.4 (a)Lower aphanitic basalt exposed at core foundation (right abutment) (b)the same lithology exposed in the left abutment with different degree of weathering andjointing

Joints vary from slightly weathered to highly weathered, smooth-rough surfaces and mostly dipping towards the river. Joint concentration was higher in the left abutment than in the right abutment, as observed during the field data collection. Joint spacing in the three sections of the dam (left and right abutments and the river bed section) varies in range 2 to 50cm. Fig. 4.4 shows the stereo plot and rose diagrams of the dam foundation from structural data collected in the field.



(a)



(b)

Fig. 2.8 Stereoplot (a) and Rose (b) diagram of dominat joint sets at Megech dam foundation

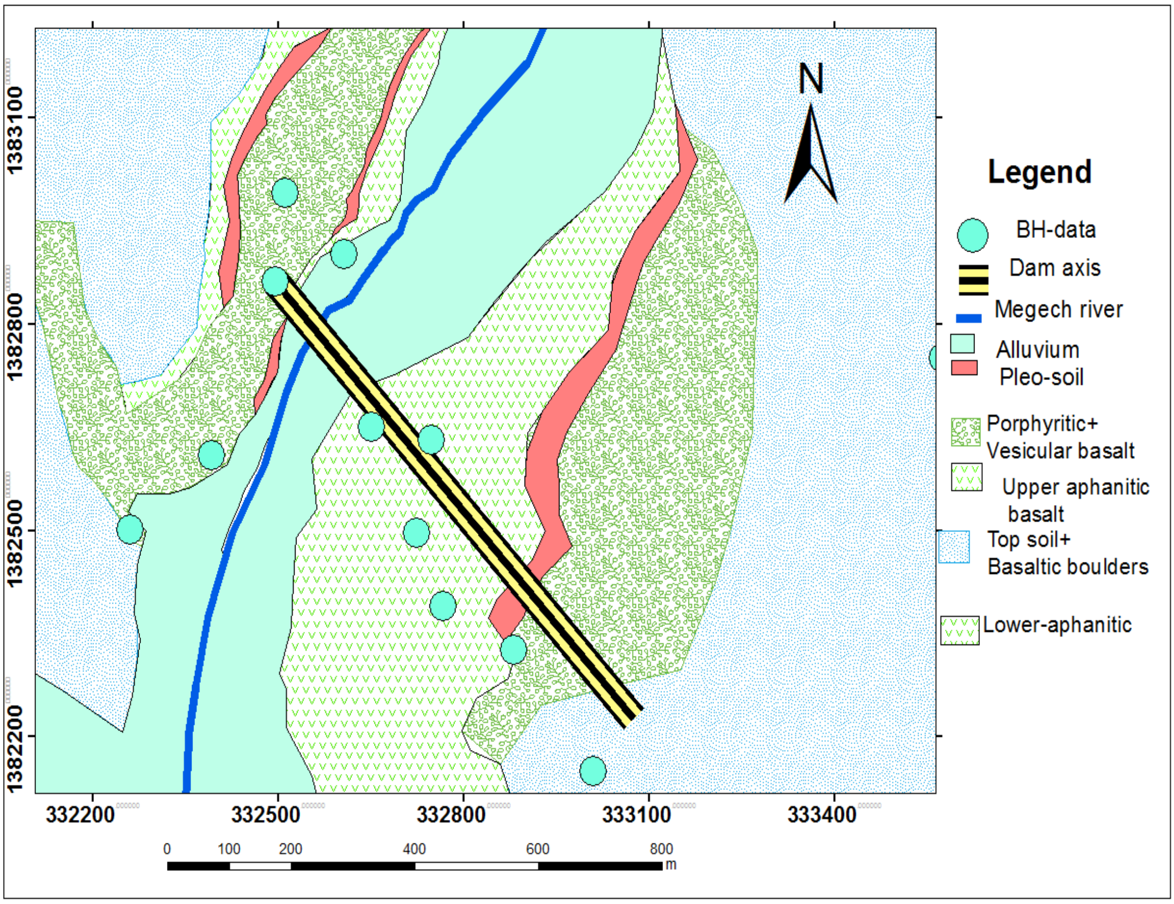


Fig: 2.9 Geological map of the dam site

CHAPTER -THREE

LITERATURE REVIEW and Methodology

3.1 Dam foundation characterization

Dam foundation is the valley floor and terraces on which the embankment and appurtenant structures of a dam rest. Both rocks and soils can be involved as a dam foundation (Umoren, et al., 2016). Engineers, geologists and engineering geologists have to work with the challenge in assessing engineering behavior of these soils and rocks involved in a project site (Solomon Mebrahetu, 2015). One of such challenges is assessing foundation problems which may constitute to about 15% of the total challenges (ICOLD, 1995). Embankment dams have been built since early times and the general philosophy to design these dams is to utilize wide range of topographic and foundation conditions as well as locally available geological materials (ICOLD, 1995). No dam can be considered fully safe; as there will never be a complete understanding of the uncertainties associated with natural and manmade destructive forces, foundation material behavior and construction processes (Abrehet Mekonen, 2017).

Dam's failure is usually a complex process that typically begins with some behavioral abnormality (foundation defects) that is not observed. That is why dam foundation assessment and monitoring, as well as rapid analysis and interpretation of data, can play a critical role in the field of dam engineering (Johansson, 1997). To address the time dependent and variable properties of geo-materials (soil, rock and water) and keep the hazard induced in to the construction industry an in-depth Geotechnical and engineering geological investigation and mapping is necessary (Gebremedhin Brehane,2010). The investigation and associated characterization of soils and rocks has to be focused on understanding the interrelationships between the geological environment and the engineering situation; the nature and relationships between the geological components, the active geodynamic processes and the prognosis of processes likely to result from the changes being made (UNESCO, 1976, as cited in Gebremedhin Brehane, 2010). These investigations should cover classification, physical properties, location and extent of soil and rock strata and variation in piezometric levels in ground water at different depths (Gebremedhin Brehane, 2010; Nehemia Solomon, 2009).Through geotechnical drilling operations, rock mass characterization (RMR, Q) methods and some indirect geophysical methods the geotechnical properties of materials at a given site can be assessed which in turn is responsible to engineering judgment (Umeron,2016).

3.2 Rock Mass Characterization

Rock mass characterization is the process of collecting and analyzing qualitative and quantitative data that brings insight to indices and descriptive terms of the geometrical and mechanical properties of a rock mass to provide a quick means to estimate the strength and deformability behavior of the rock mass (Ocepec, 2006; Caiet al., 2004; Daniel Gebremichael, 2017). It is important in rock engineering practice like excavation design, support design, foundation design and assessment along with many other applications. Thus it is necessary to obtain design input parameters like deformation moduli and strength parameters for numerical modeling. Such parameters are ultimately determined from in situ tests during preliminary design stage (Khatic and Nandi 2018; Bieniwski, 1989). However, where access to underground is limited the practical way to obtain these parameters is to apply a rock mass classification system to characterize the rock mass and estimate the rock mass properties (Cai et al., 2004).

Over the past years, many classification systems, such as RQD (Deer et al., 1967), Rock Mass Rating (RMR) (Bieniwski, 1973), Q-System (Barton et al., 1964), Rock mass index (RMI) (Pamstrom, 1995) and Geological Strength Index (GSI) (Hoek, 1994) systems, have been proposed (Cai et al., 2004). Further, suitability of the rock mass as a foundation material is evaluated through certain engineering properties such as; strength, deformability and permeability characteristics of the rock mass (Nigatu Fekadu, 2006). Different classification systems place different emphases on the various parameters and the two most commonly applied classification methods are RMR and Q-systems (Khatic and Nandi, 2018). The respective parameters for estimating RMR are uniaxial compressive strength of rock material (UCS), Rock Quality Designation (RQD), Spacing of discontinuities, Condition of discontinuities, Groundwater conditions and orientation of discontinuities (Bienwski, 1989). Similarly, for Q- system Rock quality designation (RQD), number of joint sets, discontinuity roughness, discontinuity alteration and the presence of water are the respective parameters (Biniwaski, 1989).

3.2.1 Rock mass classification

A rock mass classification system is intended to classify the rock mass based on strength, quality, discontinuity condition, weathering, structural and orientation parameters which provide a basis for estimating the deformability and strength properties of rock mass and give measureable data for support estimation (Bieniawski, 1989; Daniel Gebremichael, 2017). It is one of the only approaches estimating large scale rock mass properties and forms the basics of many empirical design and failure criteria used in many numerical modeling programs (Milne et al., 1998). Very complex nature of fractured rock mass is described simply and economically by various rock mass classification systems which quantify some qualitative information concerning the rock strength, fracture pattern, orientation, spacing, roughness, groundwater conditions and personal experience (Sen and Sadagah, 2002).

Due to the variety of rock mass strength properties, engineering design associated with rock mechanics issues is a challenging problem. This is due to the presence of fractures in rock mass which regulates the stability of surface structures and in situ stress conditions which regulate the stability of deep structures in rock mass. In addition, ground water conditions, rock mass squeezing and swelling or stability condition of rock mass and filling materials may increase their influence. Each rock parameter in these rock mass classification systems is assigned separately a value called rating depending on its weight and finally all rating are summed up to give the overall rating and characterize rock mass (Bieniwaski,1989;Khatik and Nandi, 2018; Sen and Sadagah, 2002). It is possible to retrieve many benefits from Engineering Rock Mass classification like quality site investigation using minimum input parameter's, brings quantitative information for design purpose and create effective means of communication and better engineering judgment on a given project (Bieniawski, 1989).

Various rock mass classification systems exist and modified since then to arrive on its contemporary state (Table 3.1) however; the most common ones are Rock mass Rating (RMR) (Bieniawski, 1989) and Q-systems (Barton et al. ,1964). RMR and Q-system play great role to arrive at appropriate Rock Engineering and Design and considered as the basics for developing other rock mass classification systems. Both RMR and Q-systems uses the most important ground features as input and estimate rock mass characteristics. Rock mass index, geological strength index are also rock mass characterization systems for rock engineering purposes (Palmstrom, 2009). Rock mass classifications form the backbone of empirical design and are widely used in rock engineering and become successful worldwide (Bieniawski, 1989).

Rock Mass Rating (RMR)

Rock mass rating also known as the Geo-mechanics classification system, developed first by Bieniawski (1973) is one of the most widely used classification systems (Palmstrom, 2009). The RMR system has been modified over the years based on case histories conforming to international standards, however the procedures and the basic principles remains the same (Bieniawski, 1989; Khatik and Nandi, 2018; Singh and Goel, 2011).

Table3.1 Common Rock mass classification systems (Aksoy, 2008)

Classification system	Form and type*	Originator	Application
Rock load	Descriptive and behavioristic form Functional type	Design of steel support in tunnel	Terzaghi, 1946
Stun up time	Descriptive form General type	Tunnel design	Laufer, 1958
New-Australian tunneling method	Descriptive and behavioristic form Tunneling concept	Excavation and design in incompetent ground	Rabcewicz et al, 1958; 1964
Rock-quality designation (RQD)	Numerical form General type	used in other classification systems	Deer et al., 1967
Size-strength classification	Numerical form Functional type	used mainly in mining	Franklin, 1975
Rock-structure rating	Numerical form Functional type	Design of (steel) support in tunnels	Wickham et al., 1972
Rock mass rating (RMR)	Numerical form Functional type	Design of tunnels, mines, and foundations	Bieniawski, 1973
Q-classification system	Numerical form Functional type	Design of support in Underground excavation	Barton et al., 1974
Basic geotechnical Classification (BGD)	Descriptive form General type	General applications	ISRM, 1981
Geological strength index(GSI)	Numerical form Functional type	Design of support in underground excavation	Hoek, 1994
Rock mass index system (RMi)	Numerical form Functional type	General characterization, design of support	Palmström, 1995

* Form and type - meaning:

- Descriptive form: input to the system is mainly based on descriptions
- Numerical form: input parameters are given numerical ratings according to their character;
- Behavioristic form: input is based on rock mass behavior in a tunnel
- General type: system is worked out to serve as a general characterization;
- Functional type: system is structured for a special application

RMR sustained its development until 1989 (Aksoy, 2008). The evolutions of RMR includes, reduction of classification parameters from 8 to 6; adjustment of ratings and reduction of recommended support requirements; modification of class boundaries in1974; 1975; 1976; 1978 schemes, respectively (Singh and Goel,2011). The working principle behind rock mass rating is to divide the rock mass at a given site in to a number of geological regions /units such that certain features are more or less uniform within each region (Bieniawski, 1989; Singh and Goel, 2011).The geological regions or units are then evaluated by considering the following six parameters (Bieniawski, 1989; Singh and Goel, 2011).

1 Uniaxial compressive strength (UCS) of intact rock material
2 Rock quality designation (RQD)
3 Joint or discontinuity spacing
4 Joint condition
5 Groundwater condition
6 Joint orientation

The classification parameters are then measured at each region of rock mass and assigned to an empirical rating, corresponding to its actual value measured either in the field or in laboratory according to standard tables. Adjustment for structural orientation is treated separately since its effect varies with specific engineering applications such as; Tunnel, Mines, Slope and foundation. On the basis of RMR values for a given engineering structure, the rock mass can be sorted into five classes (Singh and Goel, 2011). In this respect RMR assist in estimation of in-situ deformability of rock foundations in the case of bridge and dam structures (Bieniawski, 1978). Due to the high laboratory cost for rock material testing, limited sample size and non-representative of natural rock mass as well as measurement difficulties in in-situ tests, the value of modulus of deformation and the strength of the rock mass can be estimated indirectly from observations of relevant rock mass parameters that can be-acquired easily and at low cost (Bieniawski, 1989; Aksoy, 2008; Palmstrom and Singh, 2001; Bieniawski 1978). The various correlations can be used to approximate the modulus of deformation as forwarded by different authors at different times and presented in Table 3.2.

Good site characterizations of the rock mass and use of an effective indirect method may in many cases offer better results than expensive in situ measurements (Palmstrom and Singh, 2001). Bieniawski (1989) recommended using more than one classification systems so that the results obtained may be compared and checked for their reliability. Further, the range in which cohesion and angle of friction of a rock mass will fall is specified in RMR and Bieniawski (1976) suggested the following relationships in order to determine the cohesion (eq.3.7) and angle of friction (eq.3.8) for a specific RMR value.

$C^* = 0.05RMR$ …..eq.3.7

$\Phi^* = 0.5RMR + 5$ …..eq.3.8

The evaluation of rock mass parameters in different versions of RMR (1973 to 1989) is presented in Table 3.3. The RMR characterize the rock mass into five classes (Class I to V) and the description of these classes and corresponding range of shear strength parameters of rock mass is presented in Table 3.4.

Table 3.2 Correlations to approximate the modulus of deformation (E_m) (GPa) as forwarded by different authors at different times

$E_m = 2RMR - 100$ for RMR > 50	…eq.3.1	Bieniawski, (1978); Aksoy (2008)
$E_m = 10^{RMR-10/40}$ for RMR <50	…eq.3.2	Serafim and Pereira (1983); Aksoy (2008)
$E_m = 10^{(RMR-30)/50}$	…eq.3.3	Agarwal et al. (1991)
$E_m = 25 \log_{10} Q$	…eq.3.4	Barton et al. (1980)
$E_m = (qc)^{0.5} / 10 * 10^{(GSI-10)/40}$ Where; ‘qc’ is the UCS of intact Rock at natural moisture content, GSI ‘Geologic Strength Index’ (GSI). For RMR_{76}> 18,	…eq.3.5	Hoek and Brown (1997)

GSI = RMR_{76} and RMR_{89}> 23, GSI = $RMR_{89} - 5$		
When $\sigma_{ci} \leq 100\ MPa$ $$E_m(GPa) = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}}.10^{((GSI/10)/40)}$$ When $\sigma_{ci} > 100\ MPa$ $$E_m(GPa) = \left(1 - \frac{D}{2}\right).10^{((GSI/10)/40)}$$ Where; Em is modulus of deformation, D is rock mass disturbance factor, σci is intact uniaxial compressive strength, GSI is geologic strength index.	…eq.3.6	Hoek et al. (2002)

Table3.3 Evolution of rock mass parameters (Bieniwaski, 1973-1989) (Milne et al., 1998)

Ratings	Years of evolution				
	1973	1974	1976	1978	1989
UCS	10	10	15	15	15
RQD	16	20	20	20	20
Discontinuity-spacing	30	30	30	30	30
Continuity of joints	5	5	5	5	5
Groundwater	10	10	10	10	15
Discontinuity condition		15	30	25	30
Strike and dip of joints	15	15	15	15	15

Table 3.4 Rock mass classes and its meaning determined from total rating

Rating	81-100	80-61	60-41	40-21	<20
Class No.	I	II	III	IV	V
Description	Very good	Good	Fair	Poor	Very poor
Cohesion of rock mass (Mpa)	>400	300-400	200-300	100-200	<100
Friction angle of rock mass (deg)	>45	35-45	25-35	15-25	<15

Strength and limitations of RMR

RMR system is simple to use and the associated classification parameters are easily obtained from either borehole or underground mapping data and can serve in wide areas of applications like, in mining, tunneling, foundation stability and slope stability studies. It is addressed to determine the rock mass quality, to estimate the cohesion, internal friction angle, elasticity modulus, strength values of rock mass and to obtain net allowable bearing pressure of a foundation in range of values (Bieniawski, 1989; Aksoy, 2008). Although RMR or geo-mechanics classification works well for many applications it has certain limitations. No input parameters for rock stress despite stresses up to 25Mpa are involved in estimating RMR value missing overstressing due to rock bursting and squeezing. No clear parameter was mentioned corresponding to faults and weak zones in rock mass rating. As a result RMR does not work well for swelling rocks, fault and weak zones (Palmstrom, 2009; Palmstrom et al., 2006). It also basically depends on observations, experience and it may result in extremely safe or unsafe conditions. RMR system also discarded discontinuity orientation parameter for extremely jointed and completely crushed rock masses (Aksoy, 2008).

The Q-rock mass classification system

The Q-system was developed by Barton et al. (1974) and has been undergoing revisions since its development by incorporating case histories in underground excavations and is developed as an empirical design to characterize rock mass and estimate rock support basically for tunnels in rock mass. It is a quantitative and engineering classification system to facilitate the design of tunnel supports and its value evaluates the stability of the rock mass where high values indicate good stability and low values indicate poor stability (Bieniawski, 1989; NGI, 2013; Palmstrom, 2009). Q-value is estimated to assess the rock mass quality numerically based on the following six parameters and can range on a logarithmic scale from 0.001-1000. Based on Q value the rock mass can be characterized into various groups and classes (Table 3.5). Six parameters includes RQD; Degree of jointing (rock quality designation), Jn; Number of joint sets, Jr; Joint roughness number, Ja; Joint alteration number, Jw; Joint water reduction factor and SRF; Stress Reduction Factor (NGI, 2013; Palmstrom, 2009;Bieniawski, 1989). The six parameters are grouped in to three basic factors influencing the rock mass quality and assign a numerical value to describe the rock mass situation and given as eq. 3.9

$$Q = \frac{RQD}{Jn} * \frac{Jr}{Ja} * \frac{Jw}{SRF} \qquad \text{.......eq.3.9}$$

Where; RQD/Jn is the degree of jointing or blocks size, Jr/ Ja is the joint friction or block size and Jw/SRF is the active stress (NGI, 2013).

Table3.5 Rock mass classification based on Q-values (Barton et al., 1974)

Q-value	Group	Classification
10-40	1	Good
40-100		Very good
100-400		Extremely good
400-1000		Exceptionally good
0.1-1	2	Very poor
1-4		Poor
4-10		fair
0.001-0.01	3	Exceptionally poor
0.01-0.1		Extremely poor

As per Barton et al. (1974) description the parameters' Jn, Ja and Jr play more important role than joint orientations despite orientation is implicit in these parameters. Q-system is relatively sensitive to minor variation in rock mass properties and less subjective in description of rock mass joint conditions unlike other systems. Thus Q-method is one of the most frequently used, simple and well documented classifications systems for rock masses (Daniel Gebremichael, 2017; Barton et al, 1974; NGI, 2013).

Limitations of Q System

Case histories involved in the development of Q-system are mostly derived from hard and jointed rock masses and it needs the incorporation of other methods for support design. The Q-value is most precise when estimated from underground openings and more certain than Q-value from field mapping, core logging and borehole investigation due to difficulties encountered in estimating some parameters in these cases (NGI, 2013). In addition Q-system works best from Q = 0.1 to 40 for tunnels of 2.5 - 30m span. Despite input parameters for over stressing Q-system needs care for rock bursting, squeezing and weak zones as well (Palmstrom and Broch, 2006).

Rock mass index (RMI) rock mass classification system

The rock mass classification through rock mass index was first developed by (Palmstrom, 1995) to characterize the strength of rock mass for construction purpose and has been developed since then. The defects present within a rock mass such as discontinuities which degrade the inherent strength of intact rock were the main concern of rock mass index classification. The common inherent input parameters in the rock mass combined to express RMI are Size of the blocks delineated by joints, Vb, Strength of the block material-measured as UCS, qc, Shear strength of the block faces-given by joint characteristics, JR and JA and Size and termination of the joints, jL (Palmstrom,1995).

RMI = $\sigma c * JP$ ……..eq.3.10

Where; σc is the Uniaxial compressive strength of intact rock material in (Mpa); JP is the Jointing parameter which in turn is composed of, block volume, Joint roughness, Joint alteration and Joint size; JP $= 0.2\sqrt{JC} * Vb^{D}$, D = $0.37JC^{-0.2}$ Where; JC=JR * JL/JA (Palmstrom, 2009)

RMI value characterizes the dry rock mass characteristics and does not include the influence from rock stress and ground water. RMI works based on parameters determined through field observation and measurements and different from RMR and Q-systems in that RMI is more prone to calculation than RMR and Q-systems do (Palmstrom, 2009).

3.2.2 Hoke-Brown criteria

The Hoek-Brown failure criterion for rock mass is widely accepted and has been applied in large number of projects. This criterion solves the difficulties in finding friction angle and cohesive strength of a rock mass since 1980 and works best in designing underground excavations in hard rocks (Hoek et al, 2002). Hoek and Brown (1980) developed an empirical approach to determine the strength of the jointed rock mass and formulated a failure criterion for jointed rock mass. Based on results of number of projects this criterion was modified by Hoek& Brown in 1988 and later by Hoek et al. (1992). The Hoek-Brown criterion for jointed rock mass is (eq. 3.11);

$$\sigma_1' = \sigma_3' + \sigma_c \left(mb \frac{\sigma_3}{\sigma_c} + S \right)^a \qquad \text{........eq.3.11}$$

Where, `$m_b$'is the value of the constant 'm' for the rock mass, `s' and `a' are constant which depend upon the characteristic of the rock mass, σ_c is the uniaxial compressive strength of the intact rock pieces and σ_1' & σ_3' are the axial and confining effective principal stresses, respectively.

The original criterion works well for most of the rocks having well to reasonable quality in which the rock mass strength is controlled by tightly interlocking angular rock pieces. The failure of such rock masses can be defined by setting a = 0.5 in eq.3.11.

$$\sigma_1' = \sigma_3' + \sigma_c \left(m_b \frac{\sigma_3'}{\sigma_c} + S \right)^{0.5} \qquad \text{........ eq.3.12}$$

For poor quality rock, in which interlocking is partially destroyed by shearing or weathering, the rock mass has no tensile strength or cohesion. For such rocks, S = 0

$$\sigma_1' = \sigma_3' + \sigma_c \left(m_b \frac{\sigma_3'}{\sigma_c} \right)^a \qquad \text{........ eq.3.13}$$

To determine the material constants m_b, 'S' and 'a' used in equation 3.11, 3.12 and 3.13Hoek and Brown initially used RMR (Rock Mass Rating) of Bieniawski (1976). It could work for rock masses having RMR > 25. However, it failed for rock masses having RMR < 25. In order to overcome this limitation Hoek & Brown developed a new 'Geologic Strength Index' (GSI).

For RMR76> 18, GSI = RMR_{76} eq.3.14

RMR_{89}> 23, GSI = $RMR_{89} - 5$ eq.3.15

For RMR_{76}< 18 the value of GSI can be obtained by Barton, `Q' System.

GSI = 9 $Log_eQ + 44$ …….. eq.3.16

Thus, by using GSI material constants can be estimated; For GSI > 25 (undisturbed rock mass)

$$\frac{m_b}{m_i} = \exp\left(\frac{GSI - 100}{28}\right) \quad \text{…….. eq.3.17}$$

$$S = \exp\left(\frac{GSI - 100}{9}\right) \quad \text{…….. eq.3.18}$$

a = 0.5 …….. eq.3.19

For GSI < 25; S = 0 ……..eq.3.20

$$a = 0.65 - \frac{GSI}{200} \quad \text{…….. eq.3.21}$$

While, in general, it has been found to be satisfactory, there are some uncertainties and inaccuracies that have made the criterion inconvenient to apply and to incorporate into numerical models and limit equilibrium programs. In particular, the difficulty of finding an acceptable equivalent friction angle and cohesive strength for a given rock mass has been a problem since the publication of the criterion in 1980. Further, Hoek et al. (2002) forwarded a Generalized Hoek-Brown Criteria;

The modified expression is;

$$\sigma_1' = \sigma_3' + \sigma_{ci}'\left(m_b\frac{\sigma_3'}{\sigma_{ci}} + s\right)^a \quad \text{…………… eq. 3.22}$$

Where m_b is a reduced value of the material constant m_i and is given by ;

$$m_b = m_i \exp\left(\frac{\mathrm{GSI}-100}{28-14\mathrm{D}}\right) \quad \text{…………… eq. 3.23}$$

S and a are the constant for the rock mass given by the following expression;

$$s = \exp\left(\frac{\mathrm{GSI}-100}{9-3\mathrm{D}}\right) \quad \text{…………… eq. 3.24}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right) \quad \text{…………… eq. 3.25}$$

D is a factor which depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from 0 for undisturbed in situ rock mass to 1 for very disturbed rock mass.

3.3 Pile foundation

Pile foundations are part of an engineering structure and popular form of deep foundation often used to transfer loads from the superstructure to a competent soil/rock stratum through weak stratum or water body located at some depth below the ground surface. Pile foundations are adopted when shallow foundations cannot support structural load without excessive settlement, lateral movement or rotation (Kramer et al., 2014). It can be also used in good soils in case engineering structures are suspected to carry heavy loads or subjected to lateral movements. The maximum settlement and ultimate bearing capacity of a given pile are the main governing factors in the design of axially loaded piles (Bouafia and Derbala, 2016). Piles can be displacement and non-displacement depending on the method of placement in to the ground. Displacement piles involve driving the pile into the ground using a compaction hammer or other devices and include, driven piles, driven and cast-in-place piles, and jacked piles. Bored and cast-in-place and composite piles are non-displacement piles requiring pre-excavation of a hole for placing or casting the pile (Letsios et al., 2014). The design of bored concrete cast-in situ piles in rock is still largely based on the assessment of bearing capacity (Hamberfield and Lochaden, 2019).

Bored concrete cast-in-situ piles can be constructed in very low strength to very high strength rock and from relatively intact rock to intensely fractured rock. All loads applied from the surface to the pile must be transferred through the interface between the pile and the ground (shaft and base resistance). The performance of the pile is defined by the characteristics of these interfaces, the integrity of the concrete pile and the surrounding rock mass. Shaft and base interfaces characteristics are highly dependent on the properties and characteristics of the rock mass, pile diameter and length as well as the construction technique adopted. However, these factors are not normally considered in the design of piles in rock. Piles in rock are still predominantly designed based on estimates of ultimate bearing capacity using empirical generic (non-site specific) rules based largely on unconfined compressive strength (qu) of the rock (Zhang, 2010; 1998; Sultana et al., 2016).

Settlement is often calculated based on modulus values correlated with qu. Commonly pile foundation is used to withstand active vertical loads and usually these loads originate from gravity and separated in to live or dead load. A combination of bearing pressure on the bases of the pile (tip resistance) and shear stress developed on the vertical sides of the pile (skin friction) resist these loads. Bearing capacity of pile foundations can be estimated based on properties of piles and soils embedded, on the basis of driving resistance during installation and on the basis of in situ tests including pile load test and penetration (SPT, CPT) tests (Bouafia and Derbala, 2016; Kramer et al., 2014).

3.3.1 Pile Capacity Estimation from Pile and Soil properties

Pile capacity initiation involves mobilization of shear strength of soil below and along the sides of the piles. Pile capacity estimation based on pile geometry and soil properties is separated in to tip resistance (Q_b) and skin resistance (Q_s) to give the total capacity (Kramer et al., 2014). Thus, piles resist applied load through skin friction and end bearing and where friction piles resist majority of load by friction developed between pile surfaces and surrounding soil, end bearing piles depend mainly on the bearing of soils beneath the pile base (Wrana, 2015) as shown in the Fig. 3.1.

$$Q_{ult} = Qb+Q_s=A_b*q_b+\Sigma A_s*q_s \qquad \text{.........eq. 3.26}$$

Where; A_b is the cross sectional base area of the pile, Q_b is the effective overburden stress at the tip of the pile, A_s is the pile surface area and q_s is the unit skin friction. The parameters used in eq. 3.26 are to be derived from soil properties and pile geometry.

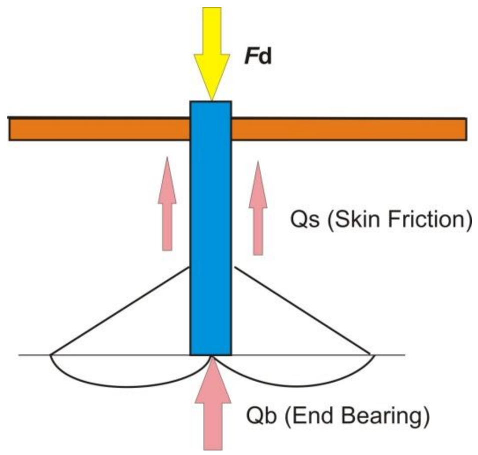


Fig.3.1 Pile's skin friction and end bearing (Source: Wrana, 2015)

3.3.2 Pile bearing Capacity Estimation from Driving Resistance

Several forms of pile capacity estimation procedures has been developed assuming the momentary excedance of pile capacity such that penetration would occurred during pile deriving process (Kramer et al., 2014). One of such earliest formulas to estimate allowable pile bearing capacity is deriving resistance, proposed by (Kramer et al., 2014).

$$Q_{all} = W_r h/FS(s + c) \qquad \text{......................eq.3.27}$$

Where; Wr is the weight of the hammer rams, h is the height of ram drop, FS is the factor of safety, S is the penetration per blow of the pile and C is an energy loss term. The other relation including pile deriving resistance and pile load test results (Kramer et al., 2014):

$Q_{ult} = 1.75\sqrt{E}\log(10N)-100$ ……..eq.3.28

Where; E is the developed hammer energy (kinetic energy at point of impact) and Nis the number of blows per inch of pile penetration.

3.3.3 Pile Bearing capacity Estimation from in situ test results

Currently estimating bearing capacity of piles from in situ testing data as a complement to static and dynamic analysis has got preference by geotechnical engineers. This is due to the fact that in situ tests are simple, faster, cheaper, rapid development of in situ testing instruments and less subjected to uncertainty than those conducted in laboratory with extensive sampling and testing to estimate the soil parameters (Shariatmadariet al., 2007; Bouafia and Derbala, 2016). Some of the commonly applied in situ testing methods used to estimate the bearing capacity of pile foundation are discussed in the following paragraphs;

Pile load test

Pile load test is generally performed when the geo-material condition at a given site is uncertain and involves applying an increasing vertical load to a test pile and measuring the downward movement of the head of the pile. This is done to abstract relevant information's like pile type, size, length, shape and installation methods (Singh, 2016) and used to resolve the difficult and costly nature of changing sub structure conditions after design and construction is completed. The basic objectives of pile load test are to determine the settlement under working load, to confirm the adequacy of design bearing capacity, as proof of acceptability and to determine allowable bearing capacity (Sultana et al., 2016). During pile load test the load-deflection behavior of the pile is measured and recorded. This may be up to large displacement (failure) or some 1.5-2 times the working load of the actual pile to be designed (Gabrielaitis et al., 2013; Kramer et al., 2014).

The observed settlement at the top of the pile may not necessarily represent the downward movement of the pile in to the ground due to the applied load. Rather it can be due to local failure of the pile above the ground surface, crushing of the ground under the test plate might be the possible factors along with material type and size of the test pile (Singh, 2016). The most commonly used method of pile capacity estimation based on pile load settlement is the Davisson procedure which states the point of failure (capacity) as the intersection of an offset sloping line with load deflection curve (Fig. 3.2). This failure point is used to define the failure load (Q_f) and failure displacement (w_f) (Fig. 3.2).

Standard penetration test

Standard penetration test is the commonly applied in situ test to determine resistance and density (sand) of materials, sub surface soil profile and associated engineering properties (Shariatmadari et al., 2008). Apart from its limitations with respect to interpretation and repeatability, standard penetration test is found to be the most amiable and reliable in-situ test (Derbala and Bouafia, 2016). Pile bearing capacity can be estimated either directly or indirectly using SPT result. In the direct method SPT-N values are applied with correction factors to determine the pile bearing capacity while in the indirect method soil parameters such as; soil friction angle and un-drained shear strength is obtained by using SPT-N values.

Direct methods face uncertainties from filtering and averaging of data relating to pile resistance and failure zone around the pile base and likewise indirect methods. Both SPT - based methods estimating the pile bearing capacity ignore the excessive pore water pressure generated during the test. Thus, the results may be reliable for sands or non-cohesive granular soils and not for soils with low permeability such as; clays and silts (Shariatmadari et al., 2008).

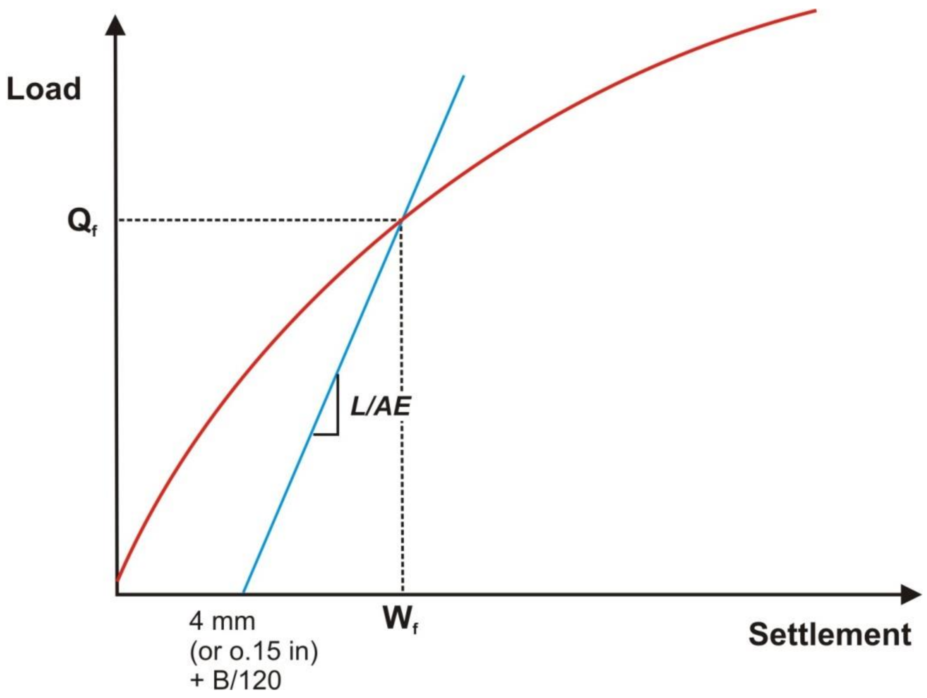


Fig.3.2 Davisson procedure to estimate vertical bearing capacity from pile load test (Source: Kramer et al., 2014)

Bearing capacity from SPT result:

$Q_s=ak/3.5*N_s$ ……..eq.3.29

$Q_b=k/1.75*N_b$ ……..eq.3.30

Where; a=14 and k = 1 for sand and a = 60 and k = 0.2 for clay; N_s is the average value of N around pile embedment depth and N_b is the average of three values of SPT blows around the pile base (Shariatmadari et al., 2008).

3.3.4 Bearing capacity Estimation for cast in-situ piles from UCS and RQD tests

Knowing rock mass parameters is important to design piles socketed in to rock. Basically modern design methods require good knowledge of young's modulus of rock adjacent to the side walls and the rock beneath the base, the average unconfined strength of the rock adjacent to the side walls and beneath the base and the average roughness of the socket sidewall. Besides, it is better to specify whether the load to be addressed in the sidewall of rock, toe of rock or shared in combined manner (Pells, 1999). The shaft resistance of a pile in rock is a frictional response that depends on the roughness of the interface between the concrete pile shaft and the rock, the base (or residual) friction angle of the interface, and the normal stresses applied to the interface. It is inferred that the higher the friction angle the higher the pile sliding resistance and the same is true for normal stress. There are several Key characteristics that affect the development of shaft resistance of a pile in rock (Haberfield, 2013 as cited in Haberfield and Lochaden, 2019).

These include:

- Intact rock strength together with large-scale roughness impacts the degradation of the interface and ultimate shaft resistance
- Intact rock strength together with short scale roughness impacts the rate of mobilization of shaft resistance
- Base friction angle of the interface together with roughness governs the shaft resistance developed as a function of the increasing normal stress
- Pile length which impacts the initial normal stress on the socket

Due to relative higher strength of rock (compared to soil), the ultimate base resistance of a pile in rock can be large and well in excess of the ultimate structural strength of the pile column (Haberfield and Lochaden, 2019). On the basis of several analyses of case studies Haberfield (2013 as cited in Haberfield and Lochaden, 2019) determined that:

- The tests (pile load test) pertinent to mobilize the ultimate bas resistance were those conducted on piles with zero embedment (Qb>5*UCS).
- Tests on piles with embedment depth greater than twice pile diameter in to rock did not reach failure and so the resulting ultimate base resistance were underestimate of it.
- The maximum base resistance measured in the tests increased with pile displacement.
- In extremely fractured rock lower bearing pressure were measured with the same pile displacement.

All these conditions were believed to be due to low porosity and permeability of rock mass which constraint to failure of rock mass below the toe of a pile and the practical limit of base resistance piles were the structural strength of pile column. In line with this low structural loads and long pile length leads to shaft resistance dominate base resistance while in high structural load and short socket pile length base resistance dominates shaft resistance. Key parameters that has to be perceived from geotechnical investigation include, lithology with layers of different rock mass strength, rock mass Young's modulus, intact rock mass strength (UCS) base friction angle of rock, groundwater and fracturing within the rock mass (Haberfield and Lochaden,2019).

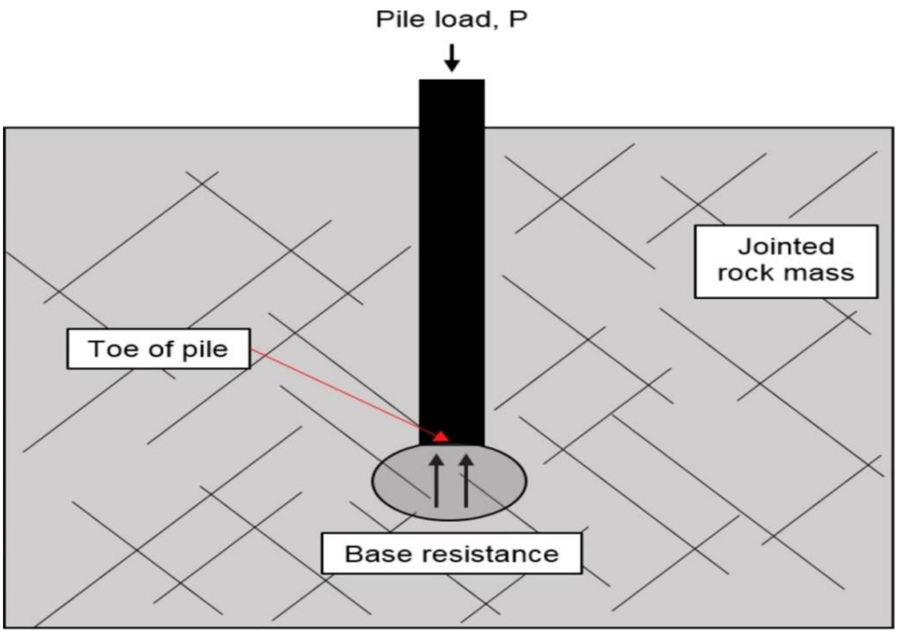


Fig.3.3 Base resistance for piles in jointed rock (Haberfield and Lochaden, 2019)

Existing empirical methods use empirical relations between unconfined compressive strength (UCS) of intact rock and rock quality designation (RQD) to determine end bearing capacity of rock socketed pile foundations. Rock socketed piles are supported by both the intact rock and the discontinuities separating intact rock blocks and thus the influence of properties of discontinuities has to be considered apart from intact rock properties in determining end bearing capacity of rock socketed piles. This is to address influence of discontinuities on rock mass strength depending on their intensity, orientation and nature of infilling material (Zhang, 2010). Zhang, (2010) developed an empirical relation between ultimate end bearing capacity and unconfined compressive strength of rock mass which explicitly addresses the effect of discontinuities represented by rock quality designation. According to Zhang's (2010) expression, the accuracy of determining UCS of rock mass was verified by comparing its estimated values with existing empirical expressions based on rock mass classifications.

Table 3.6 Equations developed so far to estimate end and shaft bearing capacity of rock mass Adopted from (Zhang, 2010)

No.	Proposed equation	Date and authors
1	$\mathbf{Q}_{max}$ =7.68($\boldsymbol{\sigma}cm$)$^{0.42}$MPa where $\boldsymbol{\sigma}$cm = $\alpha E\boldsymbol{\sigma}$c and αE=0.0231RQD-1.32=0.15	AASHTO (1996)
2	Qmax=6.39($\boldsymbol{\sigma}cm$)$^{0.45}$ where $\boldsymbol{\sigma}$cm = $\alpha E\boldsymbol{\sigma}$c and αE=0.0231RQD-1.32.=0.15	Zhang (2010)
3	T max=0.34qu 0.51 T max=0.35qu 0.5	Unit Skin Friction, Rosenberg and Journeux (1976) Williams and Pells(1981)*
4	Q ult=0.42qu Q ult=0.38qu	Unit End Bearing Zhang (1998)

3.3.5 Effect of water chemistry on concrete structures (piles)

Piles submerged in water have been designed to resist external sulfate attack, which occurs when sulfate in ground water, saline soils or marine environments reacts with calcium aluminate hydrates in the concrete (Yao and Li, 2019). External sulfate attack (ESA) is a degradation process that compromises the durability of concrete elements exposed to sulfate environments (Ikumi and Segura, 2019). Concrete piles embedded in permeable soils or rocks may be damaged by groundwater saturated by acids, alkalies, or chemical salts. The main structural effects of sulfate attack on pile foundation are; it make pile cross-sectional area loss; make the cover concrete to the reinforcing bars damage; lead to pile foundation settlement; make pile skin friction lose and so on. The deterioration of the concrete pile surface may change the friction at the affected areas and reduces the vertical load bearing capacity of a pile (Yao and Li, 2019).

The Principal factors involved in submerged pile foundation failures due to sulfate attack are composition and density of the concrete, porosity of the aggregates and concrete cover over the reinforcing steel. The deterioration of concrete piles due to sulfate in ground water can be minimized by careful formulation of the concrete mix, use of sound, hard aggregates, proper mixing, placing, consolidating, and curing to achieve hard dense concrete (Xie et al., 2019; Yao and Li,2019). The attack can be divided in four main processes such as transport, chemical reactions, expansive forces and the mechanical response. Fig 3.4 (a) show Cast in-situ pile in sulfate-rich saline soil (Xie et al., 2019) and Fig. 3.4 (b) demonstrates processes involved in external sulfate attack (Ikumi and Segura, 2019).

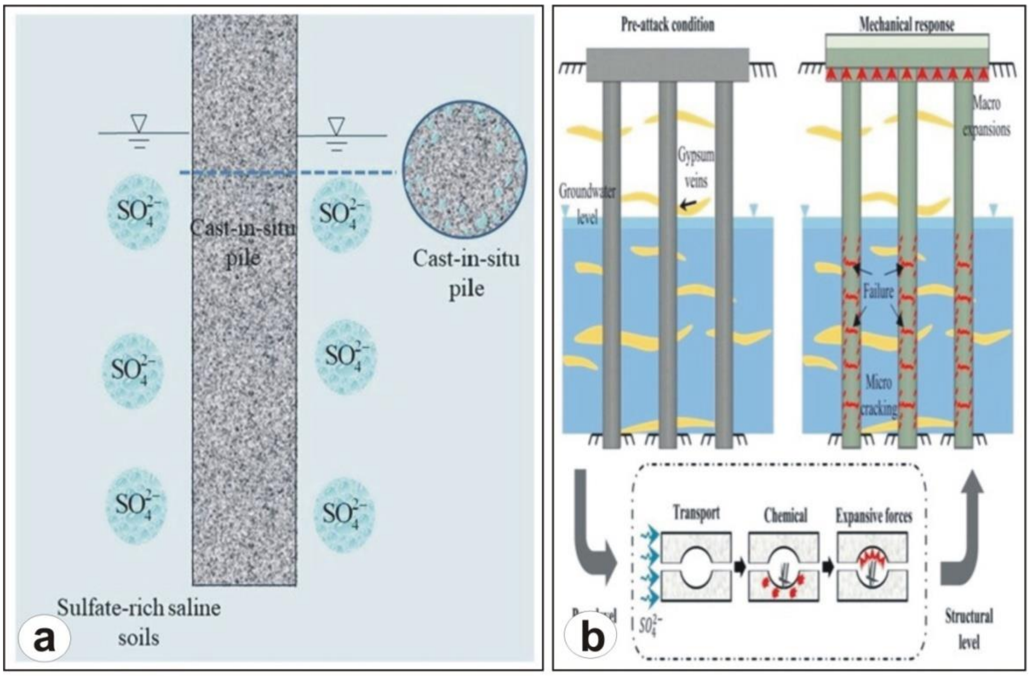


Fig 3.4 (a) Cast in-situ pile in sulfate-rich saline soil (Xie et al., 2019) and (b) Processes involved in external sulfate attack (Ikumi and Segura, 2019)

3.4 Genesis of methodology for present study

3.4.1 Preamble

Research methodology is a means which ties up all the research processes jointly and guides the researcher to achieve the aim and objectives of the research (Tibebu Solomon, 2015). The respective outcome of this study was ensured through adopting organized and systematic methodology perceived from extensive literature review which brought general conceptual frame work about this topic. Review of journal articles, books, reports both published and unpublished played a great role to outline geological and geotechnical features that have to be incorporated within dam foundation assessment. The main important topics that attract research attention for dam projects are geotechnical and engineering geological methods and same were thoroughly assessed for the present study. The objective of the present study was addressed through adopting Engineering geological and geotechnical methods relevant for the foundation assessment, encompassing respective components of the present work. The input parameters for these generic methods were obtained from secondary database more of the geotechnical aspect and primary data sources through field survey. Further, integration of engineering geological, geotechnical and hydrological studies was made to meet out the objective of the study.

3.4.2 Engineering geological and Geotechnical investigations

For the present study geotechnical as well as engineering geological investigation methods were used to obtain the relevant information for the assessment of the subsurface geological condition along the proposed Megech dam foundation. In the geotechnical framework interpretation and analysis of borehole data with corresponding in-situ and laboratory test results and data concerning engineering geological mapping and rock mass classification were accessed through the field survey.

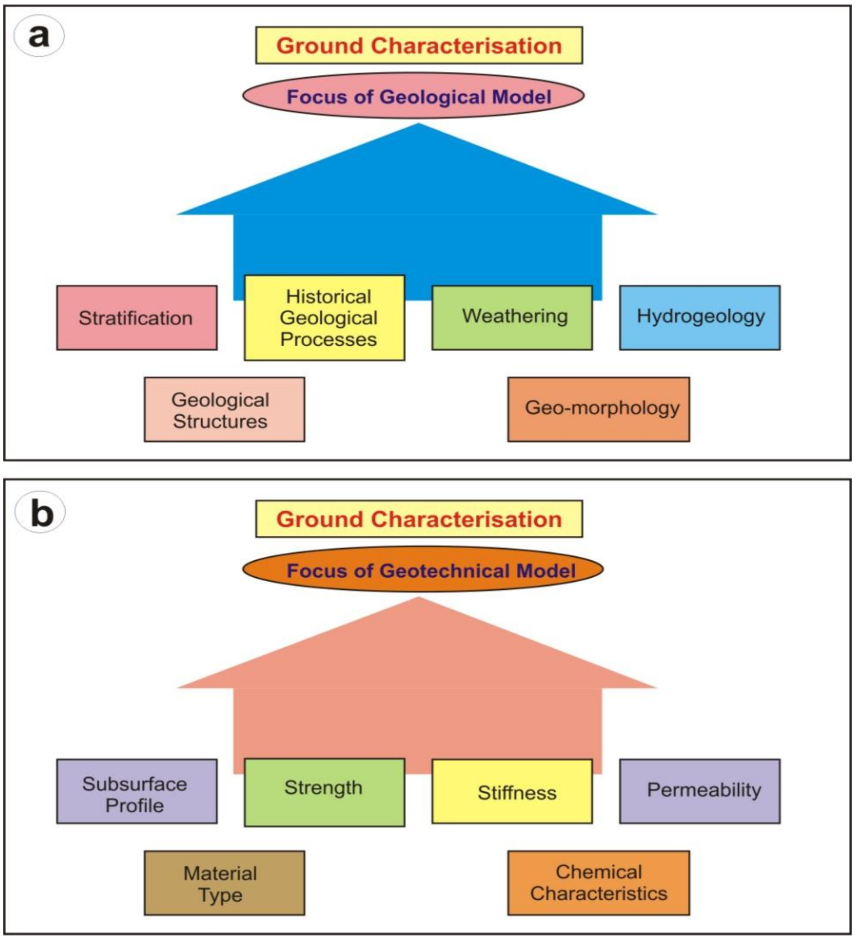


Fig 3.5 Ground modeling through (a) Geological characterization and (b) Geotechnical characterization (Source: Morgenstern, 2000)

3.4.3 Engineering geological investigation

Engineering geological investigation for the present study involved mapping, structural investigation and rock mass classification in the dam foundation as well as in the surrounding of the dam site through systematic field investigations.

Engineering geological mapping

An engineering geological map is a type of geological map, which provides a generalized representation of all those components of geological environment, which are significant to design, construction, and maintenance of civil engineering structure. Another use of the map is to identify potential problems or favorable conditions existing at the proposed project site (Daniel Gebremichael, 2017). The engineering geological mapping of Megech dam foundation and its vicinity has been carried out on the basis of engineering characterization of the rock mass and the soils that exist at the site. For mapping purpose the rocks have been characterized based on the strength, weathering grade and the jointing. Whereas the soils of the dam site area has been mapped based on geotechnical properties defined during detailed design study (WWDSE, 2010). Mapping in this study includes the dam and appurtenant structures foundation as well as the surrounding area that could influence or could be influenced by the dam. It presents types of rocks and soils exposed at the dam foundation and abutments with their mechanical and geotechnical properties (WWDSE, 2010).

Discontinuity survey

In order to understand and quantify the influence that the discontinuities may have on rock mass behavior such as; permeability, deformability and strength quantifying the relevant characteristics of discontinuities is necessary (Rajabi et al,, 2014; Nezhad et al., 2013; Ghafoori et al., 2011). The characteristics include spacing aperture, roughness, frequency, persistence, filling and orientation. In this study the discontinuity surveying was undertaken in accordance with the suggested method by the International Society of Rock Mechanics (ISRM, 1978) at dam foundation including both the abutments. Latter on the discontinuity survey results have provided basic parameters for classification of the rock mass and to determine engineering characteristics of the rock mass such as; deformability and strength of the rock mass. In general, during the present study joints of different orientation, spacing, filling, aperture and persistence were found along the river bed and abutments.

Rock mass classification

In order to assess the dam foundation in the present study and to evaluate the rock mass characteristics, rock mass rating (RMR) system was applied (Bienwski, 1989). In this respect required RMR parameters to classify the rock mass along the foundation and abutments were obtained through the field survey. Later, RMR was determined at different rock exposures in the dam foundation area (river bed and abutments). The RMR results later were used to determine the engineering geological and geotechnical characteristics of the rock mass along with the data from the secondary sources. Thus, geotechnical and engineering geological properties of the rock mass as determined for the dam foundation is dealt in detail, later in Chapter-4.

Geological cross sections

Geological cross-section in its essence shows vertical and horizontal geological (lithology) variation as well as groundwater level along the concerned axis of a dam structure. The geological cross-section during the present study was generated along the dam axis and conduit outlet works from surface geology and drilled hole data as obtained from the design investigation report (WWDSE, 2010). Seven borehole log data and five borehole log data were used to generate geological cross-sections along the dam axis and the conduit outlet structure, respectively. The geological cross section and discussion on lithological variation with groundwater depth is presented in detail in Chapter 5.

3.4.4 Geotechnical investigation

To acquire basic geotechnical data a number of boreholes were drilled on dam foundation and used to assess the subsurface structure and materials on the foundation sites including bearing capacity and water tightness. During the detailed design investigation two inclined boreholes (BH10 and BH11) were also drilled on the main dam foundation with the intention of verifying the prevalence of faulting inferred from missed lithological units on the geological x-section(WWDSE, 2010). For the present study above mentioned data on borehole logs and cross section was obtained from the geotechnical design report of the dam from the Water Works Design and Supervision Enterprise (WWDSE) and the Ministry of Water, Irrigation and Energy (MWIE). Later, by using empirical equations and input (UCS) from the design report and pile load test bearing capacity of rock socketed piles under intake and conduit outlet structures were determined.

Bearing capacity and settlement

In the present study potential settlement from pile load test and bearing capacity from UCS and empirical equations as well as pile load test of piles under intake structure and conduit outlet works was carried out. To determine bearing capacity, empirical equations were adopted and the input UCS and RQD were obtained from the design investigation report of the Megech dam project (WWDSE, 2010a). Settlement in turn was determined from the pile load test conducted on selected piles along the structures of outlet works, adopted from the design investigation report of the project as well.

Permeability and hydrological study

Permeability water pressure tests (packer) were performed by the project authorities in the sound rocks where the lithological condition was found favorable. The test was conducted for a test section between 3 to 5 m. Five pressure steps measured at the top of standpipe in bars of 33%, 67%, 100%, 67%, 33% of the effective overburden pressure at the Centre of the test section was applied. Falling head tests were conducted in soils of low permeability. In falling head test-descending water levels were measured (water level recorder) at regular intervals of 1min, 5min, 10min for 1-hour test and the results of the tests permeability was calculated (WWDSE ,2010). This data was processed according to the analysis methods to estimate the permeability of geo-materials in the dam foundation for this study. Apart from this hydro chemical analysis of groundwater and its effect on the performance of piles submerged in water through interaction was determined.

3.5 Integration of Engineering geological, geotechnical and Hydrological studies

Finally, the respective studies, analysis and results with respect to engineering geological, geotechnical and hydrological condition were integrated to meet out the objective of the present study. The required input data for the methodology and the source from where this data was procured or generated is presented and discussed in detail in Chapter-4. Through site investigation and data procurement from secondary sources with adequate analysis along with experience, appropriate model for dam foundation may be worked out. The detailed methodology followed in the present study is presented through Fig3.6.

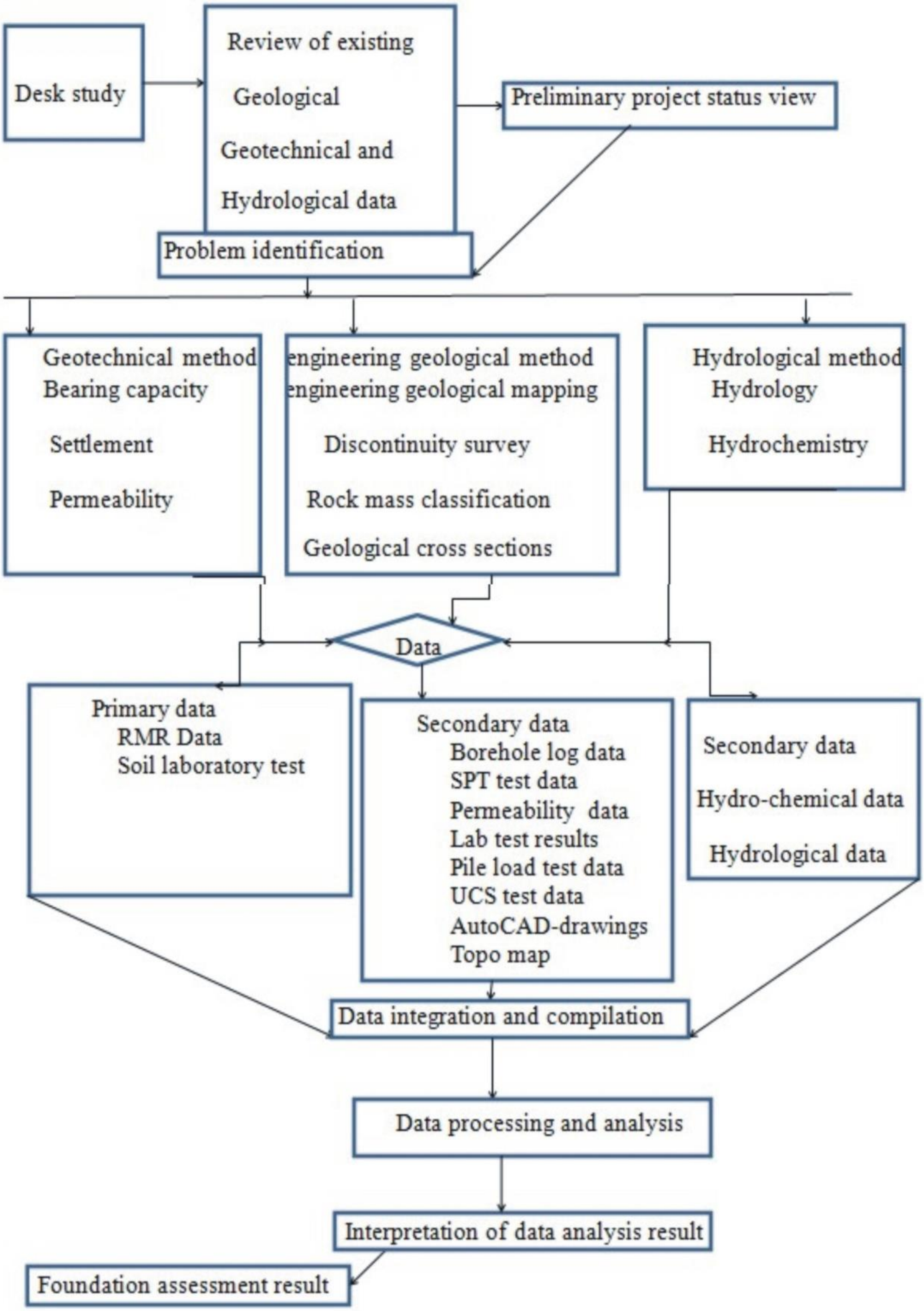


Fig. 3.6 General methodology followed in the present study

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CHAPTER –FOUR

DATA COLLECTION, PREPARATION AND ANALYSIS

4.1 Preamble

The successes of a given research largely depend on existence and reliability of data pertained to the proposed topic along with sufficient quality and quantity relative to level of study. Any study involves either collecting the relevant data purely in the field or relay on secondary data or incorporates both levels of data sources. In line with this systematic and periodical data collection makes a study relevant and accomplished.

4.2 Data collection, preparation and analysis

For the present study systematic data was collected from the secondary and primary sources. Secondary sources from where the data was procured are; Ministry of Water, Irrigation and Energy (MWIE) and Water Works Design and Supervision Enterprise (WWDSE). The secondary data that was collected for the present study mainly relate to geological, geotechnical and hydrological aspects for the Megech dam site. The primary data was collected from the field during the present study and it includes data pertaining to geological mapping, data for rock mass classification and representative sampling for the laboratory testing. Further, hydro chemical data was processed along with other secondary and primary data to assess the nature of ground water and its impact on the submerged piles for corrosion and sulfate attack. Collectively these data were used to assess the dam foundation and to address the objective of the present study.

4.3 Primary data collection

4.3.1 RMR data collection at surface outcrops

In order to know the characteristics of the rock mass at the dam foundation detailed lithological description, discontinuity survey with other relevant data pertaining to RMR was collected on rock exposures; natural or on stripped slope sections. The location for investigation and data collection during the field work was decided on the basis of variation in jointing intensity, change in degree of weathering, lithological variations or variation in ground water surface manifestations on rock exposures in dam foundation; river bed section and the abutment slopes.

In the present study rocks of right and left abutment as well as rocks in the river bed were classified according to rock mass rating (RMR) system proposed by Bieniwaski (1989). During the field investigation locations of various lithological contacts were identified and marked to finalize the geological mapping of the dam site. Besides, RMR data was also collected from various locations and representative photographs were also taken for later reference. Also, quantitative description of discontinuities including orientation, spacing, persistence, roughness, aperture and filling along with other relevant RMR parameter data were collected at different locations in accordance with ISRM (1981). In total 62 representative sites were selected for the RMR data collection. Out of these 53 sites were identified on the right abutment, 1 in the river bed section and remaining 8 were on the left abutment (Table 4.1; Fig. 4.1). Thus, from all these locations data pertaining to RMR was collected and rock mass was characterized accordingly.

Uniaxial compressive strength (UCS)

Based on Schmidt hammer value number of studies has proposed correlations to estimate UCS of intact rocks (ISRM, 2007-2015). According to ISRM (2007-2015) the UCS verses Schmidt hammer Rebound value correlations should be established using mean rebound value involving the entire set of measurements. The UCS of a given rock type are highly sensitive to slight changes in degree and style of weathering, density and orientation of micro cracks, grain size distribution as well as mineralogy (ISRM,2007-2015).

For the present study UCS has been determined by using L-type Schmidt hammer for selected rock mass and has been converted in to UCS based on relation proposed by Barton and Choubey (1977) (eq.4.1);

$$\mathrm{Log10}\ (\sigma c) = 0.00088\gamma^{*}R + 1.01 \qquad \ldots eq.4.1$$

From this; $\mathrm{UCS} = 10^{0.00088}\gamma^{R+1.01}$

Where; UCS is the uniaxial compressive strength in MPa, γ is the rock dry density in KN/m^3 and R is the Schmidt rebound number.

For the present study, dry rock density (γ) for fresh basaltic intact rock was taken equal to 30 KN/m^3, for slightly weathered basaltic intact basalt γ was taken equal to 25 KN/m^3 (WWDSE, 2010) and for moderately weathered intact basalt it was taken equal to 20 KN/m^3 (WWDSE, 2010).

Table: 4.1 Location of RMR data collection points at Megech dam site

RMR-ID*	Easting	Northing	Elevation	Geology	Location
LA-1	3331105	1382365	1945	Decomposed basalt	Left abutment
LA-2	333115	1382365	1940	Highly weathered ,,	,, ,,
LA-3	333104	1382350	1956	,, ,, ,,	,, ,, ,,
LA-4	333073	1382367	1950	,, ,, ,,	,, ,, ,,
LA-5	333032	1382456	1910	,, ,, ,,	,, ,, ,,
LA-6	332992	1382462	1911	,, ,, ,,	,, ,, ,,
LA-7	332545	1382710	1877	,, ,, ,,	River bed
LA-8	332993	1382460	1909	Highly weathered vesicular basalt	Left abutment
RA-1	332603	1382859	1877	Weathered basalt	River bed
RA-2	332615	138271	1875	,, ,, ,,	River bed
RA-3	332618	1383107	1879	,, ,, ,,	River bed
RA-4	332447	1382800	1882	Highly weathered ,,	Right abutment
RA-5	3324462	1382829	1894	,, ,,, ,,	,, ,,, ,,,
RA-6	332554	1382914	1897	Weathered basalt	,, ,, ,,,
RA-7	332563	1382924	1899	Slightly weathered ,,	,, ,, ,,
RA-8	332582	1382944	1900	Highly weathered ,,	,, ,, ,,
RA-9	332598	1382967	1902	,, ,, ,,	,, ,, ,,
RA-10	332610	1382982	1897	,, ,, ,,	,, ,, ,,
RA-11	332646	1382952	1888	,, ,, ,,	,, ,, ,,
RA-12	332661	1382976	1893	,, ,, ,,	,, ,, ,,
RA-13	332627	1382984	1884	Weathered basalt	Right abutment
RA-14	332620	1382996	1890	Highly weathered ,,	,, ,, ,,,
RA-15	332632	1383006	1890	,, ,, ,,	,, ,, ,,
RA-16	332645	1383020	1895	,, ,, ,,	,, ,, ,,
RA-17	332647	138348	1898	,, ,, ,,	,, ,, ,,
RA-18	332666	1383083	1905	Weathered ,, ,,	,, ,,
RA-19	332640	1382052	1906	,, ,, ,,	,, ,, ,,
RA-20	332630	1383087	1927	,, ,, ,,	,, ,, ,,
RA-21	332590	1383060	1944	,, ,, ,,	,, ,, ,,
RA-22	332598	1383088	1940	,, ,, ,,	,, ,, ,,
RA-23	332590	1383156	1950	Porphyritic basalt	Right abutment
RA-24	332565	1383115	1931	,, ,, ,,	,, ,,
RA-25	332535	1382971	1945	,, ,, ,,	,, ,,
RA-26	332571	1383030	1950	,, ,, ,,	Right abutment
RA-27	332568	1383071	1962	,, ,, ,,	,, ,, ,,
RA-28	332564	1383105	1958	,, ,, ,,	,, ,, ,,
RA-29	332617	1383232	1952	,, ,, ,,	,, ,, ,,
RA-30	332542	1383151	2011	,, ,, ,,	Right abutment
RA-31	332524	1383117	2010	,, ,, ,,	,, ,, ,,
RA-32	332547	1383149	1932	,, ,, ,,	,, ,, ,,
RA-33	332523	1382927	1918	Aphanitic basalt	Right abutment
RA-34	332550	1382962	1930	,, ,,	,, ,,
RA-35	332585	1383017	1932	,, ,,	,, ,,
RA-36	332559	1382976	1929	Decomposed basalt	,, ,,
RA-37	332521	1382960	1937	Aphanitic basalt	,, ,,
RA-38	332476	1382875	1917	Weathered basalt	,, ,,
RA-39	332466	1382894	1912	Vesicular basalt	,, ,,
RA-40	332470	1382919	1926	,, ,,	Right abutment
RA-41	332472	1382953	1948	Weathered basalt	,, ,,
RA-42	332480	138398	1953	,, ,,	,, ,,
RA-43	332450	1382943	1954	,, ,,	,, ,,
RA-44	332376	1382988	1957	,, ,,	,, ,,
RA-45	332480	1383061	1976	Aphanitic basalt	Right abutment
RA-46	332458	1382982	1959	Weathered basalt	,, ,,
RA-47	332452	1382999	1969	,, ,,	,, ,,
RA-48	332466	1383047	1970	,, ,,	,, ,,
RA-49	332483	1383106	1974	Upper Basalt	,, ,,
RA-50	332543	1383043	1954	Upper basalt	Right abutment
RA-51	332359	1382660	1868	Basalt	River bed
RA-51	332572	1382943	1905	Vesicular basalt	Right abutment
RA-52	332558	1382936	1991	Basalt	Right abutment
RA-53	332591	1382965	1910	Aphanitic basalt	,, ,, ,,

* LA – Left Abutment; RA – Right Abutment

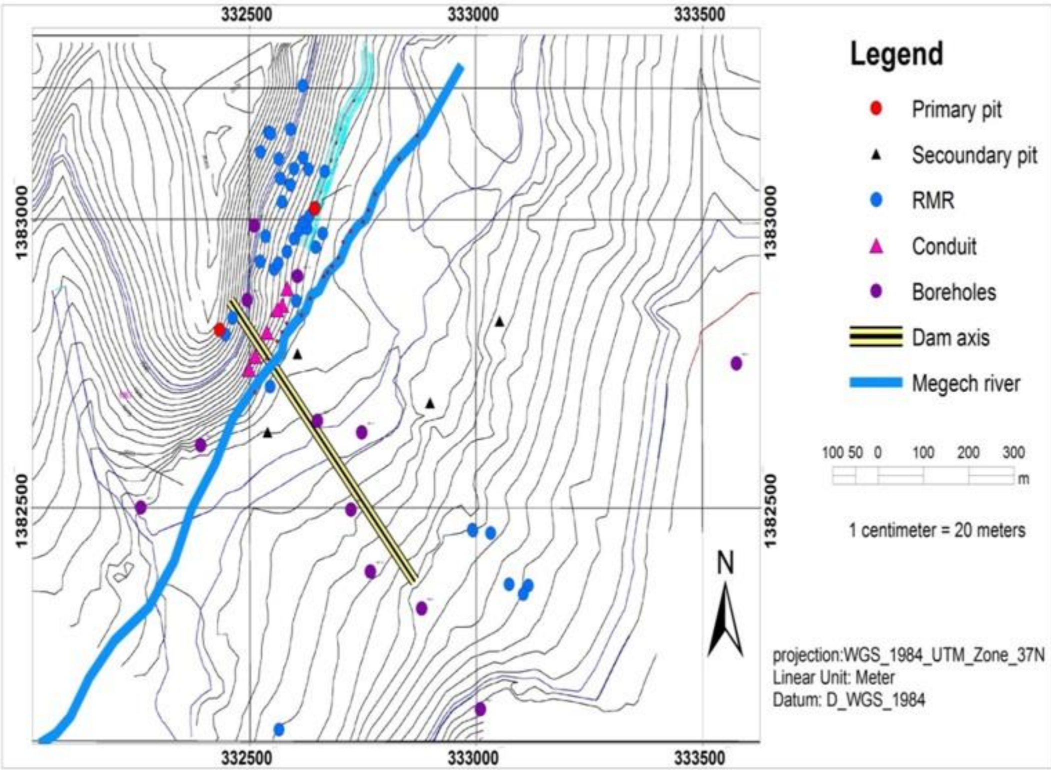

Fig 4.1 Primary and secondary data location distribution at Megech dam site – data used for the present study

Plate 4.1 Intact rock strength measurement by using L-type Schmidt hammer

Rock quality designation (RQD)

Rock quality designation (RQD) is used as an important parameter for rock mass classification systems, numerical modeling and for engineering assessment of rock mass. RQD can be determined in various ways such as direct methods and indirect methods (Singh and Goel, 2011). RQD from drill core pieces ($\geq$10cm) on one dimensional boreholes is a direct method given by RQD = sum of core pieces $\geq$10 cm/total core run*100 (Singh and Goel, 2011; Palmstrom, 2005).Indirectly, RQD can be estimated from seismic wave velocities and JV value used when core is not available. JV is partly based on joint spacing and frequency as a measure of the dominant joint set (Palmstrom, 1982).

By using in-situ compressional waves RQD can be estimated as proposed by (Singh and Goel 2011) (eq.4.2);

$RQD = (\frac{VF}{V1})^2 * 100$……..eq 4.2

Where; VF is the in situ compressional wave velocity and V1 is the Compressional intact rock core.

Other empirical relation is proposed by Palmstrom (1982) (eq. 4.3);

RQD = 115 -3:3 JV ……..eq. 4.3 Where; JV is the volumetric count;

In the present study RQD, as one of the parameter for (RMR), was determined through an indirect method using Palmstrms (1982) relation (eq.4.3).

Discontinuity spacing

Discontinuity spacing refers to the linear /perpendicular distance between two adjacent discontinuity sets (Singh and Goel, 2011; Palmstrom, 2005).The spacing between adjacent joints controls the size of individual block and size of intact rock in which close spacing implies loose cohesion rock mass behavior while those widely spaced bring interlocking rock mass condition (ISRM, 1978). Discontinuity spacing for RMR in the present study was determined by simple measurement using measuring tape.

Discontinuity conditions

Discontinuity condition is determined from five basic parameters namely; persistence, aperture, roughness, infilling and weathering of discontinuity surfaces. Collectively these five parameters constitute discontinuity condition which is one of the input parameter for RMR. The ratings for corresponding discontinuity parameters are presented in Table 4.2. During the present study discontinuity condition was assessed by visual observations in accordance with Table 4.2 at representative RMR data collection locations.

Table 4.2 Guideline for rating discontinuity conditions (Aksoy, 2008)

Parameters/ Ratings	Conditions/ Representative ratings			
Persistence (m)	<1	1-3	3-10	10-20	>30
Rating	6	4	2	1	0
Aperture (mm)	None	<0.1	0.1-1	1-5	>5
Rating	6	5	4	1	0
Roughness	Very rough	Rough	Slightly rough	Smooth	Slickenside
Rating	6	5	3	1	0
Infilling (mm)	None	Hard filling<5	Hard filling>5	Soft filling<5	Soft filling>5
Rating	6	4	2	2	0
Weathering	None	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
Rating	6	5	3	1	0

Groundwater conditions

Groundwater condition pertaining to RMR in the present study was determined through visual observation in accordance with Bieniwaski (1989). Surface manifestations of ground water were evaluated as; completely dry, damp, wet, dripping and flowing at representative RMR data collection locations.

Discontinuity orientation

Discontinuity orientation refers to the strike and dip of discontinuities. The influence of the strike and dip of discontinuities has to be considered with reference to direction of tunnel diverge, slope face orientation or foundation alignment (Singh and Goel, 2011). For the present study discontinuity orientations were measured by using Brunton compass. Several observations at representative RMR data location sites were made and later preferred orientations were worked out with the help of stereographic analysis. Finally, respective ratings were assigned as per the standard rating criteria of Bieniwaski (1989).In the present study discontinuity orientation was evaluated with respected to dam foundation alignment.

4.3.2 Estimation of rock mass rating (RMR)

Based on the observations or measurements for all six parameters pertaining to RMR at each location corresponding ratings were assigned based on the standard tables. Later, RMR was determined as algebraic sum of ratings for all six parameters at each respective site. In RMR computation basic adjusted for discontinuity orientation was also made as the effect of discontinuity orientation will have effect on type of structure to be constructed on that rock mass (Bieniwaski, 1989; Singh and Goel, 2011). Thus, on the basis of RMR values, the rock mass under the dam was classified in to different classes (Bieniwaski, 1989). Further, the rock mass classification results later were used to estimate the parameters such as; type of support required for tunnels (Bieniwski ,1989), shear strength parameters; cohesion, angle of internal friction, modulus of deformation of rock mass and allowable bearing pressure for foundation as per relations proposed by Serafim and Pereira (1983) and Bowles (1996).

4.3.3 Soil sampling and laboratory analysis

During the present study soil sample was collected from the paleo-soil, located at the main Dam foundation (core level) on the right abutment of the dam. The paleo-soil exposed is intercalated within the porphyritic basalt above and lower aphanitic basalt below it. Soil samples were collected by digging two test pits; one located at the center of the dam axis and the other downstream of the dam axis where this soil bed is exposed (Table 4.3; Plate 4.2). The necessity to sample and test this paleo-soil arises because no specific test to know its behavior was conducted earlier by the project authorities. Simply its existence along with high degree of weathered rock mass was mentioned that has resulted to adopt pile foundation in the design under the conduit outlet and intake structures. Thus, the primary aim of conducting test on this paleo soil was to assess the engineering behavior of this soil and to know the likely problems associated during construction or in the operational stage of the dam. The test on paleo soil was conducted at the Geotechnical soil laboratory of Bahir -Dar Institute of Technology. The soil properties that were determined include; Sieve analysis (wet-sieve), Specific gravity, and Atterberg limit and hydrometer analysis (Table 4.3). These tests were conducted in accordance with AASTM standard (ASTM, 1980).

Table: 4.3 Sampling location and corresponding tests conducted on paleo soil samples

Pit No.	Test type	No. of test	Test pit location		Depth of sample (m)
			Relative location	GPS coordinate (UTM-WGS-Zone-37)	
TP -1	Wet sieve analysis	1	Central dam axis	332436E; 1382973N Elevation - 1971m	1.2
	Hydrometer analysis	1			
	Specific gravity	1			
	Atterberg test	1			
TP - 2	Wet sieve analysis	1	Downstream of dam axis	332447E; 1382800N Elevation -1882	0.8
	Hydrometer analysis	1			
	Specific gravity	1			
	Atterberg test	1			

Plate 4.2 Soil sampling locations (a) Test pit (TP-1) at the core termination in between porphyritic basalt above and aphanitic basalt below on the right abutment of the dam and (b) The core termination at the right abutment in contact with paleo-soil and line of conduit outlet work structure.

4.4 Secondary data collection

4.4.1 Borehole data collection

Following the topographic survey and other previous studies conducted current dam axis was decided and extensive investigation was done by the project authorities including borehole drilling along critical locations like dam axis, conduit, intake and spillway structures (WWDSE, 2010a). In order to acquire the necessary geological and geotechnical data used to design the dam and assess the foundation in terms of bearing capacity as well as permeability several boreholes were drilled both in the preliminary and detail design investigations. Of these two were drilled with inclination during detailed design investigation with the intention of checking existence of geological discontinuity in the main dam foundation. Collectively 12 boreholes were drilled with a total depth of 702.97m in the main dam axis (8), intake structure (1), 6 in the conduit structure and two in the spillway (Table 4.4). These boreholes were drilled in conjunction with packer test (rock), falling head test (overburden), UCS test, Unit weight test, and SPT test accordingly and the depth of these boreholes varies from 21-90m (WWDSE,2010).

Table4.4 Details of drilled boreholes at different locations at dam site -dam axis, Intake tower, spillway and conduit structures

BH-ID	Location	Total depth (m)	UTM-coordinates –zone-37		Elevation (m)
			Easting	Northing	
NB-1	Dam foundation	90.07	1382150	333010	1958
NB-2	Dam foundation	80.15	1382496	332724	1888
NB-3	Dam foundation	85.00	1382650	332650	1879
NB-4	Dam foundation	90.00	1382860	332495	1961
NB-5	Dam foundation	60.00	1382325	332880	1919
NB-6	Dam foundation	60.00	1382750	333575	1888
NB-7	Dam foundation	21.05	1382500	332260	1912
NB-10	Dam, Foundation	60.00	1382389	332767	1899
NB-11	Dam Foundation	60.00	1382630	332748	1877
NB-12	Spillway	21.40	1382989	332510	1958
NB-13	Spillway	25.30	1382608	332392	1890
NB-14	Intake tower	50.00	1382901	332606	1900
Total depth		**702.97m**			
MDC-BH-5	Conduit	40	332583	1382880.23	1895.92
MDC-BH-6	Conduit	40	332512.45	1382762.89	1891.94
MDC-BH-7	Conduit	40	332572.169	1382851.362	1895.424
MDC-BH-8	Conduit	40	332561.79	1382844	1895.92
MDC-BH-9	Conduit	40	332538.42	1382805.6	1895.92
MDC-BH-10	Conduit	40.74	332499	1382740	1895.92
Total depth		**240.74 m**			

The geotechnical and geological investigation results procured from WWDSE were the theoretical as well as practical foundation of the present study. Borehole data along with surface geological mapping was used to generate the geological /geotechnical cross sections along the main dam axis and the outlet conduit structures. These cross sections have provided information on subsurface geological lateral and vertical variations. Further, the associated tests conducted in the borehole like; permeability, UCS, unit weight, RQD and pile load test were used to analyze the dam foundation in terms of its seepage potential, bearing capacity, settlement potential and to assess rock quality, respectively (WWDSE ,2010).

4.4.2 Permeability data Collection

Seepage flow within rock mass is affected by joints, weathering, fractures and other major geological structures. These structures have great influence on foundation stability and engineering performance of the structure. Thus, for adverse seepage conditions there is a need to develop practical seepage control measures used in engineering practice (Chen et al., 2010). Further, seepage through foundation resting on fissured rock mass is strongly dependent on fracture characteristics like degree of joint spacing, joint aperture, continuity and presence of weathering and infill material. Water pressure/ Lugeon permeability test is the suitable method to determine the permeability of the foundation rocks (Barani et al., 2014). Proper interpretation of permeability data in dam projects is important to determine seepage quantity through dam foundation and to outline weather there is a need to seal the rock foundation or not (Bazegari, 2017).In order to assess the permeability of the Megech dam foundation several in situ permeability tests were performed during drilling of exploratory boreholes in the feasibility as well as detail design investigations (Table 4.4). As a part of Megech dam design investigation and to assess permeability of the dam foundation falling head in the overburden soils and packer or lugeon tests in the rocks were conducted (WWDSE, 2010a).

Packer test

Water pressure tests (packer) were performed in sound rocks where the lithological conditions were favourable. Packer tests were conducted for a test section between 3 to 5 m. five pressure steps measured at the top of standpipe in bars of 33%. 67%, 100%, 67%, 33% of the effective overburden pressure at the centre of the test section was used. Flow of water was observed and recorded at every two minutes for ten minutes for each pressure step. The results presented include water intake and Lugeon unit (WWDSE, 2010a). The permeability analysis results as computed by eq.4.4 are given later inChapter-5.

$$Lugeon\ value = \frac{Water\ intake\ in\ liters/min/m * 10 bars}{Total\ test\ pressure\ (bars)} \qquad ....eq.4.4$$

4.4.3 Geotechnical laboratory test results

The suitability of a soil for a given civil engineering structure depends on its engineering properties. Therefore, the performance of any engineering work will depend on the correct assessment of engineering properties of soils. Those properties can be grain size distribution, Atterberg limits, permeability and shear strength parameters (Nigatu Fekadu, 2006).The laboratory test results adopted from dam final design report along the foundation bore holes (WWDSE, 2010a) are presented in Table 4.5.

Table 4.5 Laboratory test results adopted from dam final design report along the foundation bore holes (WWDSE, 2010a)

TP-BH ID	Location	Depth (m)	Grain size distribution					Atterberg limits				Direct shear	
			Gravel	Sand	Silt	Clay	Fine	LL	PL	PI	SL	C	φ
MP-0	FD	0.2-0.9		5.12	64.88	30	94.88						
MP-0	FD	0.9-2		2.36	41.64	56	97.64						

Mp-0	FD	0.5						60	32.3	27.7			
Mp-0	FD	1.5						68	31.4	36.7			
Mp-3	FD	0.2-3		19.5	66.5	14	80.5	46.1	29.37	16.8	8.36		
Mt-1	FD	0-0.2						60.6	39.87	20.87		68.67	29.9
Mp-1	FD	0-1.4		2.77	51.23	46	97.23	71.3	39.3	32	19.3	89.67	16.17
Mp-1	FD	0-1.4						76.5	35.2	41.28			
Mp-2	FD	0-1.75	70.56	8.37	13	8	21.07	61.8	34.87	26.93			
Mp-2	FD	0-1.75		36.77	51.23	12	63.23	49.7	31.23	18.47			
Mp-2	FD	1.75-2.3	65.83	33	1.13	0	1.13						
NB-1	LA	0.2-1						56.7	39.88	16.8	39.48		
NB-1	LA	0.2-1.64		48.16	48.84	3	51.84						
NB-1	LA	1.95-2.45										34	42
NB-1	RA	1.95-2.45										53	36.5

FD - Foundation, LA – Left abutment, RA – Right abutment, LL – Liquid limit, PL – Plastic limit, PI – Plasticity index, SL – Shrinkage limit, C – Cohesion, φ – angle of shearing resistance

4.5 Data analysis

4.5.1 Rock mass characterization

A Rock mass seldom exists without associated discontinuities inducing anisotropic deformational and strength behaviour. As a result rock mass characterizations of complex structures like dam, tunnel and others are important to recognize the vulnerable regions within the domain of construction site. In order to characterize the rock mass both the rock material and discontinuity characteristics are measured separately and later their interaction is evaluated (Brain and Chappell, 1990). In this study brief description of rock mass with Quantitative description of discontinuities including orientation, spacing, persistence, roughness, aperture and filling were done at the site by scan line method in accordance to ISRM (1981). For this lithological, weathering grade along with discontinuity were assessed.

4.5.2 Foundation assessment

Assessment through rock mass classification

Geomechanics Classification (Rock Mass Rating System) developed by Bieniawski (1973; 1989) was used for the classification and interpretation of the quality of the foundation rocks at the proposed dam site. The six parameters as proposed by Bieniawski (1989) were used to classify the rock mass using RMR system. Data pertaining to parameters used to estimate RMR were collected from the field. The rock mass was first divided in to: homogenous "geotechnical units", based on, fracture density and evenness as displayed by weighted RQD. These regions were assumed to be more or less uniform in certain geotechnical features.

Rock mass deformability assessment from RMR value

To determine deformation modulus of rock mass both in situ tests and indirect methods can be utilized. Indirect methods are getting emphasis over in situ tests as in-situ tests are time consuming, expensive and even questionable on their reliability (Ghafoori et al., 2016). Estimating deformation characteristics of a rock mass is generally based on empiricism as it is difficult to test the rock mass practically at large scale (Bertuzzi, 2019). However, the in situ deformation modulus of a rock mass can be estimated from the final results of the rock mass rating value which in turn is used to assess the rock mass characteristics (Singh and Goel, 2011). In the present study estimation of rock mass deformability was done through empirical equations proposed by different researchers at different time. There is an approximate correlation between the modulus of deformation and RMR index, as suggested by Bieniawski (1978) which is expressed as eq.4.6.

$$Em = 2RMR - 100GPa \qquad \text{.........eq.4.6}$$

The eq.4.6 is applicable for RMR greater than 50 Serafim and Pereira (1983) has also proposed relation for modulus of deformation applicable for all RMR values intended to foundation and given as eq.4.7.

$$Ed=10^{RMR-10/40} \qquad \text{.........eq.4.7}$$

Where; Em is the in-situ deformation of modulus in GPa and RMR is the rock mass rating (Bieniawski, 1989).The Serafim and Pereira (1983) relation was directly applied to assess the deformability of rocks at Megech dam foundation and results are presented in Chapter - 5.

Rock mass strength and failure criteria

As it is known that rock samples and associated laboratory results are not representative for the rock mass, besides in situ tests are also seldom practical or not economically feasible (Nigatu Fekadu,2006) empirical equations are widely applied to estimate the shear strength parameters. The shear strength of rock mass depends upon cohesion and angle of internal friction and in this regard RMR can be used to estimate the cohesion and angle of internal friction in ranges of values as well as in absolute values for representative RMR values (Singh and Goel, 2011). In the present research empirical equations to estimate the shear strength parameters from RMR including the most commonly used and accepted Hoek-Brown failure criterion (Hoek-Brown,2002) were used.

To apply Hoek-Brown failure criteria to estimate strength of jointed rock mass three properties of the rock mass are needed; uniaxial compressive strength (UCS) of intact rock, Hoek-Brown constant (mi) for intact rock and GSI for rock mass (Hoek-Brown, 1997). In turn Hoek-Brown failure criteria use RMR to determine material constants. Shear strength of foundation rock materials in the present study were determined as per Hoek-Brown failure criterion (eq.6.8);

$$\sigma'_1 = \sigma'_3 + \sqrt{m\sigma_c\sigma'_3 + s\sigma_c^2} \text{……….eq.4.8}$$

Where; $\sigma'_1$is the major principal effective stress at failure,$\sigma'_3$is the minor principal effective stress or confining pressure, m and s are the material constants, and σ_c is the uniaxial compressive strength of the intact rock and material constants were determined from RMR value for undisturbed case:

m_b=mi*exp$^{(RMR-100)}$/28

Where; mi = 17 for basalt from literature for this study and

s = exp*$^{(RMR-100)}$/9 (Hoek and Brown, 2002)

The results are presented and discussed in Chapter-5.

Rock mass allowable bearing pressure

Design of a foundation depends on the engineering performance of subsurface strata and its bearing capacity. For a foundation resting on rock the bearing capacity can be estimated from existing rock mass classifications. In the present study allowable/safe bearing capacity of rock mass foundation was calculated empirically based on the Bowles (1996) relation (eq.4.9). A factor of safety 3 was used for this purpose (Hoek and Bray, 1977).

*Safe bearing capacity (SBC) =UCS*RQD/factor of safety* ….eq.4.9

Where; UCS is the unconfined compressive strength, RQD is the rock quality designation and RMR is the rock mass rating.

The Net allowable bearing pressure (Qa) based on RMR value as proposed by Singh and Goel (2011) is presented in Table 4.6.

Table 4.6 Net allowable bearing pressure (Qa) based on RMR value (Singh and Goel, 2011)

Class No.	I	II	III	IV	V
Description	Very good	Good	Fair	Poor	Very poor
RMR	81-100	61-80	41-60	21-40	<20
Qa (t/m^2)	600-440	440-280	280-135	135-45	45-30

Foundation assessment through permeability analysis

The permeability of the foundation rocks will directly dictate the design of cutoffs beneath the dam in permeable foundations (Azimian et al., 2012). For the present study the foundation permeability data was interpreted and accordingly hydraulic domain classification was suggested (Lashkaripour and Ghafoori, 2002 as cited in Daniel Gebremichael, 2017). Data pertaining to permeability in the present study was procured from the detailed final design report for Megech dam (WWDSE, 2010a).The permeability of the dam foundation was assessed as per the following criteria, proposed by Houlsby (1976):

- Impervious if Lugeon value falls between 0-3
- Low permeability if Lugeon value fall between 3-10
- Medium permeability if Lugeon value falls btween10-30
- High permeability if Lugeon value falls between 30-60
- Very high permeability if Lugeon value is greater than 60

The interpretation of representative Lugeon value upon five lugeon values under variable pressures a, b, c, b, a in a given test section can be interpreted based on the Lugeon flow pattern, as suggested by Houlsby (1976), and presented in Fig. 4.2.

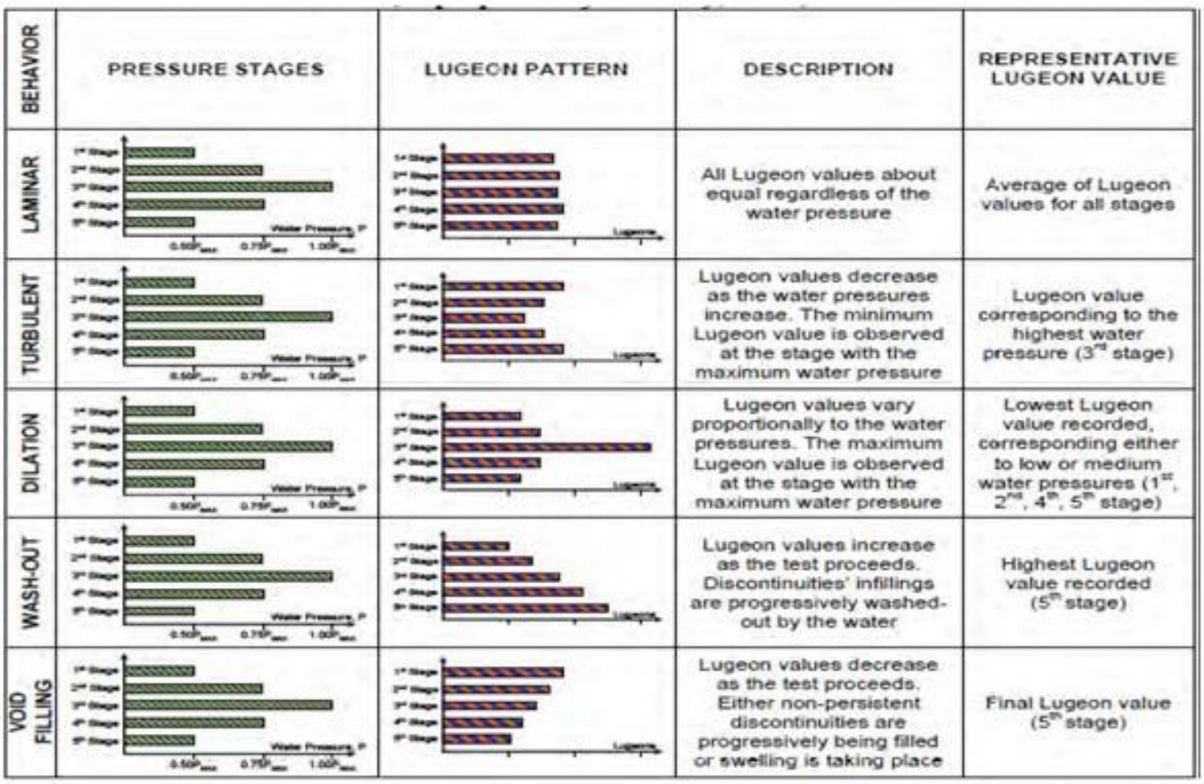

Fig: 4. 2 Lugeon pattern interpretation chart (Houlsby, 1976)

Foundation assessment through Hydro chemical analysis for concrete structures

The strength and durability of bored cast-is-situ piles largely depends on the quality of ground water. The ground water that may have extremely high or low PH, chloride ions, sulphate or organic materials may affect the piles (Ganpule and Mhaiskar, 2008). The materials that could undergo degradation due to the above mentioned agents include concrete and reinforcing steel in concrete structures. Evaluation of groundwater aggressiveness is an important element for geological, hydro-geological and engineering investigations of civil structures including dams (Larisch, 2020). For the present study (3) ground water and surface water hydro chemical analysis data (Sileshi, 2015) was procured to assess the corrosivity ratio coefficient (CR)

The Corrosivity coefficient (CR) as per Mahadevaswamy (2002) is given by eq. 4.10;

$CR=(0.028CL+0.021SO_4)/0.02\ (HCO_3+CO_3)$……………………eq.4.10

Where; CR is Corrosivity coefficient, CL is chlorine ion, SO_4 is sulfate ion, HCO_3 is hydrogen carbonate ion and CO_3 is carbonate ion.

The Corrosivity ratio was computed for all the samples and the results are presented in Chapter 5 -along with detailed discussion.

Assessment of Bearing Capacity and settlement for piles

Bearing capacity failures of structures founded on rock mass is dependent on joints and many attempts have been made to correlate the end bearing capacity with unconfined compressive strength of intact rock (Zhang, 2010). It is thought that rock socketed piles are supported by both the intact rock and associated discontinuities (rock mass) and thus in determining end bearing capacity, using rock mass strength instead of intact rock strength is advised incorporated through RQD determination. The empirical equations developed so far to determine end bearing from UCS were based on pile load test for typical case studies (Zhang, 2010; Sultana et al., 2016; Haberfield and Lochaden, 2019). Megech dam were designed with appurtenant structures resting on pile foundation with Piles are cast in situ type and reinforced concrete in composition (Table 4.7), located in the right abutment of the dam and end bearing type.

Table 4.7 Design features of piles under the intake and conduit structure for Megech dam project (Source: WWDSE,2010)

Pile type	Pile-length (m)	Pile-diameter(cm)	Test type	GW condition	Spacing
Cast in situ	38-40.74m	80	Static load	Submerged	3m

The total number of piles under the intake structure was 64 and 152 along the conduit outlet. These Piles will be submerged in water with an average depth of 15m. The main tests conducted to estimate bearing capacity of piles include unconfined compression test for rocks and Design and working pile load tests. For bearing capacity estimation only end bearing of foundation was considered by leaving skin friction and a suitable factor of safety was considered. The main challenge in constructing pile foundation under concerned structures was deflection and concrete strength failure leading to pile replacement (WWDSE, 2010a). Bearing capacity and settlement potential of pile foundation in the present study was estimated from;

- UCS Test on intact rock cores
- RQD along borehole sections
- Pile load test results: including Working pile load test result and Preliminary or design pile load test result and having these data the piles under the intake and conduit outlet work structures were assessed in terms of bearing capacity and settlement;

The ultimate end bearing capacity was computed by using eq.6.11 (Zhang, 2010; Haberfield and Lochaden, 2019);

$Q_{max} = 3(\sigma_{cm})^{0.5}$.....................................eq.4.11

Fmax=0.2$(\sigma_{cm})^{0.5}$…………………………...eq4.12

Where; $Q_{\max}$ is the ultimate end bearing capacity and σ_{cm} is the uniaxial compressive strength of rock mass and F-max is shaft bearing capacity

The uniaxial compressive strength of rock mass was determined by using RQD as a reduction factor to UCS and is given byeq.4.13

$\boldsymbol{\sigma}\mathrm{cm} = \boldsymbol{\sigma}\mathbf{c} * 10^{(\mathbf{0.013RQD-1.34})}$(Mpa) …. eq4.13

Ultimate bearing capacity (Qu) is then given by:

Qu (Mpa) =$\pi B \sum \mathrm{Li}(\,\mathrm{f(max)}i) + \frac{\pi \mathrm{B}^2}{4} * \mathrm{q(max)}$ ……………….eq4.14

Where B is pile diameter, i is number of layers and (Li) pile socket length

Besides the bearing capacity and settlement properties were determined by using Load settlement curve generated from pile load test results conducted in 2019 and 2020 by the project authorities. ******

CHAPTER –FIVE

RESULTS AND DISCUSSION

5.1 Rock mass classification at Megech dam foundation from scan line data

The present research was initiated to characterize the rock mass at Megech dam foundation with respect to relevant engineering properties. Engineering properties addressed directly or indirectly in this study to evaluate the dam foundation includes; strength, deformability, permeability, bearing capacity and settlement potential of the rock mass. These properties were assessed from geotechnical and engineering geological perspectives. Deformability, bearing capacity and strength parameters of foundation rock mass were determined indirectly from rock mass classification data (RMR). In line with this rock mass classification parameters were collected at three critical sections of the dam which were; right and left abutments and along the river section. Rocks at the dam foundation were classified from the following parameters perceived from the field investigation:

Intact rock strength

The uniaxial compressive strength (UCS) was determined from Schmidt hammer rebound value, measured in the field by using empirical equation (4.2): $UCS=10^{0.00088}\gamma^{R+1.01}$ (Barton and Choubry, 1977). The UCS values as estimated for the rocks exposed at Megech dam foundation ranges from 22.1 to 166 MPa. Where the lowest UCS value was recorded for the rocks exposed on the left abutment and the highest value was recorded for the rocks exposed on the right abutment, respectively. The strength of the rock based on the UCS values in general falls in the range of weak to very strong rocks as per Bieniwski's (1989) classification for intact rock strength. Based on the UCS values it can be said that on an average the intact rock strength of the rocks exposed on the right abutment is much better than the rocks exposed on the left abutment. The variation in UCS values of the rocks is possibly due to high degree of weathering and high joint concentrations in the rock mass exposed on the left abutment.

Rock quality designation (RQD)

RQD is simple and inexpensive index used in rock mass classification systems (Barzegari, 2017). For the present study Palmstorm's (1982) volumetric Count Method was used to determine the rock quality designation (eq. 4.3): $RQD = 115-3.3J_V$). The results show that

RQD in general varies from 9.2 to 98.5%, indicating rock mass to be of very poor to excellent quality (Barton et al., 1974). The lowest RQD (9.2%) was found in the rock mass on left abutment within highly weathered aphanitic basalt while relatively high quality rock mass (RQD 98.5%) was found in the right abutment. Poor quality rock mass was observed on the left abutment due to high relative degree of weathering and intersected joints. In general, rock mass at Megech dam foundation falls in the range of very poor (9.2%) to excellent (98.5%) quality as per the classification proposed by Bieniwski, 1989).

Discontinuity conditions and spacing

Discontinuities within a rock mass greatly affect rock mass properties and the response of rock mas to geotechnical operations (Chen et al., 2010 and Barzegari, 2017). The geotechnical parameter that is largely affected by the presence of discontinuities is the permeability of the rock mass (Barani et al., 2014). During the present study major characteristics of discontinuity such as; orientation, spacing, filling, persistence and roughness were determined at each RMR data collection points. Variable discontinuity conditions were observed at Megech dam foundations where persistent with no infill joints was observed in the right abutment whereas highly disintegrated crushed rock mass was found in the left abutment. Further, with regard to spacing, joints spacing in general varies from less than 10mm to greater than a meter as observed on both the abutments. The discontinuity orientation data was later analyzed by using dips computer program to work out the preferred orientations of the discontinuity planes (Fig2.8).

Ground water condition

From visual observation during field survey, except at two data points in the left abutment typically springs, no groundwater surface traces were observed. Thus, in general dry conditions were observed in the rock mass at dam foundation. These conditions were observed in the surface exposure of the rock mass however, shallow groundwater was reported in the borehole logs.

Discontinuity orientation

The structural features and discontinuities within the rock mass can have an important effect on the rock mass properties and its behavior in geotechnical operations (Ajalloeian and Azmin, 2013). The orientation of critical joints was favorable with respect to the dam axis as per description on effect of joints for engineering structures (Singh, 2011).

Table5.1 effect of joint orientation on stability of dam foundation (Singh, 2011)

Dip(0-10)°	Dip (10-30)°		Dip (30-60)°	Dip(60-90)°
	Dip direction			
	Upstream	Downstream		
Very favorable	Unfavorable	Fair	Favorable	Very unfavorable
Assessment of joint orientation for this study				
No data set	No data set	RA-11&12	all except,11&12	No data set

Based on RMR the rock mass at Megech dam foundation was classified based on total rating obtained from individual ratings for parameters above and the results are presented in Table 5.2. In total RMR data was collected from 62 locations at the dam foundation. Out of these 53 data points were on rock exposures at right abutment, 8 locations on left abutment and one data point in river section. Further, the rock mass classification results shows that the volcanic succession at left abutment falls in the range of poor to good, fair to good at right abutment and good for river bed rocks. The least RMR value of 37 was found for the rock mass on left abutment which falls in Group – IV rock mass whereas the maximum RMR value of 72 was found for the rock mass on the right abutment which falls in Group – II rock mass as per Bieniwski (1989) classification.

In general, rock mass exposed on left abutment has RMR that range from 37 to 58 and it can be placed in Fair to poor quality rock mass. In contrast RMR values for rock mass on right abutment vary from 45 to 72 and it can be placed in Fair to Good quality rock mass. Thus, it implies that the rock mass at dam foundation needs attention, particularly on left abutment to enhance its quality for safe engineering performance.

5.1.1 Deformability Estimation for foundation rock mass from RMR

Megech dam foundation rests on successions of volcanic rocks with different degree of weathering and jointing. Estimating deformability modulus (E_d) is of prime importance for rock foundation (Bieniwski, 1989) as many engineering operations including numerical modeling require input design parameters one of which is deformation modulus (Cai et al., 2004). In the present study rock mass rating (RMR) presented in Table 5.1 was used to estimate the deformability of foundation rock mass and the results are presented in Table 5.3. For this purpose empirical relation proposed by Serafim and Perera (1983) was used;

$E_d = 10^{(RMR-10/40)}$ ....Eq (4.7)

Based on assessment results, presented in Table 7.2, modulus of deformability for Megech dam foundation ranges from 7.5 to 34.49 GPa for rock mass on right abutment, 4.73 to 15.85 GPa for rock mass on left abutment and 25 GPa for rock mass in the river bed.

Table 5.2 Engineering rock mass classification results for Megech dam foundation

Data Location	Rock mass classification parameters										RMR-89	Rock mass class	Class-description
	UCS (Mpa)			RQD									
	SHV	UCS	Ra	JV	RQD	Ra	Spa	Con	GWC	Ori.			
Location - Right abutment													
RA-1	30	47	4	10	82	17	10	18	15	-2	62	II	Good
RA-2	28	42	4	8	88.6	17	10	16	15	-2	60	III	Fair
RA-3	20	23	2	20	50	13	10	18	15	-2	57	III	fair
RA-4	34	57.3	7	30	46	8	8	20	15	-2	56	III	Fair
RA-5	28	42.3	4	18	55.6	13	10	16	15	-2	56	III	Fair
RA-6	24	34.5	4	12	75.4	17	10	16	15	-2	60	III	Fair
RA-7	24	34.52	4	15	65.5	13	8	20	15	-2	58	III	Fair
RA-8	38	47.73	4	5	98.5	20	10	12	15	-2	59	III	Fair
AR-9	32	51.76	7	21	46	8	5	12	15	-2	45	III	Fair
RA-10	24	27	4	22	42.4	8	5	21	15	-2	51	III	Fair
RA-11	28	32	4	20	50	13	10	16	15	-7	51	III	Fair
RA-12	23	26	4	14	69	13	10	19	15	-7	54	III	Fair
RA-13	25	36.3	4	20	50	13	10	18	15	-2	58	III	Fair
RA-14	22	25	4	5	98.5	20	10	19	15	-2	66	II	Good
RA-15	23	28.2	4	10	82	17	10	18	15	-2	62	II	Good
RA-16	28	32	4	15	65.5	13	15	18	15	-2	63	II	Good
RA-17	32	37.4	4	14	68.8	13	10	17	15	-2	57	III	Fair
RA-18	24	27	4	15	65.5	13	15	24	15	-2	69	II	Good
RA-19	28	42.3	4	14	68.8	13	10	19	15	-2	59	III	Fair
RA-20	20	28.2	4	20	50	13	10	16	15	-2	56	III	Fair
RA-21	50	129	12	16	62.2	13	15	19	15	-2	72	II	Good
RA-22	26	38.2	4	16	62.2	13	15	18	15	-2	63	II	Good
RA-23	32	33.26	4	15	65.5	13	10	20	15	-2	60	III	Good
RA-24	44	61	7	24	35.8	8	10	19	15	-2	57	III	Fair
RA-25	34	40.6	4	18	55.6	13	10	21	15	-2	61	II	Good
RA-26	55	166	12	14	68.8	13	10	18	15	-2	66	II	Good
RA-27	36	63.4	7	19	52.3	13	10	22	15	-2	62	II	Good
RA-28	52	142.6	12	18	55.6	13	10	19	15	-2	67	II	Good
RA-29	34	40.6	4	22	42.4	8	8	24	15	-2	57	III	Fair
RA-30	43	90.4	7	11	78.7	17	10	22	15	-2	69	II	Good
RA-31	43	90.4	7	12	75.4	17	10	21	15	-2	68	II	Good
RA-32	52	142.6	12	10	82	17	10	21	15	-2	68	II	Good
RA-33	35	60.25	7	24	35.8	8	5	21	15	-2	54	III	Fair
RA-34	21	24	4	10	82	17	15	15	15	-2	64	II	Good
RA-35	40	51.76	7	16	62.2	13	10	20	15	-2	63	II	Good
RA-36	22	25	4	22	42.4	8	8	18	15	-2	51	III	Fair
RA-37	55	166	12	18	55.6	13	10	20	15	-2	68	II	Good
RA-38	29	44.46	4	18	55.6	13	10	19	15	-2	59	III	Fair
RA-39	30	46.77	4	14	68.8	13	10	21	15	-2	61	II	Fair
RA-40	25	29.34	4	25	32.5	8	5	19	15	-2	49	III	Fair
RA-41	35	42.3	4	14	68.8	13	10	17	15	-2	57	III	Fair
RA-42	42	86	7	12	75.8	17	10	19	15	-2	66	II	Good
RA-43	52	142.6	12	14	68.8	13	10	23	15	-2	71	II	Good
RA-44	27	30.56	4	15	65.5	13	8	20	15	-2	58	III	Fair
RA-45	34	80.8	7	22	42.4	8	10	19	15	-2	57	III	Fair
RA-46	40	77.6	7	18	55.6	13	8	20	15	-2	61	II	Good
RA-47	26	29.4	4	16	62.2	13	8	24	15	-2	62	II	Good
RA-48	36	44	4	12	75.8	17	10	18	15	-2	62	II	Good
RA-49	31	49.2	7	24	35.8	8	10	20	15	-2	58	III	Fair
RA-50	44	95	12	9	85.3	17	10	16	15	-2	68	II	Good
RA-51	39	73.8	7	24	35.8	8	10	16	15	-2	54	III	Fair
RA-52	24	34.5	4	21	47.5	8	10	14	15	-2	49	III	Fair
RA-53	40	77.62	7	30	16	3	8	16	15	-2	47	III	fair
RB-1	46	105.2	12	14	68.8	13	8	20	15	-2	66	II	Good
Location - Left abutment													
LA-1	25	23	2	15	65.5	13	10	20	15	-2	58	III	Fair
LA-2	26	28.2	4	32	9.2	3	10	11	15	-2	41	III	Fair
LA-3	19	22.1	2	30	16	3	5	14	15	-2	37	IV	Poor
LA-4	20	23	2	25	32.5	8	8	18	15	-2	49	III	Fair
LA-5	26	29.4	4	25	32.5	8	8	20	15	-2	53	III	Fair
LA-6	29	33.14	4	27	25.9	8	8	20	0	-2	38	IV	Poor
LA-7	24	27.1	4	10	82	17	5	18	7	-2	49	III	Fair
LA-8	19	26.8	4	18	55.6	13	8	18	7	-2	48	III	Fair

UCS– Uniaxial compressive strength, **SHV**– Schmidt hammer Value**, Ra**– Rating, **Jv**- Volumetric count, **RQD**– Rock quality designation, **Sp.**– Spacing of discontinuity, **Con.**– Condition of discontinuity, **GWC**– ground water condition, **Ori**– Orientation of discontinuity, **RMR** – Rock mass rating

Table 5.3 Modulus of deformation (Ed) and shear strength parameters estimated from RMR

Data Location	Elevation	RMR	Shear strength parameters (Bieniwski,1989)				E_d(GPa)
			C (KPa) (Range)	Φ(deg) (Range)	C*	Φ*	
Location: right abutment							
RA-1	1877	62	300-400	35-45	3.1	36	19.95
RA-2	1875	60	200-300	25-35	3	35	17.78
RA-3	1879	57	200-300	25-35	2.85	33.5	14.96
RA-4	1882	56	200-300	25-35	2.8	33	14.1
RA-5	1894	56	,, ,,	,, ,,	2.8	33	14.1
RA-6	1897	60	,, ,,	,, ,,	3	35	17.78
RA-7	1899	58	200-300	25-35	2.9	34	15.85
RA-8	1900	59	200-300	25-35	2.95	34.5	16.79
RA-9	1902	45	,,	,, ,,	2.25	27.5	7.5
RA-10	1897	51	,,	,, ,,	2.55	30.5	10.6
RA-11	1888	51	,, ,,	,, ,,	2.55	30.5	10.6
RA-12	1893	54	,,	,, ,,	2.7	32	12.58
RA-13	1884	58	,,	,, ,,	2.9	34	15.85
RA-14	1890	66	300-400	35-45	3.3	38	25.12
RA-15	1890	62	300-400	35-45	3.1	36	19.95
RA-16	1895	63	300-400	35-45	3.15	36.5	21.13
RA-17	1898	57	200-300	25-35	2.85	33.5	14.96
RA-18	1905	69	300-400	35-45	3.45	39.5	29.85
RA-19	1906	59	200-300	25-35	2.95	34.5	16.79
RA-20	1927	56	200-300	25-35	2.8	33	14.1
RA-21	1944	72	300-400	35-45	3.6	41	34.48
RA-22	1940	63	300-400	35-45	3.15	36.5	21.13
RA-23	1950	60	200-300	25-35	3	35	17.78
RA-24	1931	57	200-300	25-35	2.85	33.5	14.96
RA-25	1945	61	300-400	35-45	3.05	35.5	18.83
RA-26	1950	66	300-400	35-45	3.3	38	25.12
RA-27	1962	62	300-400	35-45	3.1	336	19.95
RA-28	1958	67	300-400	35-45	3.35	38.5	26.6
RA-29	1952	57	200-300	25-35	2.85	33.5	14.96
RA-30	2011	69	300-400	35-45	3.45	39.5	29.85
RA-31	2010	68	300-400	35-45	3.4	39	28.2
RA-32	1932	68	300-400	35-45	3.4	39	28.2
RA-33	1918	54	200-300	25-35	2.7	32	12.58
RA-34	1930	64	300-400	35-45	3.2	37	22.38
RA-35	1932	63	300-400	35-45	3.15	36.5	21.13
RA-36	1929	51	200-300	25-35	2.55	30.5	10.6
RA-37	1937	68	300-400	35-45	3.4	39	28.2
RA-38	1917	59	200-300	25-35	2.95	34.5	16.79
RA-39	1912	61	300-400	35-45	3.05	35.5	18.83
RA-40	1926	49	200-300	25-35	2.45	29.5	9.44
RA-41	1948	57	200-300	25-35	2.85	33.5	14.96
RA-42	1953	66	300-400	35-45	3.3	38	25.12
RA-43	1954	71	,, ,,	,, ,,	3.55	40.5	33.49
RA-44	1957	58	200-300	25-35	2.9	34	15.85
RA-45	1976	57	200-300	25-35	2.85	33.5	14.96
RA-46	1959	61	300-400	35-35	3.05	35.5	18.83
RA-47	1969	62	,, ,,	,, ,,	3.1	36	19.95
RA-48	1970	62	,,	,, ,,	3.1	36	19.95
RA-49	1974	58	200-300	25-35	2.9	34	15.85
RA-50	1954	68	300-400	35-35	3.4	39	28.2
RA-51	1868	54	200-300	25-35	2.7	32	12.58
RA-52	1905	49	200-300	25-35	2.45	29.5	9.44
RA-53	1991	47	200-300	25-35	2.35	28.5	8.41
RB	1877	66	300-400	35-45	3.3	38	25.12
Location: left abutment							
LA-1	1945	58	200-300	25-35	2.9	34	15.85
LA-2	1940	41	200-300	25-35	2.05	25.5	5.96
LA-3	1956	37	,, ,,	,, ,,	1.85	23.5	4.73
LA-4	1950	49	, ,,	,, ,,	2.45	29.5	9.44
LA-5	1910	53	,, ,,	,, ,,	2.65	31.5	6.68
LA-6	1911	38	,, ,,	,, ,,	1.9	24	5.01
LA-7	1877	49	,, ,,	,, ,,	2.45	29.5	9.44
LA-8	1909	48	,, ,,	,, ,,	2.4	29	8.91

5.1.2 Rock mass Strength Estimation from RMR

Shear strength parameters of rock mass can be determined indirectly using empirical equations from rock mass classification systems, typically Rock Mass rating system (RMR) proposed by (Bieniawski, 1989). For the present study the shear strength parameters were estimated at different location along the left and right abutments from RMR values. The Shear strength parameters of rock mass determined from RMR for various locations and the corresponding range in which the cohesion and angle of friction of the rock mass will fall as per Bieniawski (1989) classification are presented in Table 5.3. Specific values of RMR were used to determine specific values of cohesion and angle of friction (Table 5.3), as per following relations.

C* = 0.05 RMR ………. ……………………………..eq(5.1)

Φ* = 0.5RMR + 5 ………. ……………………………eq (5.2)

Where; C* and Φ* are the specific value of cohesion and angle of friction Table5.3

Perusal of results (Table 5.3) clearly shows that the cohesion (C) for the rock mass on left abutment fall in the range 200 to 300 KPa whereas the specific value for C varies from 1.85 to 2.9 KPa. Similarly, the cohesion (C) for the rock mass on right abutment fall in the range 200 to 400 KPa whereas the specific value for C varies from 2.25 to 3.6 KPa. Further, the angle of friction (φ) for the rock mass on left abutment fall in the range 25 to 35^{o} whereas the specific value for φ varies from 23.5 to 34^{o}. The angle of friction (φ) for the rock mass on right abutment falls in the range 25 to 45^{o} where the specific value for φ varies from 27.5 to 41^{o}.

5.1.3 Rock mass failure criteria (Hoek-Brown)

Many geotechnical problems are more conveniently dealt with in terms of shear and normal stresses rather than the principal stress relationships of the original Hoek-Brown criterion, defined by;

$\sigma 1 = \sigma 3 + \sigma c\left(\frac{m\sigma 3}{\sigma c} + s\right) a$ ……………………eq (4.8.)

For the present study Hoek and Brown (2002) failure criteria was used to empirically estimate the rock mass strength at the dam foundation: RMR = 0.827GSI-15.398 eq (5.3) (Zhang et al., 2019) in which GSI = 1.21 RMR-18.614 from this relation and mi = 17 for basalt (Hoek-Brown, 1997) was considered to calculate other rock mass parameters (Table5.4). After addressing all the important rock mass parameters from the above relation the effective major principal stress and associated shear strength parameters were computed and are given in Table 5.4.

The average major principal effective stress on Left abutment is 19.32 MPa and 35.83 MPa on right abutment. The rock mass characteristics of dam foundation presented in Table 7.3 were determined by using Rock Lab software and the computational sheet in MS Excel developed by Raghuvanshi (2017). The GSI was obtained from RMR (Bienwski, 1989) presented in Table 5.2. The average UCS, RMR and m_i values were used to plot major and minor principal stress relationships. The estimated cohesion (C) from Hoek-Brown failure criteria for the rock mass at dam foundation varies from 0.82 to 11.85 MPa with an average value of 3.48 MPa (Table 5.4). Similarly, the estimated angle of friction (φ) for the rock mass at dam foundation varies from 21.47 to 40.6^o with an average value of 34.67^o.

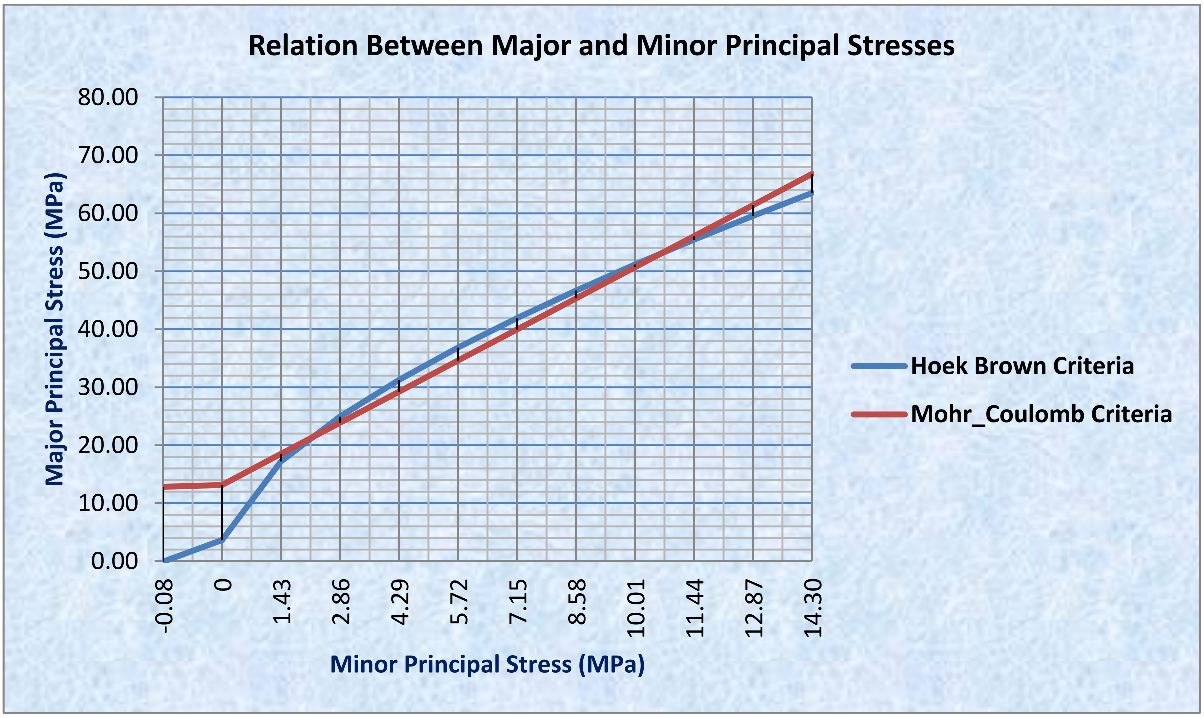

Fig. 5.1 Relationship between minor and major effective stress obtained from average RMR, UCS and mi values showing both Hoek Brown and Mohr Coulomb criteria's for the dam foundation

Table 5.4 Major effective principal stress as determined from Hoek and Brown (1997) criteria for Effective minor principal stress considered within 0<= σ3≤0.5σc

RMR	UCS (Mpa)	Material constants			Principal -stresses		σcm (mpa)	GSI	Shear strength parameters	
		s	m_b	a	$\sigma 3$	$\sigma 1$			C(mpa)	φ(deg)
Location - Right abutment										
62	47	0.009	3.74	0.5	5	35	5.788	57.6	3.06	37.37
60	42	0.006	3.23	0.5	5	31.48	4.557	54	2.6	36.3
57	23	0.004	2.89	0.5	5	23.4	3.5	50.4	1.352	35.2
56	57.3	0.004	2.75	0.5	5	32.7	3.255	49	3.3	34.78
56	42.3	.0035	2.75	0.5	5	30.33	2.4	49	2.44	34.78
60	34.5	0.004	3.29	0.5	5	29.06	2.6	53.98	2.13	36.3

58	34.5	0.004	3	0.5	5	27.87	2.43	51.6	2.06	35.57
59	47.7	0.005	3.14	0.5	5	32.8	3.37	52.77	2.9	35.9
45	51.8	0.008	1.71	0.5	5	26.5	1.3	35.8	2.43	30.78
51	27	0.002	2.2	0.51	5	22.3	1.07	43	1.425	32.9
51	32	0.0018	2.22	0.5	5	24.87	1.272	43	1.689	32.97
54	26	0.0028	2.561	0.5	5	24.34	1.313	47	1.456	34.2
58	36.3	0.004	3	0.5	5	28.63	2.4	51.6	2.17	35.57
66	25	0.013	4.25	0.5	5	28.2	2.86	61.2	1.72	38.44
62	28.2	0.009	3.74	0.5	5	28.11	2.62	57.6	1.84	37.37
63	32	0.009	3.74	0.5	5	26.65	2.99	57.6	2.08	37.37
57	37.4	0.004	2.89	0.50	5	28.55	2.3	50.4	2.2	35.2
69	27	0.020	4.85	0.5	5	30.87	3.8	64.88	1.966	39.53
59	42.3	0.005	3.14	0.5	5	30.94	2.992	52.77	2.56	35.92
56	28.2	0.004	2.75	0.50	5	24.76	1.6	49	1.6	34.78
72	129	0.03	5.52	0.5	5	68.71	22.3	68.5	9.96	40.6
63	38.2	0.009	3.74	0.5	5	25.29	3.566	57.6	2.49	37.37
60	33.2	0.008	3.29	0.5	5	28.58	2.52	53.98	2.06	36.28
57	61	0.004	2.89	0.50	5	34.94	3.76	50.4	3.59	35.2
61	40.6	0.007	3.43	0.50	5	31.6	3.3	55.2	2.56	36.65
66	166	0.013	4.25	0.5	5	64.65	19	61.2	11.4	38.44
86	63.4	0.009	3.74	0.5	5	39.95	5.92	57.6	4.136	37.37
67	143	0.015	4.44	0.5	5	58.964	17.48	62.4	9.99	38.8
57	40.6	0.004	2.89	0.50	5	29.357	2.5	50.4	2.39	35.2
69	90.4	0.020	4.85	0.5	5	53.535	12.75	64.88	6.58	39.53
68	90.4	0.017	4.64	0.5	5	52.24	11.9	63.66	6.46	39.17
68	143	0.017	4.64	0.5	5	61.312	18.77	63.66	10.188	39.17
54	60.2	0.003	2.53	0.50	5	32.8	2.988	46.7	3.36	34
64	24	0.01	3.9	0.50	5	26.766	2.399	58.8	1.59	37.73
63	51.8	0.009	3.74	0.50	5	36.5	4.83	57.6	3.376	37.37
51	25	0.002	2.22	0.50	5	21.7	0.994	43	1.32	32.97
68	166	0.017	4.63	0.5	5	70.766	21.778	63.6	11.846	39.16
59	44.5	0.005	3.14	0.5	5	31.606	3.14	52.77	2.7	35.92
61	46.8	0.007	3.43	0.5	5	28.51	3.8	55.2	2.946	36.65
49	29.3	0.001	2.04	0.51	5	22.33	1.007	40.6	1.49	32.24
57	42.3	0.004	2.89	0.50	5	29.87	2.6	50.4	2.487	35.2
66	86	0.013	4.25	0.50	5	48.89	9.85	61.2	5.92	38.44
71	143	0.026	5.29	0.5	5	70.64	23	67.3	10.79	40.24
58	30.6	0.005	3.02	0.50	5	20.24	2	51.6	1.828	35.57
57	80.8	0.004	2.89	0.50	5	39.55	4.98	50.4	4.75	35.2
61	77.6	0.007	3.43	0.50	5	42	6.3	55.2	4.889	36.65
62	29.4	0.009	3.74	0.5	5	28.88	2.744	57.6	1.91	37.37
62	44	0.009	3.74	0.5	5	33.98	4.1	57.6	2.87	37.37
58	49.2	0.004	3	0.05	5	32.37	3.25	51.6	2.94	35.57
68	95	0.017	4.63	0.5	5	53.55	12.46	63.6	6.779	39.16
54	73.8	0.002	2.53	0.5	5	35.8	3.66	46.7	4.11	34.09
49	34.5	0.001	2.04	0.51	5	23.8	1.18	40.6	1.755	32.24
47	77.6	0.001	1.87	0.51	5	32	2.3	38.26	3.8	31.53
66	105	0.013	4.25	0.5	5	53.824	12	61.2	7.24	38.44
23	58	0.004	0.66	0.59	5	19.35	0.15	9.2	1.35	21.92
28.2	41	0.0001	0.83	0.559	5	18.05	0.215	15.5	1.22	24.28
22.1	37	0.000036	0.638		5	15.866	0.083	8.1	0.82	21.47
23	49	0.00004	0.664	0.59	5	17.76	0.127	9.2	1.14	21.9
29.4	53	0.0001	0.87	0.59	5	23.312	0.322	17	1.65	24.81
33.14	38	0.0002	1.03	0.54	5	19.21	0.344	21.5	1.34	26.31
27.1	49	0.0001	0.794	0.564	5	19.613	0.228	14.2	1.393	23.82
26.8	48	0.0001	0.782	0.566	5	19	0.212	13.8	1.346	23.68
56	57.2				**Avg.**			**48.9**	**3.48**	**34.67**

Further, from Hoek Brown failure criteria by using average UCS and GSI values and mi equal to 17 (for basalt) it has been found that the Mohr Coulomb Fit shows cohesion (C) and angle of friction (φ) for the foundation rocks to be 3.292 MPa and 34.75^{o}, respectively (Fig.5.2). The average uniaxial compressive strength for the rock mass at foundation is 3.23 MPa. Similarly, average modulus of deformation for the rock mass at dam foundation comes out to be 7.09 GPa.

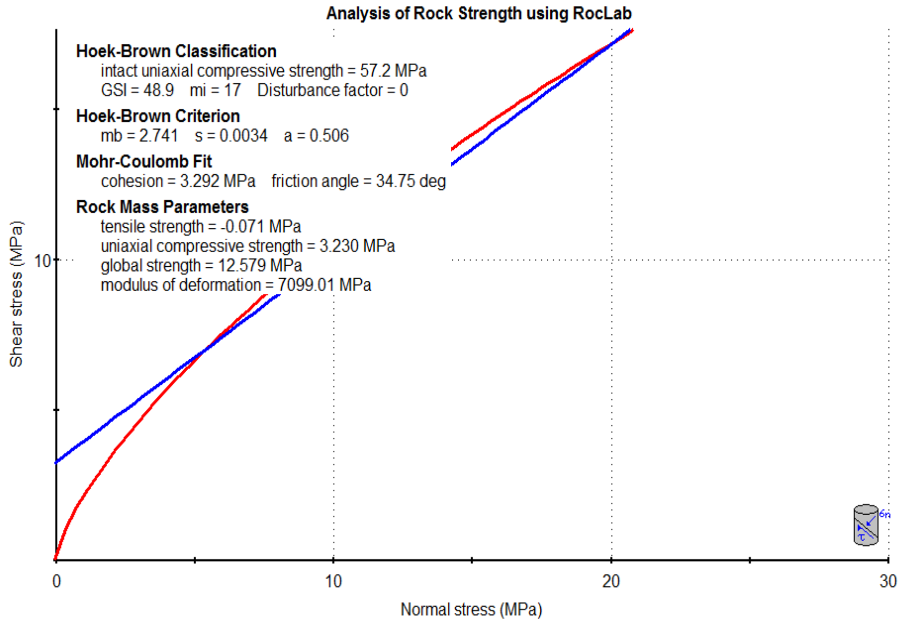

Fig.5.2 Relationship between normal and shear stress with Mohr-Coulomb envelop using average UCS and GSI values for dam foundation for all RMR data points and mi =17 for basalt

5.2 Soil characteristics from primary and secondary data

5.2.1 Soil characteristics from secondary data source

In terms of soil characteristics Megech dam foundation was addressed in to three critical sections (WWDSE, 2010). The left abutment soil was characterised as Illuvial / Talus overburden material. These overburden material includes highly plastic silty clay to clay type soil deposits with an average thickness of 5m. Perceived from the SPT test data conducted at depths of 1.9m, 3.85m and 6.5m in NB-1 borehole with "N" values of 15, 12 and 36, respectively the soils along this section range from stiff to very hard in consistency according to AASHTO (1988) consistency classification based on SPT test. The central part of the dam foundation in turn was characterised by river deposit consisting of alluvial soil and colluvial overburden materials which is very loose - medium to coarse gravel. Fine gravel and loose to firm silty clay to clay soil of intermediate plasticity is also found to certain extent within the river deposits (WWDSE, 2010). From engineering point of view this river deposit was found to be weak and loose. With regard to water tightness the very loose gravel and sand deposits found exposed at Megech river course and on the banks is unconsolidated / un-cemented material which possesses high permeability (WWDSE, 2007). SPT tests were also conducted in this unit in borehole NB3 at depths of 8.6m, 11m and 13.9m resulting "N" Values of 27, 32 and 40, respectively (Table 5.5). These ‘N’ values suggest that the material is very stiff to hard. Likewise, SPT tests conducted in borehole NB5 at depth of 2.5m, 6.4m, 9m and 13.20m have resulted "N" values of Refusal, 12, 66 and 43, respectively. Thus, these 'N' values depict the soil to be hard. Further, SPT tests conducted in borehole NB6, at depth of 3.45m has resulted "N" value to be 80 which indicate Very Hard material.The right abutment was also characterised by similar type of materials as that of left abutment. The engineering geologic materials found on right abutment are colluvial / Talus overburden with very thin, less than a meter in thickness (WWDSE, 2007).

Table 5.5 Soil characteristics based on SPT value at Megech dam foundation (Source : WWDSE,2007)

BH-ID	Test No.	BH-Depth	SPT values				Soil type
			N (each section)			N average	
			0-15cm	15-30cm	30-45cm		
NB-1	1	1.9	3	6	9	15	Stiff, silty-clay
	2	3.85	3	5	7	12	Stiff, silty clay
	3	6.5	10	14	22	36	Decomposed porphyritic basalt
NB-2	1	1.5	4	7	10	17	Highly plastic, stiff silty clay
	2	3	3	6	12	18	Highly plastic, stiff silty clay
	3	4.5	3	5	>50	Refusal	Plastic, very hard silty clay
	4	16.8	5	20	30	Refusal	Decomposed aphanatic basalt
NB-3	1	8.6	5	10	17	27	Very stiff, silty sand
	2	11	6	17	15	32	Decomposed aphanatic basalt
	3	13.9	11	21	19	40	Very hard, silty gravel
NB-5	1	2.5	21	Bounced	Bounced	Refusal	Hard, basaltic gravel
	2	6.4	7	6	6	12	Firm, gravelly silty clay
	3	9	12	21	45	66	Stiff, highly weathered basalt
	4	13.2	11	19	24	43	Completely weathered basalt
NB-6	1	1	3	5	10	15	Stiff, silty clay
	2	3.1	14	14	37	51	Weathered basaltic gravel
NB-7	1	1.5	9	17	20	37	Hard, silty clay
	2	3.45	20	44	63	80	Weathered, basaltic gravel
NB-10	1	4	4	5	6	11	Stiff silty clay
	2	7.5	8	12	14	26	Very stiff silty clay
NB-11	1	2.5	3	3	4	7	Firm silty clay
	2	4.6	1	2	3	5	Sity clay

5.2.2 Soil characteristics from primary data source (Paleo soil)

During the present study 2 representative soil samples were collected from the intercalated Paleo soil located in the right abutment at the core termination. Later these soil samples were tested to assess the grain size (wet sieve and hydrometer) distribution, consistency (Atterberg limit), and the specific gravity. The soil test results are presented in Table5.6.

Table 5.6 Test results for representative samples from Paleo soil at Megech dam foundation

TP-ID	Location	Grain size (mm)			Atterberg limits			Specific gravity	Soil Class
		% Grave	% Sand	% Fine	LL	PL	PI		
Pit-1	RA	0	95.84	4.16	77.6	0	77.6	2.54	Poorly graded Sand
Pit-2	RA	0	98.07	1.93	69.8	0	69.8	2.43	Poorly graded Sand

Table5.6 depicted both soil samples with % fines less than 5 with no gravel and to be classified as coarse grained soils typically sand with little fines(Fig 5.3 and 5.4) below.

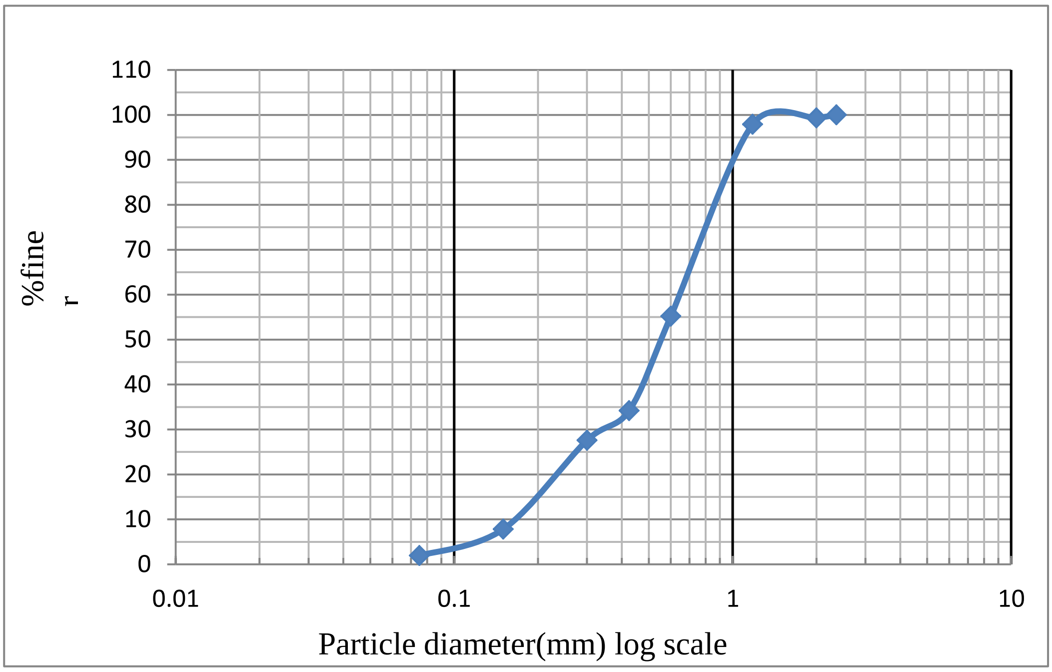

Fig5.3particle size distribution curve for soil sample taken frompit-2 at the core termination on the right abutment

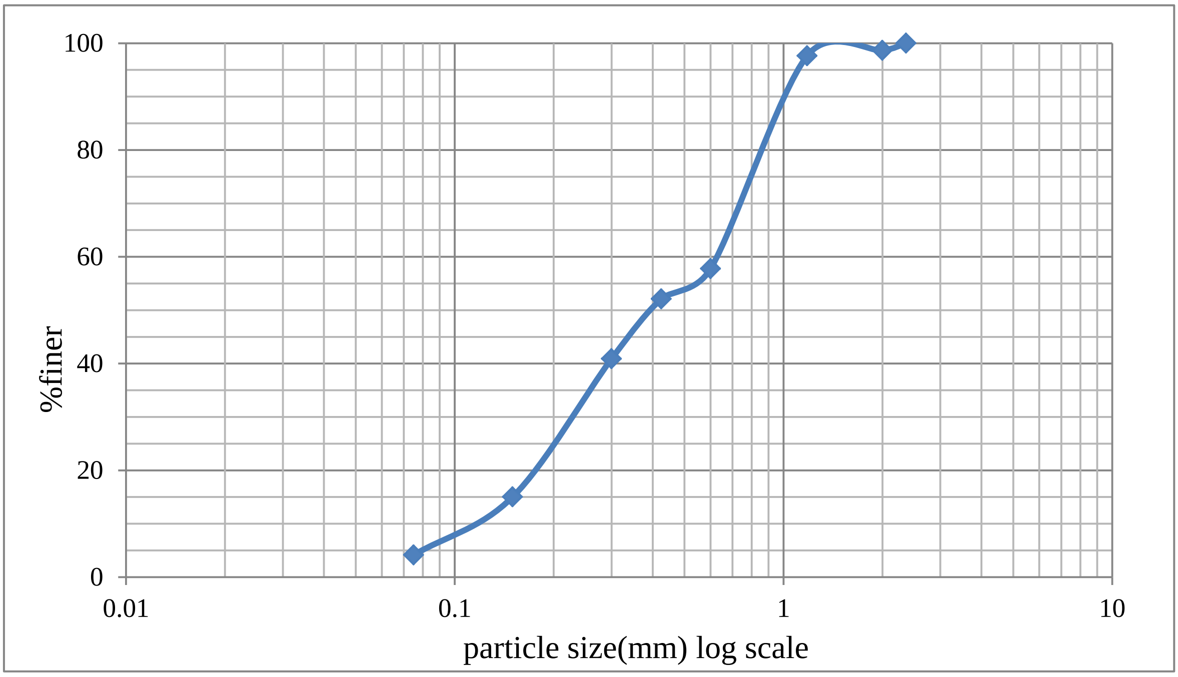

Fig 5. 4 Particle size distribution curve for soil sample taken from Pit-1 at the core termination on the right abutment

Perusal of grain size distribution results (Table 5.6) indicated that both the soil samples are poorly graded sand (Fig5.3 and 5.4) soils classified as per USCS soil classification system. The coefficient of uniformity (Cu) and gradation (Cc) used for classification were obtained (3.56, 0.98) and (4.846, 0.59) for pits 2 and 1 respectively (Fig5.3 and 5.4).

5.3 Engineering geological mapping at Megech dam site

Engineering geological mapping is a systematic recording of geologic data from field exposures and is defined as the examination of natural and manmade exposures of rock or unconsolidated materials. Engineering geological maps in general are termed storage of geological, geotechnical and engineering information and simplified model of geological facts for engineering use. For the present study geological contacts were identified and marked along the foundation, abutments and in the river section with the help of GPS survey. Later, these contacts were used to generate the engineering geological map of the dam site. As shown in the engineering geological map (Fig 5.5) the dam rests on volcanic terrain typically basaltic successions with intercalated paleo soils including overburden materials along the river bank. The rock exposures present at the dam site are mostly from excavation for investigations or for slope correction besides.

5.4 Geological cross-section along the Dam axis and conduit outlet structure

5.4.1 Geological cross-section along the dam axis

The geological cross-section along the dam axis was generated from surface geological mapping and the drill holes data, drilled during the preliminary and detailed design investigations (WWDSE, 2007; 2010). These drill holes were NB-1 to NB-7 (Table 5.4) and were drilled to the depths of 21 to 90 m. Lateral and vertical lithological variations with ground water level along the dam axis were addressed in the geological section. Thickness variation within geologic units' along the dam axis being one of the main information extracted from the geologic cross-section. Analysis of drilled holes data and associated geologic cross section (Fig.5.6) indicate that the dam foundation have basaltic varieties with different degree of weathering and jointing overlaid by overburden material of different thickness. Basaltic varieties found at the dam foundation involves porphyritic, vesicular, amygdular, aphanitic and scoraciouse basalts with different thickness and locations. Paleo soil intercalated with these basaltic units, subordinate tuff, mudstone (88.1-90m of-NB-4), siltstone and sandstone (26.3-27m of -NB-6) were also addressed. Perceived from drilled hole data analysis it is found that, in general degree of weathering and jointing decreases with depth though some anomalies in the left and right abutments of the dam were observed. The overburden material mainly comprise of river deposits, alluvial and Colluvial/ Illuvial materials that are underlain by bedrocks.

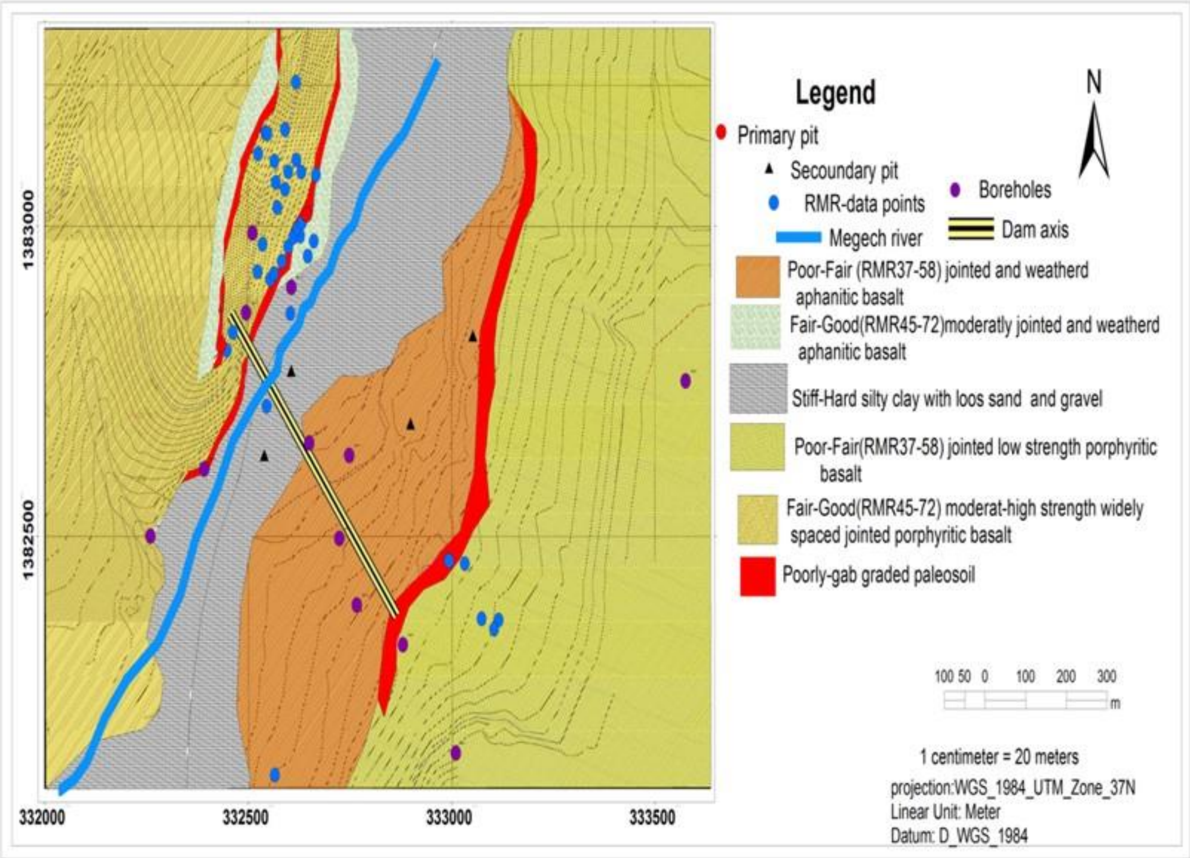

Fig.5.5 Engineering geological map of Megech dam site with distribution Of RMR data points and bore holes

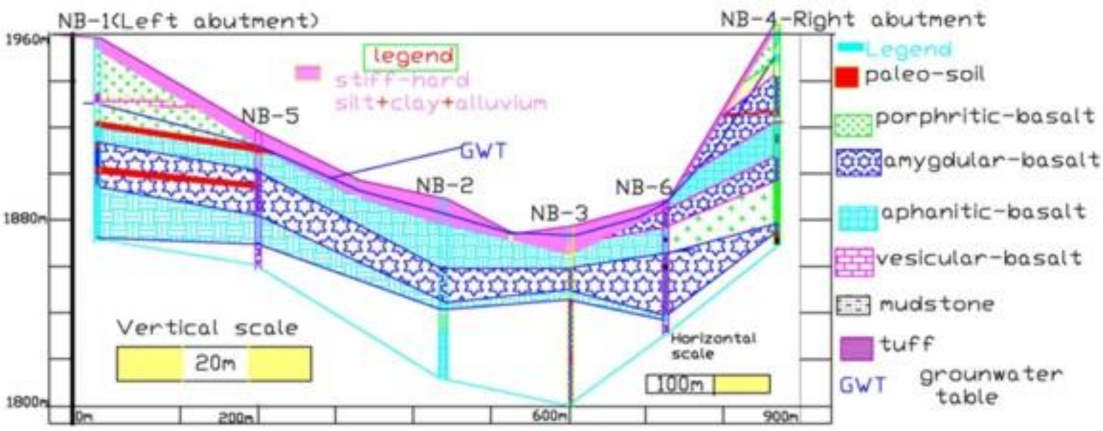

Fig. 5.11 Geological cross-section along the dam axis

The overburden material is found to be covering the bed rock incorporates stiff highly plastic silty clay to clay, loose - medium to coarse gravel, very loose unconsolidated fine to medium grained sandsoil deposits with average thickness of 5m. The overburden in general is thick and significant in the central part of the dam (NB-2, 12.6m and NB-3, 16.15m) and limited in thickness towards the right bank (Fig.5.6).

5.4.2 Geological cross-section along the dam conduit outlet work structure

The geologic cross-section along the conduit outlet portrays the nature of sub surface geology along the intake tower and conduit structures resting on pile foundation at the right abutment of the dam. The relevant data used to generate the cross-section along the conduit was obtained from the logs of bore holes BH-5, BH-6, BH-7, BH-1 and BH-10 drilled along the conduit. The drilled holes log and corresponding geological cross-section (Fig5.7) along the conduit suggests that basaltic rock successions with very thin overburden materials including alluvial silt, perceived at MDC-BH-5,is present in sub-surface. The basalts encountered through conduit cross section are mainly aphanitic, porphyritic, vesicular, scoraciouse and aymygdular basalts. Further, it was observed that in general degree of weathering varies from slightly weathered to completely decomposed rock along the conduit. From top to bottom jointing and weathering vary alternatively with depth thus, reasonable correlation was not found between depth, jointing and weathering.

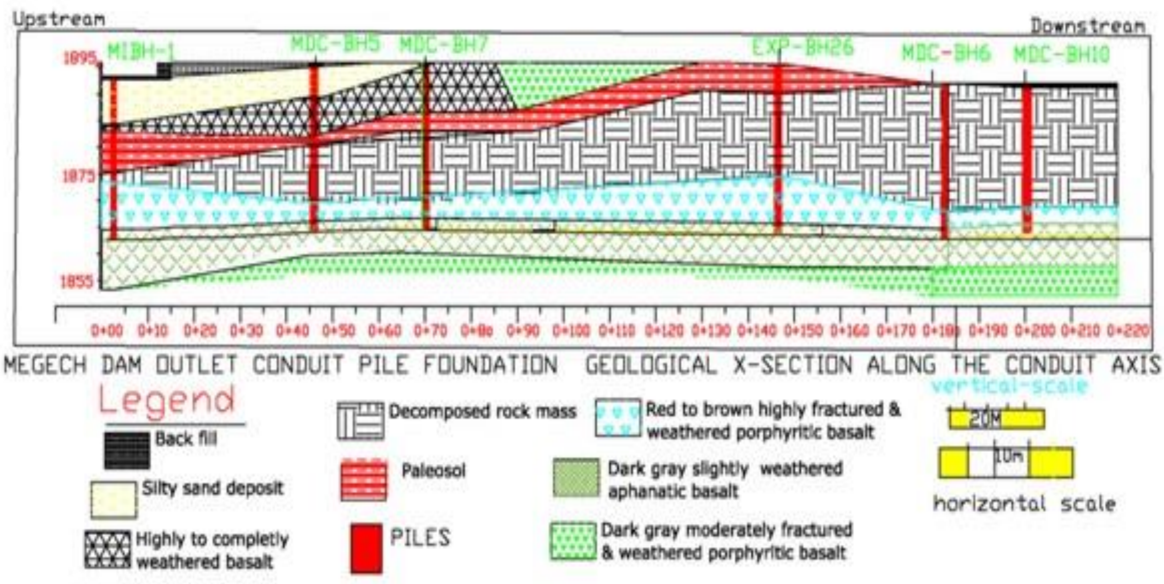

Fig. 5.7 Geological cross-section along outlet conduit structure with selected piles

5.5 Foundation permeability

No dam could be absolutely impervious with respect to its foundation, abutment or embankment rather it becomes sever when it passes certain limit (Gebremedhin Berhane, 2017). Hence taking engineering measures to address such engineering problems in dam engineering is very important (Chen et al., 2010). Grouting is a common method of sealing rock mass in dam foundation engineering to displace water and air outside the designed grouting zone (Barani et al., 2014). The execution of grouting largely depends on result of permeability (water preasure) test to delimit foundation rock mass in to zones of different rock mass quality requiring different level of treatment (Ajalloeian et al., 2014). In the present study the permeability of the dam foundation was assessed based on number of permeability test results collected from final detailed design report of Megech dam (WWDSE, 2010). The permeability test results were evaluated based on Houlsby (1976) Lugeon interpretation method and are presented in Table 5.7.

Further, assessment of permeability of dam foundation through eight investigation boreholes based on Houlsby (1976) approach revealed that 78% of the rock mass is impervious, 2% is low pervious 12% is moderately pervious, 1.3% is pervious and 6.8% is highly pervious (Table 5.8). Those geologic units responsible to high permeability included moderately-highly fractured porphyritic basalt (28-38.5m), fractured vesicular basalt (38.85-41m), highly weathered fractured aphanitic basalt (41-43m), slightly weathered aphanitic basalt (43-47m) and moderately jointed amygdular basalt (49-57m) sections of NB-1. Likewise (51-56m) section of NB-6 which includes highly weathered and moderately fractured aphanitic basalt and fractured amygdular basalt was found highly permeable zone. Similarly (17-21m) sections of NB-7 shows a record of 111 lugeon value and is amygdular basalt with interconnected cavities. In addition 0-24m section of NB-10 and 0-22m section of NB-11 were found adjacent permeable zones in the left and right abutments respectively. The respective geologic formations responsible to permeability in these adjacent zones were perceived to be loos unconsolidated sand and gravel with some fragments of rocks in the most river banks.

Amygdular basalt with joints partially filled by calcite and quartz was also found permeable (25-37m) section of NB-5. Top sections through NB- to NB-2 which are porphyritic and aphanitic basalt and bottom sections of NB-4 and NB-6 were amygdular and porphyritic basalts found medium permeable zones. Further, most of the sections tested in each borehole portray that the foundation in general is water tight though some anomaly at selected sections in selected boreholes were present, typically in the upper section of BH-1 and BH-7. Figs 5.8 to 5.10 show permeability variation with respect to depth from representative boreholes for three sections of the dam.

Table 5.7 Permeability along the dam foundation (WWDSE, 2010) and permeability class as Permeability class as per Houlsby (1976)

BH-ID	Altitude	Test Section	Test type		Permeability Class
		(m)	Lugeon-value	Type of flow	
NB-1	1958	28.45-32.00	85	Turbulent	Highly Permeable
		31.35-35.45	65	Turbulent	Highly Permeable

BH-ID	Altitude	Test Section (m)	Test type Lugeon-value	Type of flow	Permeability Class
		35.45-40.45	42	Turbulent	Highly Permeable
		55.12-60.12	0.2	Void filing	Watertight
		60.12-65.12	0.3	Void filing	Watertight
		65.12-70.07	0	Dilation	Watertight
		70.07-75.07	0.2	Dilation	Watertight
		75.07-80.07	0.2	Dilation	Watertight
		80.07-85.07	0.1	Dilation	Watertight
		85.07-90.07	0.1	Dilation	Watertight
NB-2	1888	21.00-26.00	4	Turbulent	Medium permeable
		28.00-33.00	2	Laminar	Impervious
		33.00-38.00	2	Laminar	Impervious
		38.50-43.50	1	Laminar	Watertight
		43.50-48.50	0.2	Void filing	Watertight
		49.50-54.50	0.1	Void filing	Watertight
		55.00-60.00	0.2	Void filing	Watertight
		61.00-64.80	0.1	Void filing	Watertight
		65.80-70.80	0.3	Dilation	Watertight
NB-3	1879	17.70-22.70	1	Laminar	Watertight
		22.70-27.70	1	Laminar	Watertight
		27.70-30.70	0.2	Turbulent	Watertight
		30.70-33.70	1	Laminar	Watertight
		33.70-36.70	1	Laminar	Watertight
		36.70-39.70	1	Laminar	Watertight
		39.70-43.70	0.1	Void filing	Watertight
		43.70-46.70	0.1	Laminar	Watertight
		46.70-49.70	4	Turbulent	Medium permeable
		49.70-52.70	0.1	Turbulent	Watertight
NB3	1879	52.70-55.70	0.1	Turbulent	Watertight
		55.70-58.70	0.1	Dilation	Watertight
		58.70-61.70	0.1	Dilation	Watertight
		61.70-64.70	0.1	Dilation	Watertight
		64.70-67.70	1	Laminar	Watertight
		67.70-70.70	0.1	Dilation	Watertight
		70.70-73.70	0.2	Dilation	Watertight
		73.70-76.70	0.1	Turbulent	Watertight
		76.70-79.70	1	Laminar	Watertight
		79.70-82.70	0.1	Laminar	Watertight
NB-4	1961	18.00-22.35	0.1	Laminar	Watertight
		22.40-27.40	0.1	Laminar	Watertight
		27.40-32.40	0.1	Laminar	Watertight
		32.40-37.40	1	Laminar	Watertight
		37.40-42.40	0	Turbulent	Watertight
		42.40-47.40	0.2	Laminar	Watertight
		47.40-52.40	0.1	Void filing	Watertight
		52.40-57.40	0.2	Dilation	Watertight
		57.40-62.40	0.2	Dilation	Watertight
		62.40-67.40	0.2	Dilation	Watertight
		67.40-72.40	1	Laminar	Watertight
		72.40-77.40	1	Laminar	Watertight
		77.40-82.40	1	Laminar	Watertight
		82.40-87.40	1	Laminar	Watertight
NB-5	1919	25.40-30.40	3	Laminar	Permeable
		37.50-42.40	2	Laminar	Slightly Permeable
		42.40-47.50	0.1	Void filing	Watertight
		47.25-52.00	0.1	Turbulent	Watertight
		52.00-56.50	0.1	-	Watertight
		56.50-60.00	0.1	Dilation	Watertight
NB-6	1888	7.10-11.20	2	Laminar	Slightly Permeable

BH-ID	Altitude	Test Section (m)	Test type Lugeon-value	Type of flow	Permeability Class
		11.25-16.25	1	Laminar	Watertight
		16.25-21.25	3	Laminar	Moderately Permeable
		21.25-25.70	1	Laminar	Watertight
		25.70-30.70	2	Laminar	Slightly Permeable
		30.70-36.70	0.1	Void filing	Watertight
		36.70-41.70	0	Laminar	Watertight
		41.70-46.70	0.1	Void filing	Watertight
		46.70-51.50	2	Void filing	Slightly Permeable
		51.50-56.50	12	Turbulent	Highly Permeable
		56.50-60.50	0.1	Turbulent	Watertight
NB-7	1912	12.55-17.55	2	Laminar	Slightly Permeable
		17.55-21.05	111	Wash out	Highly Permeable
BH14	1900	31.90 - 36.40	0.2	Laminar	Water tight
		36.40 - 41.80	0.2	Laminar	Water tight
		41.80 - 46.80	1	Laminar	Water tight
		46.2 - 50.0	1	Laminar	Water tight
Note: the boundary between flow types was assessed as per Houlsby 1976, Lug eon interpretation					

Table 5.8 Permeability class and frequency distribution of rock mass under Megech dam foundation (Houlsby, 1976)

Lugeon	0 - 3	3 - 10	10 - 30	30 – 60	➢ 60
Permeability class	Impervious	Low	Moderate	High	Very high
Frequency (%)	80	0	12	1.3	7

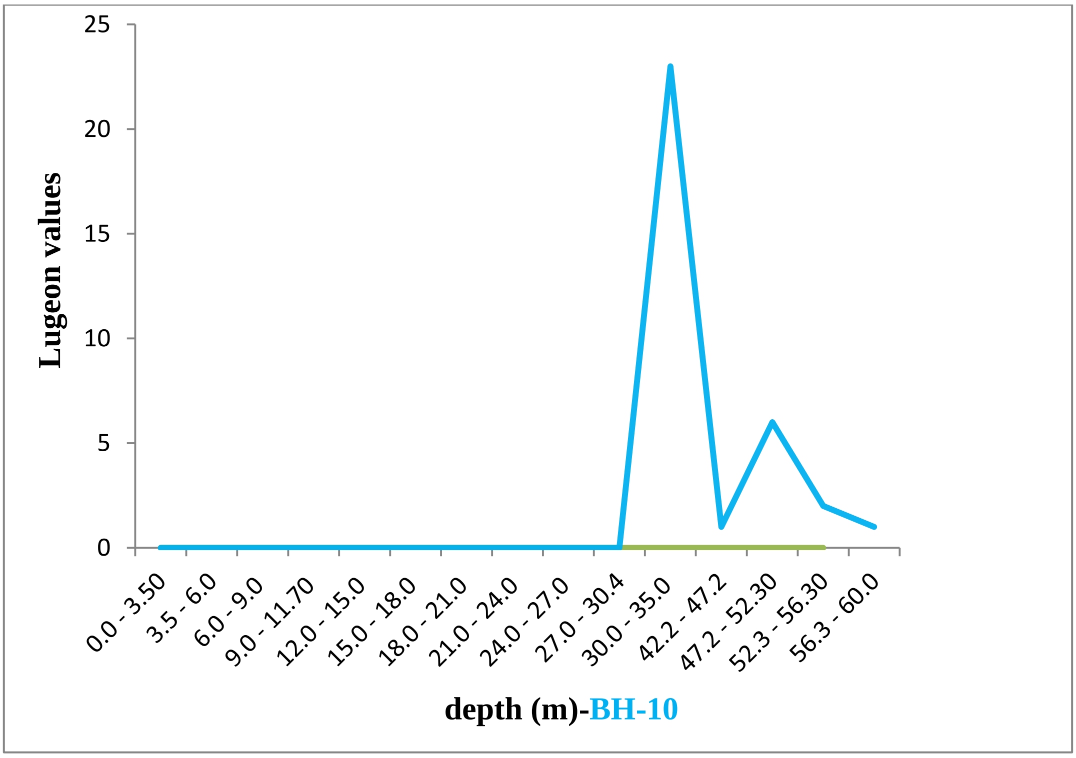

Fig. 5.8 Variation of lugeon values with respect to depth in drill -hole NB-10 drilled at the center of the dam: 0-27m implies overburden material permeability

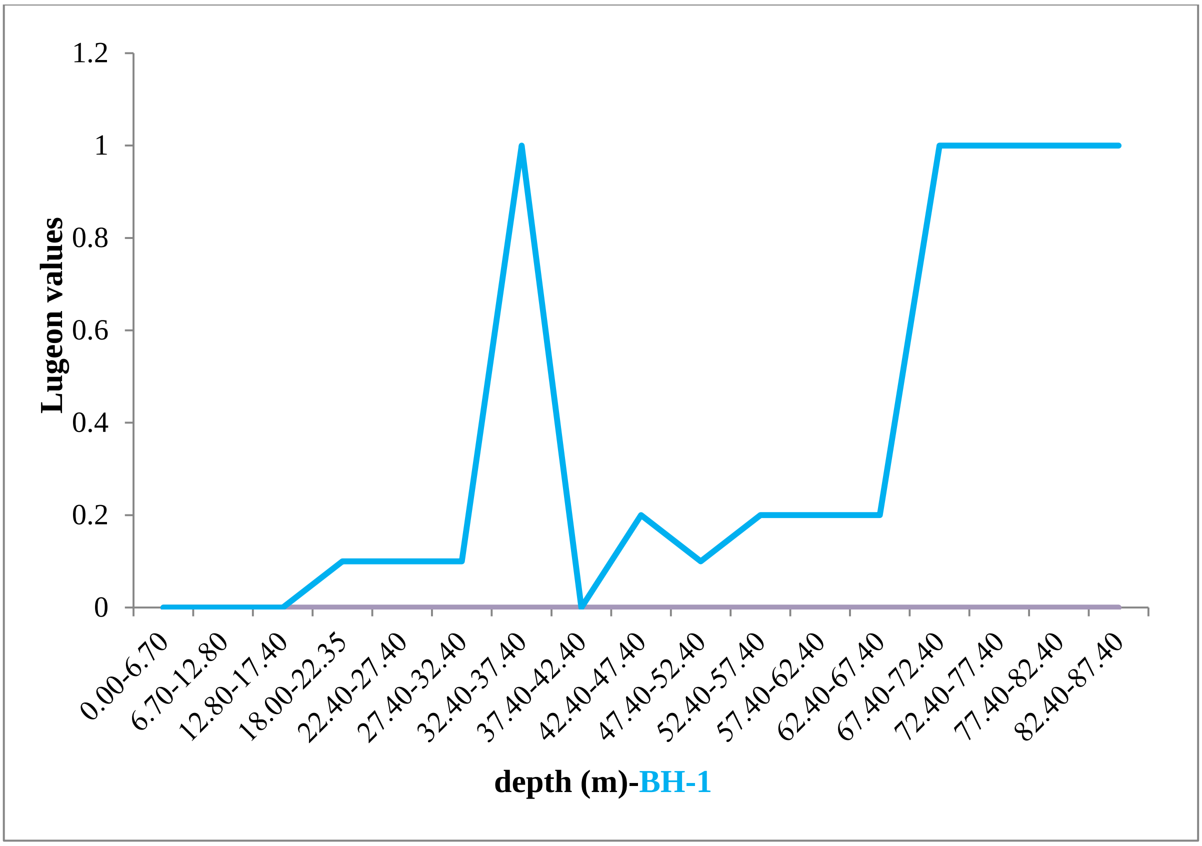

Fig. 5.9 Variation of lugeon value with respect to depth in drill hole NB-4 drilled on the right abutment of the dam

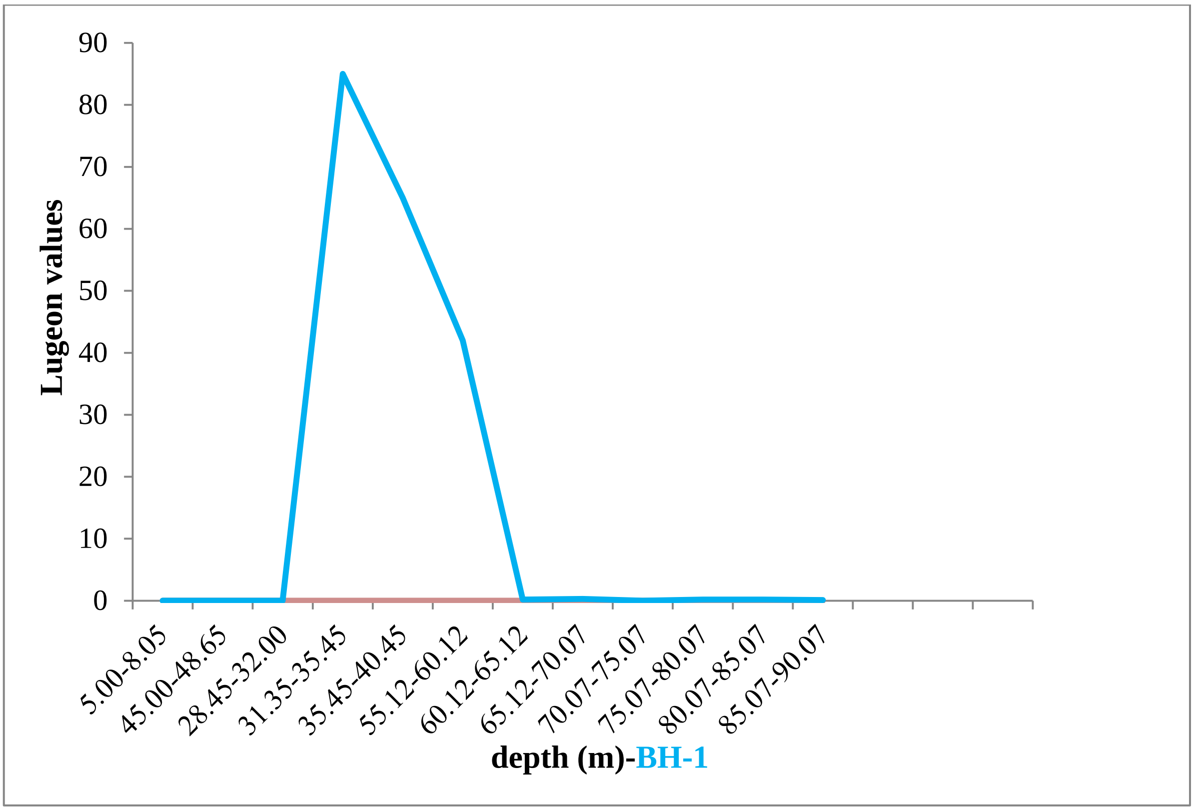

Fig. 5.10 Variation of lugeon value with respect to depth in drill hole NB-1 drilled at the left abutment of the dam: First 5 to 32m implies overburden permeability (K)

Analysis of permeability in the left abutment of Megech dam (Table 5.7-NB-1and Fig.5.10) shows general reduction in permeability with increasing depth and permeability rises in low quality rock mass. The increase in permeability was observed in test sections 28 to 40 m (Fig. 5.10) where lugeon values fall within the range of 0-85. Likewise, permeability also increases in between 30 to 35m test section for the central /axis of the dam (Fig. 5.8). On the other hand permeability of right abutment in most of the test sections is less than one lugeon which confirms good quality rock mass as compared to the left abutment. This fact is confirmed both through drill hole data as well as through field observations. In general, for the right abutment in any of the test sections permeability was not more than one lugeon (Fig.5.9) and Table5.7 (NB-4).

5.6 Foundation treatment suggestion based on permeability analysis

5.6.1 Curtain grouting

Curtain grouting is one of the commonly applied foundation treatment method in dam engineering used mainly in rocks with relatively wider joint spacing. Single or multiple rows of holes drilled along a dam axis and grouted to the depth of permeable rock in order to create a barrier for the water through an area of high permeability (Fell, 1992). Houlsby (1996) stated that for a rock mass with lugeon values 0 - 3, 3 - 10 and >10 as no grouting, single row grouting and three rows of grouting may be needed, respectively. Currently the closure requirement decides the ceasing of grouting operation in dam foundation grouting rather than predefined number of rows (Houlsby, 1977; 1978; 1985 as cited in Fell, 1992). Closure criteria was modified in the revised closer criteria (Houlsby 1985) as to continue grouting until pre-grouting lugeon value falls to 3 - 5 lugeon for earth –rock fill dams.

In Megech dam foundation lugeon tests conducted in the bore holes drilled along the dam axis indicated that the permeability values are generally low in most parts except on the very left bank, as observed in bore holeNB-1in section from 28.4 - 40.4m in which lugeon values of 85,65 and 42 were recorded. In borehole NB-1 which was drilled on the left bank, the packer tests for sections between 28 to 32 m resulted in a permeability value of over 85 Lugeon; for test sections between 32 to 35 m over 65 Lugeon, and for test sections between 35 to 40 m over 42 Lugeon, whereas the deeper sections show less than 1 Lugeon of permeability.

Further, drill hole NB-5 which was drilled on the left abutment, about 200 m to the right of borehole NB-1, the Lugeon value below the depth of 30m was generally < 2, indicating water tight rock mass while section 25 - 30m in NB-5 the lugeon values suggest permeable zone. In boreholes NB-2 and NB-3 that were drilled in the valley floor, the permeability values in general were less than 0.2 Lugeon however; about 4 Lugeon was recorded in sections 21 - 26m in bore hole NB2 and section 46 - 49m in bore hole NB-3. Further, permeability tests conducted in the right abutment in bore holes NB-4 and NB-6 show that the Lugeon values are generally <1, which indicate that water tight conditions. The lugeon value obtained in test section 30.0 - 35.0m in bore hole BH-10 was 23 which suggest highly permeable conditions. Similarly, test section 47 – 52m showed a lugeon value of 6 indicating moderately permeable conditions. Drill hole No. BH-11 drilled on the right side of the river bank revealed overburden alluvial soil to be permeable while the underlying rock mass found to be low permeable to impermeable, as perceived from the permeability test.

In view of the low-medium permeability values obtained in drill holes NB-2, NB-3, NB-4 and NB-6, moderate degree of foundation grouting appears to be sufficient for the whole section starting from NB-5 on the left abutment up to NB-6 and NB-4 on the right bank. The grouting process has to be continued until the lugeon values less than 3, as per Houlsby (1985) closure requirement for embankment dams, is achieved. The curtain grouting has to be done in split spacing or closure method in which primary, secondary and tertiary sequence is done until water pressure test provides required water tightness of the test section. Hence, extensive grouting may not be required for Megech dam foundation. According to ICOLD (2005) a typical grout curtain varies in depth from 0.35×H to 0.75×H, where "H" is the height of the reservoir above the top of the grout curtain in a specific location.

Based on Houlsby (1976) classification the rock mass may be characterized as; 0-3 lugeon as water tight, 3-10 lugeon low permeable, 10-30 lugeon medium permeable, 30-60 as high permeable and > 60 lugeon as very high permeable rock mass. Accordingly the dam foundation in the present study was zoned in to four classes and is presented in Fig.5.11 indicated large permeability difference in between right and left abutments. However, a general reduction in permeability with increasing depth can be observed in both abutments. Thus depth of curtain grouting has to vary from 27 to 57m along the central dam axis. The hydraulic head in this section will be 76m as per (ICOLD , 2005). As shown in Fig. 5.11 more depth of curtain grouting is required in the left abutment while in the right abutment progressive reduction in reservoir height confirms reduced of curtain grout depth as per ICOLD (2005). Therefore, left abutment needs special treatment with respect to permeability and ICOLD (2005) recommendations may not simply be adequate.

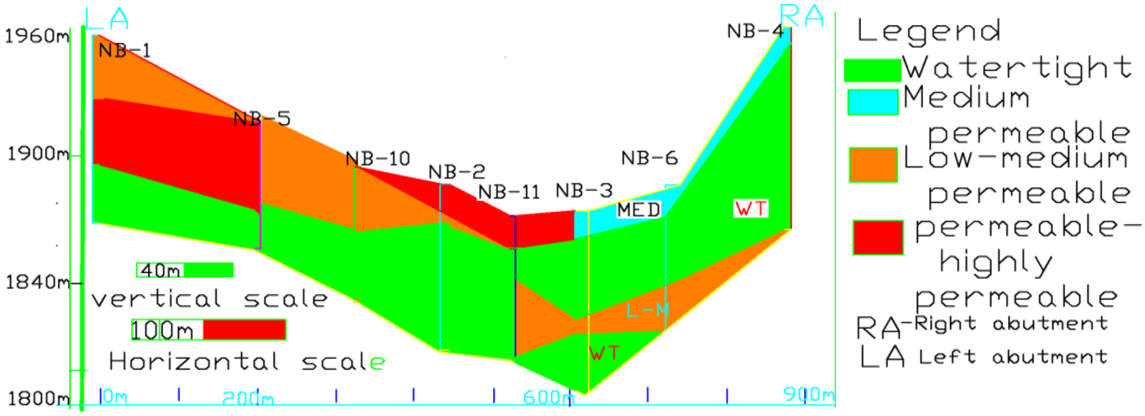

Fig.5.11 Permeability conditions in the dam foundation and abutments

5.6.2 Consolidation grouting

With regard to consolidation grouting Mistry (1983 as cited in Nigatu Fekadu, 2006) stated to extend grouting up to quarter of the base width of the dam (b/4) in the heel side and b/3 towards the toe in order to accommodate the variable stress concentrations developed due to reservoir fluctuations. For the present study permeability tests conducted in bore holes NB1 and NB5 indicated higher lugeon values in the top sections in the left abutment. Thus, it implies that sufficient extensive foundation grouting is required for these sections. Thus, consolidation grouting is required in these sections to strengthen the rock mass. With respect to permeability improvement the closure requirement of ≤ 3 Lugeon as proposed by Houlsby (1985) for embankment dams needs to be achieved.

In order to give intensive grouting of the upper layer in more fractured rock in the vicinity of the dam core, consolidation grouting to a maximum depth of 10m was proposed for the entire foundation by the Project authorities. The holes for consolidation grouting are proposed to be spaced 6 m apart and to ensure a highly efficient cutoff, a three-row grouting pattern is proposed by the project authorities. The general grouting scheme is presented in Fig. 5.12.

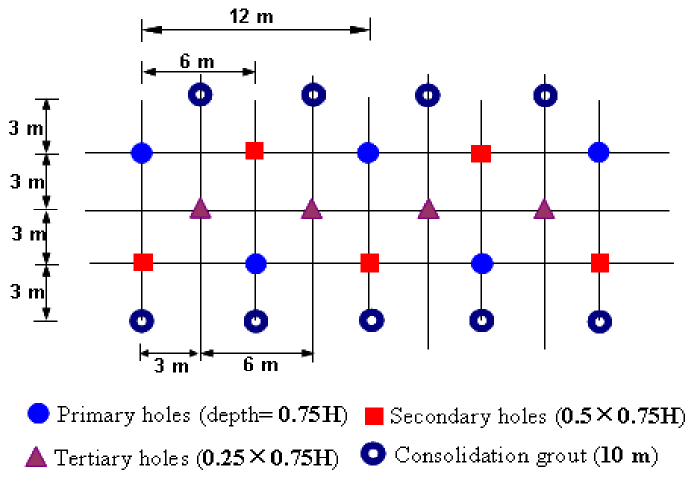

Fig.5.12 Grouting lines along the dam axis based on permeability result adopted from Megech dam final design report (WWDSE, 2010).

5.7 Bearing capacity estimation for piles under intake and conduit structures

Predictions of bearing capacity and potential settlement in rock mass are important parameters in geotechnical engineering (Zhang, 2010). Piles are cast in situ type and reinforced concrete in composition, end bearing and located in the right abutment of the dam. In total 64 piles under the intake structure and 152piles along the conduit outlet were installed and are founded on slightly weathered aphanitic basalt. As per design considerations to keep the differential settlement under control the piles depth was kept in a range 36 - 40.74m. The piles have dimensions of 80cm diameter and 3m center to center spacing. Piles are submerged in water with an average depth of 15 m. As per the design by the project authorities the main tests conducted to estimate the bearing capacity and the settlement potential of the piles includes unconfined compression test for rocks and working pile load tests. The bearing capacity estimation assumes only end bearing and skin friction was presumed to taken care of by considering a suitable factor of safety. In the present study the bearing capacity and settlement conditions of pile foundation was estimated from; (i) UCS Test results from rock cores, (ii) Pile load test results, (iii) Working pile load test result and (iv) Preliminary or design pile load test result. These tests were conducted during detailed design investigation. The preliminary pile load test refers to the pile load test conducted by using twice of the pre-defined structural load.

5.7.1 Pile bearing capacity from UCS and RQD

In order to address the rock mass properties from intact rock properties in estimating the end bearing and shaft friction the empirical correlation proposed by Kulkarni (2016) are;

$\sigma cm = \sigma c * 10^{(0.013RQD-1.34)}$ (MPa) eq.5.3

Q-max=$3(\sigma cm)^{0.5}$ (MPa) eq.5.4

F-max = $0.2(\sigma cm)^{0.5}$ (MPa) eq.5.5

Where; σcm is the unconfined compressive strength of the rock mass, RQD is the rock quality designation, σc is the unconfined compressive strength of the intact rock (MPa), Q-max is unit maximum end bearing capacity of rock socketed piles and F-max is unit socket/ shaft friction of rocks surrounding the pile shaft.

Ultimate bearing capacity (Qu) is then given by:

Qu (Mpa) $=\pi B \sum \mathrm{Li}(\,\mathrm{f(max)}i) + \frac{\pi B^2}{4} * \mathrm{q(max)}$ ......eq.5.6 (Zhang, 2010) where B is pile diameter, i is number of layers and (Li) pile socket length

The estimated bearing capacity of selected piles under conduit structure is presented in Table The bearing capacity calculation presented in Table 7.9 was estimated for single pile in which pile was inserted in drill hole and respective RQD and UCS values for representative The estimated bearing capacity of selected piles under conduit structure is presented in Table 7.8. The bearing capacity calculation presented in Table 7.8 was estimated for single pile in which pile was inserted in drill hole and respective RQD and UCS values for representative rock mass was considered. From table 5.9 it is easy to guess much of the contribution for bearing comes from skin friction than end bearing in all cases: 22.17, 22.758 and 11 with respect to 3.28, 3.55 and 4.28 for piles on BH-5, BH-6 and BH-10 respectively (Table5.9)

Table 5.9 Bearing capacity estimation of selected piles under conduit structure (MDC-BH-5, 6 and 10) based on Zhang (2010): Q-max and F-max are taken from USC of intact rock and RQD values

Core run		**σc MPa**	**σcm MPa**	**RQD**	**Q-max MPa**	**F-max MPa**	**B(m)**	**Li(m)**	**Li*f-max**	**Qu (Mpa)**	**BH-ID**
From	**To**										
19	25.7	4.2	0.4684	29.8	-	0.1368	0.8	5.9	0.807		BH-5
25.7	30.1	10.5	4.71	76.3	-	0.434	0.8	4.4	1.91		
30.1	31.1	4.6	1.23	59	-	0.22	0.8	1	0.22		
31.1	37.5	30.8	13.5	75.5	-	0.7346	0.8	6.4	4.7		
37.5	40	7.9	4.74	86	6.53	0.476	0.8	2.5	1.19		
Sum					6.53				8.827		
Gross					3.28				22.17	25.45	
1.8	25.8	0.41	0.0224	6		0.03	0.8	24	0.72		BH- 6
25.8	28.1	6.44	2	64		0.2828	0.8	2.3	0.65		
28.1	35.5	41	13.514	66		0.735	0.8	7.4	5.44		
35.5	38.2	11.36	5.444	78.5		0.4666	0.8	2.7	1.26		
38.2	40.3	7.5	5.547	93	7.065	0.471	0.8	2.1	0.989		
Sum					7.065				9.06		
Gross					3.55				22.758	26.3	
19	28	1.76	0.253	38.33		0.1	0.8	9	0.9		BH-10
28	35	20.53	4.3	50.8		0.4144	0.8	7	2.9		
35	36	18.71	8.07	75	8.52	0.568		1	0.568		
Sum					8.52				4.37		
Gross					4.28				11	15.28	
Where ,Q-max=maximum end bearing and F-max=maximum shaft friction											

5.7.2 Pile bearing capacity and settlement from pile load test

As per the final design the piles under intake tower and conduit outlet structures are circular shape with diameter and center to center spacing 800mm and 3m, respectively. Static Pile load test was conducted to determine the pile settlement under working load, to confirm design bearing capacity as well as to determine the allowable bearing capacity of piles through interpretation of load settlement curves constructed from field pile load test results. In the present study pile load test data was collected from the project authorities. Later this data was processed and analysis to develop load-settlement curve from which design load bearing capacity, gross and net settlement were determined (Fig.5.13 a, b and Fig 5.14). As inferred from the load settlement curve piles were subjected to loading and unloading phases so that behavior of the sub underlying ground can be characterized for different multiple of design load.

For a given design load of 3750 KN for a working pile under intake structure the corresponding gross settlement is equal to 14mm and net settlement equals to 0.52mm for first loading and unloading cycle while for second phase of loading gross settlement is equal to 39.23mm and net settlement equals to 0.03mm (Fig5.13a). The pile can carry even more than twice the design load and the settlement is increased linearly without failure or deflection. This is also confirmed by Singh (2016 and British standard (BS-8004, 1986) as cited in Kulkarni, 2016) accordingly the ultimate load occurs when final settlement reaches 10% of the pile diameter. In the present case pile diameter is 800mm and 10% of it is 80mm, thus the ultimate load may occurs when the final settlement reaches 80mm which was not achieved in the present case (Fig5.13a). Pile load test result and predicted pile bearing capacity of working pile E-4 under intake tower structure is presented in Table 5.10.

Table 5.10 Pile load test results and predicted pile bearing capacity of working pile E-4 under intake Tower structure

Pile length	Pile diameter	Test results					
		Load-max	Gross-sett.	Net-sett.	Predicted capacity	Test-pile	Design load
29.6m	800mm	7500KN	39.23mm	0.03mm	>7500KN	No-E-4	3750

Fig5.13b shows that the proposed pile as per the design is adequate and it performed well under the applied load. The settlement curve keeps on increasing corresponding to the applied load without failure till the load ceases at 3375kN with corresponding settlement of 3.675mm for the preliminary load test. The rate of increment of the settlement with load is also slow

indicating that it can accommodate much more load until the settlement reaches 10% of the pile diameter to achieve ultimate bearing capacity as per Singh (2016) (Fig5.13b) .

The design load for the pile P-10 was 4650kN and it was tested for a maximum load of 9300kN (Fig5.14). The gross settlement for the design load was 6.4mm and the rebound settlement was 1.5mm thus, resulting net settlement of 4.9mm for first phase of loading was recorded (Fi5.14). Likewise, the gross settlement for maximum test load was 12.59mm while the rebound settlement for this load was 2.75mm thus, giving net settlement of 9.84mm. According to Singh (2016) the ultimate bearing capacity for the pile is the load at which settlement reaches 10% of the pile diameter. In the present case settlement did not reached 10% of the diameter of the pile (80mm) (Fig5.14). Further, as can be observed that settlement increases linearly with increment in load even beyond the maximum test load adopted (9300KN). Thus, it implies that the pile can bear even more than twice the design load (4650kN). Pile load test results and predicted pile bearing capacity of working pile P-10 under conduit structure is presented in Table 5.11.

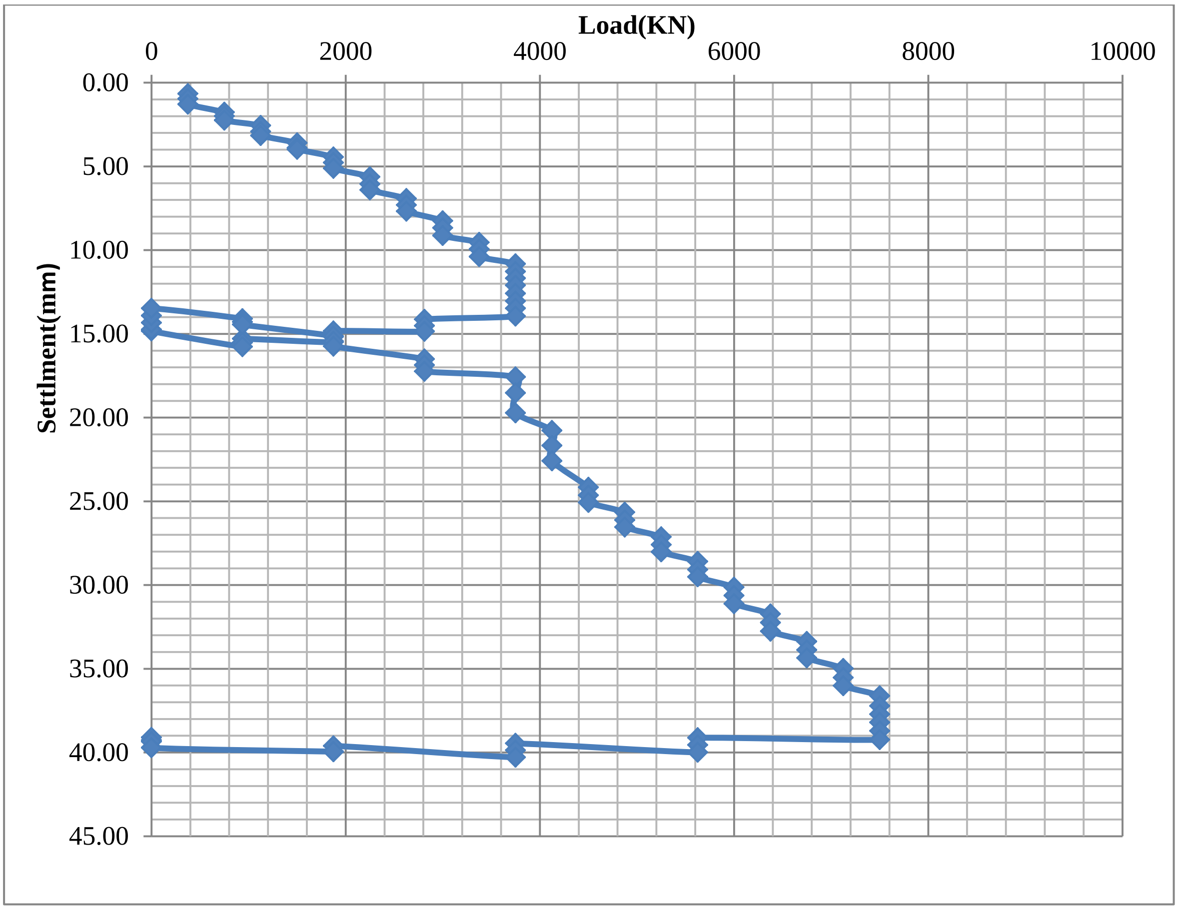

Fig. 5.13a Load verse settlement curve for pile under intake tower - derived from working load test

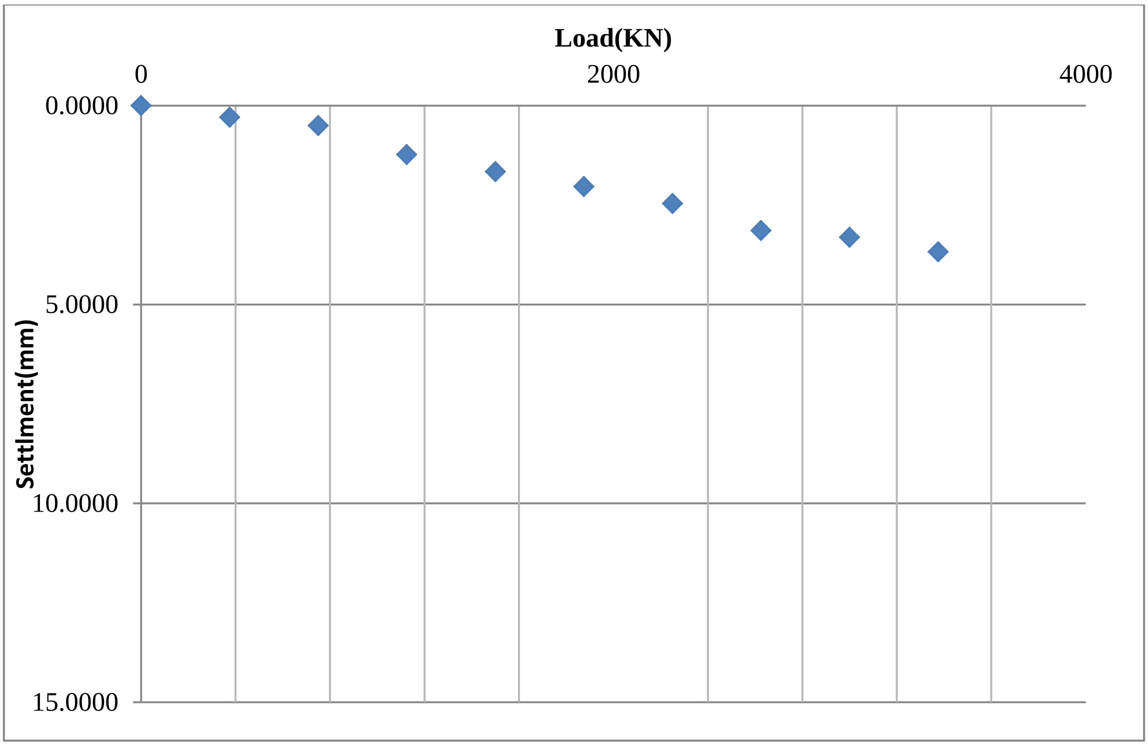

Fig. 5.13b Load verse settlement curve of pile under megech dam intake tower derived from preliminary load test (Pile-E-4)

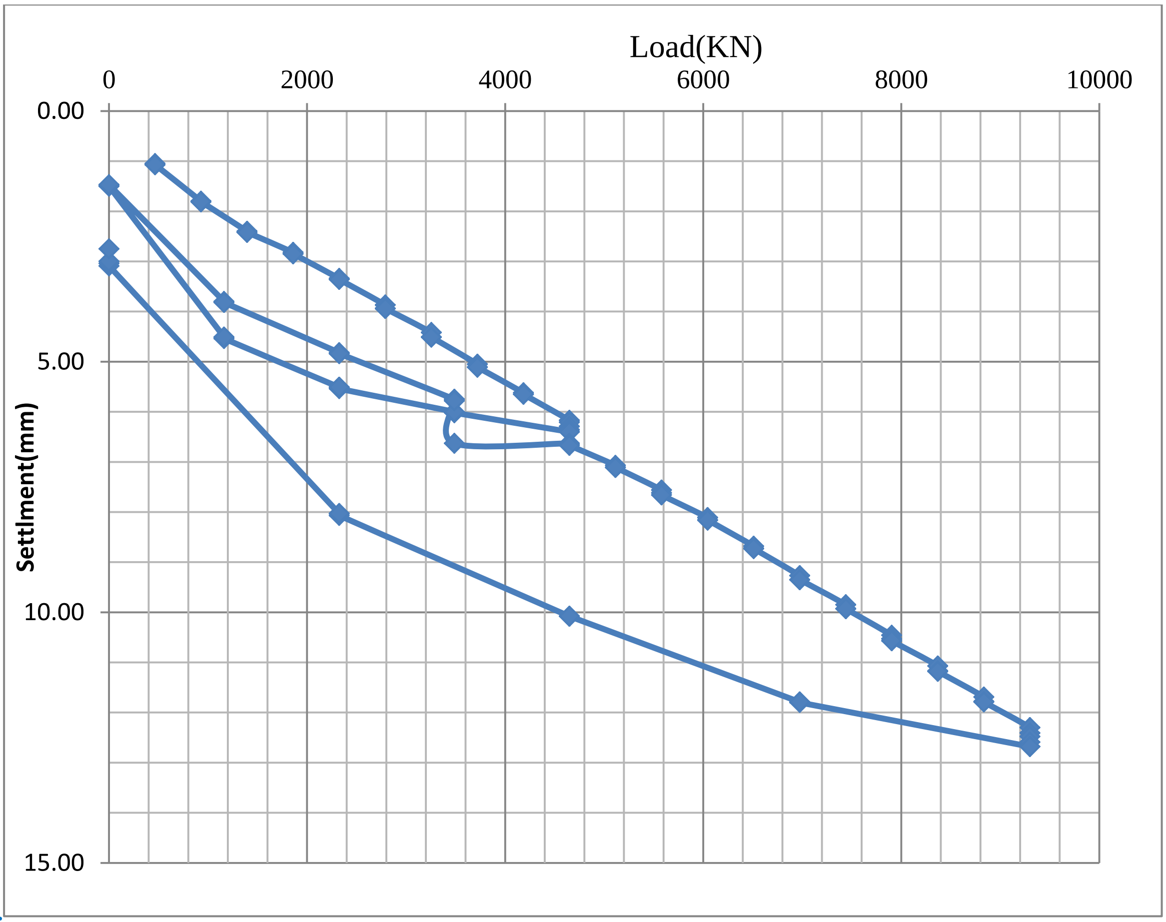

Fig 5.14 Load-settlement curve derived from pile load test under conduit structure (P-10) working pile

Table 5.11 Pile load test results and predicted pile bearing capacity of working pile P-10 under conduit structure

Pile length	Pile diameter	Design load		Max -load		Predicted-pile capacity	Test pile
		Gross settlement	Net settlement.	Gross settlement.	Gross settlement.		
31.9	800mm	6.4mm	4.9mm	12.59mm	9.84mm	>9300KN	P-10

5.8 Assessment of rock mass allowable bearing pressure from UCS and RQD

The allowable bearing pressure is the maximum net intensity of loading that can be imposed on the soil or rock with no possibility of shear failure or excessive settlement. The safe load bearing capacity of rock foundation can be calculated from UCS and RQD values considering factor of safety (Boles, 1996) and can be given as;

$$\text{Safe bearing capacity(Kpa)} = \text{UCS} * \frac{\text{RQD2}}{\text{SAFETY}}\text{Factor} \quad \ldots\ldots\ldots (4.9)$$

The average allowable bearing capacity for rock mass at dam foundation by considering a factor of safety equal to 3.0 (Hoek and Bray, 1977 as cited in Nigatu Fekadu, 2006) is presented in Table 5.12.

Table 5.12 Average allowable bearing capacity of rock mass at dam foundation computed from average UCS and RQD values for right and left abutments

Location	Average UCS (MPa)	Average RQD (%)	Factor of safety	Allowable bearing Capacity (KPa)
Right abutment	58.8	61	3	72931.6
Left abutment	26.4	44.3	3	17269.9

UCS and RQD data used for this purpose was collected from the field during the present study (presented in Table 5.2). In general, the UCS for the rock mass on the right abutment varies from 23 - 166 MPa and 23 - 33MPa for the rock mass on the left abutment (Table 5.2). Similarly, RQD for the rock mass ranges from 16 - 98.5% and 9.2 - 65.5% for the right and the left abutment, respectively. Relatively high allowable bearing capacity is achieved for the right abutment as compared to the left abutment.

5.9 Ground water interactions for sulphate attack and corrosion on Piles under Conduit and intake tower structure

Interpretation of the chemical constituents of water can provide insights into surface–groundwater interactions or the possible performance of concrete structures under submerged conditions. Some of the commonly used environmental tracers include parameters such as; the major anions and cations like calcium, magnesium, sodium, chloride and bicarbonate (Gebremedhin Berhane, 2015). The coexistence of sulphate and chloride ions in ground and surface water may cause deterioration of the concrete structures such as piles under conduit, spillway and tunnel if these structures exist for a given dam. For such conditions corrossivity of ground as well as surface water can be determined from the corrosivity ratio-coefficient as discussed in Chapter-4 (eq4.10). The Hdro chemical ions (mg/l) for boreholes at and near to the vicinity of Megech dam used to assess the interaction between pile and groundwater (Sileshi Mamo, 2015) is presented in Table 5.13.

Table 5.13 Hdrochemical ions (mg/l) for boreholes at and near to the vicinity of Megech dam used to Asses pile ground water interaction (Sileshi Mamo, 2015)

Borehole ID	Ph	Na^+	K^+	Ca^{+2}	Mg^{+2}	HCO_3	CO_3	Cl^-	F -	$SO4^{-2}$	$NO3^-$	SiO2	Br^-	Depth (m)
GW-5	7.26	20	0.5	48	19	267	0	16	0.32	13	5	61	0.26	95
SW-4	7.92	20	2	27	22	216	0	9	0.29	20	0.4	35	0.2	-
GW-21	7.86	90	1.5	25	0.1	293	0	18	0.3	12	9	52	0.76	173
GW-3	7.49	64	0.3	0.1	8	161	32	9	0.6	11	0.4	32	0.44	301

Corrosivity coefficient **(CR) = 0.135, 0.156, 0.129 and 0.125 for GW-5, SW-4, GW-21 and GW-3** respectively**, Where** Na^+=Sodium**,** K^+=potassium**,** Ca^{+2}=calcium**,** Mg^{+2}=magnesium **cations and** $HCO3^-$ =Hydrogen carbonate**,** Cl^-=chlorine**,** F^-=florin**,** $SO4^{-2}$ =sulfate**,** $NO3^-$=Nitrate **and SiO2** =silicate anions**.**

The hydro chemical analysis result as presented in Table 5.13 show that all samples are non-corrosive (CR<1). According to Mahadevaswamy (2002 as cited in Nigatu Fekadu, 2006) water (groundwater and surface water) may be corrosive for concrete structures and deteriorate concrete structure when corrosivity coefficient (CR) is greater than one. Based on this expression none of the samples show a CR value greater than unity (Table 5.13), therefore the groundwater (GW-5.GW-3 and GW-21) and surface water was found to be non-corrosive in nature and thus, will not affect the safe performance of submerged piles. This implies that the piles submerged in water under the intake and conduit outlet works will safely serve their design period without any deterioration resulted from the ground water aggressiveness. Further, the hydro chemical results were also compared to the results of Gumara (a very nearby dam project) and Tendaho dam project which is located on the eastern side of the Megech dam and all the dams are founded in same volcanic province. Respective hydro chemical analysis results for these dam projects are presented in Table 7.14.

Table 5.14 Comparison of hydro chemical analysis results for Megech, Tendaho and Gumara dam projects with respect to corosvity coefficients

Projects	Megech dam				Tendaho dam	Gumara dam
Samples	GW-5	SW-4	GW-21	GW-3	-	-
CR	0.135	0.156	0.129	0.125	0.186	0.083

Though the hydro chemical analysis result depicted (CR) value less than unity still the chemical constituents responsible to deterioration of concrete structures are dominant as compared to other constituents of water samples analyzed here. *****

CHAPTER - SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The present study was carried out at Megech dam project, an earth –rock fill embankment dam, located in Amhara National Regional State, in Northwestern Ethiopia about 740 km from Addis Ababa. The dam is founded on basaltic rock successions formed by different episodes of volcanism inferred from various intercalated paleo-soils. In the present study systematic geotechnical, engineering geological and hydro chemical studies were carried out to understand and assess the quality of rock mass, its deformability and the permeability potential of the dam foundation. Besides, bearing capacity and settlement potential of piles under the intake and conduit outlet works of the dam foundation was also assessed. In addition, effect of groundwater chemistry on submerged piles was also studied. In order to meet out the objectives of the present study both secondary and primary data was utilized.

The rock mass qualities and classifications of the dam foundation was assessed based on rock mass rating (RMR) of Bieniwski (1989). The results of rock mass classification depicted that the rock mass in general falls between poor and good rock mass classes, typically with 61% fair, 35% good and 4% poor rock mass classes. The rock mass classification results further suggests low quality rock mass on left abutment as compared to the right abutment. The discontinuity survey result at abutments showed high joint concentration and higher degree of weathering in the rock mass on the left abutment as compared to rock mass on the right abutment. Most of the joints as observed were open with aperture varying from 2 to 50cm and generally dips towards the river. The results on discontinuities were also in accordance with the drilled holes data from these sections.

Deformability characteristics of the rock mass was assessed empirically through Rock mass rating system and the results showed average modulus of deformation of 18.78, 8.25 and 25.12 MPa for rock mass on right abutment, left abutment and the riverbed section, respectively. Further, the estimated allowable bearing pressure for good and poor rock mass varies from 40 to 30 tone/m^2, respectively. Besides, quality of rock mass, as determined from rock quality designation (RQD) ranges from 9.2 to 98.5% for the rock mass on the left and right abutments, respectively.

The laboratory test results for overburden material overlying volcanic rocks, on both the abutments revealed material to be silty clay, clay; clay-silt silty sand and silty gravel soils. Significant thickness of such alluvial and colluvial deposits was also found along the central dam axis with highly decomposed rocks. The alluvial and colluvial deposits in this section generally range from medium to coarse gravel, to lose unconsolidated fine to medium grained sand with minor silty-clay deposit. The engineering properties of these soils suggest material to be stiff to hard in consistency and are water tight to permeable with respect to permeability. Further, paleo-soils that are intercalated with rocks have thickness as high as 3m in places located in the core portion of the right abutment. In the present study these soils were tested for the gain size, Atterberg limit and the specific gravity properties and these soils were found poorly graded sand soil for both soil samples.

The geotechnical drilling information presented through geological cross-sections generated along the dam axis and the outlet work structures depicted vertical and lateral alternations in the rock mass with respect to weathering grade and jointing intensity. The RQD, RMR and permeability values as recorded within boreholes confirm this variation in the rock mass in the foundation. The variation of degree of jointing, weathering grade and associated properties like permeability, RMR value and rock mass quality varied considerably with variable depths in dam foundation.

A number of permeability tests including lugeon and falling head were conducted on bedrocks and overburden materials, respectively in drilled holes along the dam axis by the project authorities. The results of permeability test for over burden material indicated that the overburden soils vary in permeability from water tight to permeable, as revealed from data from boreholes (BH-11, BH-12, BH-14 and BH-10). These permeability results are attributed due to relatively thick sand and gravel deposits in the river banks and the center portion of the dam foundation. Further, permeability tests in the rock mass underneath the overburden material revealed turbulent and laminar flow conditions with lugeon values ranging from less than one lugeon to as high as 85 lugeon, as observed in the left abutment. Besides, permeability generally rises in low quality rock mass, as observed in test sections 28 - 40 m in NB-1 bore hole. Likewise, permeability rises in between 30 - 35m test section in bore hole No along central /axis of the dam while permeability in right abutment in most of the test sections was recorded less than one lugeon, as right abutment have good quality rock mass in comparison to the left abutment. In general, permeability of the rock mass in the dam foundation was characterized based on the frequency distribution as; 78% impervious, 2.6%

low pervious ,12% moderately pervious, 1.6% pervious and 6.8 % as highly pervious. Although with regard to permeability most of the dam foundation is characterized as impervious class, however anomaly with high permeability was observed in boreholes. Particularly, high permeability was recorded in the left abutment in boreholes NB-1 and NB-5. Thus, to improve the permeability conditions there is a necessarily to provide grouting under the foundation. Extensive grouting is needed in the sections defined by bore holes NB-1, **NB-5** and top portions of sections defined by NB-10 and NB-**11.**

Empirical equations and direct pile load tests were used to determine the bearing capacity and settlement potential of the piles under conduit and intake structures. The results indicated that foundation rock mass under conduit and intake structures can carry more than the design load, as imposed by the piles. The bearing capacity as determined through pile load test suggests that piles can accommodate more than two times of the design load of the piles without undergoing any deflection and the settlement will also remain within the maximum tolerable limits. Likewise, bearing capacity estimated from empirical relation through **UCS** and **RQD** values showed that the rock mass at respective design depth can carry more than the design load, as imposed by the piles.

Hydro chemical data analysis performed to determine the possible effect of ground and surface water chemistry on the engineering performance of the submerged piles showed that none of the samples were corrosive. Therefore, submerged piles under conduit and intake structures will be safe against any possible chemical deterioration from surface or groundwater.

In general, based on the findings of the present study it can be conclude that the foundation conditions at the proposed dam site needs attention to improve the permeability conditions. The findings also showed higher degree of weathering and intense jointing in the rock mass on the left abutment as compared to the rock mass on the right abutment. Thus, left abutment needs improvement to strengthen the rock mass so that it may perform adequately for the safe performance of the dam. The foundation rock mass under conduit and intake structures is found to be suitable to accommodate the design load imposed by the piles. The bearing capacity of the foundation is sufficient and design load from the piles can be safely taken by the rock mass without undergoing any deflection and the settlement will also be within the safe limits. Further, piles under conduit and intake structures will be safe against possible chemical deterioration from surface or groundwater.

6.2 Recommendations

Based on the results and the findings of the present study following recommendations are forwarded;

- Engineering geological and geotechnical properties of the rock mass in the dam foundation; RMR, **RQD**, deformability, permeability and shear strength parameters as determined in the present study suggests relatively low engineering performance of the left abutment as compared to the right abutment. Based on the topography the dam is designed in such a way that a large part of the embankment will be resting on the left abutment. As mentioned, the weaker rock mass on left abutment may result into differential settlement with corresponding cracking in the embankment and concentrated seepage in the left abutment. Therefore, improvement of foundation rocks in the left abutment through grouting is recommended. To reduce permeability grout curtain in critical sections is necessary whereas at the same time to strengthen the rock mass, consolidation grouting in foundation is required.

- Not much information on paleosoils intercalated in between basaltic units was available. However, in the present study representative samples from paleosoils was tested. Based on the results this soil was characterized as sand soil. As per the field observations thickness of this soil was estimated to be about 3m at the core termination and along the conduit outlet in the right abutment. Thus, it is expected that these paleosoils may result into seepage and may reduce bearing capacity after reservoir impoundment. Hence, it is recommended that these paleosoils needs attention and proper strengthening measures.

- The overburden material present on both the abutments and in the river section was found to be stiff and hard in consistency. However, the permeability of this material, as observed in most of the drilled holes ranged from semi-permeable to permeable. Thus, there is a necessity to strip off this overburden material along with highly decomposed rocks just below this overburden martial, as observed in boreholes NB-1, NB-5 and NB-2.

- The bearing capacity as determined from pile load test suggests that piles can accommodate twice design load of the piles without undergoing any deflection and the settlement will also remain within the maximum tolerable limits. It was observed that load settlement curve did not reached maximum tolerable settlement up to twice the design load. Thus, it implies that load settlement curve may continue linearly with increasing load. Therefore, if ultimate bearing capacity has to be revised for possible anticipated loads imposed by impounded water it is recommended to use analytical method to extrapolate load settlement curve derived from existing pile load test.

- For the present study all efforts were made to carry out research in a systematic manner well supported by actual scientific data and realistic investigation. However, all these efforts were made under various limitations on time, resources and financial support. Further, recent pandemic crises due to Covid-19 have adversely affected the present research in the final stage which has restricted various resources. These constraints might have affected on the quality of the research result and certain component of inaccuracy may not be ruled out. Furthermore, it is strongly recommended that more detailed systematic study must be undertaken to came-up with more comprehensive and reliable research output before adopting and implementing the results of the present study.

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ANNEXURES

Annex-I: working pile load under conduit structure

Company Name: Ethiopian Construction Design & Supervision Works Corporations
Geotechnical Investigation, Geotechnical Engineering and Underground Construction Design & Supervision Works Sector

CDSMC

Pile Load Test Recording Sheet | Document No: OF/---------/----- | Issue No. | Page No.

Date 09/02/2019 G.c
Time 4:56 L.t (10:51)
- Outlet conduit pile
- Design load=4650 KN
- Pile SD= P10
- Pile Length = 31.9m
- Maximum load=9300
- Jack1 = 18013001
- Jack2: 18013002
S_1 = 65521
S_2 = 65522
S_3 = 65523
S_4: 65524

Time	Working load (4650) %	KN	Time (min) this class	Time (min) Cumulative	S1	S2	S3	S4	Average Settlement (mm) this class	Average Settlement (mm) Cumulative	Temp (°C)
10:51	0	0	0	0	0.00	0.00	0.00	0.00	0.00	0.00	24
10:57			5	5	-0.06	-0.24	-0.33	-0.17	-0.19	-0.19	24
11:02			10	10	-0.06	-0.24	-0.33	-0.17	0.00	-0.19	24
11:04	10	465	0	10	~~1.52~~	~~1.32~~	~~0.61~~	~~0.69~~	~~1.24~~ 0.00	~~1.05~~	24
11:09			5	15	1.52	1.38	0.61	0.69	1.24	1.05	24
11:14			10	20	1.54	1.41	0.62	0.71	0.02	1.07	24
11:15	20	930	0	20	1.54	1.41	0.62	0.71	0.00	1.07	24
11:20			5	25	2.32	2.17	1.28	1.44	0.73	1.80	24
11:2[illegible]			10	30	2.34	2.19	1.22	1.45	0.01	1.81	24
11:29	30	1395	0	30	2.34	2.19	1.28	1.45	0.01	1.81	24
11:34			5	35	2.32	2.80	1.84	2.14	0.59	2.40	24
11:39			10	40	2.33	2.81	1.86	2.16	0.02	2.42	24
11:39	40	1860	0	40	2.33	2.81	1.86	2.16	0.02	2.42	25
11:44			5	45	3.13	3.18	2.25	2.73	0.40	2.82	25
11:49			10	50	3.16	3.21	2.28	2.77	0.04	2.86	25
11:51	50	2325	0	50	3.16	3.21	2.28	2.77	0.04	2.86	25
11:56			5	55	3.56	3.69	2.70	3.40	0.48	3.34	25
12:01			10	60	3.59	3.72	2.72	3.42	0.02	3.36	25
12:01	60	2790	0	60	3.59	3.72	2.72	3.42	0.02	3.36	25
[illegible]:06			5	65	4.03	4.19	3.21	4.04	0.51	3.87	25
12:11			10	70	4.11	4.25	3.27	4.11	0.07	3.94	26
12:12	70	3255	0	70	4.11	4.25	3.27	4.11	0.07	3.94	26
12:17			5	75	4.49	4.72	3.74	4.75	0.48	4.42	26
12:22			10	80	4.57	4.80	3.82	4.84	0.09	4.51	26

Contractor: Name: [illegible]
Signature: ____
Date: ____

Consultant: Name: Girume A
Signature: [illegible]
Date: 09/02/19

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Company Name:
Ethiopian Construction Design & Supervision Works Corporations
Geotechnical Investigation, Geotechnical Engineering and Underground Construction Design & Supervision Works

Pile Load Test Recording Sheet	Document No: OF/---------/-----	Issue No. 1	Page No.

Project Name: Megech Dam Outlet Conduit Pile Foundation

Name of structure: Outlet conduit pile — Jack #: 18013061 — Pile Tope Elev.: ______

Design Load (KN): 4650 — Jack #: 18013007 — Pile Tip Elev.: ______

Maximum test load (KN): 9300 — Sensor S1: 65521 — Test Date: 09/02/2019 G.C

Pile Number: P10 (Working pile) — Sensor S2: 65522 — Cyclre: ______

Pile length (m): 31.9 — Sensor S3: 65523

Sensor S3: 65524

Time	Working Load (4650KN)		Observation Time (min)		Settlement Reading (mm)				Average settlement (mm)		Tempreture (0c)
	%	KN	This class	Comulat.	S1	S2	S3	S4	This class	Comulat.	
12:23	80	3720	0	80	4.57	4.80	3.82	4.24	0.09	4.51	26
12:28			5	85	5.09	5.13	4.36	5.60	0.54	5.05	26
12:33			10	90	5.16	5.19	4.42	5.67	0.06	5.11	26
12:33	90	4185	0	90	5.16	5.19	4.42	5.67	0.06	5.11	26
12:38			5	95	5.6	5.53	4.96	6.39	0.51	5.62	25
12:43			10	100	5.63	5.56	4.99	6.41	0.03	5.65	25
12:44	100	4650	0	100	5.63	5.56	4.99	6.41	0.03	5.65	25
12:49			5	105	6.10	5.94	5.54	7.10	0.52	6.17	26
12:54			10	110	6.14	5.99	5.58	7.14	0.04	6.21	26
12:59			15	115	6.2	6.06	5.66	7.22	0.08	6.29	26
13:14			30	130	6.28	6.13	5.74	7.30	0.07	6.36	26
13:44			60	160	6.32	6.16	5.77	7.34	0.04	6.4	26
13:47	75	3487.5	0	160	6.32	6.17	5.77	7.35	-0.00	6.41	26
13:52			5	165	5.97	6.02	5.33	6.74	-0.38	6.02	26
13:57			10	170	5.96	6.00	5.33	6.73	-0.01	6.01	26
13:59	50	2325	0	170	5.95	6.00	5.33	6.73	-0.01	6.00	26
14:04			5	175	5.32	5.91	4.88	6.04	-0.46	5.54	26
14:09			10	180	5.30	5.88	4.85	6.01	-0.03	5.51	26
14:10	25	1162.5	0	180	5.30	5.88	4.85	6.01	0.00	5.51	26
14:15			5	185	4.50	5.22	3.83	4.60	-0.97	4.54	26
14:20			10	190	4.47	5.18	3.80	4.59	-0.03	4.51	26
14:22	0	0	0	190	4.47	5.18	3.80	4.58	0.00	4.51	26
14:27			5	195	2.05	1.99	0.93	1.03	-3.01	1.5	26
14:32			10	200	2.05	1.99	0.92	1.02	0.00	1.5	26
14:37			15	205	2.04	1.99	0.91	1.01	-0.01	1.49	26
14:52			30	220	2.02	1.99	0.9	0.99	-0.01	1.48	27

Contractor: Name: [illegible] — Consultant: Name: Grumic

Signature: ______ — Signature: [signature]

Date: ______ — Date: 09/02/19

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CD2MC

Company Name:

Ethiopian Construction Design & Supervision Works Corporations

Geotechnical Investigation, Geotechnical Engineering and Underground Construction Design & Supervision Works Sector

Pile Load Test Recording Sheet	Document No: OF/---------/-----	Issue No. 1	Page No.

Project Name: Megech Dam Outlet Conduit Pile Foundation

Name of structure: Outlet conduit pile

Design Load (KN): 4650

Maximum test load (KN): 9300

Pile Number: P10 (Working pile)

Pile length (m): 31.9

Jack #: 18013001

Jack #: 18013002

Sensor S1: 65521

Sensor S2: 65522

Sensor S3: 65523

Sensor S3: 65524

Pile Tope Elev.: ______

Pile Tip Elev.: ______

Test Date: 09/02/2019 G.C

Cyclre: ______

Time	Working Load (4650KN)		Observation Time (min)		Settlement Reading (mm)				Average settlement (mm)		Tempreture (0c)
	%	KN	This class	Comulat.	S1	S2	S3	S4	This class	Comulat.	
14:53	25	1162.5	0	220	2.01	1.98	0.89	0.98	-0.01	1.47	27
14:58			5	225	3.87	4.66	3.15	3.53	2.33	3.8	27
15:03			10	230	3.89	4.68	3.17	3.56	0.02	3.82	27
15:04	50	2325	0	230	3.89	4.68	3.17	3.56	0.00	3.82	27
15:09			5	235	4.66	5.51	4.15	4.98	1.00	4.82	27
15:14			10	240	4.67	5.51	4.17	5.0	0.02	4.84	27
15:19	75	3487.5	0	240	4.67	5.51	4.17	5.	0.00	4.84	27
15:28			5	245	5.56	6.12	5.05	6.27	0.91	5.75	27
15:19			10	250	5.58	6.13	5.07	6.30	0.03	5.78	27
15:26	100	4650	0	250	5.59	6.14	5.08	6.31	0.85	6.63	26
15:31			5	255	6.38	6.65	5.97	7.51	0.00	6.63	26
15:36			10	260	6.42	6.69	6.0	7.54	0.03	6.66	26
15:36	110	5115	0	260	6.42	6.69	6.0	7.55	0.01	6.67	26
15:41			5	265	6.74	6.98	6.44	8.11	0.24	7.07	26
15:46			10	270	6.78	7.03	6.47	8.16	0.11	7.11	26
15:46	120	5580	0	270	6.78	7.03	6.48	8.16	0.00	7.11	26
15:51			5	275	7.10	7.41	6.99	8.73	0.45	7.56	25
15:56			10	280	7.16	7.49	7.05	8.59	0.06	7.62	25
15:59	130	6045	0	280	7.19	7.52	7.08	8.83	0.04	7.66	25
16:04			5	285	7.54	7.94	7.59	9.38	0.45	8.11	25
16:09			10	290	7.58	7.99	7.63	9.42	0.05	8.16	25
16:09	140	6510	0	290	7.59	8.00	7.63	9.42	0.00	8.16	25
16:14			5	295	7.98	8.52	8.20	10.01	0.52	8.68	25
16:19			10	300	8.03	8.59	8.25	10.06	0.05	8.73	25

Contractor: Name: [illegible]

Signature: ______

Date: ______

Consultant: Name: Gyumie A

Signature: ______

Date: 09/02/19

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Company Name: Ethiopian Construction Design & Supervision Works Corporations
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pile Load Test Recording Sheet	Document No: OF/---------/-----	Issue No. 1	Page No.

Project Name: Megech Dam Outlet Conduit Pile Foundation

Name of structure: Outlet conduit pile — Jack #: 18013001 — Pile Tope Elev.: ______

Design Load (KN): 4650 — Jack #: 18013002 — Pile Tip Elev.: ______

Maximum test load (KN): 9300 — Sensor S1: 65521 — Test Date: 09/02/2019 G.C

Pile Number: P10 (Working pile) — Sensor S2: 65522 — Cyclre: ______

Pile length (m): 31.9 — Sensor S3: 65523

Sensor S3: 65524

Time	Working Load (4650KN)		Observation Time (min)		Settlement Reading (mm)				Average settlement (mm)		Tempreture (0c)
	%	KN	This class	Comulat.	S1	S2	S3	S4	This class	Comulat.	
16:19	150	6975	0	300	8.03	8.59	8.25	10.66	0.00	8.73	26
16:24			5	305	8.46	9.13	8.84	10.63	0.54	9.27	26
16:29			10	310	8.52	9.23	8.90	10.70	0.08	9.35	26
16:31	160	7440	0	310	8.53	9.24	8.91	10.71	0.00	9.35	26
16:36			5	315	8.95	9.77	9.44	11.25	0.50	9.85	26
16:41			10	320	9.02	9.87	9.51	11.32	0.08	9.93	26
16:42	170	7905	0	320	9.02	9.87	9.52	11.32	0.00	9.93	26
16:47			5	325	9.47	10.44	10.07	11.84	0.53	10.46	26
16:52			10	330	9.55	10.52	10.14	11.91	0.07	10.53	26
16:54	180	8370	0	330	9.59	10.57	10.18	11.95	0.04	10.57	26
16:59			5	335	10.05	11.13	10.67	12.44	0.50	11.07	26
17:04			10	340	10:15	11:24	10.77	12:53	0.10	11.17	26
17:04	190	8835	0	340	10.16	11.24	10.78	12.53	0.01	11.18	26
17:09			5	345	10.63	11.81	11.27	13.03	0.51	11.69	26
17:14			10	350	10.74	11.92	11.34	13.10	0.09	11.78	26
17:15	200	9300	0	350	10.74	11.93	11.37	13.12	0.01	11.79	26
17:20			5	355	11.23	12.49	11.85	13.64	0.51	12.30	26
17:25			10	360	11.33	12.61	11.96	13.74	0.11	12.41	26
17:30			15	365	11.40	12.68	12.02	13.80	0.07	12.48	26
17:49			30	380	11.51	12.80	12.14	13.92	0.11	12.59	25
18:15			60	410	11.59	12.88	12.23	14.01	0.09	12.68	25

Contractor: Name: [illegible]

Signature: ______

Date: ______

Consultant: Name: Grumie A

Signature: ______

Date: 09/02/19

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Campany Name:

Ethioplan Construction Design & Supervision Works Corporations
Geotechnical Investigation, Geotechnical Engineering and Underground Construction Design & Supervision Works Sector

Pile Load Test Recording Sheet	Document No: OF/--------/----- 1	Issue No. 1	Page No.

Project Name: Megech Dam Outlet Conduit Pile Foundation

Name of structure: Outlet conduit pile — Jack #: 12013001 — Pile Tope Elev.: _______

Design Load (KN): 4650 — Jack #: 12013002 — Pile Tip Elev.: _______

Maximum test load (KN): 9300 — Sensor S1: 65521 — Test Date: 09/02/2019 G.C

Pile Number: P10 (Working pile) — Sensor S2: 65522 — Cyclre: _______

Pile length (m): 31.9 — Sensor S3: 65523

Sensor S3: 65524

Time	Working Load (4650KN)		Observation Time (min)		Settlement Reading (mm)				Average settlement (mm)		Tempreture (0c)
	%	KN	This class	Comulat.	S1	S2	S3	S4	This class	Comulat.	
18:19	150	6975	0	410	11.59	12.88	12.23	14.00	0.60	12.68	25
18:24			5	415	10.70	11.93	11.37	13.17	-0.88	11.80	25
18:29			10	420	10.69	11.92	11.39	13.17	-0.01	11.79	25
18:31	100	4650	0	420	10.69	11.92	11.39	13.16	0.00	11.79	25
18:36			5	425	8.98	10.40	9.72	11.23	-1.71	10.08	25
18:41			10	430	8.97	10.41	9.72	11.23	0.00	10.08	25
18:41	50	2325	0	430	8.97	10.41	9.72	11.23	0.00	10.08	25
18:46			5	435	7.39	9.04	7.40	8.43	-2.01	8.07	25
18:51			10	440	7.36	9.00	7.36	8.40	-0.04	8.03	25
18:52	0	0	0	440	7.36	9.00	7.36	8.40	0.00	8.03	25
18:57			5	445	3.74	4.8	2.13	1.70	-5.06	3.09	25
19:02			10	450	3.69	4.76	2.07	1.63	-0.05	3.04	25
19:07			15	455	3.65	4.73	2.04	1.59	-0.04	3.00	25
19:22			30	470	3.60	4.67	1.99	1.54	-1.95	2.95	

Contractor: Name: [illegible]
Signature: ___________
Date: ___________

Consultant: Name: Gyumie A
Signature: [illegible]
Date: 09/02/19

Generated by CamScanner from intsig.com

Annex: II working load test under intake structure

Campany Name:

Ethiopian Construction Design & Supervision Works Corporations
Geotechnical Investigation, Geotechnical Engineering and Underground Construction Design & Supervision Works Sector

pile Load Test Recording Sheet | Document No:OF/GIGEUCDSWS/----- | Issue No. 1 | Page No. 1

Project Name: Megech Dam intake tower pile foundation

Name of structure: Intake Tower | Jack #: ____________ | Plile Tope Elev.: _______

Design Load (KN): 3750 | Jack #: __________ | Pile Tip Elev.:_________

Maximum test load (KN):7500 | Sensor S1: __________ | Test Date: ____________

pile Number: E-4 | Sensor S2: __________ | Cycle: ______________

Pile length (m): 29.2 | Sensor S3: __________

Sensor S3: __________

Time	Working Load (3750KN)		Observation Time (min)		Settlement Reading (mm)				Average settlement (mm)		
	%	KN	This class	Comulat.	S1	S2	S3	S4	This class	Comulat.	Tempreture (0c)
			0	0				80.00			32
	0	0	5	5	80.00	80.00	80.00	80.00	80.00		32
			10	10					0	0.00	32
		375	0	10	78.76	79.39	79.65	79.56	0.66	0.66	32
	10	375	5	15	78.36	79.39	79.70	79.56	0.31	0.97	32
		375	10	20	78.35	79.38	79.69	79.54	0.32	1.29	32
		750	0	20	78.34	79.20	79.43	79.27	0.48	1.77	32
	20	750	5	25	78.32	79.17	79.41	79.27	0.23	2.00	32
		750	10	30	78.31	79.17	79.41	79.17	0.25	2.25	32
		1125	0	30	77.98	78.97	79.30	79.17	0.31	2.57	33
	30	1125	5	35	77.95	78.97	79.29	79.05	0.35	2.92	33
		1125	10	40	77.96	78.95	79.28	79.05	0.24	3.16	33
		1500	0	40	77.66	78.73	79.14	78.89	0.45	3.60	33
	40	1500	5	45	77.72	78.64	79.13	78.88	0.30	3.90	34
		1500	10	50	78.68	78.60	79.09	78.84	0.08	3.98	34
		1875	0	50	77.36	78.50	78.96	78.69	0.46	4.44	34
	50	1875	5	55	77.35	78.47	78.93	78.67	0.33	4.78	34
		1875	10	60	77.31	78.44	78.91	78.64	0.34	5.12	34
		2250	0	60	77.08	78.25	78.76	78.47	0.50	5.62	35
	60	2250	5	65	77.04	78.24	78.42	78.44	0.44	6.06	35
		2250	10	70	77.03	78.20	78.71	78.42	0.35	6.41	35
		2625	0	70	76.83	78.01	78.51	78.26	0.52	6.92	36
	70	2625	5	75	76.79	77.97	78.50	78.23	0.39	7.31	36
		2625	10	80	76.77	77.96	78.49	78.23	0.37	7.68	36
		3000	0	80	76.76	77.56	78.27	78.05	0.57	8.25	36
	80	3000	5	85	76.73	77.53	78.23	78.01	0.42	8.68	36
		3000	10	90	76.70	77.51	78.20	77.80	0.46	9.13	36

Continue ……………

		3375	0	90	76.31	77.5	78	77.76	0.41	9.54	36
	90	3375	5	95	76.26	77.47	77.94	77.74	0.41	9.95	36
		3375	10	100	76.23	77.44	77.92	77.58	0.45	10.40	36
		3750	0	100	76.06	77.26	77.71	77.58	0.43	10.82	37
		3750	5	105	76.03	77.23	77.68	77.55	0.46	11.28	37
		3750	10	110	76.03	77.23	77.78	77.55	0.40	11.68	37
	100	3750	15	115	76.04	77.23	77.7	77.56	0.42	12.10	37
		3750	30	130	75.98	77.16	77.64	77.5	0.49	12.59	37
		3750	60	160	75.95	77.12	77.61	77.48	0.46	13.05	37
		3750	90	190	75.95	77.12	77.64	77.48	0.43	13.48	37
		3750	120	220	75.93	77.09	77.59	77.44	0.47	13.95	37
		2812.5	0	160	76.16	77.32	77.88	77.66	0.19	14.14	37
	75	2812.5	5	165	76.18	77.34	77.9	77.67	0.39	14.52	37
		2812.5	10	170	76.24	77.4	77.95	77.75	0.34	14.86	37
		1875	0	170	76.7	77.85	78.42	78.13	-0.03	14.83	37
	50	1875	5	175	76.7	77.86	78.42	78.15	0.35	15.18	37
		1875	10	180	76.76	77.91	78.47	78.19	0.32	15.50	37
		937.5	0	180	77.41	78.51	78.98	78.66	-0.20	15.30	37
	25	937.5	5	185	77.45	78.56	79.02	78.7	0.23	15.53	37
		937.5	10	190	77.47	78.58	79.04	78.72	0.25	15.77	37
		0	0	190	79.71	79.77	79.44	79.7	-0.94	14.84	37
		0	5	195	80.26	80.09	79.42	79.39	-0.09	14.75	37
	0	0	10	200	80.28	80.11	79.42	79.39	-0.41	14.34	37
		0	15	205	80.29	80.12	79.42	79.4	-0.42	13.92	37
		0	30	220	80.33	80.16	79.45	79.43	-0.44	13.48	37
		937.5	0	220	78.1	78.99	79.17	78.99	0.62	14.10	32
	25	937.5	5	225	78.08	78.97	79.18	78.96	0.19	14.29	32
		937.5	10	230	78.07	78.97	79.18	78.98	0.16	14.45	32
		1875	0	230	77.33	78.41	78.84	78.59	0.69	15.13	32
	50	1875	5	235	77.31	78.4	78.83	78.57	0.31	15.45	32
		1875	10	240	77.31	78.4	78.83	78.57	0.29	15.74	32
		2812.5	0	240	76.78	77.91	78.38	78.15	0.76	16.50	32
	75	2812.5	5	245	76.76	77.88	78.36	78.13	0.37	16.87	32
		2812.5	10	250	76.75	77.8	78.36	78.13	0.37	17.24	32
		3750	0	250	76.12	77.45	78.9	78.73	0.33	17.57	32
	100	3750	5	255	76.12	77.4	78.87	78.71	0.95	18.53	32
		3750	10	260	76.1	77.4	77.86	78.7	1.19	19.72	32
		4125	0	260	76.12	77.24	78.69	78.55	1.05	20.77	32

Continue …………………………………….

	110	4125	5	265	76.12	77.24	78.69	78.55	0.90	21.67	32
		4125	10	270	76.1	77.22	78.67	78.53	0.92	22.59	32
		4500	0	270	75.93	77.02	77.5	77.37	1.58	24.17	29
	120	4500	5	275	75.87	76.94	77.45	77.31	0.48	24.64	29
		4500	10	280	75.86	76.93	77.44	77.3	0.43	25.07	29
		4875	0	280	75.7	76.75	77.28	77.13	0.58	25.66	29
	130	4875	5	285	75.66	76.69	77.24	77.07	0.47	26.12	29
		4875	10	290	75.64	76.67	77.22	77.06	0.42	26.54	29
		5250	0	290	75.49	76.46	77.07	76.89	0.58	27.13	29
	140	5250	5	295	75.43	76.4	77.02	76.83	0.47	27.60	28
		5250	10	300	75.41	76.38	77.01	76.81	0.43	28.02	28
		5625	0	300	75.25	76.2	76.84	76.64	0.58	28.60	27
	150	5625	5	305	75.18	76.11	76.78	76.57	0.48	29.08	27
		5625	10	310	75.16	76.07	76.76	76.54	0.44	29.52	27
		6000	0	310	74.98	75.79	76.58	76.36	0.61	30.13	27
	160	6000	5	315	74.9	75.76	76.5	76.29	0.50	30.63	27
		6000	10	320	74.88	75.57	76.49	76.27	0.49	31.12	27
		6375	0	320	74.68	75.57	76.29	76.09	0.61	31.73	27
	170	6375	5	325	74.6	75.46	76.23	76.01	0.52	32.24	26
		6375	10	330	74.57	75.2	76.21	75.97	0.52	32.77	26
		6750	0	330	74.39	75.24	76.03	75.79	0.61	33.37	26
	180	6750	5	335	74.31	75.16	75.95	75.73	0.50	33.88	26
		6750	10	340	74.27	75.11	75.93	75.69	0.48	34.36	26
		7125	0	340	74.07	74.91	75.74	75.52	0.63	34.99	26
	190	7125	5	345	74	74.8	75.67	75.43	0.54	35.53	25
		7125	10	350	73.96	74.77	75.63	75.4	0.49	36.02	25
		7500	0	350	73.78	74.75	75.45	75.22	0.60	36.62	25
		7500	5	355	73.65	74.45	75.31	75.08	0.60	37.22	25
	200	7500	10	360	73.61	74.41	75.27	75.03	0.50	37.72	25
		7500	15	365	73.57	74.36	75.24	75	0.49	38.21	25
		7500	30	380	73.52	74.3	75.2	74.95	0.51	38.71	25
		7500	60	410	73.48	74.23	75.11	74.9	0.52	39.23	25
		5625	0	410	74.03	74.79	75.77	75.45	-0.11	39.12	24
	150	5625	5	415	74.04	74.8	75.78	75.46	0.43	39.55	24
		5625	10	420	74.05	74.8	75.78	75.47	0.43	39.99	24
		3750	0	420	75.02	75.78	76.78	76.41	-0.53	39.46	24
	100	3750	5	425	75.03	75.8	76.76	76.42	0.41	39.87	23
		3750	10	430	75.03	75.81	76.76	76.42	0.41	40.28	23

Annex-III: UCS test result

Company Name:
በኢትዮጵያ የኮንስትራክሽን ዲዛይንና ሱፐርቪዥን ሥራዎች ኮርፖሬሽን
የምርምር ላቦራቶሪ እና የሥልጠና ማዕከል
Ethiopian Construction Design &Supervision Works Corporation
Research Laboratory & Training Center Laboratory Test Process

Tel.:0116 614501/0116-610105 Fax: +251-116-615371/610898, e-mail/wsdse@ethionet.et Po.Box 2561

Title:	Material Testing Report	OF/RTLC/603	Issue	Page No.: Page 1of 1

CODE: 5743/2019 Client Ref No.: ==========
SUBMITTED BY: Zhongmei Engineering Group LTD
PROJECT: Megech Dam Construction Project
SAMPLE OF: Core Rock
DATE SAMPLED: 17/05/2019
STATION/LOCATION: As stated below
TEST REQUESTED: Compressive Strength
REPORTED TO: Zhongmei Engineering Group LTD

Reported On: 16/07/2019

Marking	BH ID	Depth	ID	Dimension - m		Unit Weight, Kg/m³	Compressive Strength	
				D	H		Mpa	PSI
1	MDC-BH 10	19 - 28	Z1.1	0.070	0.140	1566	2.86	414
2	MDC-BH 10	19 - 28	Z1.2	0.070	0.140	1536	0.00	0
3	MDC-BH 10	19 - 28	Z1.3	0.070	0.140	1549	2.42	350
4	MDC-BH 10	28 - 35	Z3.1	0.070	0.140	2487	24.42	3541
5	MDC-BH 10	28 - 35	Z3.2	0.070	0.140	2086	16.63	2411
6	MDC-BH 10	28 - 36	Z3.3	0.070	0.140	2206	18.71	2712

Remark: The Samples were Collected and Submitted to the laboratory by the client.

Reported by: Checked by: Approved by:
Date : 16-07-19 Date : 16/7/19 Date : 16/7/19
Behayilu Kebede, Material Engineer — Tirunesh Yirga, Senior Material Engineer — Rediet Gashaw, Material Lab.S/P Manager

Among the major services rendered by the Geotechnical and Material Laboratory Testing S/processes of Ethiopian Construction Design & supervision Works Corporation are:-
·In Geotechnical Laboratory:- Testing the engineering properties of Soil Mechanics and Rock Mechanics
·In material Testing Laboratory: - Cements Rocks, Water, and Reinforcement Steel Bars. Hollow blokes, Bricks, Ceramics, Tiles., Asphalt and Concrete Core Tests, Concrete Mix Designs, Asphalt Mix Designs, sampling of the Soil and Construction Materials and so on.

BH-ID	Core run		UCS(Mpa)	BH-ID	Core run		UCS (mpa)
	From	To			From	To	
MDC-BH-5	19	19.8	4.2	**MDC-BH-6**	1.8	2.9	0.41
	19.8	21.3	4.2		2.9	4	0.41
	21.3	22.2	4.2		4	4.7	0.41
	22.2	22.6	4.2		4.7	6	0.41
	22.6	24.2	4.2		6	6.8	0.41
	24.2	25.7	4.2		6.8	7.9	0.41
	25.7	26.4	10.5		7.9	9.8	0.41
	26.4	27.8	10.5		9.8	11	0.41
	27.8	30.1	10.5		11	11.3	0.41

	30.1	31.1	4.6		11.3	13.5	0.41
	31.1	33.1	30.8		13.5	15.8	0.41
	33.1	34.2	30.8		15.8	19	0.41
	34.2	37	30.8		19	22.7	0.41
	37	37.5	30.8		22.7	23.6	0.41
	37.5	39.2	7.9		23.6	25.8	0.41
	39.2	40	7.9		25.8	28.1	6.44
					28.1	30.3	41
					30.3	32.3	41
					32.3	35.5	41
					35.5	36.3	11.36

Annex –IV:Paleosoil–laboratory result A. specific gravity-test-result

		Pit-1			
pychnometer	**Weight of bottle**	**Weight of soil**	**Weight of pychnometer+H_2o**	**Wpws**	
45	45.53	25	102.86	117.32	
42	42.8	25	99.32	115.13	
Gs=2.54					
3.5	43.51	25	100.63	115.6	Pit-2
3	46.34	25	96.96	11.34	
Gs=2.43					

B.hydrometer test result

Pit-1	**G1** =1.00	**Hydrometer 151H**	**Temp.°c =**	25				
	Gs=	2.54	**K =**	0.01408				
Time (min)	RHS	RHC	Temp. °c	RHC-1	R	Effective Length, L (cm)	Particle Diameter , (mm)	%finer
2	1.009	1.002	25	0.002	1.007	13.9	0.0371	23.09
5	1.008	1.002	25	0.002	1.0060	14.2	0.0237	19.79
15	1.006	1.002	25	0.002	1.004	14.7	0.0139	13.19
30	1.005	1.002	25	0.002	1.003	15	0.0100	9.90
60	1.004	1.002	25	0.002	1.002	15.2	0.0071	6.60
250	1.004	1.002	25	0.002	1.002	15.2	0.0035	6.60
Pit-2								
2	1.009	1.002	25	0.002	1.007	13.9	0.0371	23.09
5	1.008	1.002	25	0.002	1.0060	14.2	0.0237	19.79
15	1.006	1.002	25	0.002	1.004	14.7	0.0139	13.19
30	1.005	1.002	25	0.002	1.003	15	0.0100	9.90
60	1.004	1.002	25	0.002	1.002	15.2	0.0071	6.60
250	1.004	1.002	25	0.002	1.002	15.2	0.0035	6.60
1440	1.003	1.002	25	0.002	1.001	15.5	0.0015	3.30
RHS=actual hydrometer reading ,RHC=hhydrometer reading at suspension ,RHC-1=composit correction and R=correcte hydrometer reading								

C. wet seive test result

Pit-1	Weight of soil before was=1500gram			Weight of soil after wash=1186 gram		
Seive size(mm)	seive.Wt(gm)	seive+ soil(gm)	Wt retained(gm)	% retained	comm.%retained	% of pass
2.36	337.0	337	0	0.00	0.00	100.00
2	320.0	336	16	1.35	1.35	98.65
1.18	280.0	292.0	12	1.01	2.36	97.64

0.6	265.0	738	473	39.88	42.24	57.76
0.425	377.0	444	67	5.65	47.89	52.11
0.3	262.0	395.0	133	11.20	59.09	40.91
0.15	265.0	572	307	25.88	84.97	15.03
0.075	253.0	382	129	10.87	95.84	4.16
0	315.0	358	43	3.63	99.47	0.53
		Sum of retained	**1180.0 gram**			
pit -2	Wb	2077 gram		Wf	1616 gram	
Seive-size(mm)	seive.Wt(gm)	seive+ soil(gm)	Wt retained(gm)	% retained	comm.%retained	% of pass
2.36	337.0	337	0	0.00	0.00	100.00
2	320.0	331	11	0.68	0.68	99.32
1.18	280.0	303.0	23	1.42	2.10	97.90
0.6	265.0	955	690	42.70	44.80	55.20
0.425	377.0	718	341	21.00	65.80	34.20
0.3	262.0	369.0	107	6.60	72.40	27.60
0.15	265.0	585	320	19.80	92.20	7.80
0.075	253.0	348	95	5.87	98.07	1.93
0	315.0	344	29	1.80	99.87	0.13
		Sum of retained	**1616.0 gram**			

D. atterberg limit test result(plasticity zero)

Pit -1	Can no.	Can(weight)	Can+wet soil(weight)	Can+dry soil(weight)	penetration	LL=77.6 %
	930	36.79	97.6	72.23	15.5	
	999	36.08	97.91	71.1	19	
	821	36.2	97.79	70.1	21.9	PL=0
	568	35.91	97.78	69.41	25	
Pit-2						69.8%
	A-1	36.88	90.18	68.38	15.5	
	T	37.25	90.69	69	19.5	
	23	36.63	92.76	69.53	21.5	PL=0
	A-3	36.34	90.47	67.05	25.2	

Annex-V

A. Classification parameters and their ratings in the RMR system

	PARAMETER		Range of values // RATINGS				
1	Strength of intact rock material	Point-load strength index	> 10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range, uniaxial compr. strength is preferred
		Uniaxial compressive strength	> 250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa; 1 - 5 MPa; < 1 MPa
		RATING	**15**	**12**	**7**	**4**	**2**; **1**; **0**
2	Drill core quality RQD		90 - 100%	75 - 90%	50 - 75%	25 - 50%	< 25%
		RATING	**20**	**17**	**13**	**8**	**5**
3	Spacing of discontinuities		> 2 m	0.6 - 2 m	200 - 600 mm	60 - 200 mm	< 60 mm
		RATING	**20**	**15**	**10**	**8**	**5**
4	Condition of discon-tinuities	a. Length, persistence	< 1 m	1 - 3 m	3 - 10 m	10 - 20 m	> 20 m
		Rating	**6**	**4**	**2**	**1**	**0**
		b. Separation	none	< 0.1 mm	0.1 - 1 mm	1 - 5 mm	> 5 mm
		Rating	**6**	**5**	**4**	**1**	**0**
		c. Roughness	very rough	rough	slightly rough	smooth	slickensided
		Rating	**6**	**5**	**3**	**1**	**0**
		d. Infilling (gouge)	none	Hard filling		Soft filling	
			-	< 5 mm	> 5 mm	< 5 mm	> 5 mm
		Rating	**6**	**4**	**2**	**2**	**0**
		e. Weathering	unweathered	slightly w.	moderately w.	highly w.	decomposed
		Rating	**6**	**5**	**3**	**1**	**0**
5	Ground water	Inflow per 10 m tunnel length	none	< 10 litres/min	10 - 25 litres/min	25 - 125 litres/min	> 125 litres /min
		$p_w / \sigma 1$	0	0 - 0.1	0.1 - 0.2	0.2 - 0.5	> 0.5
		General conditions	completely dry	damp	wet	dripping	flowing
		RATING	**15**	**10**	**7**	**4**	**0**

p_w = joint water pressure; $\sigma 1$ = major principal stress

B. RMR rating adjustment for discontinuity orientations

		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
RATINGS	Tunnels	**0**	**-2**	**-5**	**-10**	**-12**
	Foundations	**0**	**-2**	**-7**	**-15**	**-25**
	Slopes	**0**	**-5**	**-25**	**-50**	**-60**

C. Rock mass classes determined from total RMR ratings

Rating	100 - 81	80 - 61	60 - 41	40 - 21	< 20
Class No.	I	II	III	IV	V
Description	VERY GOOD	GOOD	FAIR	POOR	VERY POOR

D. Meaning of ground classes

Class No.	I	II	III	IV	V
Average stand-up time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span
Cohesion of the rock mass	> 400 kPa	300 - 400 kPa	200 - 300 kPa	100 - 200 kPa	< 100 kPa
Friction angle of the rock mass	< 45°	35 - 45°	25 - 35°	15 - 25°	< 15°