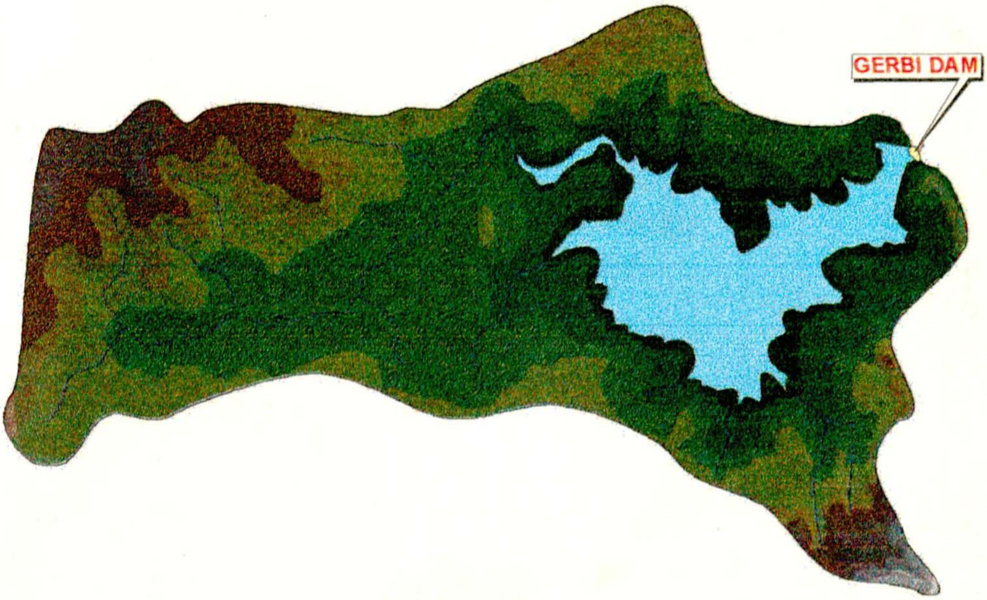
ENGINEERING GEOLOGICAL CHARACTERIZATION OF GERBI DAM SITE, RESERVOIR AND CATCHMENT AREA



In Partial Fulfillment for the Degree of
Master of Science in Geology
(Engineering Geology)

By

REDIET GASHAW

June 2004

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Addis Ababa University
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Abstract

Constructions of dams require the input of thorough engineering geological investigation. To fully understand and characterize the soils and rocks of the study area a detailed investigation has been carried out taking into account many disciplines such as geomorphological, geological, hydrogeological and geophysical investigations.

Gerbi dam project envisages construction of a 19 m high earth dam across Gerbi river between villages Daka and Dibdibe, defined by longitudes 38^0 35'-38^0 50'E and latitudes 9^0 00'- 9^0 20'N. The elevation at which the dam site is situated ranges between 2630m and 2672 m. The proposed Gerbi dam has a crest length and width of 250m and 9m, respectively. The storage volume of the dam is 49.88 million cubic meter and maximum flood level is at 2659.7m and the full supply level is at 2658 m (AAWSA, 1997).Gerbi catchment covers 82 square kilometer.

Aerial photos (taken in December, 1971 and January, 1993), topographic maps and detailed field investigation has been utilized to produce slope map, land use land cover map, soil map and drainage map of the study area at a scale of 1:50,000.

Geological maps of the whole catchment and the dam site at a scale of 1:50,000 and 1:1000, respectively were produced to clearly describe the geological set up of the area. Gerbi dam site is located within Shoa plateau, which is comprised of Tertiary volcanic rocks. Gerbi dam site is comprised of volcanic rock belonging to Ashangi Group of the trap series. This Group consists of alkaline basalts with interbedded pyroclastics and trachytes. The volcanic rocks of the area are overlain by Quaternary soils of residual, alluvial and colluvial types.

Basalts (usually in boulder form) are commonly found with in the basin. Extensive trachyte outcrops are found near the dam site. Except few basaltic and trachyte outcrops the remaining area of the catchment is covered by clays of alluvial, residual and colluvial origin.

Areal photos at a scale of 1:8000 and 1:50000 are used to map the structural features of the basin. The Lineament analyses made on aerial photographs of the basin indicate that most of the structures are continuous trending in NE-SW, E-W and N-S directions.

Only two outcrops are present in the vicinity of the dam site and all the joint measurements are taken on these outcrops. Joints trending in NW-SE and NE-SW are the dominant joint sets exposed on these outcrops.

Hydrometerlogical data of Gerbi catchment is collected from National Meteorological Services Agency (NMSA) of three nearby stations; Addis Ababa Bole, Chancho and Sululta stations have been used to determine the hydrological parameters.

Results from water quality analysis are also analyzed to determine the quality of water for drinking purpose and the effect of water on the engineering structure. From results of the chemical analysis conducted on water from Gerbi river some results (TDS, total alkalinity, total hardness, Nitrate and Ammonia, Chloride, Fluoride, Copper, Sodium)are below the WHO's guidelines and some results(Iron, Manganese, Phosphate) are above the WHO's guidlines. It is therefore important to treat the water (physically and chemically) to make the result match with the WHO's quality requirement.

Soils of the study area are thoroughly investigated. Detailed field investigation has been carried out to classify and characterize the soils. The soils are grouped into broad categories based on their origin. The Unified soil classification system (USCS) is used to further classify soils of the study area. Laboratory analysis and field investigation have been carried out, different empirical approaches have also been used to determine the physical, index and engineering properties of soils of the study area.

Basalts and trachytes are the dominant rock types that are found throughout the catchment. Pycroclastic deposits are also found at depth. Data from boreholes and the few available outcrops has been used to classify and characterize rocks of the study area. Different empirical methods, laboratory test results and softwares are utilized to define the engineering properties of the rocks.

Engineering Geological maps for the dam site and reservoir are then produced from the results of the above mentioned methods and analysis at a scale of 1:1500 and 1:50,000 for the dam site and reservoir respectively.

Assessment of the construction materials required for the construction of the dam has been conducted to make sure the materials are adequately found with in a reasonable distance from the dam site and the quality of the materials meets the quality requirement.

Stability analysis utilizing SARC software has also been conducted to assess the stability of the reservoir rim and the abutments. The right abutment provides stable slope but the left abutment has factor of safety below the acceptable value.

Suggestions based on the results of the present study are also presented in this work.

CHAPTER- ONE

Introduction

Plans and designs of many civil engineering projects require the knowledge of properties of soils and rocks. Engineering geological investigation serves as a basis to plan and design a sustainable engineering structure. The investigation will generate sufficient information to eliminate major uncertainties and provides information about the foundation condition, suitability and availability of construction materials, slope stability of the abutments and reservoir rim, groundwater condition, etc.

Constructions of dam (a major engineering structure) require the input of thorough engineering geological investigation.

Dam is a barrier constructed to keep back water for storage purposes. Every dam should accomplish the following objectives under all anticipated loading conditions:

- hold back or store water safely
- contain the water and resist leakage
- maintain its shape and configuration
- resist movement in any direction
- safely pass maximum design flood events

1.1 Justification

Water is a fundamental natural resource. The expansion of Addis Ababa and the rapid population growth has resulted in shortage of water. The demand for water has increased in the city. It has become very difficult to satisfy the growing demand from Gafersa and Lega-Dadi reservoirs. Hence the city should be provided with sufficient water from other sources.

This requires construction of water harvesting structures such as dams. The construction of a dam requires extensive geological and engineering geological investigation before planning, design and construction.

The Addis Ababa Water and Sewerage Authority (AAWSA) came up with a large project to alleviate the problem. Gerbi dam site is chosen with an aim to store and convey raw water to the Weserbi (located North of Entoto hill) treatment plant prior to distribution in Addis Ababa. This research will provide information about the engineering geological characteristics of rocks and soils of the proposed dam site, reservoir and catchment area.

1.2 Objectives

A, General objectives

-Engineering geological appraisal of the dam site

-Water tightness study for the reservoir area

-To assess the quality and availability of the construction materials

B, Specific Objectives

- To carry out geological mapping of the dam site on 1:1000 scale

-To determine the engineering properties of the rocks and soils at the dam site

-To carry out stability studies for the dam abutment

-To study the permeability properties of the foundation rock

-To assess hydrological and hydro-geological conditions of the proposed dam site

-To determine quality of both surface and ground water to know the corrosive property for mortar

-To provide engineering geological maps of the dam site at a scale of 1:1000 and for the reservoir on 1:50000

1.3 Methodology

In order to achieve the above said objectives the following methodology has been adopted.

-Collecting all information from published and unpublished literature

-Collecting, analyzing and interpreting hydrometerological data

-Interpretation of aerial photographs and satellite imageries

-Procuring topographical map and preparing geologic and engineering geologic maps for dam site and reservoir area

-Field study to map the area and collect soil and rock samples (water sample for quality analysis was taken; Schmidt hammer test has been done)

-In situ tests to evaluate the engineering properties of surface and subsurface materials

-Stability analysis for abutments for existing and possible worst conditions by using computer programs such as SARC, DIPS.

-Laboratory analysis to determine engineering geological and geo-technical properties of the soil and rock has been conducted.

-Soil map, land use, geological and engineering geological maps were produced from existing data and the filed data.

1.4 Previous Works

In 1991, Messrs. SEURECA of France finalized a 10 volume Feasibility Study and Preliminary Design, indicating additional sources of water, both surface and groundwater, to meet the needs of the Addis Ababa Metropolitan Area (AAMA) to the year 2020. Dams at Sibilu and Gerbi were recommended. However the study also proposed further hydrological and hydrogeological studies to increase the statistical data, and thereby improve the estimation of yield of each source. Recommendations were also made to start land acquisition steps for water supply areas contemplated for various development needs: dams, reservoirs, treatment plant, pumping stations, transmissions mains, etc.

In 1994, AAWSA initiated a follow up study meant to continue the source research and prepare detailed design and tender documents for Project implementation. The latter Project started in May 1995 and was entrusted to a Joint Venture of Messrs. Associated Engineers (AE) and HBT AGRA both of Canada. These consultants prepared studies mainly on surface water resources recommended by the earlier feasibility study, i.e., Gerbi and Sibilu dams, about 30 km north of Addis Ababa on the other side of the Entoto Hills, and on groundwater supply in the southern part of the town from the Akaki aquifer. This Contract was terminated in November 1998 due to contractual problem.

At present, AAWSA selected a new consultant, TAHAL CONSULTING ENGINEERS LTD. having sub-consultants Messrs. SMEC/WWDSE/HYWAS to complete the design and prepare tender documents.

Many researchers and the Ethiopian Institute of Geological Survey have made lots of geological works to explain the regional geology of the area. Mohr (1971), Zanettin et al (1973, 1974, 1977), Kazmin et al (1975), Morton et al (1979), and Haile Sellase Girmay and Getaneh Assefa (1989) are some.

1.5 Location and Accessibility

The dam site is located 30 kms North West of Addis Ababa with in the vast land of Sululta plain. The dam site is located North West of Addis Ababa branching to the left at about 12 Kms on the way to Sululta and found after traveling nearly 20 kms on the flat grazing land. It is situated about 20 kms off the road branching to the left at Wosserbi village found about 12 kms away from the capital on the way to Sululta. It lies between longitudes 38^0 35'-38^0 50'E and latitudes 9^0 00'- 9^0 20'N in the Ethiopian Mapping Agency topo-sheet number 0938 D3. The location map is presented in Figure 1.

The river rises from the Entoto range and meander through out Sululta plain. The elevation at which the dam site is situated ranges between 2630m and 2672 m.

The area is characterized by vast grassland and localized hills. The reservoir of Gerbi dam is found on Shoa plateau in the upper part of Muger catchment in the Blue Nile basin. The dam site is located in the area of Awasso Farmers Association, between the villages of Daka and Dibdibe located North West and South East of the dam site respectively

There is an asphalt road up to Woserbi. Accessibility from the main road up to the dam site is good; the local people have made a road to allow cars that come to pick the hay from the area. But during rainy seasons it is impossible to reach to the dam site by car.

FIG. 1.1 Location map of Gerbi basin

1.6 Physiographic Setting

Physiography is "physical geography, the study of the natural features of the earth's surface, including landforms, oceans, seas, soils, atmosphere, and the distribution of fauna and flora" (Webster's Dictionary 1990).

Gerbi dam site lies in the central part of the Ethiopian plateau. The area is nearly flat except the fewer scattered hills and ridges present in the study area. It is surrounded by Entoto rhyolitic and basaltic ridges.

Vegetation is sparse consisting mainly of Eucalyptus and Acacia not to mention fewer scattered bushes and other plant varieties along the mountainside of Entoto ridge. The flat area of the catchment is mainly a pastureland besides crops with scattered trees are also found. The Gerbi reservoir covers the flat lying areas used mainly for grazing and small-scale farming; hay is also grown on the flat land by the local people for sale.

CHAPTER-TWO

Climate and Geomorphology of Gerbi Catchment

2.1 Climate

The climate of Ethiopia ranges from desert to hot and cool steppe, from tropical savanah and rain forest to warm temperate, and from hot low land to cool high lands (Tenalem Aynew and Tamiru Alemyehu, 2001). As part of the Ethiopian highlands, the Gerbi catchment is characterized by warm temperature and high rainfall with higher altitudes. Gerbi area has similar climatological characteristics as that of Addis Ababa.

Hydrometrological data such as rainfall, temperature, humidity, etc of three stations (Chancho, Sululta and Addis Ababa Bole stations) is collected to represent the climatological characteristics of the area.

According to the results of the analysis, the annual rainfall of Gerbi catchment is calculated to be 1153.5 mm. The average mean monthly minimum temperature and the mean average monthly maximum temperatures are 3.8^0C and 21.6^0C respectively. Based on the above results it can be concluded that the area has humid tropical climatic conditions.

2.2 Geomorphologic Characteristics

The study of geomorphology is based on the principle that all land forms can be related to a particular geologic process or sets of processes. The different landforms of the study area are results of volcanism, tectonism and structure that are later modified by weathering processes. The extent to which a regional landscape is controlled by volcanism depends on:

-The nature of the extruded materials (basic lavas tend to spread over larger areas, producing landforms with more subdued slopes than do silicic lavas;

-The distribution of vents and fissures;

-The volume of outpourings,

-The duration of volcanism,

-The age(s) of volcanic activity relative to the present and to associated stratigraphic units; and

-The intensity and stage of subsequent erosional activity.

In some regions, volcanic outpourings were confined to a limited time period, leading to flows that cap older non-volcanic units. The resistance of such volcanic rocks to erosion strongly influences the subsequent history of landscape development as streams penetrate into the underlying more erodible bedrock, causing a distinctive assemblage of lava- capped hills and mesas. Likewise, lava flows and/or thick tephra deposits that accumulate over larger areas may partially to completely bury preexistent topography.

Volcanoes develop from extrusion or expulsion of fluids, congealed fragments, and gases that collect or distribute at or near the Earth's surface to produce a variety of forms, chief of which are conical, often mountain-like structures, thin to thick piles of flow sequences, and sheets of air fall deposits of tephra (fragmented particles of volcanic material). The forms are controlled in part by the mode or types of volcanic activity. This ranges from quiet emission to explosive ejection, depending mainly on the gas content and viscosity (related to composition) of the initial magma and resultant lava.

The geomorphologic characteristics of the study area have been interpreted from aerial photographs, topographic maps and field investigation. Gerbi catchment is part of the Ethiopian central plateau consisting of plains covered by red clays with few ridges. Flat to undulating topography dominate in the area. The elevation at which the dam site is situated ranges between 2630m and 2672 m the lowest being at the riverbed at the dam site.

The different landforms of the study area are results of volcanism, tectonism and structure that are later modified by weathering processes. The area is bounded by East West trending Entoto ridge (built by the accumulation of flow lavas and sheets of airfall deposits (ash) (Lulseged Ayalew, 1990) and an assemblage of mountain-like structures (hills).

The north western and upper south eastern portion of the dam site is covered by boulders of basalt intermingled with red clay (this formation goes up to the bottom of the hill on the north western part and up to the upper portion of the trachyte unit on the south eastern portion). Highly weathered to slightly weathered trachyte underlie the basalt. It can be concluded that successive lava flows are responsible for the formation of such volcanic terrain.

Flood basalt plateaus and plains cover most of the area. While generally of low relief after the final eruptive emplacement of lava onto a thickening pile, the terrain is affected by later erosion that produced notable relief characterized by localized hills and cliffs.

Most volcanic forms do not experience significant weathering until the activity causing them has either ceased or been long dormant. Likewise, lava flows of the study area generally did not experience significant weathering or removal until after the last flows or ejects has covered them.

Not to mention, the weathering processes, slope movements (such as landslide, fall, and flow) further modified the landscape of the area. Generally, vast plains covered dominantly by Quaternary sediments (alluvial and residual soils) dominate throughout the area. Small relatively steep hills are found scattered with in the catchment. Colluvial soils are found at the foot of almost every hill. The south eastern and southern portions (near the boundary of the catchment) are characterized by high forested cliffs.

2.3 Relief and Landforms

Different landforms have developed with in the study area owing to both the petrogenic mechanisms and modifying geomorphic processes. The forms are controlled in part by the mode or types of volcanic activity and the resistance of such volcanic rocks to erosion.

2.3.1 Slope Map

The steepness of the ground surface can be represented on a map in several ways. The most familiar and widely used is the conventional topographic map, on which points of equal elevation above sea level are joined by contour lines. The most easily visualized and explicit representation of the steepness of the ground is the slope map.

Preparation of slope maps for local areas is made possible by modern computers and GIS software (Arc View) and the expanding availability of high-quality digital elevation data. Slope maps are relevant to land-use planning, in addition slope maps are of great importance for the overall assessment of landslide hazard potential areas and slope instability.

In hilly terrain, it is often the slope or inclination that plays the dominant role in land use planning and slope stability analysis. In a slope map, it is not the elevation of an area that is displayed, as in conventional topographic contour maps, but rather the inclination of the ground surface to the horizontal. It can be expressed as the percent rise in elevation along a given horizontal distance. [A 10 percent slope rises 10 feet over a horizontal distance of 100 feet (This percentage is actually the trigonometric tangent of the vertical slope angle)].

The area is classified into different slope classes based on the observed slope characteristics from topographic maps and field observations. The slope maps prepared, display four classes (from Class 1 up to 4) in the increasing order of slope angle range. The land form classification is according to UNESCO-FAO (1997) guideline for slope and slope profile. The classification is given in Appendix E.

1. Flat land

This portion comprises land forms with slopes from 0 to 2%.This portion is extensive and covers 52.38 % of the total catchment area. It covers the entire reservoir area in the central part and some portions in the north eastern part. This section is bounded by gently slopping land in every direction and is bisected by tributaries of Gerbi river. Gerbi river meanders across this flat land.

Thick alluvial, residual and at some portions colluvial soils characterize this section. The elevation ranges from 2640 to 2700m.

2. Gently slopping to undulating Land

This portion comprises land forms with slopes from 2 to 8% .It covers 33.3 % of the entire catchment area. It comprises the north western and south eastern portions of the basin. Gentle slopes and hills characterize this portion of the basin. The elevation for this portion of the slope class ranges from 2720 - 2780m.

3. Rolling

This portion comprises land forms with slopes from 8 to 15%. Its aerial coverage is 4.76 % of the total area of the basin. It comprises both the abutments where the body of the dam rests. The right abutment (facing downstream) has relatively lower slope than the left portion. The elevation of this land form varies from 2680 to 2740m.

4. Moderately steep to very steep land

The slope of this class is greater than 15 %. It covers 9.52 % of the total catchment area. This landform covers a very small aerial proportion and is found on the southeastern parts of the basin beyond the gently slopping land.

The elevation ranges from 2680 – 2880m.

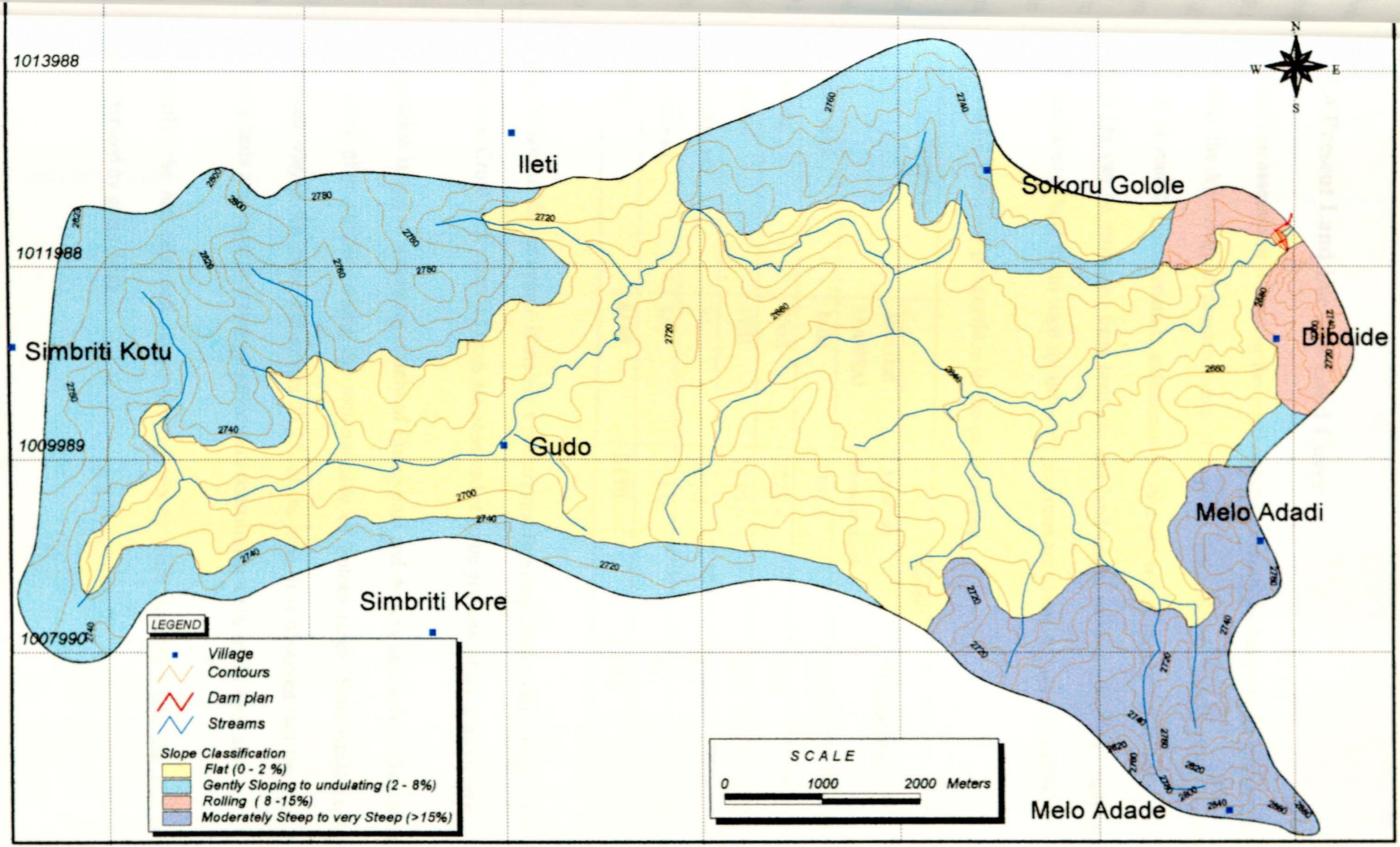


Fig. 2.1 Slope Map of Gerbi Catchment

2.4 Present Land Use and Land Cover

Land use assessment indicating the relationships between the land and its use can be used to assess the hydrological characteristics of a given basin. Some hydrological characteristics such as runoff characteristics, sedimentation rate etc which have relation with the existing land use can be estimated from land use and land cover assessment of the study area. The proportions of land forms used for different purposes are given in Table 2.1 below.

Table 2.1 Land use proportions in the project area

Land use	*Gerbi*		
	Area (ha) (AAWSA)	**Area(ha) Present study**	**% proportion**
Cultivated	2468	2705.8	33.2
Grazing	3555	3227.4	39.6
Forested	1731	1336.6	16.4
Waterlogged	54	505.3	6.2
Homestead	341	374.9	4.6
Total	**8149**	**8150**	**100**

The flat area of the catchment is mainly a pastureland (grazing land) constituting 39.6% of the total area. Crops with scattered trees are observed at some portions of the grazing land.

Vegetation is sparse consisting mainly of Eucalyptus and Acacia and fewer scattered bushes and other plant varieties along the mountainside of Entoto ridge. Small-scale farming is practiced with in the basin and covers 33.2% of the area. It is observed that the majority of housing units are found in scattered settlements constituting 4.6% of the total area.

Generally, the hill tops and ridges support woody plant species while the flat area is characterized by grass lands.

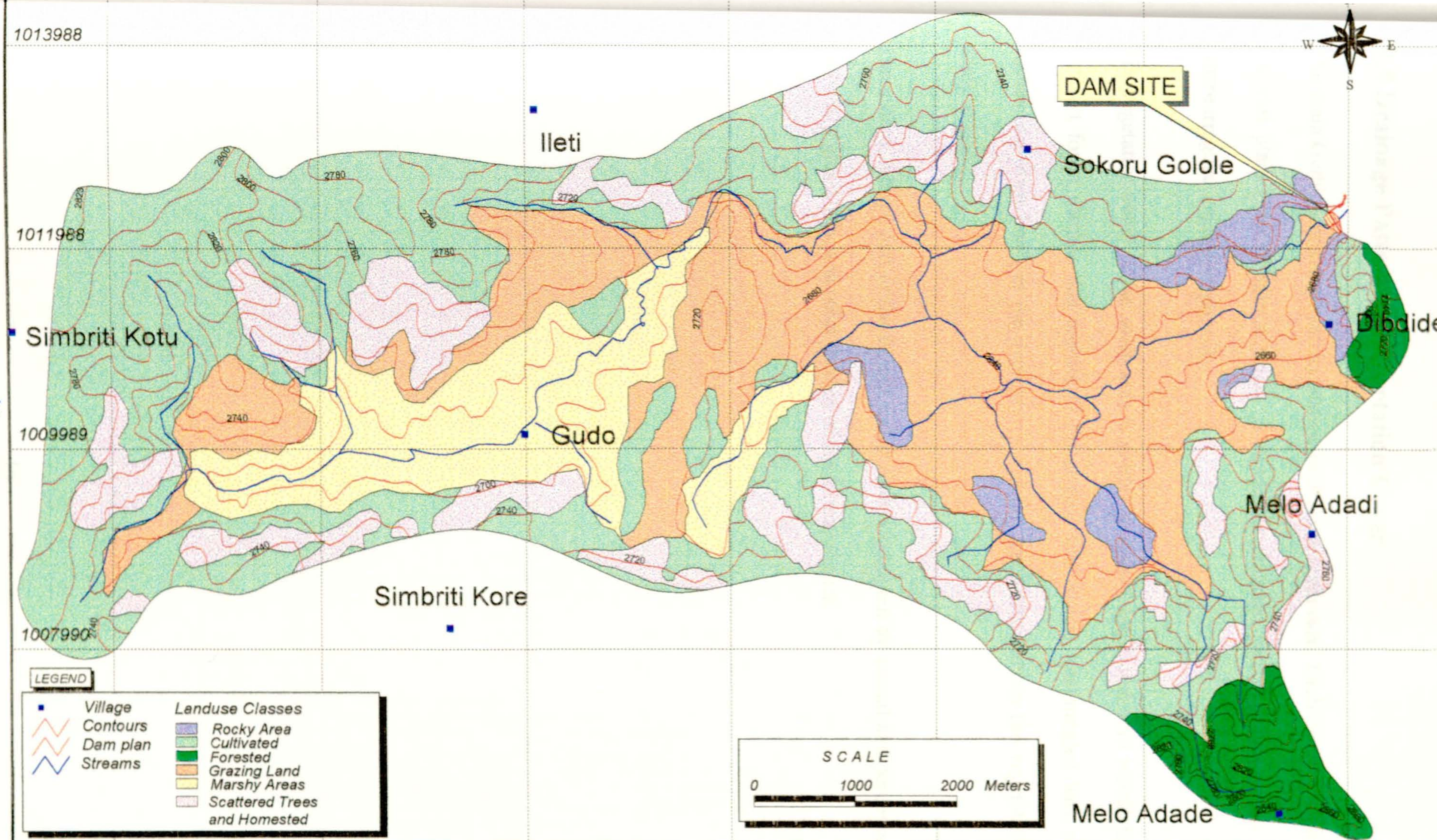


Fig. 2.2 Land use Land cover map of Gerbi catchment

2.5 Drainage Pattern and Vegetation Cover

The main Gerbi river along with its tributaries drain to north east. Gerbi river meanders along the vast plain (Part of sululta plain). A denderitic drainage pattern characterizes the area where irregular landforms prevail.

The vegetation cover of the study area is mainly grassland associated with woodland. Some remnant forest patches that consist of Eucalyptus and Acacia intermingled with scattered bushes and grass lands in the depressions and flat lands characterize the vegetation of the study areas.

These days, a large part of the former forest zone is occupied by annual and perennial crops, secondary grasslands, and relict patches of the former forests.

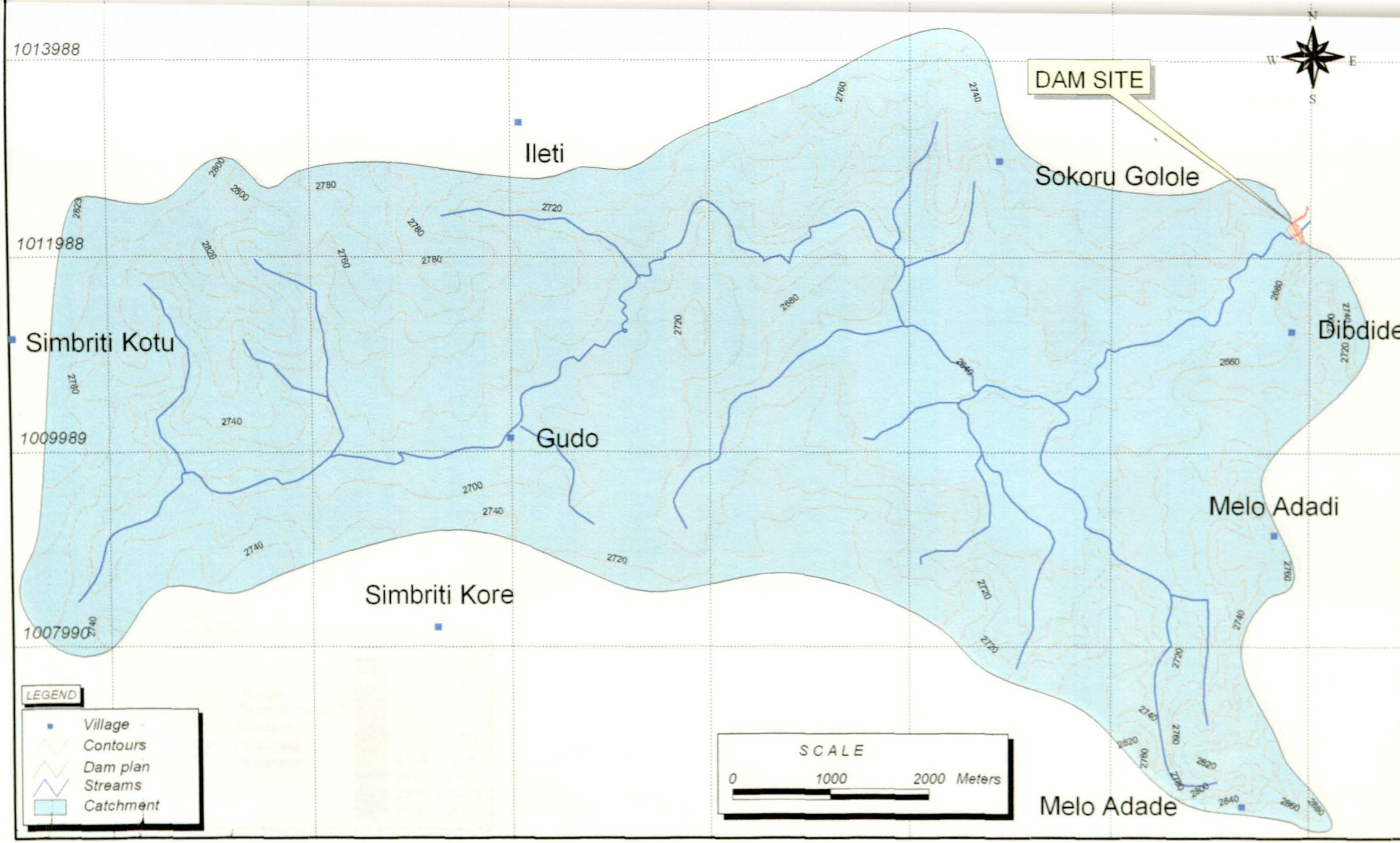


Fig. 2.3 Drainage pattern of Gerbi catchment

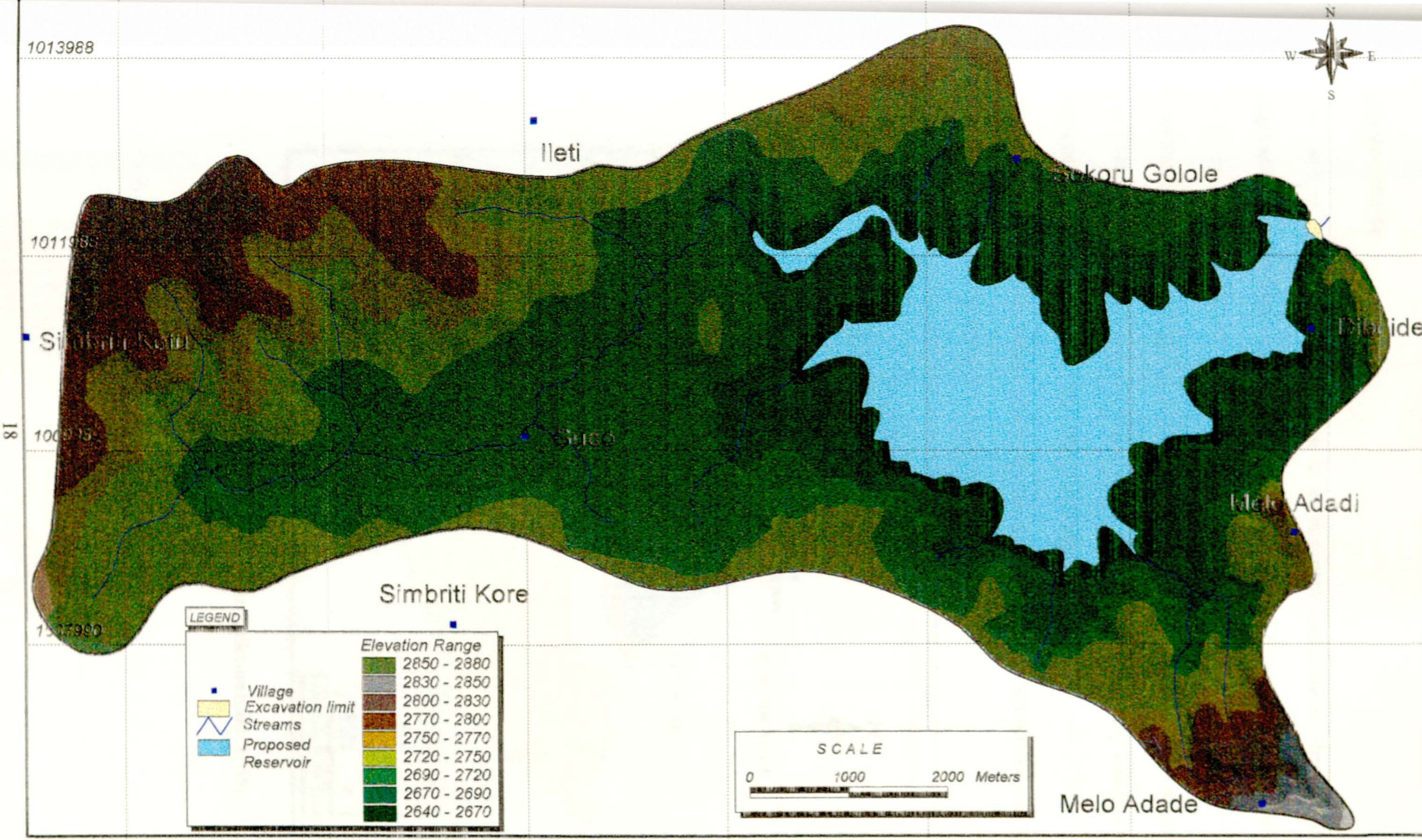


Fig. 2.4 Triangular Irregular Network (TIN) Model of Gerbi River Catchment

2.5 Seismicity

The study of seismicity of a given area is an important consideration when constructing engineering structures since earthquake forces may cause significant damage and distruction on the engineering structure.

Based on the historical records and assessment of the regional and local geology Laikemariam has produced seismic risk map of Ethiopia. According to this map the study area lies with in the highest risk zone which is related to the main Ethiopian rift.

The design of the dam therefore should take into consideration the high seismic risk and the dam to be constructed has to be earthquake resistant.

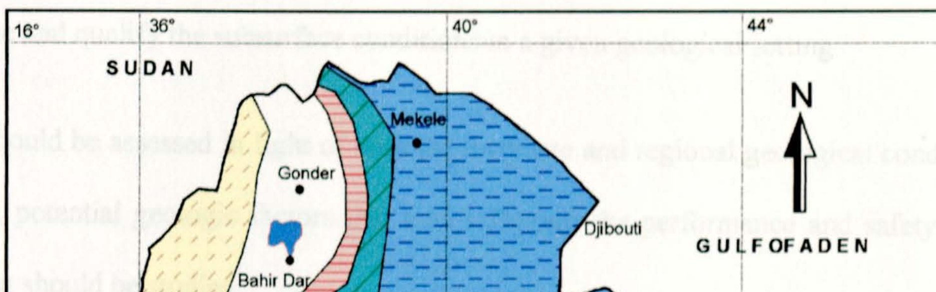


Figure 2.5 Seismic risk map of Ethiopia (After Laikemariam. 1988)

CHAPTER-THREE

Geology

3.1 Preamble

The study and description of the geological setting is very important when designing dams, or when trying to troubleshoot problems or safety deficiencies. The geological features and conditions of the site should be fully understood to better assess problems and deficiencies of the dam site. Site-specific information obtained from a geotechnical exploration program will better define and qualify the subsurface conditions in a given geological setting.

All dams should be assessed in light of both the local site and regional geological conditions. In addition, potential geologic factors that may influence the performance and safety of an existing dam should be studied.

3.2 Regional Geology

The proposed project (Gerbi catchment) is underlain at great depth by Archean (Early Precambrian) gneisses of the Alghe Group intruded by younger granites and syenites. These basement rocks occur to the north-west in the Abay River valley and to the south-east (south and east of the Ethiopian Rift Valley) (TAHAL, 2003).

Successions of sedimentary rocks uncomformably overlie (at an elevation of about 1,000 m) the Archean rocks. The sedimentary rock successions are described below.

The oldest of these sedimentary successions is the Adigrat Formation which occurs in the Abay and Muger gorges and to the south of the Rift Valley (TAHAL, 2003).

These rocks unconformably overlain by Abay Formation. The Abay Formation is of Middle Jurassic Age and is comprised of limestones, calcareous sandstones, shale and gypsum.

The Abay Formation is in turn overlain by the Antolo Formation which is of Middle to Late Jurassic Age (165 to 135 Ma).It is comprised of oolitic, detrial micro grained limestone and marls.

Amba Aradom Formation (of Late Cretaceous Age (135 to 90 Ma)) conformably overlay the Antalo limestones. It is comprised of sandstones, shale and marl.

The Abay, Antolo and Amba Aradom Formations all occur in the Abay and Muger gorges and almost certainly underlie the project area. The top of these sedimentary rocks can reach up to an elevation of about 2,100 m (TAHAL, 2003).

Following a period of uplift during the Jurassic and Cretaceous, the earliest flood basalts (Cenozoic volcanic rocks) associated with this uplift and associated faulting covered the above sedimentary rocks.

Gerbi dam site is located within Shoa plateau, which is comprised of Tertiary volcanic rocks.

On a regional scale the Cenozoic volcanic rocks of the study area comprise:

-Volcanic rocks of the Trap series of early and middle Tertiary age and

-Younger volcanic rocks

Volcanic rocks of the Trap series include Ashangi group and Shield group. Volcanic rocks younger than rocks of the trap series include Magdala and Afar group.

The shield group is Miocene in age. This group is comprised of alkali olivine basalt, tuff and agglomerates.

I. Ashangi Group

Consists predominantly of alkaline basalts with interbedded pyroclastics and rare rhyolites erupted from fissures. They are intruded by dolerite sills, acidic dykes and gabbro-diabase intrusions. The Ashangi group has a Paleocene to Miocene age range (Kazmin, 1975).

II. Magdala Group

Magdala Group is abundant with in the Ethiopian rift and on the adjoining plateau. Acidic rocks including acidic tuffs, ignembrites, rhyolites and trachytes are commonly found. These rocks are interbeded with lavas and agglomerates of basaltic composition. This group is suggested to be upper Pliocene in age (Kazmin, 1975).

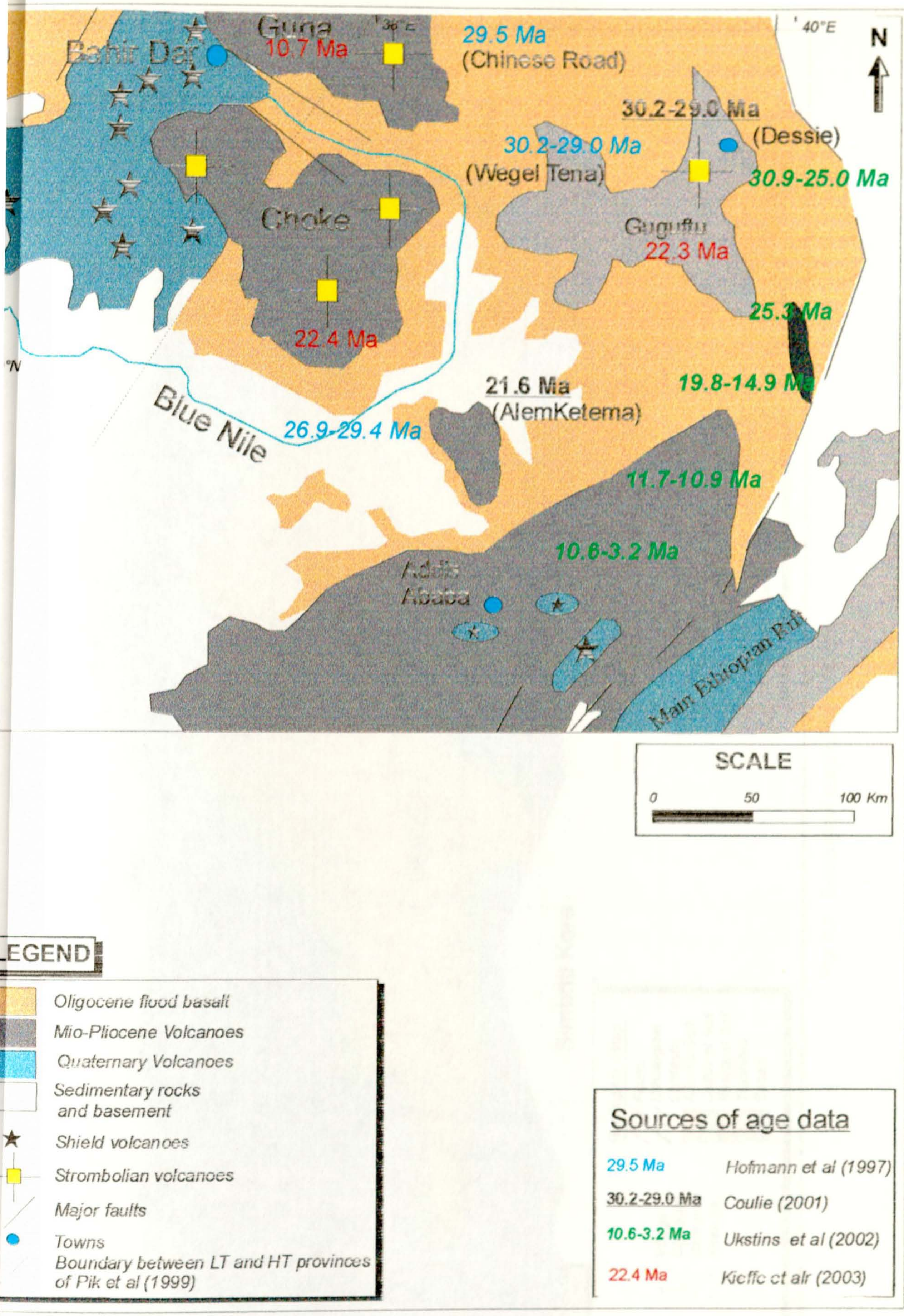
III. Afar Group

The Afar Group is comprised of basalts which are Miocene to Pliocene in age. Silicic rocks occur throughout the sequence. The Afar basalts are alkaline and resemble those of the Ashangi Group (Mohr, 1971; Kazmin, 1975).

According to a recent study by Kieffer et al (2004), the study area lies with in the Mio-Pliocene volcanoes. According to Hofmann et al. (1997), most of the Ethiopian flood basalts erupted 30 Myr ago, during a short 1 Myr period, to form a vast volcanic plateau volcanic plateau. After this activity, a number of large shield volcanoes developed on the surface of the volcanic plateau, after which subsequent volcanism was largely confined to regions of rifting (Mohr and Zanettin, 1988). The lava flows of the shield volcanoes are thinner and less continuous than the underlying flood basalts. These volcanoes contain sequences of alternating basalts, rhyolitic and trachytic lava flows, tuffs and ignimbrites.

According to Kieffer et al (2004) there is no evidence of deformation to distinguish a lower deformed formation (Ashangi) and upper undeformed formation (Aiba) as the type used in Meral et al (1979) and Berhe et al (1987) used in other parts of the plateau. Hence Kieffer et al (2004) avoided the formation names Ashangi and Aiba and speak only of the upper and lower flood basalt formations.

The geologic set up of the study area afte Kieffer et al (2004) is given in figure 3.1.



g 3.1 The regional geological setting of the study area (After Kieffer et al, 2003)

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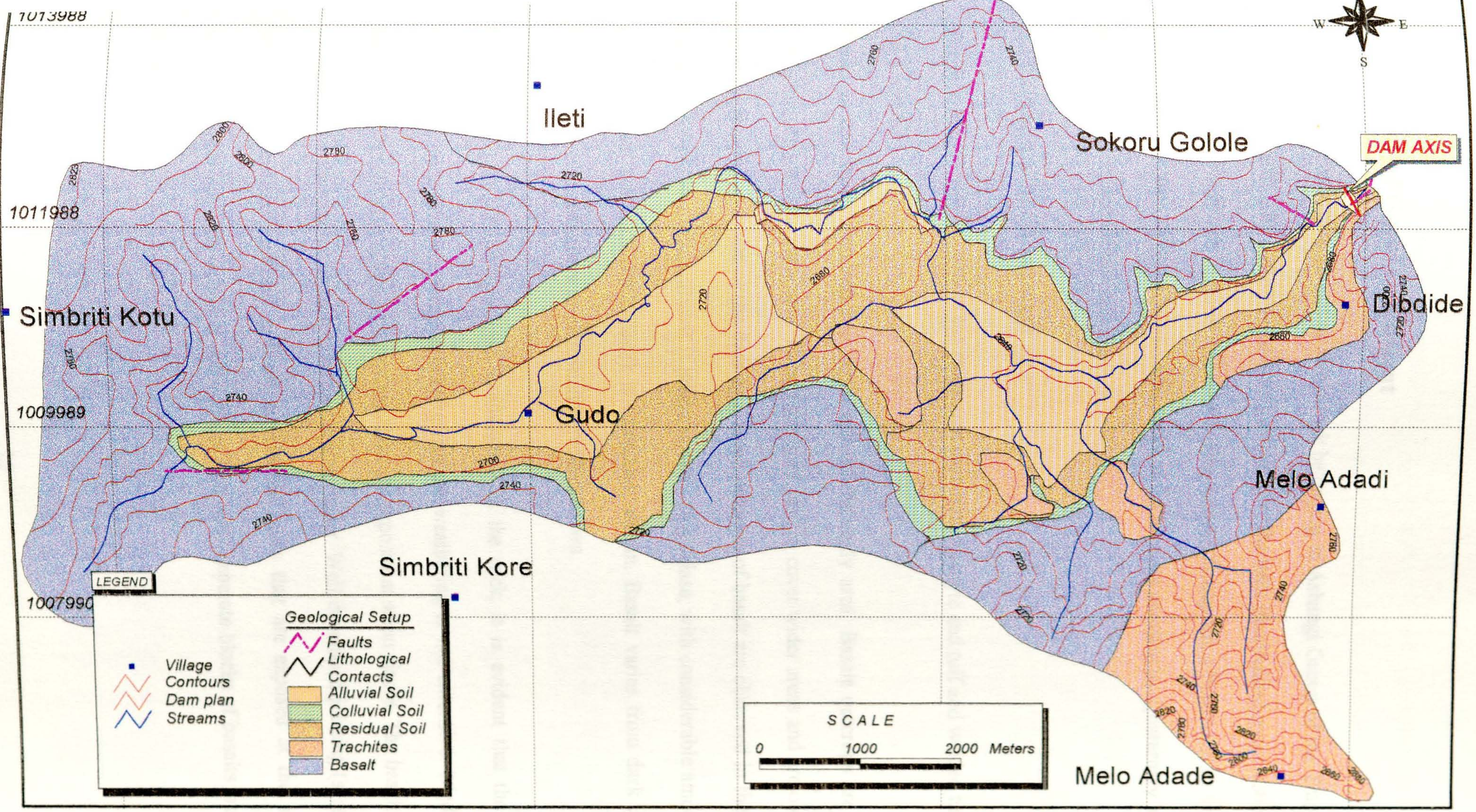


Fig 3.2 Geological map of Gerbi basin

3.3 Geology of Gerbi Catchment

Gerbi dam site is comprised of volcanic rock belonging to Ashangi Group of the Trap series (AAWSA, 1996). This Group consists of alkaline basalts with interbedded pyroclastics. The upper part of this Group is more tuffaceous. The Ashangi Group has a Paleocene to Miocene age range (Kazmin, 1975). The volcanic rocks of the area are overlain by Quaternary soils of residual, alluvial and colluvial types.

3.4 Lithology of Gerbi dam site

The lithological successions of the study area are basalt, trachyte and tuff and welded tuff.

3.4.1 Basalt

Basalt is the most abundant rock type with in the study area. Basalt outcrops are found overlying the trachyte rock unit. These rock units tend to cover wider areas and are observed more on the western part of the dam site. Small outcrops of basalt are observed in the North Eastern part of Gerbi dam. Basalt is predominantly plagioclase, with considerable amounts of pyroxene, and some olivine. Quartz is completely absent. Basalt varies from dark gray to black in color, but weathers to yellowish to reddish-brown.

From outcrop and hand specimen examination of the rock, it is evident that the basalt observed is dark grey to black indicating that the weathering is not extensive. Medium to highly weathered basalts having rusty red color are exposed at some places. The basalt is fine grained and aphanitic in texture. The basalt is vesicular. Well developed jointing (systematic and non-systematic jointing) prevails on basaltic rocks that are exposed at the surface. Columnar jointing is observed at some places as a result separate blocks of basalts are found at the surface.

The joints observed are open joints. Basalt may present serious problems in foundation design, especially for dams. Quite often soil horizons form on top of one basalt flow, and are then buried by a subsequent flow. This provides well-developed plane of material contrast, and may expedite or inhibit groundwater flow. In addition, basalt is often cavernous, which may cause excessive leakage.

Figure 3.3 Basaltic rock outcrop

1- Highly weathered basaltic rock outcrop (left reservoir rim), 2-Closer look of 1

3- Huge basaltic boulders on top of the left abutment

4- Smaller basaltic boulders at the foot of the left abutment

5- Closer look of 4

3.4.2 Trachyte

The trachyte is found underlying the basalt. It is located in South and South Eastern part of the area. Outcrop and hand specimen examination of the rock indicates that the trachyte is slightly to highly weathered. Portions of the rock where weathering is not significant are light grey to dark grey in colour while the highly weathered ones are pinkish grey in colour. The trachyte predominantly consists of feldspar and minor maffic minerals. The joints observed are not very open as those of the basalt.

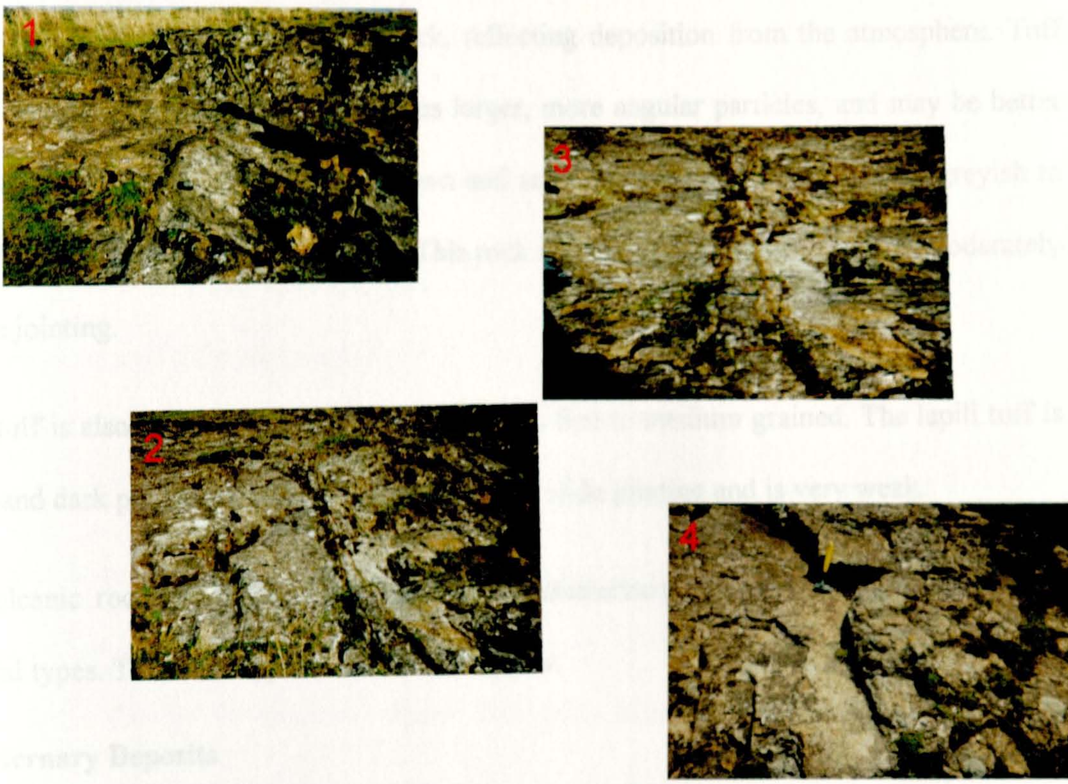


Figure 3.4 Trachyte rock outcrop

1- The top portion of the trachyte outcrop (quary site)

2- Photo showing the joint structure ,3- Closer view of 2

4- Photo showing the joint characteristics

3.4.3 Tuff and welded tuff

These types of deposits are found underlying the trachyte unit. Pyroclastic deposits are dubious foundation materials because of low density, high porosity and collapsible structure. Tuff and welded tuff are the most common deposits that are observed with in the study area. Tuff results from volcanic explosions of high-silica lavas. It usually consists of a mixture of rock fragments of various sizes cemented into a fine grained, porous matrix. Tuffs are commonly stratified like sedimentary rock, reflecting deposition from the atmosphere. Tuff can grade into volcanic breccia, which has larger, more angular particles, and may be better cemented. The tuff is brown to light brown and sometimes the color ranges from greyish to green. It is fine to medium grained rock. This rock is weak and is characterized by moderately to close jointing.

Lapili tuff is also found together with the tuff. It is fine to medium grained. The lapili tuff is brown and dark pink in color. It is characterized by wide jointing and is very weak.

The volcanic rocks of the area are overlain by Quaternary soils of residual, alluvial and colluvial types. These soil types are described below.

I. Quaternary Deposits

A. Residual soils

This type of deposit is the result of insitu weathering of basalts, trachytes and tuff. Residual soils are observed on the top part of the hills characterized by relatively horizontal surfaces. Almost every part of the higher topography beyond the Gerbi reservoir rim (where agriculture is practised) is covered by residual soils. Residual soils are also observed with in the reservoir area. Residual soils are more extensive at left upstream portion of the dam site which are bounded by a hill (at the left reservoir rim) and at the pipe line saddle (downstream).Residual soils are also observed at the down stream portions of the dam site. At some places (top of hills and some portions of the reservoir) the residual soil is found intermixed with colluvial deposits and it has all material size ranging from clay to rock boulders size. The soil is reddish brown in color, it is usually thin but at some places the thickness can reach up to six meters.

B. Alluvial soils

Alluvial soils are formed by alluvial deposition. The alluvial sediments of the study area are present along the main river of Gerbi and along the tributaries of Gerbi river. These sediments are more extensive in portions of the course of the river near the upstream part of the dam axis.The thickness of the alluvial deposit is more in this portion of the basin than other areas. The soil thickness varies from 2 - 5m.

Reddish clay soil, silty clay soils and little sandy clay soil mixed with gravel are the alluvial soil types observed in the study area.

C. Colluvial soils

Colluvial soils are those deposited primarily through the action of gravity. The rock fragments are readily removed from the slopes by gravity, due to which the surface of the slopes remains exposed and undergoes further weathering. This results in the accumulation of hillside waste or talus at the foot of the mountain slopes. This slope deposit (the colluvial deposit of the study area) is classified into two; coarse colluvial and fine colluvial. The coarse colluvial deposit consists reddish brown clay soil intermingled with basaltic blocks, cobble and pebble sized rock fragments. The fine colluvial deposits consist of predominantly reddish brown clay with few rock fragments of basalt and trachyte.

Colluvial sediments with in the study area are more extensive along the left side of Gerbi river. Gravely clay and clayey gravel are the most common colluvial soil types observed in the study area. The soil thickness varies from 2 - 5m.

3.5 Geological Structures

The study of geological structures (joints, faults, contacts, shear zones) is of great importance to assess the rocks with respect to water tightness, stability and soundness of the rock mass. The engineering behavior (strength and stiffness) of a rock mass is greatly affected by the characteristics of joints and faults.

3.5.1 Joints

Joints are characteristic features of nearly all rocks. Joints are fractures along which there has been no appreciable displacement parallel to the fracture and only slight movement normal to the fracture plane, (Hatcher ,1995); blocks may move apart, but do not slip past each other. Systematic or non-Systematic, joints affect the engineering properties of rocks and are also responsible to many failures in civil constructions.

Analysis of joints is not easy as they are easily activated, relative dating of the time of formation is usually unknown, represent virtually no measurable strain and there are many possible mechanisms of origin. As a rule, rectangular and columnar jointing is a characteristics feature of volcanic terrain especially basaltic rocks. In fact genesis of joints is generally considered the result of tensile failure due to cooling of an igneous rock or due to relaxation of the rock mass due to erosional up lift (Price, 1996). Joints often form regular patterns (joint sets) in the rock mass. Representation of important discontinuities on engineering geological maps and presentation of concentration of discontinuities on diagrams gives important clue on the engineering assessment of the rock mass. Hence many measurements of joints from the reservoir, dam site and representative areas with in the basin should be taken. Since Gerbi dam site is comprised of volcanic rock units covered by thick over burden consisting of Quaternary soils of residual, alluvial and colluvial types, it is difficult to take many discontinuity measurements.

Only two outcrops are present in the vicinity of the dam site and all the measurements are taken on these outcrops. The first outcrop is the basaltic rock located at the left of the reservoir rim and the second one is the trachyte outcrop found at the right abutment (quarry site).

The joint measurements taken on these outcrops and their stereographic plots are presented and thoroughly discussed in Chapter 6 and Chapter 7.

3.5.2 Faults

Faults are discontinuities in which one block has slipped past another. Faults (fractures having appreciable movement parallel to the plane of fracture) occur in many forms and dimensions. They can be few centimeters to several kilometers long and their traces can be straight or curved (Hatcher, 1995).

Understanding faults is important during the construction of major civil engineering projects (dams, bridges, multistory buildings, etc).

The east-west running Entoto ridge manifests regional tectonic events that could possibly be related to the tectonic events that prevail with in the catchment. The Gerbi area and its reservoir do not exhibit such regional tectonic features. However, the localized hills are aligned (approximately north-south) which could give attention to the occurrence of tectonic events associated with volcanism (AAWSA, 1997). From interpretation of aerial photos and field assessment for geological structures during the present study the following faults have been suggested. The interpretations made on aerial photographs of the basin indicate that most of the structures are continuous trending in NE-SW, E-W and N-S directions.

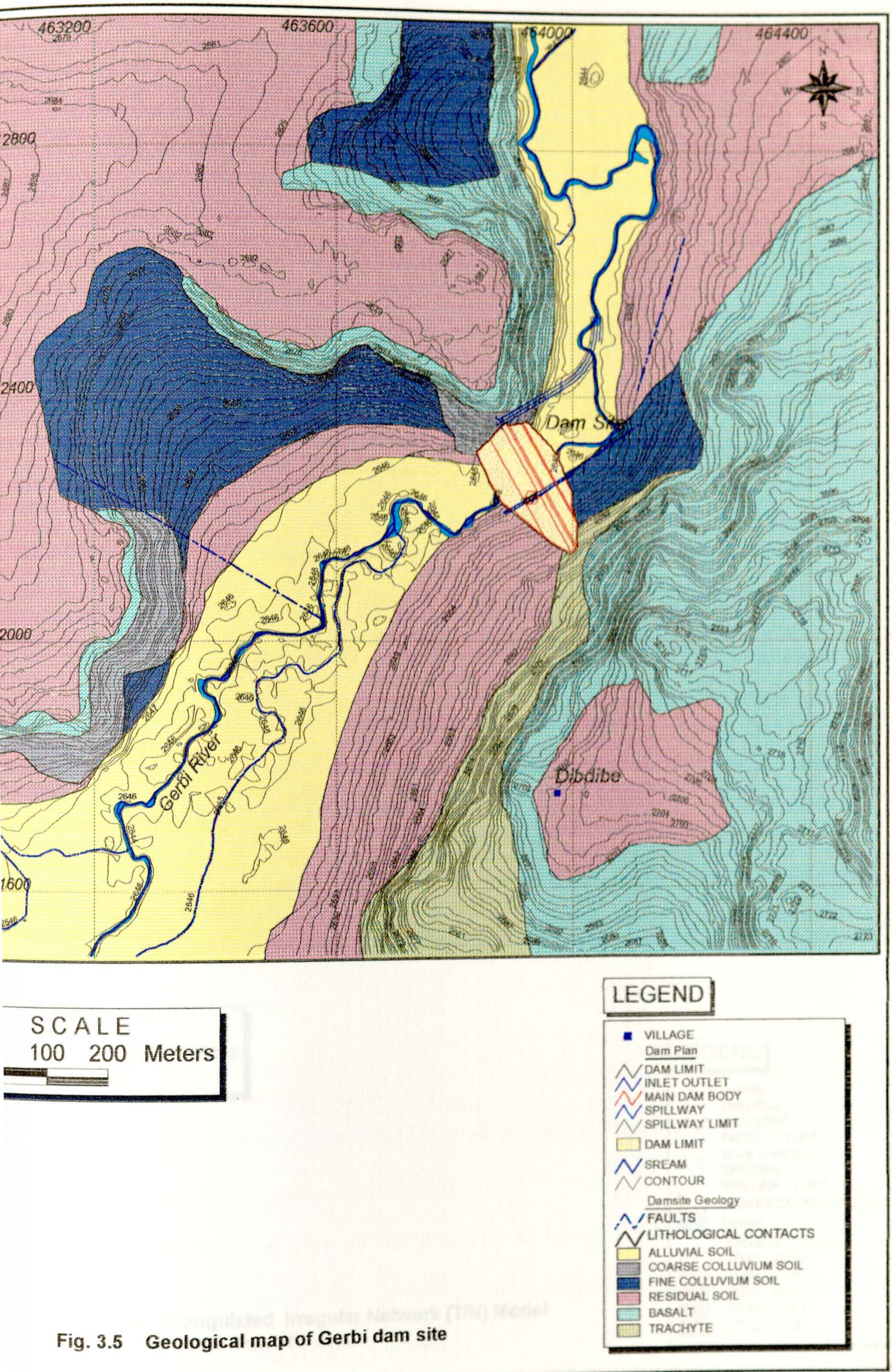


Fig. 3.5 Geological map of Gerbi dam site

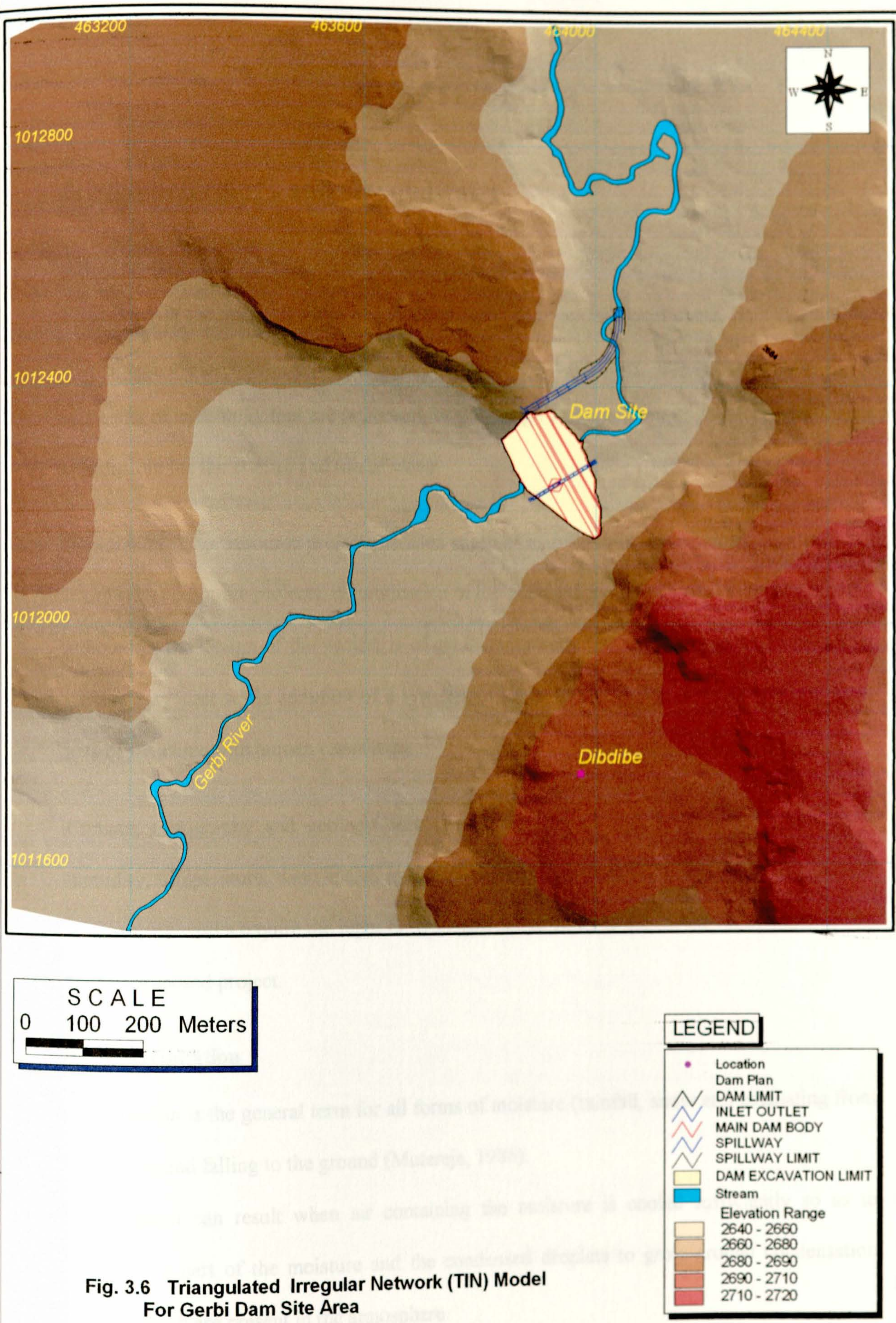


Fig. 3.6 Triangulated Irregular Network (TIN) Model For Gerbi Dam Site Area

CHAPTER-FOUR

Hydrometerology and Groundwater

4.1 General

Hydrology is the study of three important phases of the hydrological cycle, namely rainfall, runoff and evaporation (Mutereja, 1986). Engineering hydrology, however, includes those segments of hydrology that are important for the design and operation of engineering projects responsible for the control and use of water.

In planning water resource projects detailed study of hydrology is very important. In fact, for many water resource projects, determination of the peak and magnitude of flood that has to be adopted in the design of the project is of great importance. Too high design flood results in unnecessary cost while adoption of a low design flood can (if higher flood occurs) result in loss of structure with human causalities.

Climate, topography and geology determine hydrology of a given region. Precipitation, humidity, temperature, wind speed, evaporation and sunshine duration are important factors. Engineering works require the input of hydrogeological study to assess the suitability of a site for the proposed project.

4.1.1 Precipitation

Precipitation is the general term for all forms of moisture (rainfall, snow etc) emanating from the clouds and falling to the ground (Mutereja, 1986).

Precipitation can result when air containing the moisture is cooled sufficiently so as to condense a part of the moisture and the condensed droplets to grow around condensation nuclei, which are present in the atmosphere.

All forms of precipitation are usually expressed as the vertical depth of water that would accumulate on the earth's surface if there were no losses. The precipitation data is basic in planning and management of water resource projects and is used to extend records of run-off both in time and space. Nevertheless, proper network of rain gauges is necessary to collect precipitation data of the water shade.

Data on precipitation of Gerbi catchment is collected from National Meteorological Services Agency (NMSA) of three nearby stations; Addis Ababa Bole, Chancho and Sululta stations.

Table 4.1 Location of the three stations

Station	Elevation (m.a.s.l)	Location
Addis Ababa Bole	2480	$9^0 02^{''}$N and $38^0 45^{''}$E
Chancho	2620	$9^0 20^{''}$N and $38^0 45^{''}$E
Sululta	2604	$9^0 11^{''}$N and $38^0 46^{''}$E

The mean monthly precipitation of the three stations is given below.

Table 4.2 Mean monthly precipitation in mm of the stations

Stations	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual Mean
A.A Bole	11.8	33.1	66.8	99.1	69.2	122.7	228.0	248.9	130.8	32.7	2.1	4	1049.0
Chancho	27.8	29.8	69.7	68.1	65.1	144.4	368.6	334.6	125.5	25.8	12.3	9.9	1282.1
Sululta	17.7	22.0	70.1	91.0	54.8	161.4	300.2	274.9	107.3	18.6	6.8	4.7	1129.6
Average	19.1	28.3	68.8	86.0	63.0	142.8	298.9	286.1	121.2	25.7	7.1	6.2	1153.5

(Source: National Meteorological Agency)

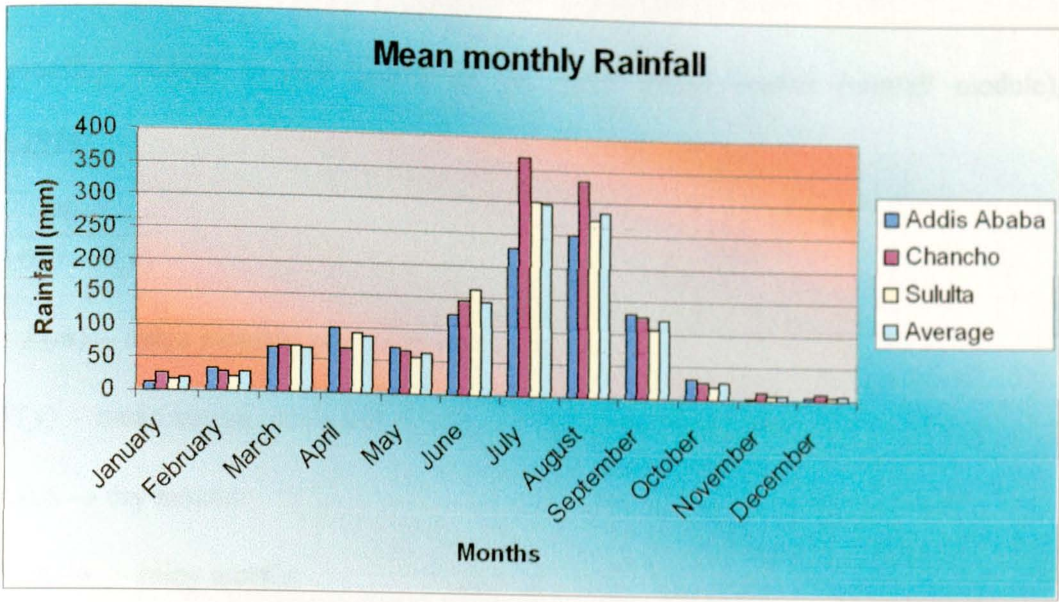


Figure 4.1 Mean monthly precipitation of Addis Ababa Bole, Chancho and Sululta stations and their arithmetic mean

A rainfall measurement is a point observation and may not represent the area under consideration, so the point measurements have to be averaged over the area except in its immediate vicinity. The arithematic mean of the three stations is taken as the mean annual average rainfall depth of the basin and it is found to be 1153.5 mm.

The knowledge of the intensity and distribution of the maximum rainfall that may occur on the area under consideration with in a given period of time is the basis to determine the flood flow that will result. A study of precipitation is essential for the solution of some problems where runoff data is not available. Run-off data is available so the flow characteristic is basically derived from the run-off data of Gerbi river.

The basin is characterized by a unimodal rainfall pattern. In fact the twelve months are grouped into two rainfall groups based on their rainfall coefficient .The second group for months March, April and May has relatively higher rainfall than the remaining months but is not high enough to be considered as a rainfall group. The rainfall coefficient is the ratio of mean monthly rainfall to one twelfth of the mean annual rainfall (rainfall module), (Daniel,1974).

$$C_{rf} = \frac{P(m)}{0.083 \times P(y)} \qquad \text{Eq.4.1}$$

Where, $P(m)$ – mean monthly precipitation

$P(y)$ - mean annual precipitation

If $C_{rf} < 0.6 \rightarrow$ dry months

If $C_{rf} > 0.6 \rightarrow$ rainy months

If $0.6 < C_{rf} < 0.9 \rightarrow$ little rain

If $C_{rf} > 1 \rightarrow$ heavy rainfall

Table 4.3 Rainfall coefficient of the mean monthly average

Stations	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Average mean Rf	19.1	28.3	68.8	86	63	142.8	298.9	286.1	121.2	25.7	7.1	6.2
Rainfall coefficient	0.20	0.30	0.72	0.90	0.66	1.49	3.12	2.99	1.27	0.27	0.07	0.06

4.1.2 Temperature

Increase in temperature, increases rate of evaporation by which the precipitated water is returned back to the atmosphere as vapor to perpetuate the hydrologic cycle. Generally, maximum amount of evaporation occurs as a function of temperature independent of existence of other gases.

Monthly minimum and maximum temperature data of Addis Ababa Bole and Sululta stations is collected from NMSA (Appendix A) and the mean monthly maximum, mean monthly minimum and average monthly temperature of these stations is given below.

Table 4.4 Monthly mean maximum and minimum and average temperatures of the stations

Stations	Temperature (°C)	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
A.A Bole	Monthly Minimum	7.0	8.6	10.4	11.2	10.9	10.6	11.0	10.9	10.0	8.2	5.7	6.2
	Monthly maximum	24.0	24.8	25.3	24.9	25.2	23.5	21.3	21.1	21.9	23.1	23.2	23.4
	Average	15.4	16.7	17.8	18.0	18.0	17.0	16.1	16.0	15.9	15.6	14.4	14.7
Sululta	Monthly Minimum	3.2	3.6	4.0	4.3	4.1	3.6	3.8	3.6	3.8	3.7	3.1	3.2
	Monthly maximum	22.0	22.5	21.9	22.9	22.9	21.4	20.2	20.4	21.1	21.4	21.5	21.7
	Average	12.6	13.0	13.0	13.6	13.5	12.5	12.0	12.0	12.5	12.6	12.3	12.4

Temperature data from sululta station (the nearest station) is used to represent temperature of the study area. The average minimum temperature of the study area is between 3.2 and 3.6 °C from November to February and the average maximum temperature is between 21.8 and 22.9 °C from December to June.

4.1.3 Relative Humidity and Sunshine Durations

4.1.3.1 Relative Humidity

The state of the atmosphere in relation to the amount of water vapour it contains is called humidity (Mutereja, 1986). Humidity is closely related to temperature; the higher the temperature the more vapor it can hold. Humidity is extremely important factor since atmospheric water is the source of precipitation and controls the rate of evaporation.

Relative humidity is the ratio between amount of water vapor actually contained per unit volume and the maximum amount of moisture it can hold when saturated and at the same temperature.

Table 4.5 Mean monthly Relative humidity

Stations	Relative Humidity (%)	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
A.A Bole	Mean Monthly at 600	75.33	71.33	70.00	70.78	74.33	85.44	90.33	91.11	87.00	74.56	69.00	75.00
	Mean Monthly at 1200	43.44	41.33	43.89	47.56	47.00	59.11	70.11	72.44	62.44	43.56	40.44	43.67
	Mean Monthly at 1800	50.67	43.56	44.89	52.67	51.67	68.67	77.89	79.11	70.56	52.78	47.22	49.89
	Mean monthly Average	56.48	52.07	52.93	57.00	57.67	71.07	79.44	80.89	73.33	56.96	52.22	56.19

(Source: National Meterological Agency)

The mean monthly relative humidity of Addis Ababa has been used to represent the relative humidity of the study area. From the monthly relative humidity data collected from NMSA (Appendix A), it can be concluded that the mean monthly humidity varies from 52 to 80. Months June, July, August and September are very humid months (and hence evaporation is lower) while February, March and November are relatively dry (higher evaporation).

4.1.3.2 Sunshine Duration

Monthly sunshine duration data of Addis Ababa Bole station from 1992 to 2003 is collected from NMSA. The mean monthly sunshine duration is given in table 4.6. The mean monthly sunshine duration of Addis Ababa has been used to represent the sunshine duration of the study area.

Table 4.6 Sunshine duration of Addis Ababa Bole Station

Stations	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Mean monthly sunshine duration (hrs)	8.9	8.2	7.8	6.8	7.9	5.9	3.8	4.2	5.9	8.5	9.3	9.9

(Source: National Meteorological Agency)

Months June, July, August and September have short sunshine durations while months from October to May have long durations indicating minimum cloud cover.

4.1.4 Evapotranspiration

Evapotranspiration is the total water loss (free water evaporation, transpiration from plants, soil moisture evaporation) from a basin.

To determine the maximum flood that will occur with a certain frequency and to determine the yield expected from a certain basin, the run-off should be evaluated. To estimate the run-off characteristics of a basin knowledge of both the water input and output (water loss in the form of evaporation, transpiration etc) elements is necessary.

4.1.4.1 Potential Evapotranspiration

Potential evapotranspiration is the water loss which will occur if at no time there is a deficiency of water in the soil for vegetation use (Thornthwaite, 1944). This term implies an ideal water supply to vegetations.

Various methods are used for estimating evapotranspiration but there is none, which is generally acceptable under all circumstances. The various methods use different factors (climate, soil and vegetation) to come up with an estimate of evapotranspiration. Sululta station has only rainfall and temperature data so Thornthwaite's method is used. The method uses the mean daily temperature, the latitude of the place and months of the year to compute the evapotranspiration. It assumes that the mean temperature is correlated with radiation, atmospheric moisture and wind. There is no crop constant included.

The formula developed by Thornthwaite is:

$$PET = CT_m^a \qquad \text{Eq.4.2}$$

Where, PET – monthly potential evapotranspiration (cm), C – Coefficient

T_m - mean monthly temperature (°C), a – constant

$a = 67.5 \times 10^{-8} I^3 - 77.1 \times 10^{-6} I^2 + 0.0179 I + 0.492$

I – is the annual heat index and is given by:

$$I = \sum_{m=1}^{12} \left(\frac{T_m}{5} \right)^{1.51} \qquad \text{Eq.4.3}$$

If each month has 12 hours of sunshine each day and 30 days a month which is similar to the case in the basin, the equation reduces to:

$$PET = 1.62 \left[10 \frac{T_m}{I} \right]^a \qquad \text{Eq. 4.4}$$

Potential evapotranspiration for Gerbi basin using Thornthwaite method is given in table 4.7. The result is extremely low compared to other estimates (usually above 1000 mm) under similar climatic conditions.

Table 4.7 PET of Gerbi basin

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
T_m	**12.6**	**13.0**	**13.0**	**13.6**	**13.5**	**12.5**	**12.0**	**12.0**	**12.5**	**12.6**	**12.3**	**12.4**
PET (cm)	**5.34**	**5.56**	**5.56**	**5.88**	**5.83**	**5.29**	**5.03**	**5.03**	**5.29**	**5.34**	**5.18**	**5.24**

4.1.4.2 Actual Evapotranspiration

Actual evapotranspiration (AET) is the amount of evapotranspiration that occurs when the soil moisture is less than the PET or under actual field conditions. AET is usually less or equal to PET.

AET is computed from PET values. The amount of moisture available with in the reach of roots (root depth) is considered in this method. This method (monthly water balance model (Thornthwaite and Mather, 1957)) uses monthly precipitation and PET values.

To compute AET, nature and type of the soil and the vegetation cover with in the basin should be known and the corresponding estimated water capacities are given based on the correlation between soil texture and vegetation type and water capacity developed by Thornthwaite and Mather (1957) as cited in Dune and Leopold (1978). The average AET is 780 mm.

As stated earlier the evapotranspiration data is used to estimate the run off characteristics and hence the yield expected. It is also used to estimate the flood that will occur with a certain interval. However evapotranspiration and precipitation data are used to estimate the run off characteristics when run off data is not available. Gerbi river is gauged (flow data is available) and hence the flow data has been used to estimate the yield expected.

4.1.5 Surface run-off

Run-off is the residual of precipitation after the demands of interception, infiltration, depression storage and evapotranspiration are met and constitutes overland flow (surface run off) and inter flow (subsurface run-off (depends on geology) and base flow (ground water flow that contributes to the river) based on the characteristics of the streams.

Surface run-off is water that flows over the soil surface when precipitation rate exceeds the infiltration capacity.

Run-off characteristics of a basin are affected by the rainfall input, physical, vegetative and climatic characteristics of the basin. The catchment characteristics are greatly determined by its geology, geomorphology, area, slope and drainage basin dynamics.

Gerbi river is a perennial river fed by a number of intermittent tributaries. The river is gauged a few meters upstream of the dam site. The mean monthly flow data is given in Table 4.8.

Table 4.8 Mean monthly flow data

Stations	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Flow (m^3/s)	0.080	0.075	0.079	0.118	0.090	0.153	2.838	5.311	2.288	0.297	0.092	0.074

(Source: Addis Ababa Water and Sewerage Authority)

The direct runoff consists of the overland flow. Discharge from excess runoff constitutes the direct runoff hydrograph. The initial rise of direct runoff hydrograph is due to arrival of runoff at the outlet. As enough time elapses flow from distant areas of the basin adds to the flow at the outlet.

The highest flow occurs in June, July, August and September and the lowest in December corresponding to the highest and lowest rainfall months respectively.

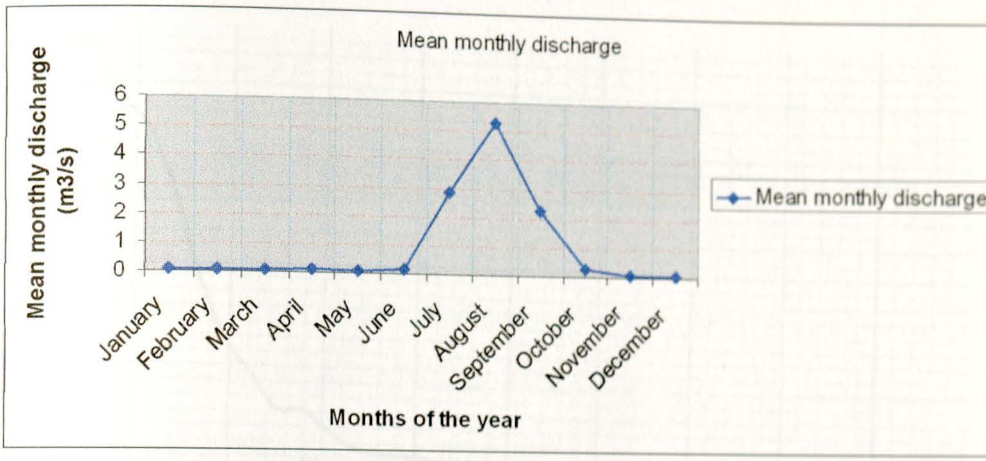


Figure 4.2 Hydrograph of Gerbi river

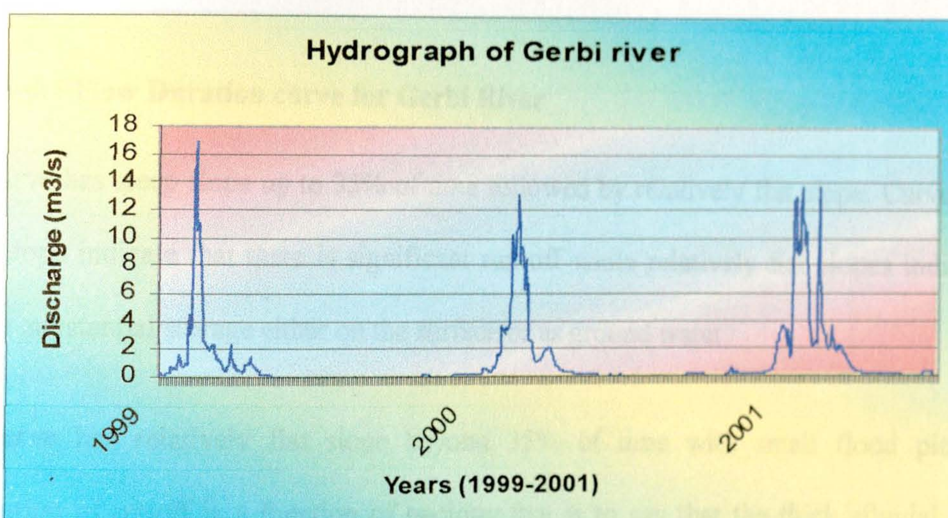


Figure 4.3 Hydrograph of Gerbi river based on daily data

4.1.6 Flow duration for Gerbi river

The characteristics of a certain basin (whether there is a large amount of runoff or not) can be established by analyzing the shape of the Flow duration curve.

A flow duration curve shows the percentage of time a specified discharge is equaled or exceeded (Bell, 1980).

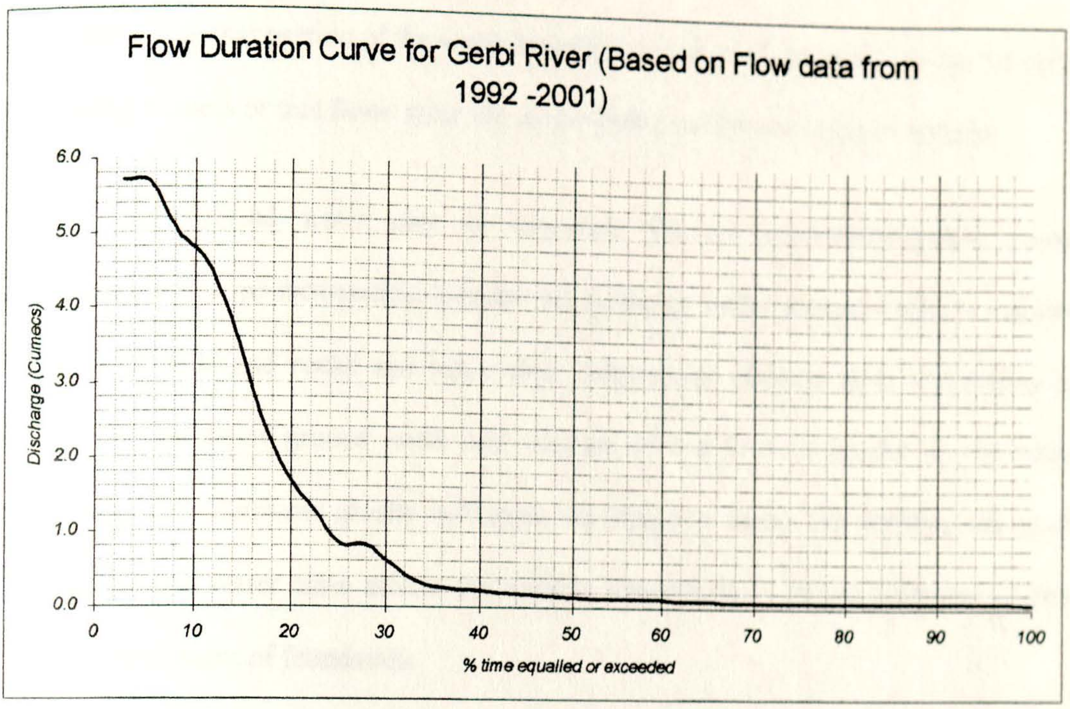


Figure 4.4 Flow Duration curve for Gerbi River

The curve has steep slope up to 35% of time followed by relatively flat slope. Curves having steep slope indicate that there is significant run-off while relatively flat slopes indicate that there is substantial storage either on the surface or as ground water.

The curve has relatively flat slope beyond 35% of time with small flood picks. The distribution of runoff is a function of geology that is to say that the thick alluvial deposits along the course of the river absorb the water resulting in small flood picks.

The mean annual runoff depth is calculated using the formula give below.

$$RD = Q \times \frac{t}{A} \qquad \text{Eq.4.5}$$

Where: RD- Runoff depth, Q-mean annual runoff, t-time in second and A- area of catchment. The mean annual runoff depth which is the combination of both the surface runoff and ground water flow is found to be 0.355cm (355mm).

4.1.7 Ground Water

I. General

Ground water is that portion of the water beneath the surface of the earth that can be collected with wells, tunnels or that flows naturally to the earth's surface via seeps or springs.

Surface and Ground water play an important role on geomorphologyical processes, weathering and slope movements. Ground water (usually under pressure) affects engineering behavior of soils and rocks and hence their deformation characteristics and failure (Bell, 1993). The nature of ground water and variation of one form to another in the reservoir, foundation and abutments greatly influences the design of dams. The position and nature of existing ground water table affects the stability characteristics of the abutment slopes and potential settlement of foundation.

Ground water flowing in to excavations affects the construction of the dam by increasing the uplift pressure. The corrosive action of ground water has an impact on the engineering structure itself. The interaction of rock and soil with ground water can result in a change in the properties of the rocks and soils which in turn may possibly impose adverse conditions on the engineering structure.

II. Ground water levels and fluctuations

The ground water observations of the site were made during the drilling of boreholes. The ground water level is observed by placing piezometers in drill holes. Piezometer observations are usually made over a period of time to record fluctuations with the seasons or to observe its relationship with the rise and fall of river levels. Many ground water levels show a seasonal variation affected by recharge from rainfall and discharge from pumping. Ground water is not in use at the site. Fluctuation due to pumping out of ground water is therefore insignificant.

From the analysis of rainfall data, highest level of ground water occurs about July and the lowest about April.

To Asses the ground water conditions of the study area, some hydrogeological data especially static water levels were collected in the field. The depth of penetration, and boreholes logs were collected from AAWSA.

The observation made at the site is only for the dry season, and hence it is difficult to analyze the effect of ground water fluctuation on the stability at the dam site. The ground water levels of the boreholes are given in Appendix A.

The rocks exposed in the study area are mainly basaltic lava flows which are overlain by Quaternary sediments and exhibit a wide range of hydrogeological properties. These rocks range from fresh, dense and massive type to highly porous, fractured, jointed and vesicular type. The variation in the rock's property and the variation in the degree of weathering with the different stress conditions with in the study area is the possible cause of the variation in their hydrological properties.

Aquifer (the ground water bearing formations sufficiently permeable to transmit and yield water) with in the study area are basically the unconsolidated sand and gravels (found in the alluvial valleys) and the weathered and/or fractured and porous basalts and trachytes.

III. Ground water flow directions

The characteristic of ground water flow can be determined from a full map of the water table. The direction that the water table or potentiometeric surface is sloping can be determined by using graphical methods. The gradient is in the direction of decreasing head and is perpendicular to the contour lines.

Static water levels from boreholes are used to determine the flow direction. The results indicate that the ground water flow follows local slope directions. The flow direction is from NW to SE. Generally flow is towards the valley.

There are springs at the right side some 150m of the dam axis. The springs come at the contact of the basalt and trachyte layer. There is also a spring at the down stream portion of the dam.

Figure 4.5 A spring at the down stream portion of the dam

The spring found at the down stream portion is perennial and has high discharge. The level of the spring remains the same throughout the whole year.

Presence of springs at the down stream portion of a dam site mean there is excessive leakage of water either through the abutment or foundation creating excess loss of water even endangering the stability of the dam. Therefore, it has been tried to investigate the spring and describe the probable type of spring, its source and its relation with the reservoir water.

The spring is a contact spring. The spring is found at higher elevation to the left of the main river and it is very clear that the source of the spring water is not the river. The water coming out of the spring joins the river few meters down. It seems the water is coming following a contact between the basalt and trachyte from the elevated land downstream left portion of the dam site. It seems the spring has no relation with the reservoir. But further investigation must be carried on to describe its source and its relation with the reservoir water.

4.1.8 Water Chemistry

The study of water chemistry is important since water can cause deterioration of engineering materials and structures.The chemistry of both surface and ground water of the site should be studied to assess the impact of water on the structure. To analyze the impact of the water, samples are collected from Gerbi river, springs and boreholes and quality analysis is done considering some major water quality determinants (Turbidity, color, pH, alkalinity, hardness, total dissolved solids, total suspended solids, etc) (Adinew Adam,1996).

A. Physico-Chemical qualities of Gerbi river

The physical analysis includes temperature, turbidity, color, taste and odour. The results of the physical tests are given below.

Table 4.9 Results of the physical qualities carried out from of Gerbi river

Parameters	Results (Feb.to Mar.,1996)
Temperature	$18.1^{0}C$
Turbidity	40.5 FTU
Color	226.3 Pt-Co
Taste and odour	Objectionable

The results of the physical analysis indicate that the values exceed the WHO's guideline value of turbidity, color. The water requires physical and chemical treatments to bring down the values to the acceptable limits. The offensive taste and odour of the river do not fit with the quality standards for drinking water hence the water should be treated so that the result matches with the quality requirement.

WHO's guideline for drinking water is given in Appendix B.

B. Hydro-chemical analysis

Many hydro-chemical analysis such as pH, conductivity, TDS, alkalinity etc are conducted between February and March, 1996 on water samples of Gerbi river by AAWSA. The results of the hydrochemical analysis are given in the table 4.10.

Table 4.10 Results of the chemical qualities of Gerbi river water (AAWSA)

Parameters	Result
pH	8.1
Total Dissolved solids	94.5 mg/l
Conductivity	0.136 mS/cm
Total alkalinity	61.5 mg/l as $CaCO_3$
Total hardness	54.1 mg/l $CaCO_3$ 34.4 mg/l Calcium hardness 19.7 mg/l Magnesium hardness
Carbon dioxide	2.5 mg/l
Dissolved oxygen	6 mg/l
Nitrate, Nitrite and Ammonia	0.2 mg/l NO_3 0.002 mg/l NO_2 0.08 mg/l NO_3
Iron	0.09 mg/l
Manganese	0.09 mg/l
Chloride	2.7 mg/l
Fluoride	0.36 mg/l
Chromium Hexavalent	0
Copper	0
Phosphate	0.34 mg/l
Silica	8.8 mg/l
Sulfate	0
Sodium, Potassium	-
Saturation Index	-0.48
Sediment load	60.9 ton/km^2/year

(Source AAWSA)

The suitability of water for drinking can be assessed from physico-chemical analysis of water samples. From results of the chemical analysis conducted on water from Gerbi river some results (TDS, total alkalinity, total hardness, Nitrate and Ammonia, Chloride, Fluoride, Copper, Sodium)are below the WHO's guidelines and some results(Iron, Manganese, Phosphate) are above the WHO's guidlines. It is therefore important to treat the water (physically and chemically) to make the result match with the WHO's quality requirement.

Besides the results of the physico-chemical analysis helps to assess the impact of water on the engineering structure and construction materials. The chemistry of water is usually decisive in causing the deterioration of engineering structures and construction materials.

The following tables show the impact of water on engineering structures and construction materials by considering their chemistry.

Table 4.11 Effects of sulphate on concrete (Hunt, 1984)

Degree of attack	*Sulphate in water (mg/l)*
Negligible	0-150
Positive	150-1000
Considerable	1000-2000
Severe	>2000

The results of the water chemistry analysis (Table 4.10) indicate that the sulphate content of the water is negligible and hence do not cause significant damage on the concrete structure of the dam.

Table 4.12 Effects of aggressive CO_2 on concrete (Hunt, 1984)

Degree of Aggressiveness	P^H	*Aggressive CO_2 (mg/l)*
Neutral	6.5	<15
Weak	6.5-5.5	15-30
Strong	5.5-4.5	30-60
Very strong	<4.5	>60

From the results of the analysis (Table 4.11) the value of CO_2 is 2.5 mg/l which is with in the neutral aggressiveness range indicating that it may not cause deterioration. The pH value also gives the same aggressiveness range as that of aggressive CO_2.

Table 4.13 Hardness classification of water, Todd (1980)

Hardness, mg/l as $CaCO_3$	**Water class**
0-75	Soft
75-100	Moderately hard
150-300	Hard
Over 300	Very hard

According to the chemical analysis result (Table 4.10) the water of Gerbi River can be classified as soft water.

The results of the analysis also indicate that the saturation index has negative value indicating that the water is aggressive in causing corrosion. It is therefore necessary to add lime before and after treatment to control corrosion (Adinew Adam, 1996).

CHAPTER- FIVE

Engineering Geological and Geotechnical characterization and mapping of soil of the study area

5.1 General

The word 'soil' has different meanings for different professions. To a Geologist, soil is a material in the top thin zone with in which roots occur. Soil in engineering sense is a mineral material where individual particles are not sufficiently bonded and hence lack strength. Soils are the weathering products of rocks.

Soil is a natural aggregate of mineral grains, with or without organic constituents that can be separated by gentle mechanical means such as agitation in water (Murthy, 1989).

The importance of soil in engineering geologic works stems from the fact that soil is the most common and abundant building material and structures are commonly founded on soil and this affects the design of a structure.

5.2 Origin and Description of soils

The process of weathering of the parent rock forms soils. The process of weathering of the rock decreases the cohesive force binding the mineral grains and leads to the disintegration of bigger mass to smaller and smaller particles.

Weathering is a surface process. It occurs at low temperature and low-pressure environments. This process is the result of the interaction of atmosphere with existing rock bodies. During this interaction existing rock bodies are broken and altered because the surface condition does not favor stability of the minerals forming the rock bodies. The formation of soils is the result of the combination of various processes. There are many factors that control formation of soils. Factors such as: climate, mechanical action of freezing water and roots, topography, type of vegetation cover, nature of parent rock, greatly influence the formation of soils and the properties of the resulting soil. Hence classification and description of soils based on their actual field condition in addition to laboratory analysis is necessary since the formation mechanism of soils greatly influence the engineering properties of soils.

On the basis of the origin of their constituents, soils of the study area are classified in to two broad groups; residual soils and transported soils. The transported soils are further grouped into alluvial and colluvial soils.

5.2.1 Residual soils

Residual soils are those that remain at the place of their formation. They form on surfaces of the earth where the surface is approximately horizontal; this horizontality favors formation of soils without being removed from their original position. Residual soils are found only in regions that have not been greatly disturbed during recent geological time. The soils are the actual residues of the original rock.

This type of deposit is observed on the top part of the hills characterized by relatively horizontal surfaces. Almost every part of the higher topography beyond the Gerbi reservoir rim is covered by residual soils. Residual soils are also observed with in the reservoir area. Residual soils are more extensive at left upstream portion of the dam site which are bounded by a hill (at the left reservoir rim) and at the pipe line saddle (downstream). An important characteristic of residual soils is that the sizes of grains are indefinite. Particles are sharp edged and not rounded. At some places (top of hills and some portions of the reservoir) the residual soil was found intermixed with colluvial deposits and it has all material size ranging from clay to rock boulders size.

Some residual soils are silty, but residual clays are more common than silts. Due to the absence of organic material in the topsoil and the consequent lack of humic acid to act as a leaching agent, such soils have high alumina and iron oxide content. Clays of this nature are soft when naturally moist but harden on exposure to the atmosphere.

The depth of the residual soils depends primarily on climatic conditions and the time of exposure. In some areas this depth is considerable. In most places the residual soil is very thin. At some places the thickness can reach from 2.5 m up to 4 m.

The most common types of residual soils encountered with in the study area are: clay, silty clay and gravely clay with sand.

A, Clay

This type of residual soil covers most parts of the study area. The clay encountered at the surface is light brown clay. It is formed by residual weathering of underlying bed rock. With increase in depth the rock disintegration decreases and finally the natural unaltered rock is reached.The color of the clay changes from light brown to brownish brown to dark brown with depth. From analysis of the representative samples collected from all auger holes and test pits the following soil conditions are found. This clay soil is moist (but the moisture decreases with depth), expansive and highly plastic; its strength is found to be firm to stiff.

Shrinkage cracks are observed at some places. Soil mass classification is possible based on the degree of weathering of the parent rock, in this case basalts and trachytes. The weathering grade of the material is Grade IV to V (Johnson, 1988), given in Annex D.

B, Silty Clay

This type of residual soil is found together with the highly expansive clay that covers most part of the study area. The colour of silty clay soil of the study area is usually light brown; the moisture content of this type of soil is insignificant hence they are dry. This soil is loose. The thickness is usually thin; it reaches from 0.3 m to 0.5 m. Sometimes it is found mixed with organic matter.

C, Gravelly clay with sand

These types of soils are found mixed with the colluvial soil. They are usually encountered in portions of the study area characterized by moderate slopes. These soils have relatively higher proportions of gravely material. The strength of these soils is found to be stiff. They are reddish brown in color, stiff and moderately plastic.

5.2.2 Transported soils

Transported soils are soils that are found at locations far removed from their place of formation. The transporting agencies of such soils are wind, water and glaciers. Erosion is one of the great modifying influences at work today on the surface of the earth. The disintegrated rocks by erosive power are transported and deposited by the action of glaciers, wind and water. Gravity is considered as an agency responsible for the movement of disintegrated rock mass. Transported soils of the study area thus can be classified based on their mode of transportation. Alluvial and colluvial soils are the transported soil types that are observed with in the study area.

5.2.2.1 Alluvial soils

Running water is one of the most important agents of transportation of soils. The running water carries a large amount of sediment either in suspension or by rolling along the bed.

Alluvial soils are of great depth and extent. The deposits show stratification. The alluvial sediments of the study area are observed along the main river of Gerbi and along the tributaries of Gerbi river. These sediments are more extensive in portions of the course of the river near upstream part of the dam axis.The thickness of the alluvial deposit is more in this portion of the basin than other areas. The soil thickness varies from 2 - 5m.

Reddish clay soil, silty clay soils and little sandy clay soil mixed with gravel are the alluvial soil types observed in the study area. As part of the investigation program many auger holes were drilled and representative samples were collected and they are classified and analysed in the laboratory to determine the soil properties.

A, Reddish clay soil

This type of soil is found in most parts along the course of Gerbi river. This type of material is encountered from surface to 0.30m and from 2 to 5m depth and at some portions it is only found at the surface. The clay soil is reddish brown in color. The soil is moist and highly expansive; its strength is found to be soft to firm.

B, Silty clay soil

The silty clay soils are the most abundant alluvial deposits with in the reservoir and are usually encountered along the course of Gerbi river between the top 0.06 and 030m. The soils consist of more or less uniform sized and rounded grains of rock flour with little or no plasticity. Laboratory analyses of the samples indicate that these soils are characterized by the following soil conditions: the soils are dry and are slightly to moderately plastic. They are reddish brown in color. Hand identification of samples show that these soils can be moulded easily by finger pressure indicating that the soils are soft. Results of laboratory analysis also confirm that the soils are soft

C, Sandy clay soil mixed with gravel

These types of soils are ecountered at some portions along the course of Gerbi river, Sandy and gravelly material and weathered rock is also encountered before 2 to 5m. The sandy CLAY soils display little cohesion and plasticity. They are reddish brown in color, and their strength is found to be soft to firm.

5.2.2.2 Colluvial soils

Colluvial soils are those deposited primarily through the action of gravity. The rock fragments are readily removed from the slopes by gravity, due to which the surface of the slopes remains exposed and undergoes further weathering. This results in the accumulation of hillside waste or talus at the foot of the mountain slopes. This accumulation sometimes is very thick and can cover large areas. The fragmental material derived from the destruction of the bed rocks and has crept downhill under the action of gravity and deposited at the foot of the slope is called colluvial deposit. These deposits consist of irregular, coarse particles. It is good source of coarse-grained soils for many engineering works.

Colluvial sediments in Gerbi area are more extensive along the left side of Gerbi river than the right side. Gravely clay and clayey gravel are the most common colluvial soil types observed in the study area.

A. Gravely clay soil

This type of soil is found at the bottom of hills where the steppeness of the slope is relatively lower. The proportions of gravel sized particles are with in the range of 30 to 35 % in the soil samples. The material encountered at the surface is dark brown, modertely plastic, highly expansive clay. It is formed by colluvial deposition.

B. Clayey gravel soil

This type of soil is found on hill sides characterized by relatively higher slope. The proportions of gravel sized particles are with in the range of 50 to 60 % in the soil samples. This material is reddish brown in color and non-plastic to moderately plastic. The soil is moderately expansive; its strength is found to be soft to firm.

Soils of organic origin are chiefly formed in either by growth and subsequent decay of plants such as peat mosses, or by the accumulation of fragments of skeletons or shells of organisms. Organic soils can be found with in the reservoir area of Gerbi dam characterized by relatively flat slope. The prevalence of organic soils decreases as tropical areas are approached. Organic soils are rarely of great depth, and are sponge like; hence drainage of water from organic soils is difficult. They cause real problem to engineering work.

The laboratory results carried out on representative soil samples are given in table 5.1.

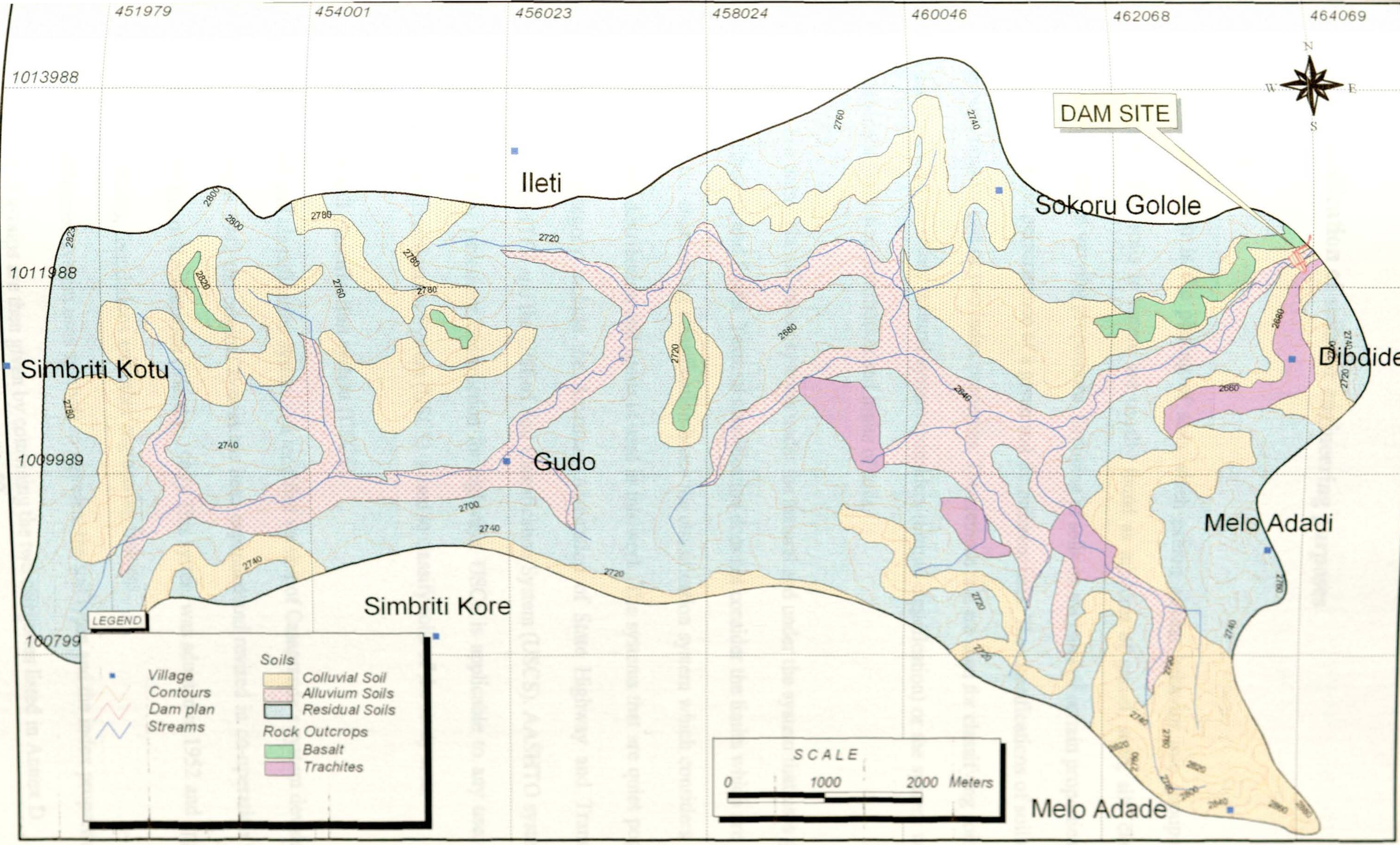


Fig 5.1 Soil Map of Gerbi basin based on their origin

5.3 Classification of soils for engineering purposes

I. General

Soil classification is the placing of soils which exhibit similar behavior into groups and subgroups. The soils in nature are usually found as mixtures of gravel, sand, silt, clay or organic matter in varying proportions. Grouping of soils on the basis of certain properties will help to rate the performance of a given soil for different uses. The classifications of soils into groups employ one or two index properties. The methods that are used for classifying soils are usually based on either textural system (considers textural classification) or the system which considers grain size distribution and limits of soils.

There are many classification systems under the textural and under the system that uses grain size distribution and limits. Textural classification does not consider the limits which are very important for engineering classification hence the classification system which considers both the grain size distribution and limits is used in this work. The systems that are quiet popular under this category include the American Association of State Highway and Transport Officials (AASHTO) and the Unified Soil Classification System (USCS). AASHTO system is mostly used for pavement construction for highways. USCS is applicable to any use. The Unified Soil Classification System (USCS) is used to classify soils of the study area.

II. Unified soil classification system (USCS)

The unified soil classification system is a modified version of Casagrande's system developed in 1942. Since 1942 the original system has been expanded and revised in co-operation with the U.S. Bureau of Reclamation (Murthy, 1989). This system was adopted in 1952 and applies to embankments, foundations and other engineering features.

This classification system uses symbols to represent the soil types and the index properties of the soils. Soil groups are then given by combining the two categories listed in Annex D.

The laboratory analysis was conducted at the laboratory of the Department of Geology and Geophysics, results from Laboratory report by AAWSA is also used for the purpose of classification of the soil samples. The locations of all test pits where the samples were taken are shown on the engineering geologic map given in figure 5.7.

Table 5.1 Soil classification of the study area based on USCS

SAMPLE TAKEN FROM	DEPTH SAMPLED (RANGE) (m)	Particles > 2mm (%)	Sand	Silt (%)	Clay (%)	L.L	P.I	soil classification
STP8-2A*	1.0 - 2.0	0	4	62	34	55	25	MH
STP6-2A*	1.0 - 2.0	0	5	42	53	57	27	MH
STP5-2A*	1.0 - 2.0	0	11	38	51	68	37	CH
STP-10-2A*	1.0 - 2.0	1	3	56	40	65	28	MH
GB-96-S-22*	1.30-1.90	-	-	-	-	75	32	MH
GB-96-S-23*	1.20-1.80	-	-	-	-	49	19	MH
GB-96-S-23*	2.00-2.60	-	-	-	-	71	30	MH
GB-96-S-24*	1.45-2.05	-	-	-	-	76	44	CH
GB-96-S-24*	2.25-2.85	-	-	-	-	68	31	MH
A1	0.3 - 1.3	1	10	30	59	53.2	20.34	CL
AA1	0.5 - 1.5	0	10	28	62	51	16.8	ML
C1	0.5 - 1.5	52	8	18	22	57.8	28	GC
CC1	0.6 - 1.2	60	9	15	16	58.3	11.8	GC
R1	0.3 - 1.2	0	10	39	50	56	26	CL
RR1	0.3 - 1.5	0	8	38	54	46	12	ML

* Indicates samples and results taken from AAWSA

The plasticity chart for soils of the study area is given in Fig.5.2.

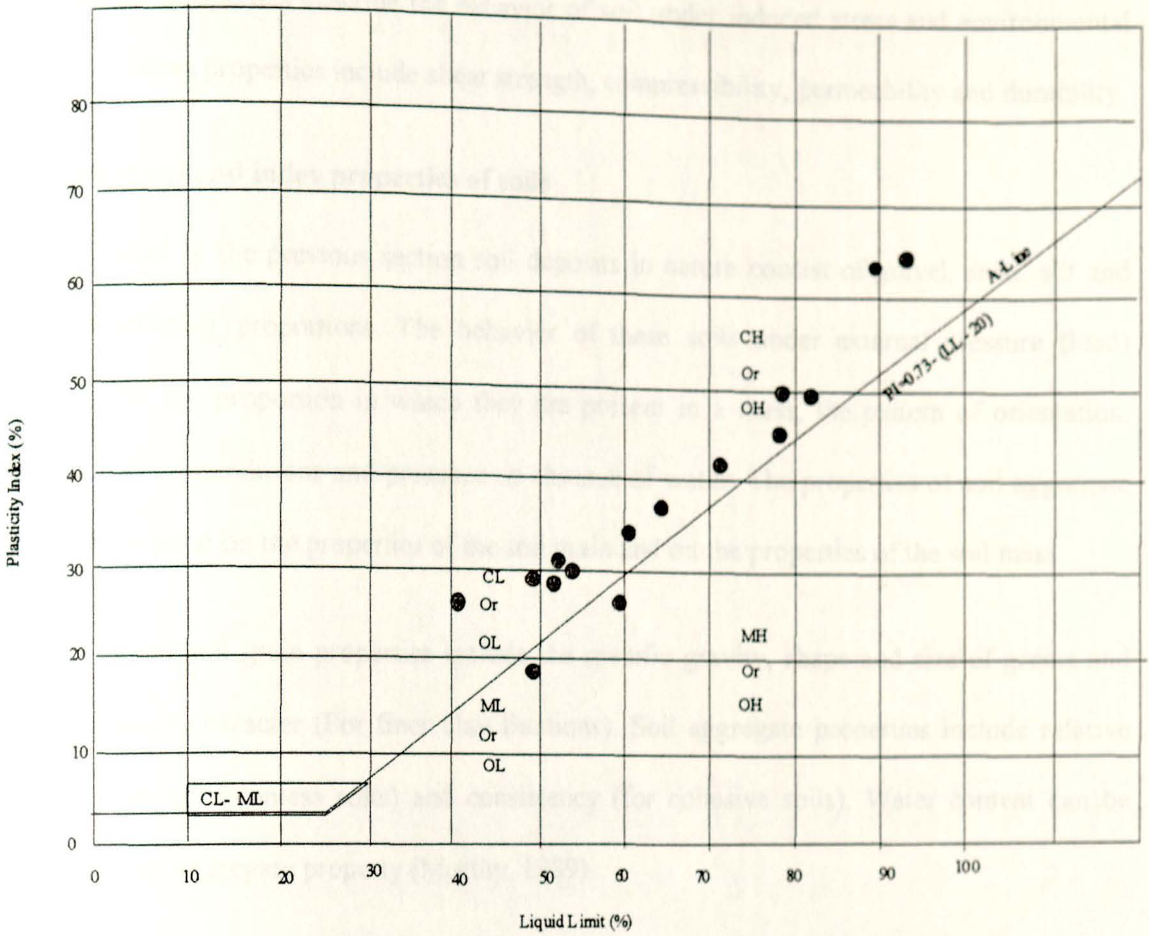


Figure 5.2 Plasticity Chart of soils of the study area based on USCS

5.4 Properties of soils

For a proper evaluation of suitability of soils for use as a foundation or construction material, information about its property should be known. Soil properties are classified as either physical, engineering or index.

Physical properties (properties that describe the state of a soil) include specific gravity, grain size, color, texture, structure, water content, unit weight and consistency. The principle indices used in geotechnical engineering include: plastic limit, liquid limit and shrinkage limit.

Engineering properties describe the behavior of soil under induced stress and environmental changes. These properties include shear strength, compressibility, permeability and durability.

5.4.1 Physical and index properties of soils

As discussed in the previous section soil deposits in nature consist of gravel, sand, silt and clay in different proportions. The behavior of these soils under external pressure (load) depends on the proportion in which they are present in a mass, the pattern of orientation, environmental conditions and presence or absence of water. The properties of soil aggregate basically depend on the properties of the soil grain and on the properties of the soil mass.

The principal soil grain properties include the specific gravity, shape and size of grains and mineralogical character (For finer clay fractions). Soil aggregate properties include relative density (for cohesionless soils) and consistency (for cohesive soils). Water content can be studied as an aggregate property (Murthy, 1989).

Thus the physical and index properties of soils can be used in identifying and classifying between different types of soils and to some extent in assessing their suitability for a particular engineering use as a foundation or construction material.

I. Specific gravity

Specific gravity is the ratio of the unit weight of a given volume of solid to the weight of an equal volume of water at 4^0C. Specific gravity is an important factor used in computing soil properties (such as unit weight, void ratio, consolidation tests on clays, degree of saturation etc). The specific gravity of soils within the study area is given in the table 5.2.

Table 5.2 Specific gravity and moisture content of soils of Gerbi basin

Soil type	Specific gravity (G)	Average Natural moisture content (%)
Brown soft silty CLAY (Alluvial soil)	2.625	36
Brown soft CLAY (Colluvial soil)	2.69	27.9
Brown gravely CLAY (Colluvial soil)	2.92	27
Brown soft CLAY (Residual soil)	2.765	30
Light greyish soft CLAY (Residual soil)	2.685	36
Greyish soft CLAY (Residual soil)	2.68	38

Specific gravity values for the study area ranges from 2.62 for brown soft silty CLAY to 2.92 for Brown gravely CLAY. Specific gravity values obtained are used in computing the properties indicated above.

II. Moisture content

Moisture content of a soil mass is the water content of the mass which is the ratio of the mass of water present to the mass of solids. The moisture content greatly influences the engineering properties of soils and their significance to a particular use. Soils with almost negligible moisture content are stiff. Soils with high moisture content act as water held suspensions.

Seasonal ground water fluctuations and rainfall variations can result in moisture content variation which has to be controlled since it has direct effect on strength and compaction properties and in determining consistency limits of the soil. It also affects the swelling and shrinkage properties of the soil.

Laboratory analysis to determine the natural moisture content of representative samples was conducted at the engineering geology laboratory in Addis Ababa University.

The results (given in table 5.2) indicate that the moisture content of the representative samples varies from 27 for gravely CLAY with silt and sand to 38 for greyish soft CLAY found in conjunction with alluvial soils. The results reveal that most soil samples are moist except the colluvial and some residual deposits found on higher topography beyond the dam axis of Gerbi river. Some higher moisture values could be due to the unseasonal and unusual rain during the collection of samples.

III. Grain size analysis

The size and shape of particles affects the behavior of soil mass. This size distribution analysis is the simplest means of describing soils. Analysis of grain size also helps to determine the engineering properties of soils (especially coarser grains).

The physical separation of a sample and classifying soils as gravel, sand, silt and clay is done as per ASTM standards.

Table 5.3 ASTM standard for classification of soils according to their grain sizes

Grain size (mm)	Soil name
> 4.5	Gravel
4.5 - 0.075	Sand
0.075 - 0.002	Silt
< 0.002	Clay

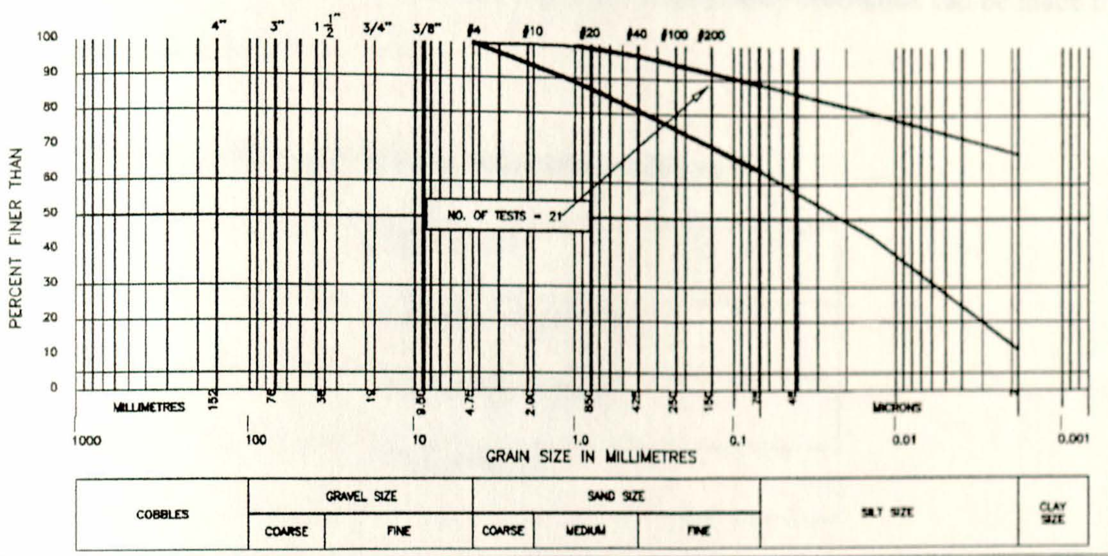
Mechanical analysis (determination of the weight of the material in fraction containing only particles of certain size) provides information on the uniformity or gradation of the material with in established size ranges. It also provides information for textural classification.

Mechanical analysis is divided into two:

I. Sieve analysis- For coarse fractions

II. Wet mechanical analysis (sedimentation or hydrometer analysis) - for fine fractions(less than 0.075mm in diameter).

Grain size analysis has been done for representative soil samples. The results of the grain size analysis are then represented on grain distribution curves with the grain size, D as abscissa on logarithmic scale and percent finer, P as ordinate on the arithmetic scale.



(Source: AAWSA, 1997)

Figure 5.3 Gradiation curve of representative soil samples

The shapes of the gradation curves have been used to describe the nature of soil tested. Nearly vertical lines represent uniformly graded soils. The shapes of the curves indicate that soils of the study are well graded and gap graded soils.

The grain distribution curves are also used to infer certain grain size characteristics such as uniformity coefficient (C_u) and coefficient of curvature (C_v) of soils.

These curves can also be used to estimate the hydraulic conductivity (permeability) of clean filter sandy soils (Hazen, as cited in Murthy, 1989).

The uniformity coefficient is expressed as:

$$C_u = \frac{D_{60}}{D_{10}} \qquad \text{Eq.5.1}$$

Where: C_u - Uniformity coefficient,

D_{60} -diameter of the particle at 60% finer on the grain size distribution curve

D_{10} -diameter of the particle at 10% finer on the grain size distribution curve

Determination of type of soil according to the value of uniformity coefficient can be made by considering table 5.4.

Table 5.4 Soil grading according to uniformity coefficient

C_u	Type of soil
<5	Uniform size particles
5 – 15	Medium graded soil
>15	Well grade soil

The gradation of particles can also be determined from coefficient of curvature (C_v) which is expressed as:

$$C_c = \frac{D_{30}^2}{D_{10} \times D_{60}} \qquad \text{Eq.5.2}$$

Where: C_c - Coefficient of curvature

D_{30}^2 - Size of particles at 30 % finer on the gradation curve

If C_c lies between 1 and 3 the soil is said to be well graded.

Calculated results of the uniformity coefficient for most soils of the study area are above and for some soils between 5 and 15 indicating most soils are well graded with few medium graded soils. Calculated results of the coefficient of curvature for most soils of the study area fall between 1 and 3 indicating that most soils of the study area are well graded.

IV. Atterburg limits and index values

Consistency is a term used to indicate the degree of firmness of cohesive soils. The consistency can be expressed by terms as soft, stiff, very stiff and hard. The physical properties of cohesive soils (clays) vary at different water contents; however two clay samples with the same water content may have different properties. Hence water content alone is not an adequate index to express consistency. Consistency can be expressed in terms of Atterberg limits and unconfined compressive strength (Murthy, 1989).

When the moisture content of a soil (fine grained) sample is reduced continually, the clay water mixture undergoes changes from liquid state through plastic state to solid state. The water content corresponding to the transition from one state to another is the Atterberg limit. Liquid limit is the transition state from the liquid state to plastic state. At this moisture content, the soil possesses small (probably the smallest) shear strength.

Plastic limit is the transition state from the plastic state to semi-solid state. Further decrease of water content leads to the point where there is no further decrease in volume.
Shrinkage limit is the moisture content below which there is no further decrease in volume. At this point the sample begins to dry and the color begins to change from dark to light.
The limits expressed above are expressed by their percentage water contents.

The Atterberg limits (liquid limits and plastic limits) of soils are determined in the laboratory according to ASTM procedure. Plasticity indices are calculated from these limit tests.

The liquid limit and plastic limits are determined using Casagrande's apparatus in the laboratory. The laboratory analysis of some soil samples was conducted at the laboratory of the Department of Geology and Geophysics, Addis Ababa University. Laboratory test results of some soil samples conducted by AAWSA are also used .The results of the Atterberg limit tests of representative soil samples are given in the table 5.1. Atterberg limit test results of the proposed borrow areas are given in table 8.4.

According to the result of the analysis the liquid limit of residual, alluvial and colluvial soils ranges from 48 to 86, 40 to 96 and 40 to 77, respectively. The average plastic limit values of the soil samples are 36, 33 and 40, respectively.

Plasticity index, which is an important measure of plastic behavior, is the range of water content between the liquid and plastic limit. Higher plasticity index indicates that the sample is more plastic and contains more clay. Usually, higher compressibility and higher swelling potential values are characteristics of plastic soils. These soils will have lower permeability. According to the result of the analysis the plastic index of residual, alluvial and colluvial soils ranges from 12 to 56, 16.8 to 65 and 11.8 to 56, respectively.

Description of plasticity of fine soils in terms of their range of liquid limits and plasticity index are given in Appendix D. Comparison of the liquid limit and plasticity index of the soils of the study area ranges given in these tables show the most of the fine soils are characterized by high plasticity.

The consistence of undisturbed sample is determined by an index called liquidity index or water plasticity ratio. The liquidity index is expressed as:

$$LI = \frac{w_n - w_p}{PI} \qquad \text{Eq. 5.3}$$

where; PI -is the plasticity index

w_p -is is the plastic limit

w_n - is the natural moisture content

LI - is the liquidity index

The liquidity index is calculated only for those soil samples tested at the laboratory of the Department of Geology and Geophysics, Addis Ababa University. The results are given in table 5.5.

Table 5.5 Liquidity and Consistency index values for representative soil samples

Sample	Natural moisture content (%)	Liquid limit (%)	Plastic limit (%)	Plasticity Index	Liquidity Index	Consistency Index
A1	53	53.2	32	20.34	1.03245	0.0098328
AA1	51	51	34	16.8	1.0119	0
C1	53	57.8	36	28	0.60714	0.1714285
CC1	58	58.3	46	11.8	1.01695	0.0254237
R1	52	56	40	26	0.46154	0.1538461
RR1	46	46	35	12	0.91667	0

The liquidity index of soils can vary from less than zero to greater than one. The following table gives the consistency of undisturbed sample according to LI values.

Table 5.6 Consistency of soil based on values of LI

LI	Consistency
<0	Semisolid or solid state
0	Very stiff state ($w_p = w_n$)
1	Very soft state (LL$= w_n$)
>1	Liquid state (when disturbed)

Liquidity index (LI) is a useful indicator of the behavior of fine grained soils when sheared.

Table 5.7 Properties of soils according to liquid index values

LI	Property of the soil at natural moisture content
Less than 0	Exhibit brittle stress strain behavior if sheared
0< LI <1	Will behave like a plastic
Greater than 1	Act like viscous liquid when sheared

The results indicate that soils of the study area are soft and plastic.

Consistency index (CI) is used to determine the field behavior (consistency) of clay soils. CI is expressed as:

$$CI = \frac{LL - w_n}{PI} \qquad \text{Eq.5.4}$$

Where; CI - consistency index

LL -the liquid limit

w_n -the natural moisture content

PI -is the plasticity index

Table 5.8 Soil property related to CI values

CI	*Soil property*
CI< 0	The soil behaves as liquid
O< CI <1	The soil is in a plastic state
CI >1	The soil is in a semi solid state and will be stiff

Soils having consistency index nearly equal to zero behave as a very soft soil with negligible shear strength. The consistency index of the representative soil samples fall between 0 and 1 indicating the soils are in plastic state.

V. Shrinkage limit

Shrinkage limit is the moisture content below which there is no further decrease in volume with the decrease in moisture content. Silts and sands are not susceptible to shrinkage as clay soils. The amount of shrinkage usually depends on the clay content and its mineralogy. The structural arrangement and the forces between clay particles in its natural state also affect the amount of shrinkage, so the test should be carried out on undisturbed specimen (Barnes, 1995). Shrinkage limit can be used to determine the degree of expansiveness of the soil as given in (Altmeyer, 1955) given in Annex D.

Soils of the study area have higher degree of expansiveness falling into the critical to marginal range. From the table which relates the degree of expansiveness and Shrinkage limit (given in Annex D) it can be inferred that the shrinkage limit of most soil samples of the study area is between 9 and 12.

VI. Free swell

Dry Clay soils swell when they come in contact with water and shrink when dry. Such expansive soils cause considerable damage on structures built on them. Hence swelling characteristics of clay soils should be determined.

Free swell is the ratio of the change in volume to the initial volume. Gibbs and Holtz (1956) proposed a test to determine free swell values.

The amount of clay content, initial condition of the soil sample and the time allowed for swelling influence the swelling potential. Swelling potential can be determined in the laboratory. It can also be determined from plasticity index values of the soil samples.

The formula developed by Chen, (1988) to calculate the swelling potential from plasticity index is given below.

$$S_p = 60K(PI)^{2.44} \qquad \text{Eq.5.5}$$

Where:

S_p- Swelling potential

K- constant , K= 3.6×10^{-5}, PI-Plasticity index

The swelling potential of representative soil samples as determined from plasticity index values is given in table 5.9.

Table 5.9 Swelling potential of soils as determined from Plasticity index

Sample	Plasticity Index (PI)(%)	Swelling potential (%)
A1	20.34	3.36
AA1	16.8	2.10
C1	28	7.33
CC1	11.8	0.8
R1	26	6.12
RR1	12	0.92
STP5-2A*	37	14.8
GB-96-S-24*	44	22.1
GB-96-S-24*	31	9.40

Table 5.10 Expansiveness based on swelling potential

SWELLING POTENTIAL (%)	EXPANSIVENESS
0- 1.5	Low
1.5- 5	Medium
5- 25	High
>25	Very High

The swelling potiential of most soil samples fall between 5 and 25 indicating that the soils are with in high swelling range which in turn indicates that most soils are expansive.

The swelling pressure which is the vertical pressure required to prevent volume change of laterally confined sample when the sample is allowed to take in water has a linear relationship with the liquid limit of that sample. A graph showing this relationship is given in figure 5.4.

From the graph it can be concluded that the swelling pressure required to prevent the volume change increases linearly with an increase in the liquid limit of the soil sample.

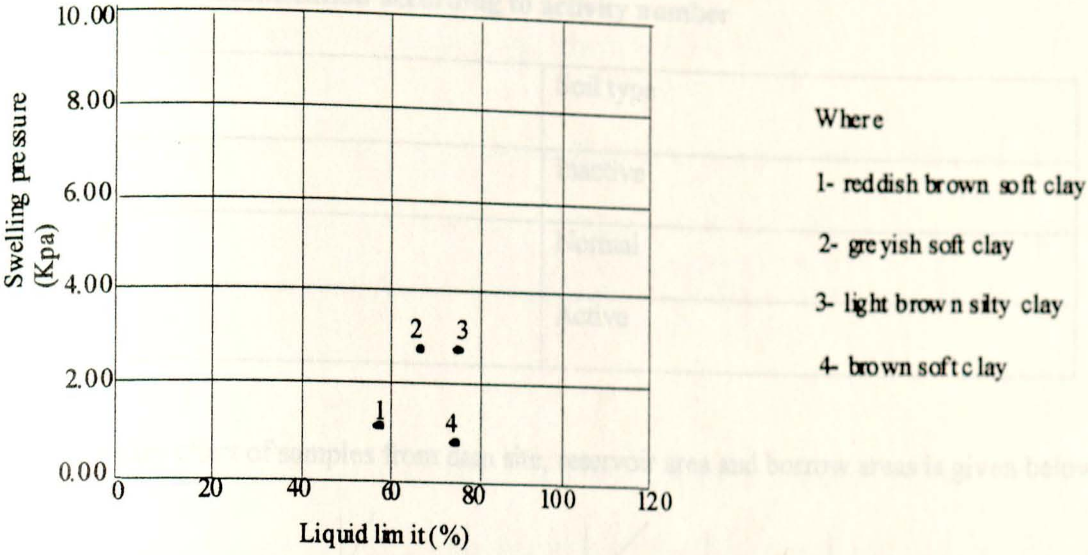


Figure 5.4 Swelling pressures - liquid limit relationship

VII. Activity of the clay soils

The significant change in the volume of a clay soil during shrinkage or swelling is a function of plasticity index and the quantity of colloidal clay particles (Skempton as cited in Murthy, 1989).

The activity of clay soil is the ratio of the plasticity index to the clay fraction in percent finer than two Microns.

$$A_c = \frac{PI}{CFractions} \qquad \text{Eq.5.6}$$

Where; A_c is the activity number

PI is the plasticy index

$CFractions$ - is clay percent finer than two microns

Type of soil according to value of A_c is given in table 5.11.

Table 5.11 Soil classification according to activity number

A_c	Soil type
<0.75	Inactive
0.75-1.40	Normal
>1.40	Active

The plasticity chart of samples from dam site, reservoir area and borrow areas is given below.

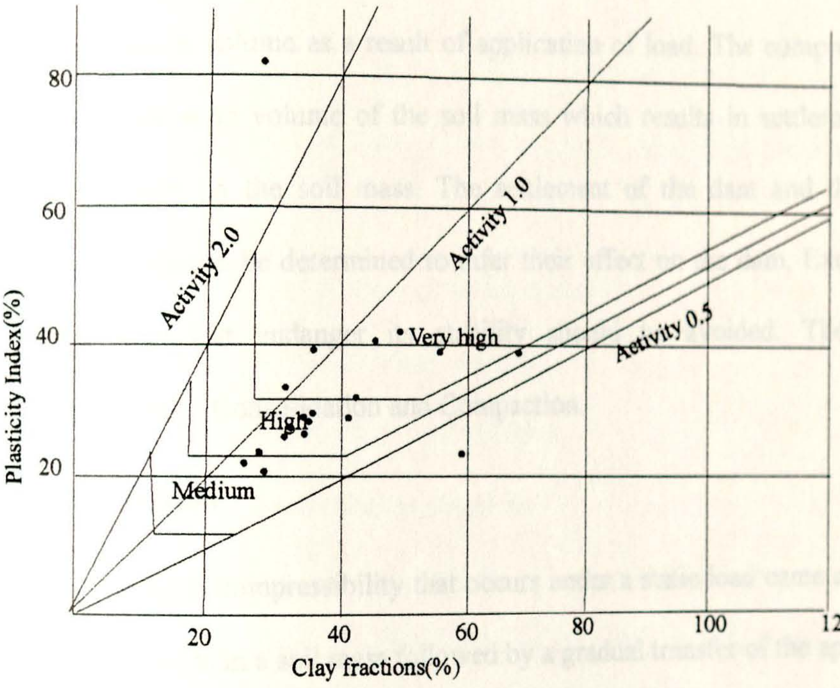


Figure 5.5 Plasticity chart of soil samples from dam site and reservoir area

The activity of the clay soils of the study area (AAWSA, 1997) is with in the range of 0 and 1 the average value being between 0.65 and 0.86. The results indicate that clay soils of the study area fall into the normal and active group indicating that most soils of the study area are very expansive when in contact with water and they have to be treated with care.

5.4.2 Engineering properties of soils

The suitability of a soil for a particular use depends on its response to that use. The suitability usually depends on one or more engineering properties of soils.

Engineering properties of soils are properties which help to determine the suitability of that soil for a particular use and to predict the performance of the soil. The engineering properties of soils include compressibility, shear strength and permeability.

I. Compressibility

Compressibility is the degree to which soil will change volume under a load (Bell, 1980).It is the decrease in volume as a result of application of load. The compression of the soil mass causes decrease in volume of the soil mass which results in settlement of the engineering structure built on the soil mass. The settlement of the dam and the compression of the foundation should be determined to infer their effect on the dam. Excessive settlements and deformations that endanger its stability should be avoided. There are two types of compressibility: Consolidation and Compaction.

A, consolidation

Consolidation is compressibility that occurs under a static load came about as water is driven away from voids in a soil mass followed by a gradual transfer of the applied pressure from the pore water to the soil skeleton Consolidation may occur due to external static loads from structures; due to self-weight of the soil such as recently placed fills; due to lowering of the ground water table; due to desiccation (Murthy, 1989).

The total compression of a saturated clay sample occurs as a result of immediate, primary and secondary consolidations. The portion of the settlement of a structure which occurs more or less simultaneously with the application of loads is the initial or immediate compression.

Primary consolidation occurs when the rate of compression of the soil layer is controlled solely by the resistance of the flow of water under the induced hydraulic gradients. Secondary consolidation occurs as a result of compression of the clay layer and starts after the primary compression ceases.

One dimensional consolidation theory (by Terzaghi as cited in Murthy, (1989)) is applied to estimate the primary consolidation of the soil mass. Confined compression or consolidation tests (which simulate Terzaghi's theory) were done on selected undisturbed soil samples taken from different soil units in the basin. The test is conducted by using one-dimensional oedometer (an apparatus devised by Terzaghi) according to ASTM procedures.

The main purpose of this test is to obtain necessary information on the compressibility properties and rate of settlement of the soil mass. Results of the analysis (Oedometer test) help to determine:

- the coefficient of primary consolidation (C_v)
- Coefficient of volume compressibility
- Coefficient of secondary consolidation

The following data should also be obtained:

-Moisture content and weight of the soil sample before the commencement of the test.

-Moisture content and weight of the sample after the completion of test.

-The specific gravity of the solids.

-The temperature of the room where the test is conducted.

I. Compression Index (C_c)

Compression Index represents the linear portion of the void ratio versus log δ graph. C_c is given as:

$$C_c = \frac{e - e_0}{Log\left(\frac{\delta_1}{\delta_0}\right)} \qquad \text{Eq.5.7}$$

Where:

- C_c is the compression index

- e and e_o represent the final and initial void ratios, respectively and

- δ_0 and δ_1 represent the initial and final effective stress, respectively

Settlement characteristics can be computed by analyzing the compression index. A linear relationship between the compression index and the liquid limit is developed by Terzaghi and Peck (1925). They developed an empirical relation for remolded clays and is given by:

$$C_c = 0.007 \times (LL - 10) \quad \text{Eq.5.8}$$

Where: LL is the liquid limit

C_c-is the compression index

The value of the compression index as determined from the liquid limit for representative soil samples is given in table 5.12.

Table 5.12 Compression index as determined from the liquid limit

Sample	Liquid limit (LL)	Compression Index (C_c)
A1	53.2	0.3024
AA1	51	0.287
C1	57.8	0.3346
CC1	58.3	0.3381
R1	56	0.322
RR1	46	0.252
GB-STP5-2A	68	0.406
GB-96-S-22	75	0.455
GB-96-S-24	76	0.462

II, Coefficient of volume compressibility (M_v)

Coefficient of volume compressibility is calculated using the formula given below (Mc-Carthy et al, 1993).

$$MV = \frac{\Delta e}{1+e_0} \times \Delta e \qquad \text{Eq.5.9}$$

where:

M_v- coefficient of volume compressibility

e_0 -initial void ratio and Δe is the change in void ratio.

Result of the coefficient of compressibility as calculated from void ratio values is given in Table 5.13.

III. The coefficient of primary consolidation (C_v)

The coefficient of consolidation relates the time elapsed for a given degree of consolidation induced for any pressure increment. Two graphic methods; the square root of time method (Taylor, 1942) and log time method (Casagrande and Fadum, 1940) are used to estimate the coefficient of consolidation. The results of the analysis are given in table 5.13.

IV. Coefficient of secondary consolidation (αc)

When the stress is increased the void ratio will decrease. The ratio of decrease in void ratio to increase in effective stress gives coefficient of compression or secondary consolidation (David, 1993). It is calculated from the slope of void ratio versus effective stress graph and is given by:

$$\alpha c = \frac{-\Delta e}{\Delta \delta} \qquad \text{Eq.5.10}$$

where:

αc - is coefficient of compression or secondary consolidation

$-\Delta e$ -decrease in volume

-$\Delta \delta$ - increase in stress

Table 5.13 Consolidation test results for samples taken from the dam site

Sample type and station	Depth	Presure (KPa)	Void ratio (e)	Coefficient of primary consolidation (C_v)-(Cm^2/s)	Coefficient of volume compressibility (M_v) Cm^2/Kg	Coefficient of secondary consolidation (αC)
Browinish silty clay (GB-96-S-21)	1.00 – 1.30 m e^0=1.124	100	1.086	2.519×10^{-3}	1.786×10^{-2}	4.599×10^{-3}
		200	1.055	2.300×10^{-3}	1.479×10^{-2}	5.396×10^{-3}
		400	0.966	2.255×10^{-3}	2.219×10^{-2}	7.198×10^{-3}
		600	0.9109	2.094×10^{-3}	1.413×10^{-2}	1.093×10^{-3}
		400	0.9131			
		100	0.9235			
Brown to red soft Clay (GB-96-S-21)	1.60-2.20 m e^0=0.80	100	0.749	4.260×10^{-4}	3.278×10^{-3}	2.078×10^{-4}
		200	0.786	3.900×10^{-4}	4.391×10^{-3}	6.721×10^{-4}
		400	0.764	8.411×10^{-4}	5.944×10^{-3}	1.072×10^{-4}
		600	0.744	8.119×10^{-4}	5.648×10^{-3}	1.547×10^{-4}
		400	0.746			
		100	0.756			
Greyish soft Clay (Gb-96-S-22)	3.10- 3.70 m e^0=1.203	100	1.10	4.447×10^{-3}	0.052	1.103×10^{-3}
		200	1.056	1.825×10^{-3}	0.020	1.644×10^{-3}
		400	0.970	6.688×10^{-4}	0.021	1.460×10^{-3}
		600	0.904	6.005×10^{-4}	0.016	1.450×10^{-3}
		400	0.908			
		100	0.937			
Light Greyish soft Silty Clay (Gb-96-S-23)	2.00- 2.60 m e^0=1.24	100	1.196	2.012×10^{-4}	1.917×10^{-2}	1.789×10^{-3}
		200	1.141	1.068×10^{-4}	2.468×10^{-2}	3.977×10^{-3}
		400	1.052	6.071×10^{-5}	2.123×10^{-2}	1.053×10^{-2}
		600	0.985	2.975×10^{-5}	1.649×10^{-2}	1.515×10^{-2}
		400	0.993			
		100	1.051			
Light brown silty Clay (Gb-96-S-23)	2.90- 3.50 m e^0=0.081	100	0.760	3.856×10^{-3}	2.284×10^{-2}	4.02×10^{-4}
		200	0.745	2.698×10^{-3}	8.427×10^{-2}	6.27×10^{-4}
		400	0.718	1.976×10^{-3}	7.704×10^{-3}	1.368×10^{-3}
		600	0.696	7.200×10^{-4}	6.359×10^{-3}	1.268×10^{-3}
		400	0.698			
		100	0.714			
Browinish soft Clay (Gb-96-S-24)	1.45- 2.05 m e^0=0.913	100	0.903	8.504×10^{-4}	5.151×10^{-3}	5.876×10^{-4}
		200	0.892	1.261×10^{-3}	5.699×10^{-3}	6.028×10^{-4}
		400	0.864	2.471×10^{-4}	7.363×10^{-3}	2.120×10^{-3}
		600	0.841	1.126×10^{-4}	6.124×10^{-3}	1.314×10^{-3}
		400	0.844			
		100	0.863			
Light greyish silty Clay (Gb-96-S-24)	2.25- 2.85 m e^0=0.913	100	1.077	2.581×10^{-4}	4.568×10^{-2}	3.233×10^{-3}
		200	1.032	9.544×10^{-5}	2.205×10^{-2}	3.586×10^{-3}
		400	0.957	6.964×10^{-5}	1.859×10^{-2}	8.798×10^{-3}
		600	0.903	5.054×10^{-5}	1.401×10^{-2}	1.019×10^{-2}
		400	0.913			
		100	0.966			

(Source: AAWSA, 1997)

B, Compaction

Compaction is an artificial densification of a soil and is considered when the soil is used as a construction material. Compaction is obtained by vibrating and loading and unloading the soil mass.

Compaction of a soil mass decreases the void ratio and increases the density of the mass. Compaction helps to improve the quality of the fill material used in dams; it increases the shear strength, decreases permeability and settlement. It is measured in terms of dry density of the soil which is the dry weight divided by one plus the moisture content.

Moisture content and the compactive effort play a major role in compaction of a soil mass. Method of compaction and type of soil also affect compaction.

The main aim of compaction is to attain maximum dry density. However, for a particular compactive effort there is only one moisture content which gives the maximum dry density. Increased compactive efforts at water contents higher than the optimum create excess pore pressures in the water filling the voids which facilitate shearing of the entire soil mass. So the soil mass should be properly compacted.

These properties are determined by laboratory tests such as standard compaction test and modified compaction tests carried out on representative samples (A.A.S.H.O. Test).

Compaction tests are carried out on soil samples collected from the borrow area proposed located 600m upstream of the dam axis (with in Gerbi reservoir) (Figure 5.7) towards the left of Gerbi river.

The standard proctor density and moisture content relationship of the soil samples is given in figure 5.6.

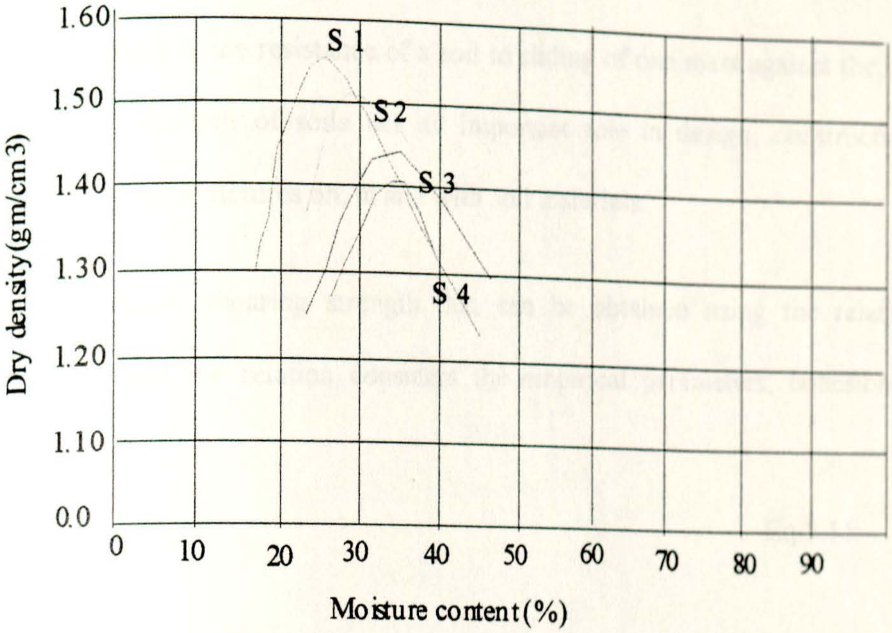


Figure 5.6 Standard proctor density and moisture relationship

Table 5.14 Compaction test results showing M.D.D and O.M.C

Soil type	Maximum Dry Density (M.D.D)	Optimum Moisture Content (O.M.C)
Brown soft CLAY (Colluvial soil) (S 2)	1.51	29.8
Brown soft CLAY (Alluvial soil) (S 4)	1.40	33.9
Dark brown soft CLAY (Residual soil) (S 3)	1.43	31.1
Brown soft Silty CLAY (Residual soil) (S 1)	1.54	28.5

(Source: AAWSA, 1997)

The laboratory results indicate that the dry density of soil samples increases when the moisture content increases up to a single point which is the optimum moisture content. Beyond this point further increase of water content reduces the dry density because the extra water starts to occupy the void spaces that should have been occupied by soil particle creating excess pore water pressure. Hence compaction of the soil must be performed at its optimum moisture content.

II. Shear strength

Shear strength is the resistance of a soil to sliding of one mass against the other (Bell, 1980). The shear strength of soils has an important role in design, construction and long-term stability of the structures on, in and with soil materials.

The maximum shearing strength that can be obtained using the relation developed by Coulomb and the relation considers the empirical parameters; cohesion (C) and internal friction (ϕ).

$$\tau_f = C + \sigma \tan\phi \qquad \text{Eq.5.11}$$

Where:

τ_f - shear strength

C- Coehsion

ϕ - angle of internal friction

σ - normal stress

For purely granular soils that are cohesionless(C=0) and the coulomb equation becomes:

$$\tau_f = \sigma \tan\phi \qquad \text{Eq.5.12}$$

The shear strength parameters can be determined in the undisturbed or remoulded state by any of the following methods.

-Laboratory methods include direct or box shear test, triaxial compression test or laboratory vane shear test.

-Field methods include the vane shear method, Standard penetration tests, dynamic cone penetrometer, pocket penetrometer test or any other indirect field strength tests method.

In-situ shear strength estimates for soils of Gerbi basin are derived and analyzed from standard penetration tests.

I. Standard penetration (SPT) tests

Standard penetration test helps to determine relative density of insitu cohesionless soils. SPT test data is given in the table below. SPT tests can also give an estimate of unconfined compressive strength of in situ cohesive soils.

Table 5.15 Relation between consistency, unconfined compressive strength, and penetration resistance (N), (USBR, 1998)

Consistency	*Unconfined compressive strength (q_u) (KN/m^2)*	*Penetration resistance (N)*
Very soft	<25	0-2
Soft	25-50	2-4
Medium	50-100	4-8
Stiff	100-200	8-15
Very stiff	200-400	15-30
Hard	>400	>30

Standard penetration tests were conducted at the dam site in vertically drilled borholes by Transport Construction and Design Enterprise (TCDE). The results are given in table 5.16.

Table 5.16 SPT results in vertically drilled Bore holes

Bore Hole number	Depth	Number of blows (N)
GB-96-7	1.60- 2.05	1/2/3
GB-96-8	1.60- 2.55	1/1/1
GB-96-13	2.10- 2.55	1/2/4
GB-96-14	2.45- 2.90	2/4/4
GB-96-15	2.60- 3.05	2/2/3

According to the consistency classification given in table 5.15 most soil samples of the study area have lower UCS value (<50) and are grouped as soft to very soft soils.

II. Laboratory tests

Triaxial compression tests were conducted by AE-HBT joint venture on soil samples of the study area. In the triaxial compression test, two or more identical samples of soil are subjected to uniformly distributed fluid pressure around the cylindrical surface. The sample is sealed in a water tight rubber membrane. Then axial load is applied to the soil sample until it fails. Although only compressive load is applied to the sample, it fails by shear on internal faces; and it is possible to determine the shear strength of the soil from the applied loads at failure.

In order to interpret the results of a triaxial compression test, it is necessary to analyze the stress relationships which exist at a point in the interior of a stressed body (Murthy, 1989). The test results are given in table 5.17.

Table 5.17 Shear strength parameters of soil samples taken from the dam site and reservoir area

Soil type	Cohesion (C) in KPa	Angle of internal friction (ϕ) (in degree)
Brown soft silty CLAY (Alluvial soil)	35	16
Brown soft CLAY (Colluvial soil)	55	21
Brown soft CLAY (Alluvial soil)	60	23
Greyish soft CLAY (Residual soil)	50	19

Shear failure occurs when the Shear stress exceeds the shear strength. The laboratory analysis indicates that the clay samples have lower shearing strength.

III. Permeability of soils

A material is permeable if it contains continuous void. The permeability of soils greatly affects the seepage loss through embankments or reservoirs. Hence, permeability of soils should be determined to come up with an appropriate design.

The coefficient of permeability can be determined from constant and variable head permeability tests or empirical methods can be used. The emperical method makes the use of coefficient of primary consolidation, coefficient of volume compression and unit weight of water. The formula is given as:

$$K = C_v M_v \gamma_w \qquad \text{Eq.5.13}$$

Where:

K-is the coefficient of permeability, C_v-is coefficient of primary consolidation

M_v- is coefficient of volume compression and γ_w- is the unit weight of water

The coefficient of permeability for representative soil samples is given in table 5.18.

Table 5.18 Coefficient of permeability for representative soil samples

Samples	Coefficient of Permeability (K)
Brownish silty clay	4.5×10^{-5}
Grayesh soft clay	1.4×10^{-5}
Light brown silty clay	1.3×10^{-6}
Brownish soft clay	1.4×10^{-9}

The permeability of the foundation soils ranges from 4.5×10^{-5} to 1.4×10^{-9}. The coefficient of permeability of most clay soils of the study area falls with in this range. The laboratory permeability teat results done by AAWSA also conform to these values. The results indicate that most soil samples have low permeability.

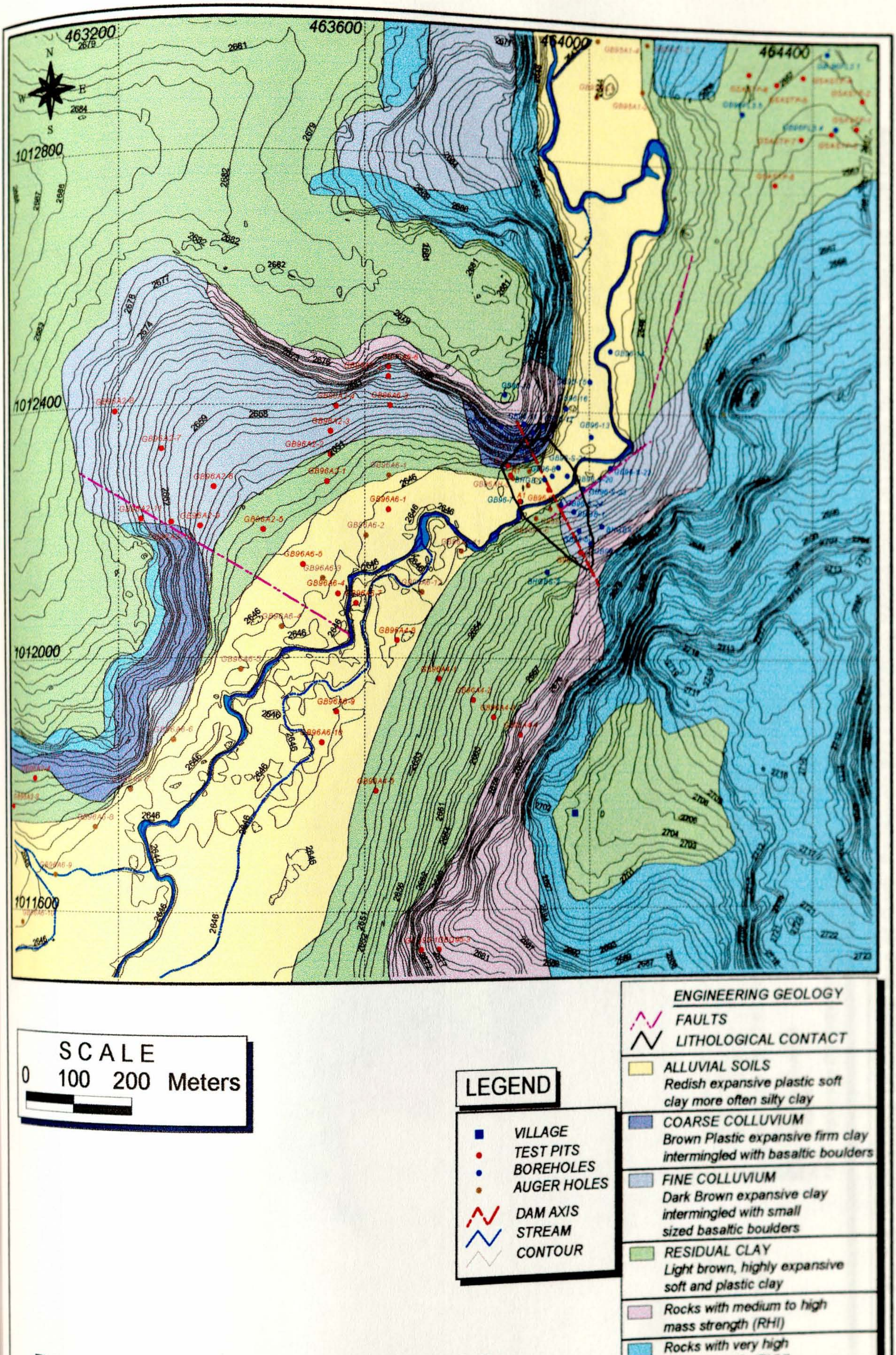


Fig 5.7 Engineering geologic map of Gerbi dam site

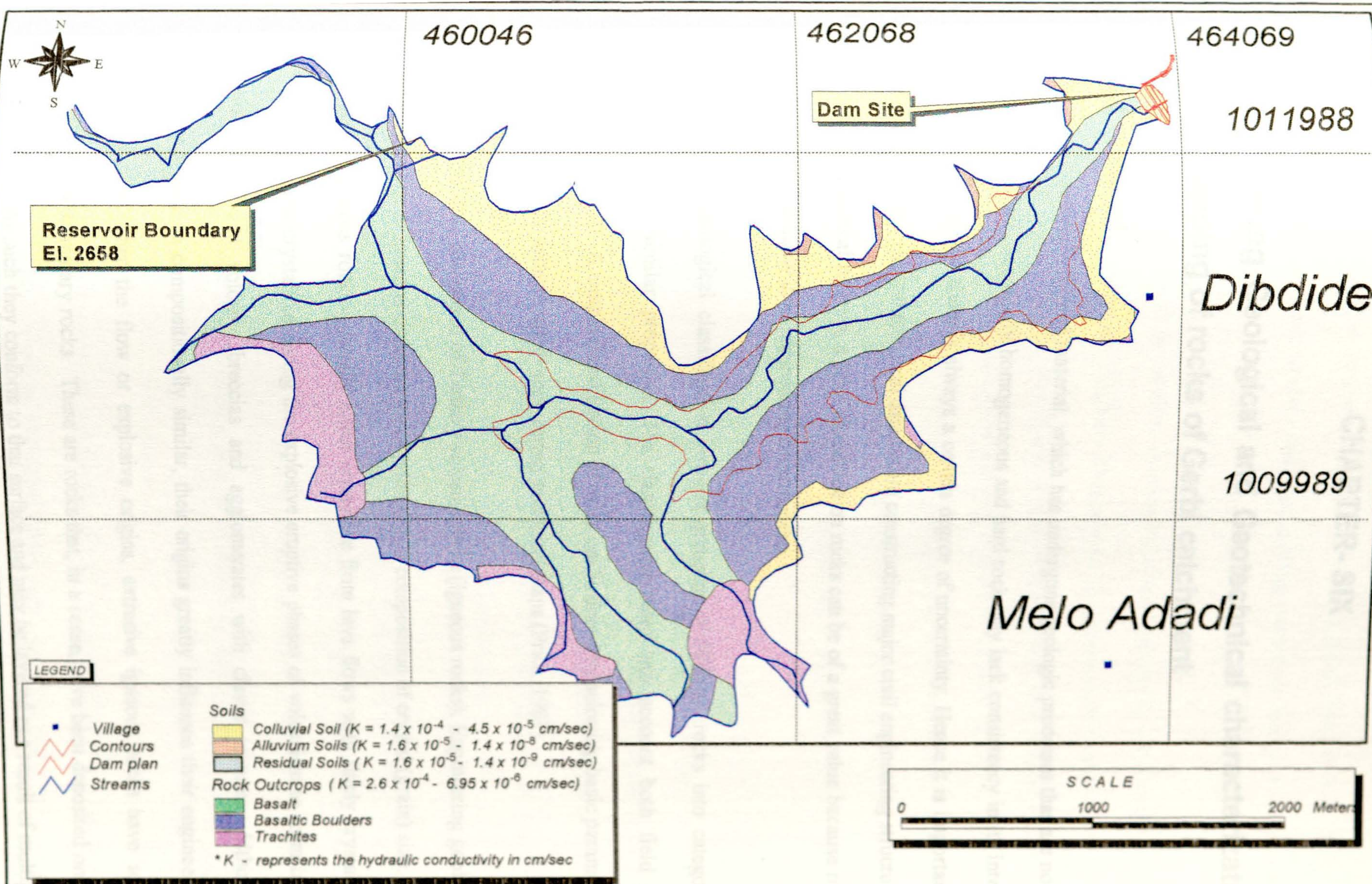


Fig 5.8 Engineering Geological Map of Gerbi Reservoir Area indicating Permeability (K) range for various soil and rock types

CHAPTER- SIX

Engineering Geological and Geotechnical characterization and mapping of rocks of Gerbi catchment

6.1 General

Rock is a naturally formed material, which has undergone geologic processes that are not yet known in detail. Seemingly homogeneous and hard rock may lack consistency in its internal state. This is to say, there is always a certain degree of uncertainty. Hence it is important to acquire geologic and geotechnical data before constructing major civil engineering structures. For many practical reasons, a brief discussion on rocks can be of a great value because rocks are involved in many civil engineering constructions.

Engineering geological classification involves grouping of different rocks into categories which possess similar properties. The classification takes into account both field and laboratory results. A simple classification of rocks can be drawn by selecting basic parameters such as strength, genesis, etc in engineering geological maps (Price, 1983).

Gerbi dam site is comprised of Tertiary volcanic rocks (Igneous rocks). Engineering geologic classifications of igneous rocks are based primarily on composition of crystal (grain) size. Extrusive igneous rocks generally either crystallize from lava flows with finely crystalline textures or they crystallize during the explosive eruptive phases of volcanism to form tuffs, welded tuffs and volcanic breccias and agglomerates with clastic textures. Although extrusives may be compositionally similar, their origins greatly influence their engineering properties. In either the flow or explosive origins, extrusive igneous rocks have some attributes of sedimentary rocks. These are rocks that, in a sense, have been deposited on the earth surface. As such they conform to that surface and may be layered as a result of multiple eruptions. Closely spaced columnar jointing characterizes many lava flows, whereas more widely spaced; sometimes crudely columnar jointing occurs in the more heterogeneous pyroclastic rock masses.

The emplacement mode of intrusive igneous rocks has engineering significance. Massive intrusive bodies tend to have relatively homogeneous composition and textures that are three dimensional throughout. On the contrary, tabular intrusive bodies such as dikes and sills may lack inherent three-dimensional continuity that is found in massive intrusives and hence it creates more construction or rock-utilization problems than massive intrusives. The typically sharp contact of intrusives with surrounding country rock may create stability problems where relatively planar contacts intersect tunnels and rock slopes in addition to normally occurring jointing.

The cooling histories of tabular intrusives that have different thickness introduce variations in crystal size and resultant differences in crushing strengths. Jointing perpendicular to the contact surfaces may result from cooling and additional jointing from other causes. All things considered, tabular intrusives present more problems in mapping and in engineering construction and rock use than the massive intrusives.

With in the study area rock outcrops are rare. Trachyte outcrop has been observed at the south eastern areas of the dam site where the proposed quarry site is located. Small basaltic outcrops have also been observed along the left bank of the proposed reservoir rim. Except these areas the rest of the dam site and reservoir area is covered with soils of residual, alluvial and colluvial types.

In order to assess the engineering significance of rock the engineering properties of intact rock and the rock mass must be evaluated. Intact rock represents rocks containing no discontinuities while rock mass is interrupted by discontinuities; it may be said that rock mass is the combination of intact rock and the discontinuities (joints, bedding planes, dykes, etc). In general, detailed description of intact rock and the rock mass is of profound importance to fully understand the engineering geologic conditions of the rocks of the study area.

6.2 Rock description

Full Description of rock mass includes description of the rock material and natural discontinuities. The description of the rock mass includes:

I. Rock name, lithological name (petrographic properties are included)

II. Properties of the rock material

III. Rock mass properties

Taking into account the above mentioned description methodology the lithologic units of Gerbi catchment are grouped in to three; basalts, trachytes and tuff and welded tuff.

Basalt is the most abundant rock type with in the study area. Basalt outcrops are found overlying the trachyte rock unit. Well developed jointing (systematic and non-systematic jointing) prevails on basaltic rocks that are exposed at the surface. Columnar jointing has been observed at some places as a result separate blocks of basalts are found at the surface.

The trachyte is found underlying the basalt. Trachyte outcrops are also observed in South and South Eastern part of the area. The trachyte is slightly to highly weathered. The joints observed on the trachyte outcrops are not very open as those on the basalt.

Tuff and welded tuffs are found underlying the trachyte unit. Tuffs are commonly stratified like sedimentary rock, reflecting deposition from the atmosphere. This rock is weak and is characterized by moderately to close jointing.

The lithologic units were discussed in chapter three under the title lithology of Gerbi dam site.

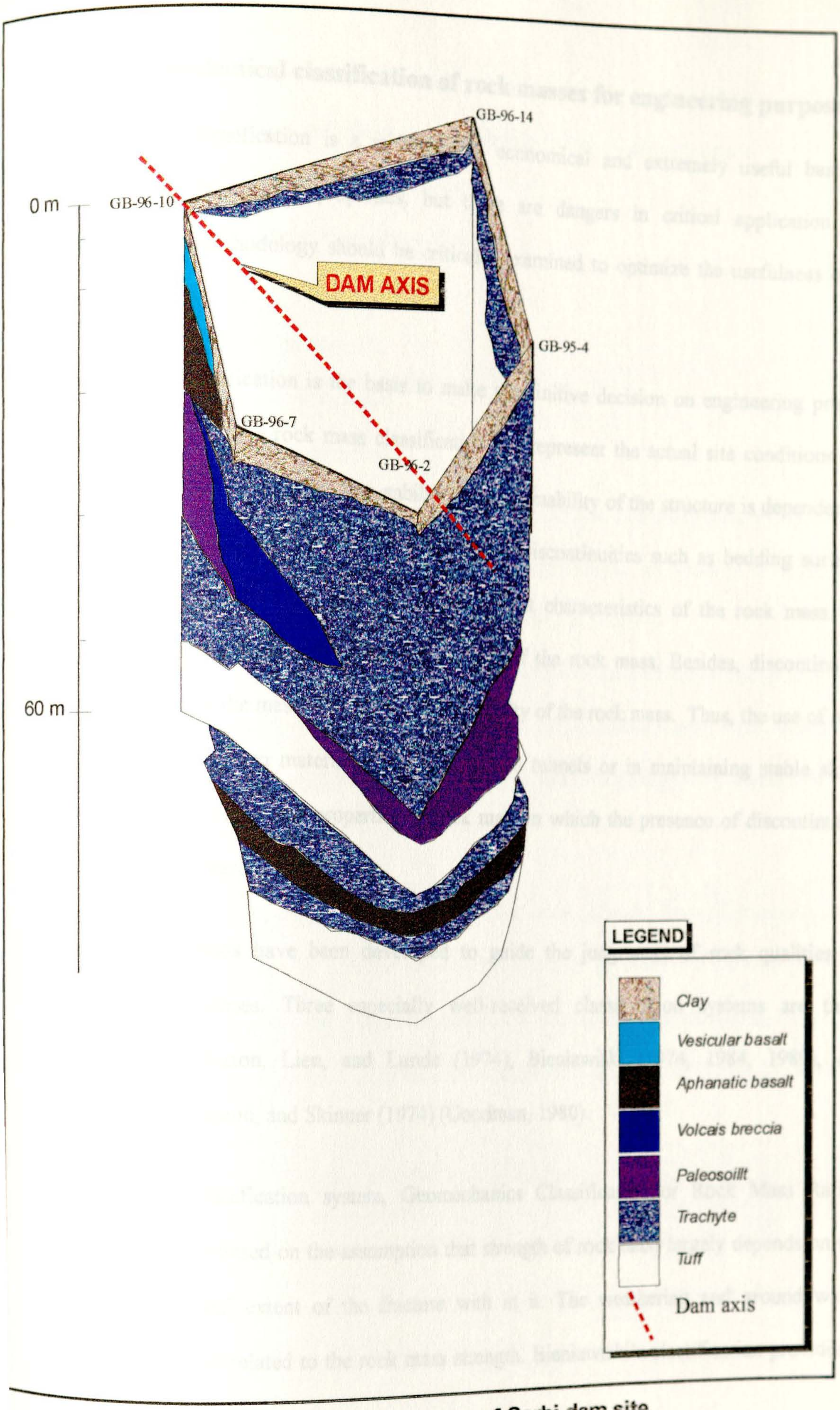


Fig. 6.1 Fence diagram of Gerbi dam site

6.3 Geomechanical classification of rock masses for engineering purposes

Rock mass classification is a widely used, economical and extremely useful basis for determining engineering properties, but there are dangers in critical application. The classification methodology should be critically examined to optimize the usefulness of the measured data.

Rock mass classification is the basis to make a definitive decision on engineering projects involving rock, since rock mass classification can represent the actual site conditions than intact rock classifications do. The stability and deformability of the structure is dependent on the strength and deformability of the rock mass. Discontinuities such as bedding surfaces, faults, joints and foliations are the most important characteristics of the rock mass. The presences of discontinuities reduce the Strength of the rock mass. Besides, discontinuities primarily control the mass strength and deformability of the rock mass. Thus, the use of rock, whether as foundation material in excavations and tunnels or in maintaining stable slopes involves determinations of properties of rock mass in which the presence of discontinuities govern the engineering character.

Numerous methods have been developed to guide the judgments of rock qualities for engineering purposes. Three especially well-received classification systems are those developed by Barton, Lien, and Lunde (1974), Bieniawiski (1974, 1984, 1989), and Wickham, Tiedemann, and Skinner (1974) (Goodman, 1980).

Bieniawiski's classification system, Geomechanics Classification or Rock Mass Rating System (RMR) is based on the assumption that strength of rock mass largely depends on the density, nature and extent of the fracture with in it. The weathering and ground water conditions are also related to the rock mass strength. Bieniawiski's classification provides a general rock mass rating (RMR) increasing with rock quality from 0 to 100 based on six parameters. The RMR classification system considers the following parameters:

-Unconfined compressive strength (UCS)

-Rock Quality Designation (RQD (%))

-Mean fracture spacing

-Discontinuity condition

-Ground water conditions and

-Orientation of discontinuities

The engineering evaluation of an area, considers measurement of discontinuity characteristics. Discontinuity characteristics mainly depends on, i) Orientation , ii) Spacing, iii) Continuity, iv) Surface characteristics and v) Separation and infilling of fractures.

The conditions of discontinuities, spacing and orientation of discontinuities and ground water conditions are determined in the field. The spacing of the discontinuities were measured at each locality and the condition of discontinuities and the groundwater conditions were visually assessed.

For the present study, data pertaining to RMR has been collected from 32 locations in the limited out crops. Schmidit hammer rebound value is used to determine the strength of the rock (UCS). When using the Schmidt hammer as a uniaxial strength measuring device, the rock strength will be calculated using the following formula proposed by Barton and Choubey (1977):

$$\log_{10} (\sigma_c) = 0.00088\gamma R + 1.01 \qquad \text{Eq. 6.1}$$

Where: σ_c =Uniaxial compressive strength in MPa , γ= Dry rock density in kN/m^3

R= rebound number of Schmidt hammer (to be evaluated according to the guidelines given in ISRM, 1978).

RQD values have been empirically estimated by using Palmstorm's volumetric count method, 1983 as cited in Bell, 1983. In this method RQD index is determined by counting the total number of discontinuities present in a one cubic meter of rock mass. The Palmstorm's relation is defined by;

$$RQD=115-1.5J_v \qquad \text{Eq. 6.2}$$

Where, J_v= total number of discontinuities in a 1 cubic meter of rock mass.

Based upon the measurements/observations at each locality each of the parameters was assigned ratings as per the standard tables of the RMR system. The sum total of the ratings for each parameter provides the RMR value which is presented in tables 6.1 and 6.2

Field joint measurements/ observations were taken from the trachyte out crop at the right bank of the dam site and the basalt on the left bank. These were the only two outcrops present around the dam site where discontinuity measurements were possible.

The RMR classifications of the two out crops are given in the following tables.

Table 6.1 RMR classification of the Aphanatic basalt

Location	RQD			Spacing	Rating	Discontinuity condition	GW Condition	Strength			RMR	Class
	Jv	RQD	Rating					Schimidt rebound number	UCS	Rating		
L27	5	98.5	20	30	8	30	15	41	69.16	7	80	II
L28	5	98.5	20	30	8	30	15	32	45.47	4	77	II
L29	6	95.2	20	30	8	30	15	38	60.13	7	80	II
L30	6	95.2	20	25	8	25	15	45	83.33	7	75	II
L31	5	98.5	20	25	8	25	15	38	60.13	7	75	II
L32	5	98.5	20	25	8	25	15	38	60.13	7	75	II

Table 6.2 RMR classification of the Trachyte

Location	RQD			Spacing	Rating	Discontinuity condition	GW Condition	Strength			RMR	Class
	Jv	RQD	Rating					Schimidt rebound number	UCS	Rating		
L1	8	88.6	17	25	10	25	10	34	49.91	4	66	II
L2	9	85.3	17	20	10	20	10	36	54.78	7	64	II
L3	11	78.7	17	30	8	30	12	38	60.13	7	74	II
L4	6	95.2	20	25	8	25	12	37	57.40	7	72	II
L5	10	82	17	30	8	30	12	39	63.00	7	74	II
L6	11	78.7	17	25	10	25	0	40	66.01	7	59	III
L7	8	88.6	17	30	10	30	15	36	54.78	7	79	II
L8	5	98.5	20	30	10	30	15	40	66.01	7	82	I
L9	13	72.1	13	25	8	25	15	32	45.47	4	65	II
L10	5	98.5	20	30	10	30	15	40	66.01	7	82	I
L11	11	78.7	17	30	8	30	15	36	54.78	7	77	II
L12	12	75.4	17	30	8	30	13	37	57.40	7	75	II
L13	13	72.1	13	30	8	30	15	38	60.13	7	73	II
L14	13	72.1	13	25	8	25	13	31	43.40	4	63	II
L15	6	95.2	20	25	10	25	12	36	54.78	7	74	II
L16	9	85.3	17	25	10	25	13	40	66.01	7	72	II
L17	5	98.5	20	25	10	25	10	36	54.78	7	72	II
L18	6	95.2	20	30	8	30	15	36	54.78	7	80	II
L19	13	72.1	13	30	8	30	15	36	54.78	7	73	II
L20	9	85.3	17	25	10	25	15	34	49.91	4	71	II
L21	6	95.2	20	25	8	25	15	38	60.13	7	75	II
L22	9	85.3	17	30	8	30	13	44	79.54	7	75	II
L23	6	95.2	20	30	8	30	7	35	52.29	7	72	II
L24	10	82	17	30	8	30	15	31	43.40	4	74	II
L25	9	85.3	17	30	10	30	15	35	52.29	7	79	II
L26	9	85.3	17	30	10	30	15	42	72.46	7	79	II

According to the RMR classification all the aphanatic basalts and the trachites are within the range of good quality (class II).

6.4 Engineering properties of rocks

The concepts of engineering properties of rocks are highly applicable in the design of major civil engineering works. These properties can serve as a basis in explaining, clarifying or interpreting geological structures affecting civil engineering constructions. The main engineering properties considered include: modulus of deformation, shear strength and permeability characteristics of the rock mass.

Reliable estimates of strength and deformation characteristics of rock masses are required for almost any form of analysis to be applied in the design of slopes, foundation and underground excavations. Even though evaluations of these properties are very difficult it is essential to estimate these properties and come up with reliable results.

Hoek and Brown (1980) proposed a method for estimating the strength of jointed rock mass based on an assessment of the interlocking of rock blocks and the condition of the surface between these blocks. This method was modified over the years to solve problems related to parameters that are not considered when the original criterion was developed. The application of this method to very poor quality rock masses required further changes and the development of a new classification called the Geological Strength Index (GSI) (Hoek, 1995).

The generalized Hoek-Brown failure criterion for jointed rock mass is given by the equation:

$$\sigma_1' = \sigma_3' + \sigma_c\left(m_b\frac{\sigma_3'}{\sigma_c} + s\right)^a \qquad \text{Eq. 6.3}$$

Where:

- m_b -the value of the constant m for the rock mass

- s and a - constants which depends on the characteristics of the rock mass

- σ_c - the uniaxial compressive strength of the intact rock pieces and

- σ_1' and σ_3' - the axial and confining effective principal stresses, respectively.

The failure of good to reasonable quality rocks (similar to most rock types found within Gerbi basin) can be defined by setting a=0.5 in the equation 6.3:

$$\sigma_1' = \sigma_3' + \sigma_c\left(m_b \frac{\sigma_3'}{\sigma_c} + s\right)^{0.5} \qquad \text{Eq. 6.4}$$

For poor quality rock masses where the interlocking has been partially destroyed by shearing or weathering, the tensile strength or cohesion has to be approximately zero and rock specimens will fall apart. For such cases the value of's' is set to be zero (s = 0) in Eq. 6.3 giving:

$$\sigma_1' = \sigma_3' + \sigma_c\left(m_b \frac{\sigma_3'}{\sigma_c}\right)^{a} \qquad \text{Eq. 6.5}$$

In order to use Hoek-Brown failure criterion to determine the strength and deformation characteristics of the rock mass the following parameters have to be estimated/assessed.

-the uniaxial compressive strength of the intact rock pieces(σ_c)

- m_b'- the value of the Hoek-Brown constant

- the value of the Geological Strength Index (GSI) for the rock mass

-s and a- values of the material constants

Reliable results of the strength of intact rock can be obtained from triaxial tests. When time or budget constraints do not allow a triaxial testing both uniaxial compressive strength and material constant of the rock mass can be estimated from the standard tables given by Hoek and Brown (1992). The tables are developed based on analysis of triaxial test results on intact rock (Hoek, 1983; Doruk, 1991; Hoek et al., 1992).

Table 6.3 Field Estimates of Uniaxial Compressive Strength

Grade	*Term*	*Uniaxial Compressive Strength (MPa)*	*Point Load Index (MPa)*	*Field Estimate of Strength*
R6	Extremely Strong	> 250	> 10	Rock material only chipped under repeated hammer blows
R5	Very Strong	100 - 250	4 - 10	Requires many blows of a geological hammer to break intact rock specimens
R4	Strong	50 - 100	2 - 4	Hand held specimens broken by a single blow of a geological hammer
R3	Medium Strong	25 - 50	1 - 2	Firm blow with geological pick indents rock to 5mm, knife just scrapes surface
R2	Weak	5 - 25	***	Knife cuts material but too hard to shape into triaxial specimens
R1	Very Weak	1-5	***	Material crumbles under firm blows of geological pick, can be scraped with knife
R0	Extremely Weak	0.25 - 1	***	Indented by thumbnail

* Grade according to ISRM (1981)

*** Rocks with a uniaxial compressive strength below 25 Mpa are likely to yield highly ambiguous results under point load testing

Table 6.4 Values of the constant m_i for intact rock, by rock group. The values in parenthesis are estimates.

Rock type	Class	Group	Texture			
			Course	Medium	Fine	Very Fine
Sedimentary	Clastic		Conglomerate (22)	Sandstone 19	Siltstone 9	Claystone 4
	Non clastic	Organic		←Chalk→ 7 ←Coal→ (8-21)		
		Carbonate	Breccia (20)	Sparitic Limestone (10)	Micritic Limestone 8	
		Chemical		Gypsum 16	Anhydrite 13	
Metamorphic	Non foliated		Marble 9	Hornfels (19)	Quartizite 24	
	Slightly foliated		Migmatite (30)	Amphibolite (31)	Myolinite (6)	
	Foliated*		Gneiss 33	Schist (10)	Phylites (10)	Slate 9
Igneous	Light		Granite 33 Granodiorite (30) Diorite (28)		Rhyolite (16) Dacite (17) Andesite 19	Obsidian (19)
	Dark		Gabbro 27 Norite 22	Doletrite (19)	Basalt (17)	
	Extrusive pyroclastic type		Agglomerate (20)	Breccia (18)	Tuff (15)	

*These values are for intact rock specimens tested normal to foliation. The value of m_i will be significantly different if failure occurs along a foliation plane (Hoek, 1983).

To come up with values for the material constants 'm_b', 's' and 'a' Hoek and Brown (1988) suggested an empirical method that uses RMR values from the 1976, 1989 version of Bieniawiski's rock mass rating assuming completely dry condition and favorable joint orientations.

Using Bieniawiski's 1976 Rock mass rating:

For $RMR_{76}>18$ $GSI=RMR_{76}$

For $RMR_{76}<18$ Bieniawiski's 1976 Rock mass rating can't be used to estimate GSI instead Q value by Barton, Lien and Lunde is used.

Using Bieniawiski's 1989 Rock mass rating:

For $RMR_{89}>23$ $GSI=RMR_{89}-5$

For $RMR_{89}<23$ Bieniawiski's 1989 Rock mass rating can't be used to estimate GSI instead Q' value by Barton, Lien and Lunde is used. The value of Q' can be used to estimate the value of GSI using the following equation:

$$GSI = 9Log_e Q' + 44 \quad \text{Eq. 6.6}$$

The minimum value of Q' is 0.0208 with GSI value 9.

The method (using Bieniawiski's 1989 Rock mass rating) is applicable for rock masses with RMR>25, it doesn't work for poor quality rock masses with RMR<18. To overcome this limitation Geological Strength Index (GSI) is introduced and its value ranges from about 10 (for poor quality rock mass) to 100 (for intact rock).The relationship between m_b/m_i, a, s and GSI is given below:

For GSI>25(Undisturbed rock mass)

$$\frac{m_b}{m_i} = \exp\left(\frac{GSI - 100}{28}\right) \qquad \text{Eq.6.7}$$

$$s = \exp\left(\frac{GSI - 100}{9}\right) \qquad \text{Eq.6.8}$$

$$a = 0.5$$

For GSI<25(Undisturbed rock masses)

$$s = 0$$

$$a = 0.65 - \frac{GSI}{200} \qquad \text{Eq.6.9}$$

For the present study, the calculated values of σ_c, s, a, GSI and m_b are given in the following table.

Table 6.5 Calculated results σ_c, s, a, GSI and m_b

Hoek and Brown constants	Basalt	Trachyte
σ_c(MPa)	59.33	57.47
m_i	17	17
GSI	71	67
s	0.000599	0.000215
a	0.5	0.5
m_b	1.5657	1.1268

To find values of the shear strength parameters a Mohr-circle based on the calculated values has to be drawn. To draw the circles, different σ_3 values are considered, the maximum being one fourth of the uniaxial compressive strength, and the corresponding values for σ_1 is determined by using Eq. 6.3.

Table 6.6 Values of the normal stress

Tests	Basalt		Trachyte	
	σ_3	σ_1	σ_3	σ_1
1 ($=\sigma_c/4$) max value	14.83	51.9756	14.36	44.86722
2	12	45.41995	12	39.89001
3	10	40.51386	10	35.46231
4	6	29.65379	6	25.73023

The Mohr-envelope relating normal and shear stress can then be drawn by the method proposed by Hoek and Brown (1980). For any particular problem the effective stress is calculated using an appropriate stress analysis technique and the shear strength developed at that value is then calculated using the equation developed by Hoek and Brown (1997).

Balmer (1952) published a solution to estimate the equivalent set of strength parameters (cohesion and angle of internal friction) for given Hoek and Brown values in which the normal and shear stresses are expressed in terms of the corresponding principal stresses given below.

$$\sigma_n = \sigma_3 + \frac{\sigma_1 - \sigma_3}{\partial\sigma_1 / \partial\sigma_3 + 1} \qquad \text{Eq.6.10}$$

$$\tau = (\sigma_n - \sigma_3)\sqrt{\frac{\partial\sigma_1}{\partial\sigma_3}} \qquad \text{Eq.6.11}$$

For GSI > 25, when a=0.5:

$$\frac{\partial\sigma_1}{\partial\sigma_3} = 1 + \frac{m_b\sigma_c}{2(\sigma_1 - \sigma_3)} \qquad \text{Eq.6.12}$$

And for GSI < 25, when s=0:

$$\frac{\partial\sigma_1}{\partial\sigma_3} = 1 + am_b^a\left(\frac{\sigma_3}{\sigma_c}\right)^{a-1} \qquad \text{Eq.6.13}$$

Where σ_n and τ are the normal and shear stress respectively. The set of σ_n and τ values are given in the following tables.

Table 6.7 Set of σ_n and τ values for basalt

σ_n	τ
26.257	17.143
21.858	15.240
18.663	13.758
11.967	10.273

Table 6.8 Set of σ_n and τ values for trachyte

σ_n	τ
24.325	14.307
20.823	12.970
17.782	11.730
11.418	8.806

Once a set of (σ_n, τ) is determined then linear regression analysis in which the best fitting straight line is calculated for the range of (σ_n, τ) pairs can be used to calculate the average cohesion and friction angle. The cohesion and friction angle can also be determined by drawing mohr circles from (σ_1, σ_3) pairs.

For the present study the linear regression analysis has been used. The results obtained indicate that the angle of internal friction is 40° and 38° and the value of cohesion is 4 and 4.5 for the trachyte and basalt, respectively.

I. Modulus of deformation

Rocks deform when they are subjected to stresses. The deformation can be elastic or plastic and results in the deformation of the rock material and closure of discontinuities.

There is a close relation between geological structure of a rock mass and a factual situation, and the generated deformations must not create danger to the engineering structure. The structure should be absolutely safe. Thus, when engineering designing is done it should consider investigations that include the geological structure of a rock mass.

However, the application of such methods requires a detailed analysis of a rock mass on the base of geological survey, and also their verification using measurements of deformation. A rock mass is a very complex medium, and its thorough study with regard to the rock's geo-mechanical properties is practically difficult.

Deformability of rock mass can be determined using direct in situ tests or if it is not feasible to perform the direct in situ test indirect empirical methods are used. Hence, some parameters from RMR results are used to determine the deformation characteristics of the rocks of the study area.

RMR values are related to shear and deformation parameters of the rock. Thus, RMR results are used to determine the modulus of deformation using the relations developed by Serafim and Periera (1983).

For rock masses where 20< RMR<85 Serafim and Periera (1983) developed a formula relating the RMR value and the modulus of deformation given as:

$$E_d = 10^{\frac{(RMR-10)}{40}} \qquad \text{Eq.6.14}$$

Where E_d - is the modulus of deformation in Gpa.

For rock masses having lower RMR values Serafim and Pereia (1983) developed a formula given as:

$$E_d = 10^{\frac{(GSI-10)}{40}}, \text{ where GSI>25} \qquad \text{Eq.6.15}$$

Where E_d - is the modulus of deformation in Gpa.

Since the RMR values of rock outcrops of the study area are greater than 25 the Eq.6.14 is used to calculate the modulus of deformation.

The result indicates that values of the modulus of deformation are between 15 and 63 GPa indicating that both rock types provide a sound foundation condition.

II. Shear strength

Near any geotechnical construction (e.g. slopes, excavations, tunnels and foundations) there will be both mean and normal stresses and shear stresses. The mean or normal stresses cause volume change due to compression or consolidation.

The shear stresses prevent collapse and help to support the geotechnical structure. Shear stress may cause volume change. Failure will occur when the shear stress exceeds the limiting shear stress (strength).

To determine the shear strength of rock masses, the shear strength parameters should be determined.

Hoek and Brown (1980) relation is used (discussed in section 6.2) to estimate the strength parameters of jointed rock mass, based on an assessment of the interlocking of rock blocks and the condition of the surface between these blocks. Based on the analysis of the result obtained the angle of internal friction is 40° and 38° and the value of cohesion is 4 and 4.5 for the trachyte and basalt, respectively.

RMR values can also be used to determine the shear parameters; cohesion(c) and angle of internal friction (Φ) (Bieniawiski, 1989).Tables 6.9 and 6.10 present the Cohesion (C) and angle of internal friction (Φ), as determined from RMR. The Cohesion and angle of internal friction for trachytes, as determined from RMR, falls in the class from 2.95 to 4.1 and 34.5° to

46°, respectively whereas, Cohesion and angle of internal friction for basalts falls in the class from 3.75 to 4 and 42.5° to 45°, respectively.

Bieniawiski, also proposed the following relation to determine the specific values for cohesion and angle of internal friction for a given RMR value (Bell, 1983). Thus, the shear parameters are calculated using the following formulas;

Φ=0.5 RMR + 5, Φ - angle of internal friction Eq.6.16

C=0.05 RMR, C - Cohesion (Kg/cm^2) Eq.6.17

The results of the shear parameters for the two outcrops are given in the tables below.

Table 6.9 Shear strength parameters of the Aphanatic basalt

Location	*RMR*	*Shear strength parameters*	
		C(Kg/cm2)	*Φ*
L27	80	4	45
L28	77	3.85	43.5
L29	80	4	45
L30	75	3.75	42.5
L31	75	3.75	42.5
L32	75	3.75	42.5

Table 6.10 Shear strength parameters of the Trachyte

Location	*RMR*	*Shear strength parameters*	
		C(Kg/cm2)	*Φ*
L1	66	3.3	38
L2	64	3.2	37
L3	74	3.7	42
L4	72	3.6	41
L5	74	3.7	42
L6	59	2.95	34.5
L7	79	3.95	44.5
L8	82	4.1	46
L9	65	3.25	37.5
L10	82	4.1	46
L11	77	3.85	43.5
L12	75	3.75	42.5
L13	73	3.65	41.5
L14	63	3.15	36.5
L15	74	3.7	42
L16	72	3.6	41
L17	72	3.6	41
L18	80	4	45
L19	73	3.65	41.5
L20	71	3.55	40.5
L21	75	3.75	42.5
L22	75	3.75	42.5
L23	72	3.6	41
L24	74	3.7	42

A perusal of Table 6.9 and 6.10 indicates that the average values of C and Φ for basalt is 3.8 Kg/cm^2 and 44°, respectively. Whereas, for the trachyte C and Φ values are 3.6 Kg/cm^2 and 42°, respectively. The shear strength parameters obtained using Hoek and Brown (1980) failure criterions are also close to these results.

Further, for the present study an empirical method developed by Hoek and Brown (1980) is used to determine the rock mass strength. The formula developed is given as:

$$UCS_{RM} = \frac{2 \times C \times \cos\phi}{1 - \sin\phi} \qquad \text{Eq.6.18}$$

Where, C is the cohesion, ϕ is the angle of internal friction and UCS_{RM} is the unconfined compressive strength for rock mass.

Thus, the UCS for basalt and trachyte is calculated to be 6 and 2.2 Kg/cm^2, respectively indicating that the basalt is stronger than the trachyte. However both rock types provide good strength.

III. Permeability of the rock mass

In highly permeable rocks excessive seepage beneath a dam may damage the foundation. Seepage rates can be lowered by reducing the hydraulic gradient beneath the dam by incorporating cut-off in to the design (Bell, 1980). Cutoff walls are placed along the bottom of the cutoff trench to decrease seepage along the plane of contact between natural slope of rock and the rolled fill. The rate of seepage can also be reduced by placing an impervious earth fill against the lower part of the upstream portion of the dam.

The dam must be designed properly so that there is no dangerous seepage both through the dam and the foundation. Thus the impermeability of rocks should be assessed so that the harmful effect of seepage water is controlled.

Permeability tests to determine the hydraulic conductivity in rocks is done using single and double packer test techniques. In these testings water is added (pumped) into the boreholes at different pressure and the rate at which water flows to the ground is measured in the section sealed by packer glands. The test is conducted by Transport Construction and Design Enterprise (TCDE) staff.

The units observed with depth in the boreholes are clay, vesicular basalt, gravely clay, aphanatic basalt, paleosoil, trachyte and tuff.

In most boreholes value of the hydraulic conductivity (K) decreases with depth, the section found at depth is found to be quiet tight. K values will show an increase when the trachyte rock unit ends and tuff unit begins. Some variation of permeability is also noted with in the same rock unit at different depth. The decrease of K value with depth is due to the closure of joints with depth. No dangerous joint orientations (joints dipping downstream) were observed. K value of a representative borehole is given in the table below.

Table 6.11 Hydraulic conductivity of Borehole GB-96-6.

Test interval (m)	Permeability (K) (cm/sec)
7- 11.85	$(0- 2.76) \times 10^{-5}$
11.85- 17.10	$(1.84- 2.16) \times 10^{-5}$
17.10- 21.15	$(5.24- 7.45) \times 10^{-5}$
21.15- 25	$(1.99- 22.6) \times 10^{-7}$
25- 30	$(1.21- 1.71) \times 10^{-5}$
30- 35.2	$(4.86- 6.06) \times 10^{-5}$
35.2- 40.1	$(1.38- 1.73) \times 10^{-5}$
40.1- 45	$(6.25- 37.9) \times 10^{-7}$
45- 50	$(8.62- 29.5) \times 10^{-7}$
50.0- 55	$(6.77- 10.2) \times 10^{-6}$
55- 60.0	$(0.84- 1.20) \times 10^{-5}$

(Source:TCDE)

It was generally considered that rock showing permeability of less than 10^{-9} cm/sec would not require grouting (Izharul et al, 1983). The results of permeability tests indicate that K values are lower indicating the dam foundation exhibits low permeability. Therefore, the dam requires no grouting but clay blanketing is wise.

6.5 Engineering Geological classification of rocks

Engineering geological classification involves grouping together rock masses having similar properties such as strength, degree of weathering, discontinuity properties etc. This classification is vital in mapping of engineering geological rock units.

The principle that the physical and engineering geological properties of rocks in the present state are dependent on the origin, subsequent diagenetic, metamorphic and tectonic history and on weathering processes should be used in the classification of rocks and soils (Price and Rangers, 1983).

This Classification is a basic principle in engineering geological mapping which makes it possible to determine the lithological and physical characteristics of the rocks and their spatial distribution.

Thus classification of rocks of the study area is made based on strength, degree of weathering, discontinuity properties of the rock mass. These properties are determined from field observations and when possible from laboratory test results.

Based of the field and laboratory test results the rocks of the study area are classified into three rock mass strength units.

I. Rocks with very high mass strength (RVH)

Basalt is the most abundant rock type with in the study area. Basalt outcrops are found overlying the trachyte rock unit. These rock units tend to cover wider areas and are observed more on the western part of the dam site. Small outcrops of basalt are observed in the North Eastern part of the dam site. Fresh basalts found with in the study area have dark gray to black color, but the weathered basalts are yellowish to reddish-brown in color.

The basalt is fine grained characterized by aphanitic texture. The basalt is vesicular. Basalts show dark tone in aerial photographs and texturally are aphanitic. The Joints observed are open and joint spacing ranges from 15cm 98 cm. Boulders of basalt are commonly observed in the basin. Basalts have very high strength. The strength for basalts is with in the range 38-70 MPa (Johnson, 1988).

II. Rocks with Medium to High mass Strength (RHI)

Highly jointed basalts, trachytes , trachybasalts and the welded tuff fall with in this group. The trachyte is found underlying the basalt. It is located in South and South Eastern part of the area. This rock unit is abundantly found along the dam axis and the quary site. The trachyte is slightly to highly weathered. Portions of the rock where weathering is not significant are light grey to dark grey in colour while the highly weathered trachites are pinkish grey in colour. The joints observed are not very open as those of the basalt and the joint opening ranges from 5 to 50 cm. The strength for trachytes is with in the range 32-66 MPa (Johnson, 1988).

The welded tuff is found underlying the trachyte unit. This rock unit is not exposed at the dam site or in the reservoir area.

IV. Rocks with low mass strength (RLO)

These types of deposits are found underlying the trachyte unit. Pyroclastic deposits are dubious foundation materials because of low density, high porosity and collapsible structure. Tuff and welded tuff are the most common pyroclastic deposits that are observed with in the study area. The tuff is brown to light brown and sometimes the color ranges from greyish to green. It is fine to medium grained rock. This rock is weak and is characterized by moderately to close jointing. Lapili tuff is also found together with the tuff. It is characterized by wide jointing and is very weak.

Moisture easily weakens these rock units; therefore these rocks have low water resisting property. Their mineralogical composition determines their response to weathering.

This classification based on strength has been used to produce the engineering geologic map.

CHAPTER-SEVEN

Slope stability analysis

Dam failures are of particular concern because the failure of a large dam has the potential to cause more death and destruction in the downstream areas than the failure of any other man-made structure. This is because of the destructive power of the flood wave that would be released by the sudden collapse of a large dam. When a dam fails, the pent-up water can be suddenly unleashed and have catastrophic effects on life and property downstream. Homes, bridges and roads can be demolished in minutes.

Many dams, both large and small, have failed but only a few have had a significant impact on the practice of dam design and engineering geology. Failures of dams range from complete catastrophic failure to relatively minor ones that cause minor damage.

The principal cause of damage include failure due to overflowing of water, piping, sloughing, cracking, geological problems with the dam foundation, earthquakes and stability of reservoir banks.

Properties of rocks which make up the valley sides (particularly those that are with in the range of water fluctuation) affect the safety of dams. The disturbance of banks is generally due to one of two causes:

I. Flooding

Flooding changes the stability conditions of some rocks; when the rocks are saturated with water, their weight decreases due to uplift but their loading above the water level remains the same. The cohesion of fine-grained soils may decrease. Sudden lowering of the water level will put the rock under pressure of water seeping into the reservoir. All these factors may cause sliding of the reservoir banks that causes failure of dams.

II. Wave action of the reservoir water:

Wave action of the reservoir water also disturbs the bank by erosion. Reservoirs display great fluctuation in water level depending on the operation of the power plant. As a result sediments may be displaced down slope towards the bottom of the reservoir creating steep scarp that may collapse.

Valley sides that are made up of firm bedrock are not disturbed by fluctuations of the reservoir water. However, banks covered with rock debris, sand or clayey sediments are highly affected by fluctuations of the reservoir water and hence particular attention should be paid to.

7.1 Stability analysis of the reservoir rim

The banks of the reservoir should be examined for undercutting, erosion, depressions, and any other evidence of the initiation of a possible slip or landslide. Depending on the size of the reservoir, a landslide could displace enough water to overtop the dam, block the reservoir, head pond, race or spillway, or reduce the reservoir capacity. All of these events could jeopardize the safety of the dam.

For the present study a reconnaissance survey has been carried out all along the reservoir rim to identify the potential slopes having the stability problems. Since, the topography of the reservoir area is undulating with very gentle slopes (Fig 2.1), therefore no such slopes has been identified in the reservoir rim area. However, the proposed querry site may pose problems of instability during the construction stage, when excessive excavation for construction material will be done. Therefore an attempt has been made to carry out the stability analysis of the slope at the proposed quarry site.

I. Stability Analysis for querry site

The trachyte rock units of the study area have relatively stable slopes. The proposed quarry site is located on the trachytes about 200 m upstream of the reservoir rim, on the right bank of the reservoir rim. Excessive excavations will be under taken at the quarry site; therefore there is possibility that the slope at quarry site may pose some stability problems. These may trigger slide movements and rock fall. Hence the design of excavation should include the choice of safe angles for slopes, width of benches and the overall shape (Goodman, 1980).

Hard rock is usually strong but in regularly jointed rock there are many possibilities for block movement along planes of weakness and a large number of models are exhibited. Failures of rock blocks along discontinuities comprise one or more of the three basic failure models: plane failure, wedge failure and toppling. Kinematic model studies prove very useful in determining whether a slope is stable or not and if the slope is not stable it helps to determine which type of failure is expected.

II. Kinematic check for possible mode of failure

According to Markland a plane mode of failure occurs under gravity alone if a rock block rests on an inclined plane that daylight into free space or the slope face. The inclination of the plane of slip must be greater than the angle of internal friction of that plane (Goodman, 1980). Whereas, for a possible mode of wedge failure the plunge of the line of intersection of the wedge forming planes must be less than the slope angle measured in the line of intersection and should be greater than the angle of internal friction (Hoek and Bray, 1977). Hence the resistance to sliding along the sliding surface and margins of the slide (joints often provide the release along the margins in hard rock) has to be overcome.

Wedge slide occur when two planes of weakness intersect and this intersection line daylight on the surface. The failure can occur without release features.

Steeply inclined rock layers into the hillside trigger formation of cracks resulting in toppling failure. Plane sliding, wedge sliding and toppling can occur simultaneously in highly jointed and bedded rocks.

The Kinematic conditions are;

For plane failure

$\alpha_f > \alpha_p > \phi$

For wedge failure

$\alpha_f > \alpha_I > \phi$

Where: α_p-dip of potential failure plane

α_I-plunge of the line of intersection of the two wedges forming plane

ϕ-angle of internal friction

Discontinuity plane, slope face and angle of internal friction along sliding planes can be plotted on circular projections for kinematic analysis. Once all the structural elements are plotted it is possible to examine the kinematic requirements to anticipate the expected rock failure. The spatial relationship of these planes can be interpreted for the possible mode of failure.

To perform the kinematic analysis for the slope (where the quarry site is located) structural measurements as many as possible are taken and then the measurements are plotted on a stereonet. Stereographic projections of planes are very helpful for the purpose of rock mechanics. Problems in slope stability and rock foundation make the use of such projections. Measurements of Dip and Dip direction of the joints are taken and analyzed using Spherista software to determine the preferred orientation of the discontinuity planes.

For the present study, around 112 discontinuity data, in terms of dip amount and dip direction has been collected from the slope at the quarry site. The data has been fed to the Spherista software. The analysis has been carried out to get the preferred orientation of the discontinuity planes. From the analysis three major preferred joint orientations (joint sets) were identified, which are given in table 7.1.

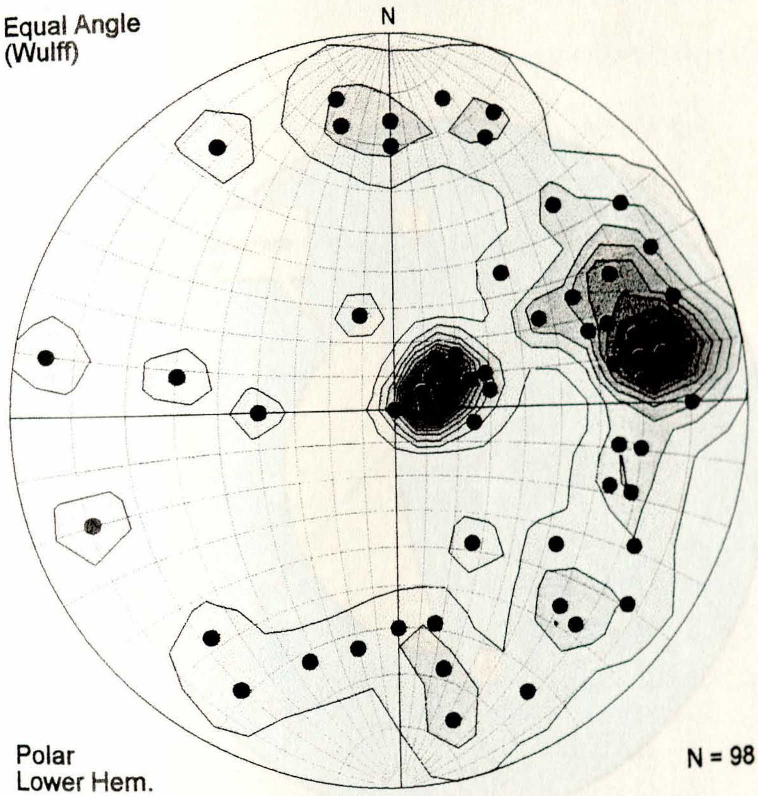


Figure 7.1 Contour diagram showing concentration of joints

Table 7.1 Concentration of major joint sets

Major plane	*Orientation (Dip direction/angle)*	*Strike*	*Concentration (in %)*
P1	N250/10	N20°W-S20°E	30.30
P2	N260/70	N10°W-S10°E	23.37
P3	N320/70	N50°E- S50°W	5.08

For the purpose of kinematic check the Slope face angle and direction should also be determined. The measurements are taken in the field and are found to be N261/65.

From the RMR data it has been found that the angle of internal friction (ϕ) value ranges from 35^0 to 45^0 but to be on conservative side the most critical friction angle (35^0) is taken to draw the ϕ-circle. These planes are plotted as great circles using Dips software.

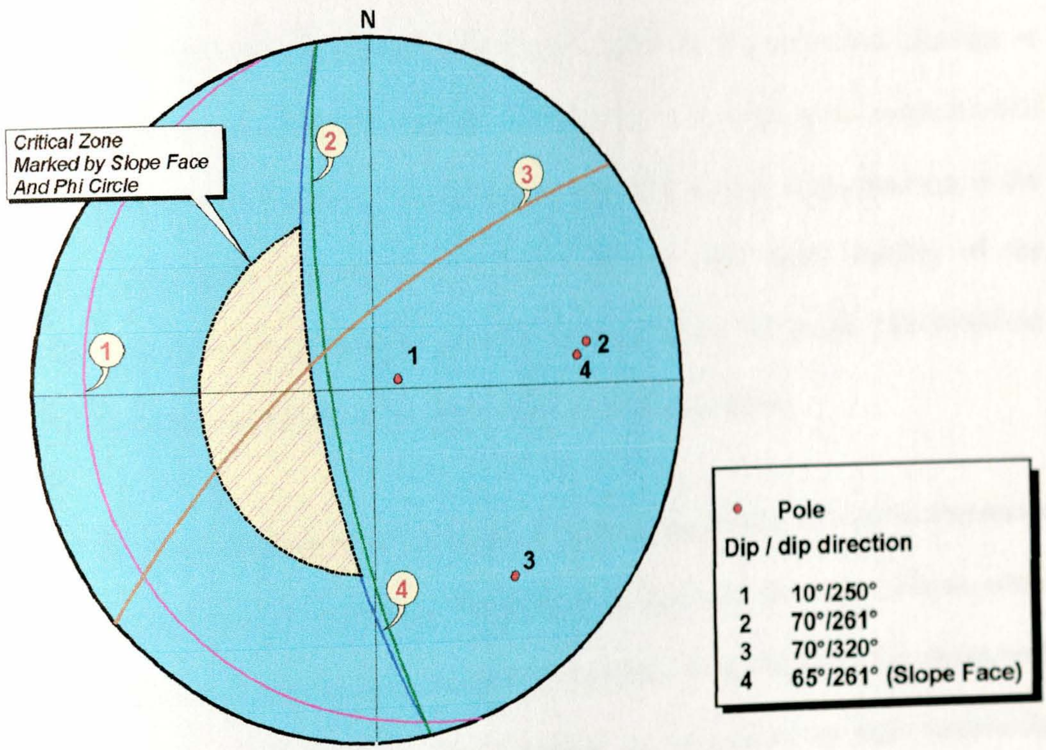


Figure 7.2 Slope stability kinematic check

A perusal of figure 7.2 indicates that none of the discontinuity plane or the intersection falls within the critical zone marked by the phi circle and the slope face. Thus, no discontinuity plane or the intersection is kinematically unstable. From this it may be concluded that the slope at proposed querry site is stable. Further, even if the slope are cut steep during the excavation for the construction material, it is not going to fail as the angle of internal friction (35°) is more than that of the Joint N250°/10°. Therefore, the kinematic condition will never be satisfied for plane or wedge mode of failure.

7.2 Stability analysis of the abutment slopes

The abutment is that part of the valley side against which the dam is constructed. The contact between the abutment and the embankment slope is called the slope-abutment-interface or groin. The abutments and groins are designated as left or right when facing downstream while standing on the crest of the dam. The abutments must offer support to the structure in the lengthwise, upstream-downstream, and vertical directions. And hence stability of the abutment slopes should be assessed for possible modes of failure and proper care should be taken during construction based on the results of slope stability analysis.

For the present study an attempt has been made to work out the stability of both the abutments under the static and dynamic conditions. The analysis has been carried out for different water saturation conditions. For the analysis the present condition of the abutments are represented as dry static whereas, the possible worst conditions are represented as static-moderately saturated, static-fully saturated, dynamic-dry, dynamic-moderately saturated and dynamic-fully saturated.

The stability analysis of a slope is not an easy task. Evaluation of variables such as the soil stratification and its in-place shear strength may prove difficult. Water seepage through the slope and the choice of a potential slip surface add to the complexity of the problem. In fact, one should keep in mind the words of specialists: "Slides may occur in almost every conceivable manner, slowly and suddenly, and with or without any apparent provocation. Usually, slides are due to excavation or to undercutting the foot of an existing slope. However, in some instances, they are caused by a gradual disintegration of the structure of the soil, starting at hair cracks which subdivide the soil into angular fragments. In others, they are caused by an increase of the pore water pressure in a few exceptionally permeable layers, or by a shock that liquefies the soil beneath the slope. Because of the extraordinary variety of factors and processes that may lead to slides, the conditions for the stability of slopes usually defy theoretical analysis." (Terzaghi and Peck, 1967).

The stability analyses require an estimate of the strength parameters of the soil comprising the abutment. In fact, before trying to analyze stability of slopes factors that affect the value of shear strength parameters used in the analysis should be considered.

The strength parameters involved in the stability analysis of the abutment are given below

Table 7.2 Design parameters adopted for slope stability analysis

MATERIAL	UNIT WEIGHT(KN/m^3)	COHESION (KN/m^2)	ANGLE OF INTERNAL FRICTION (degrees)
Clay with boulders of basalt (Right abut.)	18	3	35
Basaltic boulders with clay (Left abut.)	22	0	40

In the present study, the stability analysis has been carried out utilizing slope stability software known as SARC, Developed by Bhawani Singh, Department of Civil Engineering, Indian Institute of Technology, Roorkee, India. The SARC helps in the analysis of the circular mode of failure. This software utilizes the limit equilibrium theory to compute the Factor of Safety (FS) of the soil slopes for a variety of methods. A detailed description on SARC is given in Annex C.

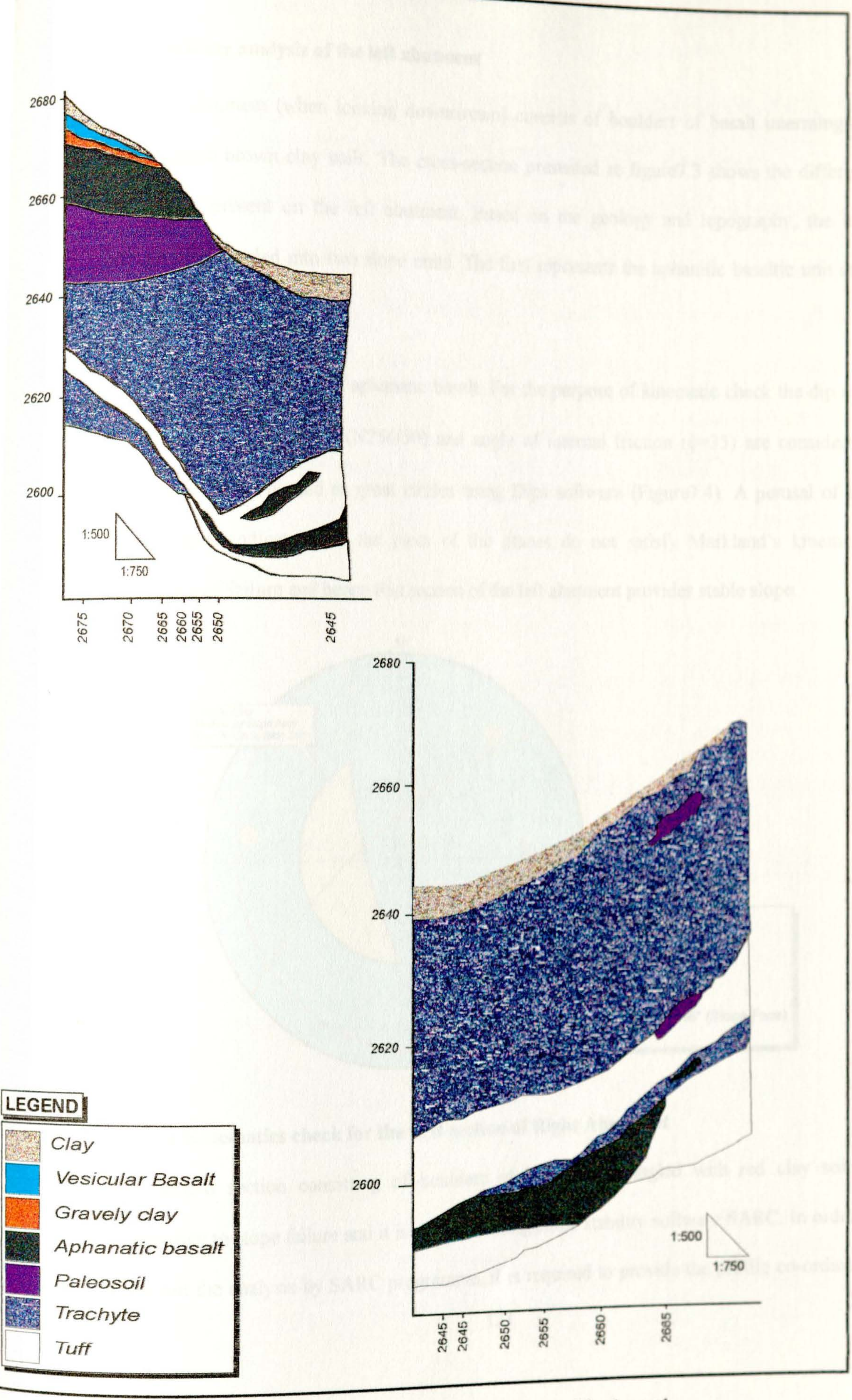


Fig.7.3 Geological cross section of Left and Right Abutment

7.2.2 Stability analysis of the left abutment

The left abutment (when looking downstream) consists of boulders of basalt intermingled with reddish brown clay soils. The cross-section presented as figure7.3 shows the different formations present on the left abutment. Based on the geology and topography, the left abutment is divided into two slope units. The first represents the aphanitic basaltic unit and the second the colluvial soil.

The first section consists of aphanatic basalt. For the purpose of kinematic check the dip and dip direction of the slope (N256/50) and angle of internal friction (ϕ=35) are considered. These planes are plotted as great circles using Dips software (Figure7.4). A perusal of the great circles indicates that the plots of the planes do not satisfy Markland's kinematic conditions of failure and hence this section of the left abutment provides stable slope.

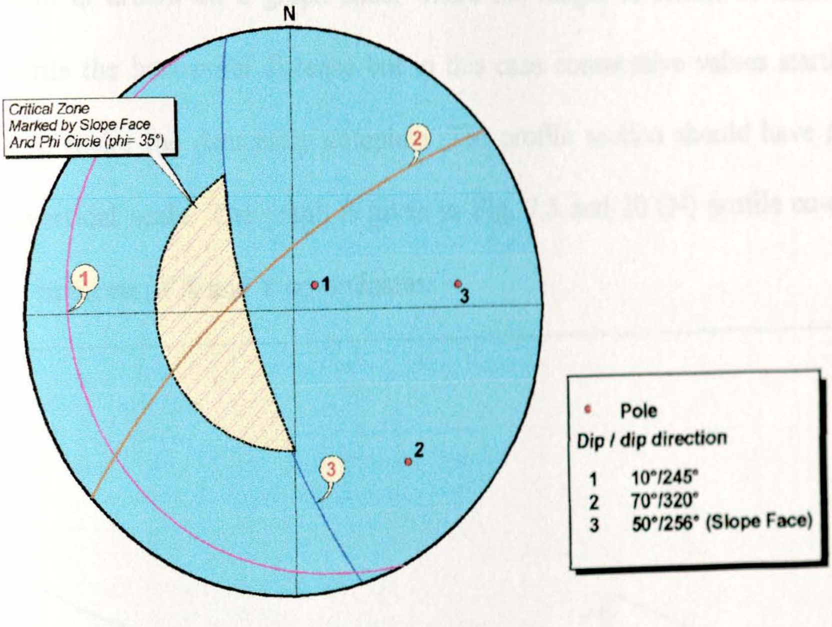


Fig. 7.4 Kinematics check for the first section of Right Abutment

The second section consisting of boulders of basalt intermingled with red clay soil is susceptible to slope failure and it is analyzed using slope stability software SARC. In order to carry out the analysis by SARC programme, it is required to provide the profile co-ordinates of the slope. For the left abutment slope cross section was drawn on the graph sheet (Fig. 7.5) and 20 profile co-ordinates (N) were delineated in terms of X and Y co-ordinates.

The angle of internal friction has been considered as 40° and cohesion has been considered as 0. The unit weight of the clay material has been considered as 22kN/m^3 (Adopted from AAWSA laboratory test results). The water saturation conditions are represented as BBAR, for a dry slope condition the BBAR is considered as 0.0, for moderately saturated conditions it is 0.1 and for fully saturated conditions it is 0.2. The horizontal earthquake acceleration has been considered as 0.25, and the vertical earthquake acceleration has been taken as 0.5, these values have been adopted from a report on embankment stability analysis by AH-HBT joint venture (AAWSA, 1997).

The cross section is drawn on a graph sheet where the height is drawn as abscissa. The ordinate represents the horizontal distance but in this case consecutive values starting from zero are given to make the data entry complete. The profile section should have the same horizontal and vertical scale. The graph is given in Fig. 7.5 and 20 (N) profile co-ordinates were delineated in terms of X and Y co-ordinates.

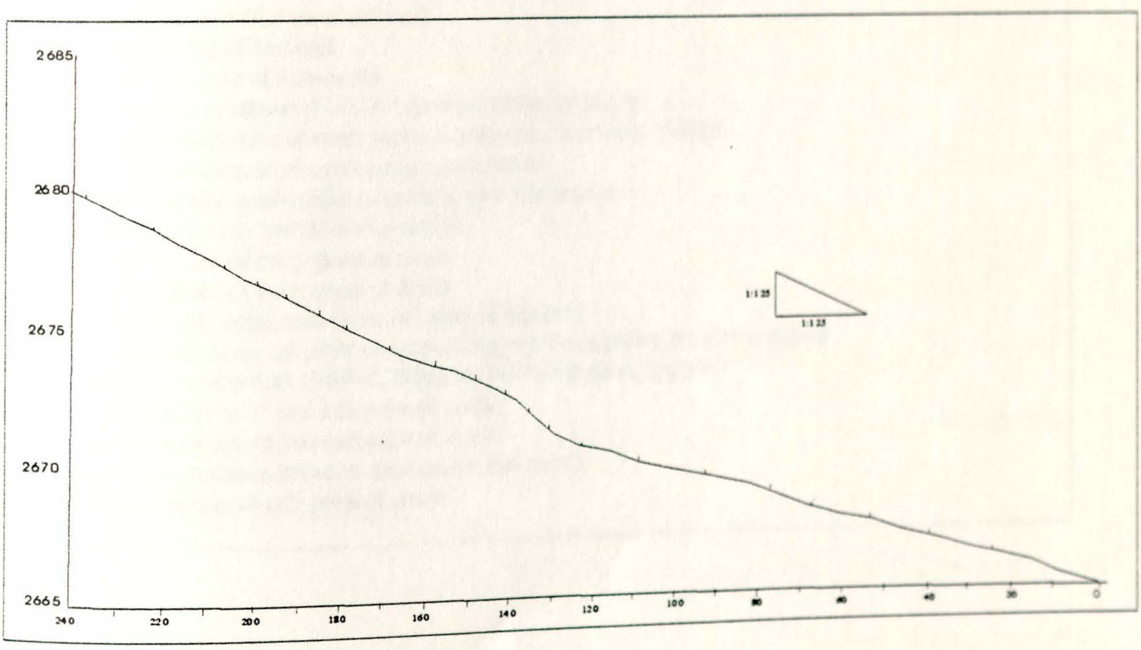


Figure 7.5 Cross-section of the left abutment

Table 7.3 Input data used for the stability analysis of the left abutment

Paramters	Values
N	24
X (I), Z (I), I=1 up to N	(0.0, 2665), (25, 2666), (40, 2667), (55, 2667.5), (70, 2668), (78, 2668.5), (94, 2669), (110, 2670), (124, 2671), (131, 2671.5), (136, 2672), (142, 2672.5), (150, 2673), (159, 2674), (170, 2674.5), (180, 2674.5), (182, 2675), (187, 2675.5)(194, 2676), (200, 2677), (208, 2677.5), (223, 2678), (237, 2679), (240, 2680)
Rock, RWL, XS, WI, ZC, ZWR	0, 0.0, 230, 0.0, 2.0, 1.0 respectively
C, PHI, GAMA, GAMAW, BBAR, AH, AVR, EQM	0, 40, 2.2, 1.0, 0.0, 0.25, 0.5, 7 BBAR will have values 0.2, 0.1 and 0.0 for saturated, moderately saturated and dry conditions respectively. And the values of AH, AVR and EQM will be respectively 0.25, 0.5 and 7 for dynamic conditions and zero for static conditions.
NENP, <ENTX<I>, ENTY, I=1 TO NENP>	1, 0.0, 2665
NEP, NOPT	0, 0
XEXITI, XEXITL, GAP	140, 240, 100

Where:

N- Number of profile coordinates, X (I), Z (I)- Coordinates of profile points,
ROCK-Reduced level of hard strata with respect to origin
RWL= Reduced level of GWT/reservoir water with respect to origin
XS= X-coordinate of point from where surcharge starts
WI= Uniform surcharge intensity
ZC= Depth of tension crack
ZWR= Depth of water in tension crack (ZC)
C= Cohesion of soil/rock
PHI= Angle of internal friction of soil/rock
GAMA= Unit weight of soil/rock
GAMAW= Unit weight of pore water
BBAR= Pore water pressure/ (GAMA* Average height of slices)
AH=Horizontal component of earth quake acceleration near crest of slope
AVR= Vertical component of earth quake acceleration
EQM= Corresponding earth quake magnitude on Richter scale
NENP= Number of entry points of slip circles
ENTX= X-coordinate of entry point of circle
ENTY= Y-coordinate of entry point of circle
NOPT=0, When only minimum factor of safety is required
=1, when all factor of safety corresponding to all exit points are also required
NEP= Number of exit points (NEP=0, When no individual point is given)
XEXITI= X-coordinate of first exit point of circle
XEXITL= X-coordinate of last exit point of circle
GAP= Horizontal distance between consecutive exit points
XEXIT= X-coordinate of exit point of circle

Table 7.4 Factor of safety of the left abutment

		Static			Dynamic		
	Water saturation	Dry	Moderately saturated	Saturated	Dry	Moderately saturated	Saturated
Left abutment	Ww (T)	0.11E+07	0.11E+07	0.11E+07	0.11E+07	0.11E+07	0.11E+07
	D disp (M)	0.0	0.0	0.0	0.309	26.432	26.432
	AH critical	0.54	-0.037	-0.115	0.048	0.0	0.0
	F_s	1.0808	0.9405	0.7644	0.730	0.6081	0.4377

The results thus, obtained are presented in Table7.4. A perusal of table 7.4 indicates that the slope is critically stable (F_s=1.08) for the existing conditions i.e dry slope under static condition. However, as the results indicates that it is unstable for possible worst conditions in static and dynamic conditions. During the construction stage this clayey material will be removed and the dam will be founded over the sound aphanitic basalt. At the excavation stage proper care has to be taken to remove this material, as the cohesion for this clayey material is zero, means it has got a very poor strength and also the stability analysis results indicates that it is critically stable. Therefore, the excavation must be planned from top to the bottom, so that removal of toe support does not trigger the failure of this slope.

7.2.1 Stability analysis of the right abutment

The abutment at the right side is composed of basalt overlain by trachyte. It is gently slopping. Clay soils overlay the trachyte. The stability analysis is performed for the clay soil. This clay soil is also found intermingled with basaltic boulders but the size of boulders is not as big as those of the left abutment.

To perform the stability analysis utilizing SARC software 20 profile coordinates are taken from the profile section of the right abutment. The profile section for this portion is given in Figure 7.6.

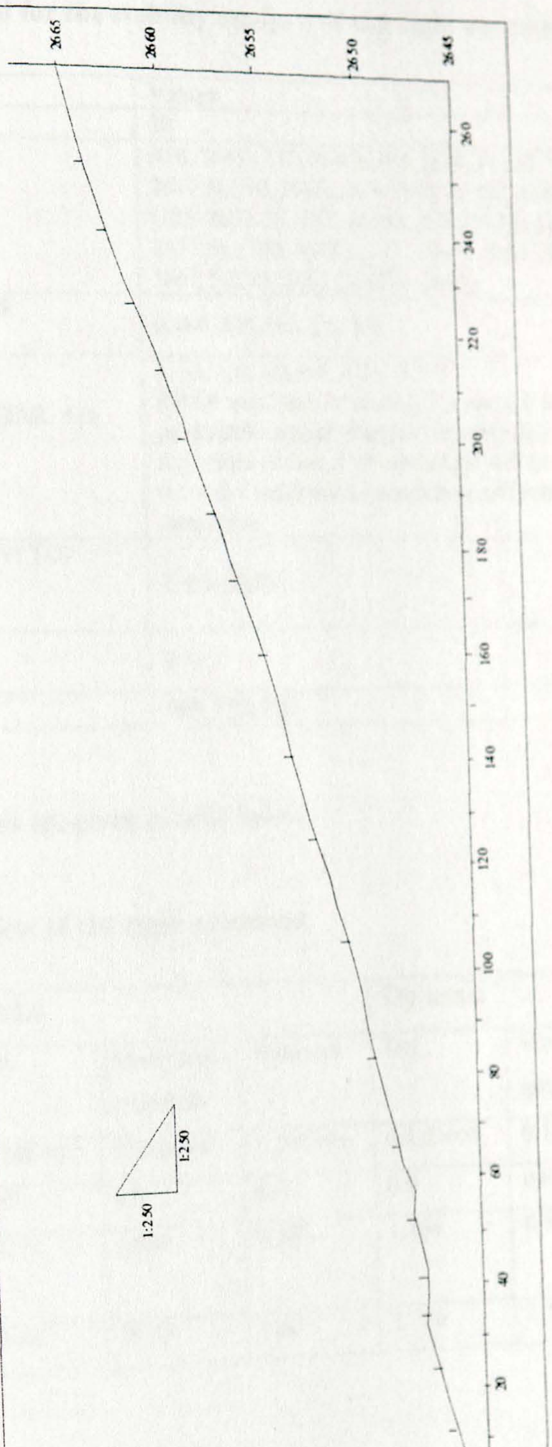


Figure 7.6 Cross-section of the right abutment

The data for this portion of the abutment is given in table 7.4. Their order is exactly similar to the order of data entry shown in table 7.3

Table 7.5 Input data used for the stability analysis of the right abutment

Paramters	Values
N	20
X (I), Z (I), I=1 up to N	(0.0, 2645), (12, 2646), (23, 2646.5), (34, 2647), (40, 2647.5), (50, 2648), (65, 2648.5), (84, 2650), (105, 2651.5), (125, 2652.5), (140, 2653), (160, 2655), (173, 2655.5), (187, 2657.5), (200, 2658), (213, 2660), (226, 2661.5), (238, 2662.5)(250, 2663.5), (270, 2665)
Rock, RWL, XS, WI, ZC, ZWR	0, 0.0, 210, 0.0, 2.0, 1.0
C, PHI, GAMA, GAMAW, BBAR, AH, AVR, EQM	3, 35, 1.8, 1.0, 0.0, 0.25, 0.5, 7 BBAR will have values 0.2, 0.1 and 0.0 for saturated, moderately saturated and dry conditions respectively. And the values of AH, AVR and EQM will be respectively 0.25, 0.5 and 7 for dynamic conditions and zero for static conditions.
NENP, <ENTX<I>, ENTY, I=1 TO NENP>	1, 0.0, 2645
NEP, NOPT	0, 0
XEXITI, XEXITL, GAP	160, 270, 100

The results of the analysis are given in table below.

Table 7.6 Factor of safety of the right abutment

		Static			Dynamic		
	Water saturation	Dry	Moderately saturated	Saturated	Dry	Moderately saturated	Saturated
Right abutment	Ww	0.24E+05	0.97E+04	0.10E+05	0.12E+05	0.12E+05	0.10E+05
	D disp	0.0	0.0	0.0	0.0	0.0	0.0
	AH critical	0.729	0.658	0.587	1.104	0.944	0.787
	F_s	11.64	10.64	9.60	2.756	2.501	2.234

A perusal of table 7.6 indicates that the slope is stable (F_s=2.234) even for the possible worst dynamic and saturated condition. During the construction stage this clayey material will be removed and the dam will be founded over the sound trachyte. At the excavation stage proper care has to be taken even if the slope is very stable. Therefore, the excavation must be planned from top to the bottom, so that removal of toe support does not trigger the failure of this slope.

CHAPTER-EIGHT

Construction Materials

One of the major objectives of engineering geology is to make sure that the construction materials required for the construction of the engineering structure is adequately found with in a reasonable distance from the dam site and the quality of the materials meets the quality requirements. Most of the materials used for construction are obtained directly or indirectly from constituents of the earth's crust; hence assessing, evaluating and characterizing the potential sources of natural construction materials (rocks and soils) should be carefully done. Geologic materials, which are to be used as a construction material, must satisfy certain fundamentally economic requirements. The source of the construction material should be with in an economic distance so that the cost of transport charges is kept to a minimum.

The geological conditions at the source should also be economically favorable: for example there should be no excessive overburden, there should be enough concentration and quantity of the required material besides separating the required material from the waste should not be very expensive. It is usually advisable to obtain construction materials from areas where excavation is a must such as cut-off trenches, spill way, outlet sites, saddles or from the reservoir. In addition it is advised to use the materials at the site rather than look for materials found too far with ideal properties. However, this rule should not be considered in the search for filters and drains where quality requirements are high.

For each construction material the location and accessibility, quantity, quality, method of processing, overburden and waste materials quantities should be described.

8.1 Dam Embankment Type and Internal Zoning

Gerbi dam is to be constructed of thin selected rock-fill shell material, thick central impermeable clay core and, an internal chimney drain made of a graded filter.

The previous embankment design consisted of a broad earth core with an internal chimney drain and downstream filter blanket, the earth core is protected by narrow rockfill shoulders. A geotextile is used as a separator between the earthfill and rockfill. The crest is capped with rockfill and a thin layer of road embankment fill over the earthfill. The project authorities consider that a zoned earthfill / rockfill type dam is the most appropriate for the site, because of the variable weathering, fracturing and low strength of the foundation rock, and large quantity of fill materials available near the site (AAWSA, 1997).

As shown in the sketch (figure 8.1) the type of dam to be constructed is zoned earthfill dam.

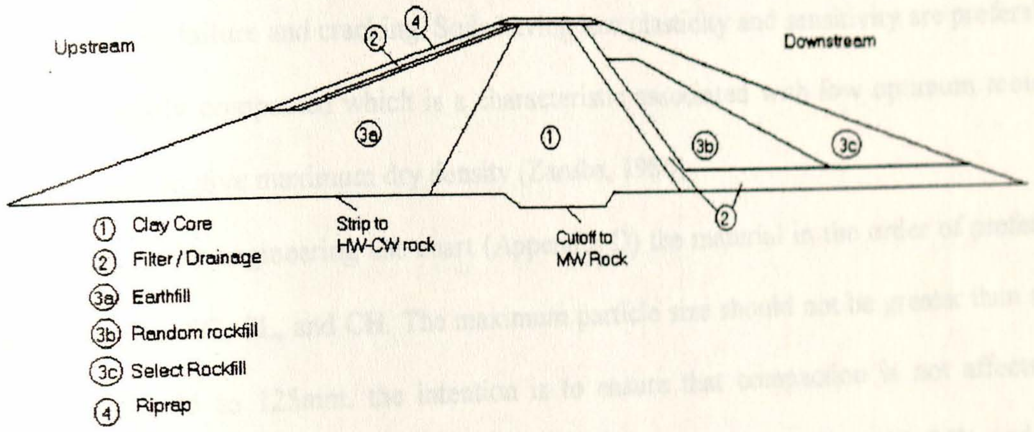


Figure 8.1 Design of Gerbi Dam

The design incorporates six main components: clay core, filter, earthfill, random rockfill, selected rockfill and riprap. Typical construction materials used for the different zones of Gerbi dam are discussed below.

I. Clay core

It is the most important element in the structure of a dam which controls seepage through the dam. The core should act as an impermeable membrane. The core could be rolled clay or sometimes concrete. It consists of clayey soil having low permeability to prevent leakage.

A good impervious material contains between 20 and 30 percent clay, with the balance made up of some silt, sand and gravel. Too much clay results in the dam being weaker and prone t expand and contract with moisture changes. On the contrary if the clay is not sufficient seepage problems may prevail. The clay material used in dam construction should:

-Be impervious enough (to prevent excessive leakage) when compacted. Impervious materials have permeability below $0.01X10^{-6}$ m/s

-shearing strength characteristics of the soil when compacted and saturated should be acceptable permitting relatively steep slopes and reducing construction costs; thus the clay should contain sufficient amount of sand and gravel.

-Have low compressibility when compacted and saturated which will reduce the danger of piping failure and cracking. Soils having low plasticity and sensitivity are preferable.

-Be easily compacted which is a characteristic associated with low optimum moisture content to give maximum dry density (Zaruba, 1986).

According to the engineering use chart (Appendix-D) the material in the order of preference should be GC, SC, SL, and CH. The maximum particle size should not be greater than a size in the range 75 to 125mm. the intention is to ensure that compaction is not affected by oversize material. They should be above "A" line in the plasticity chart with 25% and more liquid limit. Classifications of core materials for dams on the basis of the capability of resistance to concentrated leaks (after Sherard et al, 1976) are given in Appendix-D.

Visual inspections showed presence of impervious clay material source in close proximity to the dam and a further detail field investigation was conducted at these areas. The details of the investigation carried out are discussed in the following sub-sections. The clay borrow areas investigated are located between the dam axis and up to a maximum of 1600 m downstream and 1300 m upstream . Except Area 1 and the saddle, all clay borrow areas are located with in the Gerbi reservoir. An investigation was conducted utilizing test pits, auger holes and trenches (AAWSA, 1997). Summery of each borrow area is given below.

Table 8.1 Summary of the proposed borrow areas

Area	Elevation (masl)	Area of borrow (ha)	No. of auger holes	No.of test pits	Estimated volume (m^3)
Area 1	2641-2653	35	20		700,000
Area 2	2648-2670	11	-	11	200,000
Area 3	2650-2658	4	-	4	80,000
Area 4	2647-2663	8	-	5	160,000
Area 5	2648-2664	15	-	10	300,000
Area 6	2645-2666	39	13	10	600,000
Saddle	2654-2658	9	-	10	180,000

A laboratory testing program was undertaken at different laboratories with in Addis for the various construction and foundation materials. In the laboratory, sample preparation and testing was conducted to the procedures of the Engineering Geological Society working group report as much as possible (AAWSA).

Table 8.2 Summery of borrow material properties

Areas	1			2			3			4			5			6			Saddle		
Test	In %			In %			In %			In %			In %			In %			In %		
	NMC	LL	PI	NMC	LL	PI	NMC	LL	PI	NMC	LL	PI	NMC	LL	PI	NMC	LL	PI	NMC	LL	PI
Max	51	96	54	40	77	51	50	71	56	50	61	32	53	86	56	61	96	65	64	71	57
Avg	30	62	36	27	56	32	28	55	34	29	53	28	38	68	40	36	63	35	37	58	4[illegible]
Min	19	40	21	14	40	22	17	45	20	20	48	20	24	47	21	17	43	20	28	45	2[illegible]

Table 8.3 Summery of lab tests of all borrow areas

	Max.	Avg.	Min.
O.M.C (%)	45	24	23
MDD(g/cc)	2	1.41	1.18
Activity	1	0.69	0
Plasticity	CH	MH	CV
Sp.G	3	2.73	2.55
Permeability	7.44e-6	2.10e-7	4.24e-8

All of the clay borrow areas are Quaternary deposits of residual, alluvial and colluvial soils. Various tests were conducted to classify and study property of these materials with respect to their use as a construction material.

-The results of the laboratory analysis indicate that permeability of clay materials in all borrow areas is low and is in the order of 10^{-7}cm/s.

-The shear strength of the clay is generally low and its use shall generally be limited to minimum in the core section.

-Piping is another problem on embankment soils. Clays with a plasticity index above 15 are the most piping resistance materials. The piping resistance of the clay material is generally medium to high.

-The optimum moisture content of clay material ranges from 23 to 45, higher values being at the surface (Rainfall effect).Increased compactive efforts at water contents higher than the optimum create excess pore pressures in the water filling the voids which facilitate shearing of the entire soil mass. Hence, great care should be taken when compacting the clay.The maximum dry densities of the clays are in the range of 1.18 to 2 gm/cc.

-Activity curves of the clay indicate that all the borrow materials fall into medium to very high activity clays. Dispersion tests conducted using chemical methods, pinhole, and double hydrometer showed that the clay material is not dispersive.

-The borrow materials lie close to the A-line on the plasticity chart (most of it above the A-line) and could be described as CH/MH clays. The dam foundation material could also be classified as CH/MH. The plasticity of the clay material although is good for making the dam flexible, a problem of compaction and workability is expected.A general summery would be that all silts (ML & MH) are relatively poor materials. For inorganic clays, the higher the liquid limit and the higher the position above "A" line on the plasticity chart, the higher the leakage resistance.

There is no clear pattern of variation between liquid limit (LL) and depth. There is, however, a general trend of decrease of the LL with depth. This could be due to the fact that the deeper materials are close to the parent rock. It is generally recommended to remove the top one metre and use the underlying material to construct the core.

Cracking in embankment dams is influenced by soil properties used. Sherared (1953) indicated that inorganic clays with plasticity indices of less than 15 are more susceptible to cracking when compacted drier than optimum moisture content than either finer or coarser materials. By contrast clays with plasticity indices exceeding 20 can under go much larger deformation with out cracking. There is a high likelihood of cracking in embankment formed of residual soils that contain coarse particles of soft rock, which break down when compacted. It is better to cover the top exposed section of the clay core with non plastic material in order to avoid shrinkage cracks within the core. In general the residual soils (especially the red clay soils) are proved to provide a good construction material than alluvial soils. Clay materials from the selected borrow areas can be used as a clay core with some precautions.

II. Filter material

Filter materials prevent erosion of clay core to rockfill. Although the quantity of pervious materials required for filters and drains is usually small, quality requirements are high. Filters and drains are used to prevent piping and reduce hydrostatic uplift pressures. Therefore, the material must be able to dissipate relatively high hydraulic heads with out movement of either filter material or protected soil.

Filter aggregates may be obtained from alluvial sand and gravel deposits, or from quarries. Generally suitable aggregates are of igneous and less commonly metamorphic origin. It is very unusual to manufacture from sedimentary rocks, as these rocks are usually not sufficiently durable and often have poor shape (as measured by flakiness index).

For large dams it is usually necessary to establish a separate crushing and screening plant for manufacture of filter. Minerals contained in pervious materials should be evaluated for potential degradation as water percolates through the filter. Likewise, attention should be given to soundness and durability of particles to be sure no significant change occurs in gradation due to particle break down as the material is compacted.

Filter and drain material must meet filter design criteria. According to USBR, the filter material should fulfill the following criteria.

I. Filter material should be much more pervious than the base material.

That is D15F/D15B = 5 to 40 provided that the filter not contain more than 5% passing 0.075 mm sieve. The fines should be cohesionless.

II. The compacted filter must be thick enough to prevent base material from penetrating the filter. This will cause clogging of the filter with the base material. This is ensured by the following criteria.

-D15F/D85B $\leq$ 5

-The grain size curve of the filter should be roughly parallel to that of the base material.

III. Maximum size particle in filters = 75mm to prevent segregation due to placement.

IV. For base materials which include gravel particles, the base material D15B, D85B etc. should be analyzed on the basis of the gradation of the soil finer than 4.75m.

V. Filter material should be clean, cohesionless sands or gravels with coefficient of uniformity (Cu) between 1.5 and 8.

Where: F and B-represent filter and base material respectively.

Filter aggregates may be obtained from alluvial sand and gravel deposits, or from quarries. But there is no natural sand with in an economic distance so it is necessary to establish a separate crushing and screening plant for manufacture of filter from sufficiently available boulders of basalt.

III. Earthfill

Control seepage through the dam and also provide stability of the dam.

Clay, sandy clay, clayey sand and silty sand possibly with some gravel usually 15% passing 75μm can be used as an earthfill. Weathered silt stone, shale and sandstone can be compacted in thin layers to give sufficient fine material. The majority of the materials available from necessary excavation in the spillway and saddle areas can be used as an earthfill.

IV. Rockfill (Random rockfill and selected rockfill)

Rockfill provide stability, commonly free draining to allow discharge of seepage through and under the dam. Rock products have many specialized applications in construction. Mineral composition, texture, reaction to alkalis, susceptibility to breakdown, and resistance to abrasion are among the properties used to evaluate a rock for a particular application.

An understanding of rock characteristics, available test procedures, engineering applications, construction methods and in-service conditions is critical to the proper use of rock in construction. Rock suitable for the various applications must be sufficiently massive to satisfy the size requirements of the particular application. Thus, discontinuity analysis and rock mass characteristics identification should be carried out in the quarry site. Drilling can be done if there is no out croup available in the area.

Examination of the rock quality at a potential extraction site includes durability, petrography analysis, freeze-thaw, sulfate, absorption, LA abrasion, and bulk specific gravity.

The Gerbi quarry site is located 300m upstream of the dam axis to the left reservoir rim. The rock type in the quarry site is slightly weathered trachyte (on the surface). The trachyte at the quarry is underlain by basalt. The trachyte is exposed on the surface and has reddish brown colour and is slightly weathered on the surface. Quartz and feldspars are seen as phenocrysts. The foot of the hill composed of residual and alluvial soil. Three boreholes extending to a depth of 15m were drilled to explore the subsurface condition of the quarry site.

Table 8.4 Summary of borehole drilled at Gerbi rock quarry site

No.	Boreholes No.	Depth (m)	Easting	Northing	Elevation (masl)
1	GQ95-1	15	463701.00	1011537.15	2673.81
2	GQ95-2	15	463727.93	1011192.03	2674.25
3	GQ95-3	15	463728.75	1011544.20	2679.36

The total area of quarry site was estimated to be 85,000m^2. The total volume of quarry at Gerbi was calculated with three different heights of 10, 15, and 20m and the average volume available from the area was estimated to be 1,250,000m^3.

In order to exploit the quarry site, blasting will be required. The trachyte in the area has a sheeting appearance and has horizontal jointing. Care shall be taken not to have a flat unsuitable rock during the exploitation. As the rock quarry site is located upstream of the dam site, no social problem is expected.

Core samples were collected and laboratory tests were conducted on them. Thin section analysis was also done to determine rock type. The following table shows the laboratory test conducted on the rock samples:

Table 8.5 Summary of Rock quarry lab test results

Test	Result
LA abrasion	50%,Grading A
Soundness loss	63.2% (solution used is sodium sulphate)
Bulk specific, Gr	2.513% or 2.5 gm/cc
% Water Ab	3.35
Slake durability	98.8%
Reduction in alkalinity(Rc)	342.5Mi/L
Dissolved silica (SC)	55.6Mi/L

In order to check the suitability of the trachytic rock as aggregate for concrete, alkali-aggregate reaction test was conducted. The test revealed that there is no reaction between the alkali of Portland cement and the aggregate.

It was seen from the investigation conducted that sufficient amount of suitable rock is available for use as rock fill. As there is no precedence of using the trachytes as concrete aggregate, basalt has to be picked up for crushing for use as concrete aggregate. A lot of basaltic boulders are available in river streams and at the foot hills as colluvial deposits at left of Gerbi river.

The alkali-aggregate reactivity test conducted showed that the material is not reactive with alkali of Portland cement. The strength of the rock against abrasion is excellent. The slake durability tests showed that the material is suitable for use as rock fill and capable of resisting wave action without damage.

V. Riprap

Riprap prevents erosion of the up stream face by wave action of the reservoir water. It is required to protect earth embankments or exposed excavations from the action of water either as waves, turbulent flow, or heavy rainfall. Material from rock sources for such purposes should satisfy two main requirements:

-The rock source should produce rock fragments in suitable sizes to the required use

-The rock fragments should be hard and durable enough to with stand the processes involved in procuring and placing them and to withstand normal weathering processes and other destructive actions associated with their place of use. Selected dense, durable rocks found by sorting larger rocks from the materials prepared for rockfills can be used as a riprap.

CHAPTER- NINE

Conclusion

Gerbi dam project envisages the construction of a 19 m high earth dam across Gerbi river between villages Daka and Dibdibe between longitudes 38° 35'-38° 50'E and latitudes 9° 00'-9° 20'N. The elevation at which the dam site is situated ranges between 2630m and 2672 m. The engineering geological appraisal of Gerbi dam project has been carried out using surface and subsurface investigation, stability analysis on the reservoir rim and abutment slopes. The findings of the present study are summarized below.

Soils of the study area were classified into three broad groups (alluvial, residual and colluvial soils) based on their origin and they are further classified in to sub groups to provide full engineering geological description of the soil. Grain size distribution and some consistency limits (USCS system) were used to describe the soils. Based on this method the soils of the study area further described as Clay, Clayey Gravel, Silty Clay, Gravelly Clay and Sandy Clay. The consistency limits and strength properties of soils of the study have been used to produce the engineering geologic map of the dam site.

Basalts and trachytes are the dominant rock types that are found throughout the basin. Tuff and welded tuff are also found at depth. There are very few outcrops with in the study area; in fact there are only two sites where structural measurement is possible hence borehole data from the dam site and quarry site has been used to characterize the rocks of the study area. For engineering purpose the rocks should be classified based on basic engineering properties. Even though the presence of few outcrops with in the study area make engineering classification of rocks difficult, it has been tried to classify the rocks based on their strength property. This classification is the basic principle used in the engineering geological mapping of the study area. Rocks of the study area are classified in to three units, rock with very high strength (RVH), rocks with medium to high mass strength (RHI) and rocks with low strength (RLO). The basaltic rocks are grouped under RVH, the highly jointed basalts, trachytes and the welded tuff are grouped under RHI and the pycroclastic deposits fall under RLO group. The above mentioned classification of soils and rocks are adopted to produce the engineering geological map of the study area.

I. Engineering Geological evaluation of the foundation conditions

Settlement analysis for Gerbi dam has been carried out to determine the maximum amount of settlement that will occur and to make sure the settlement is with in the acceptable limit.

The section along the dam axis indicates that the foundation material beneath the dam is mainly a sound basaltic rock and trachyte hence it will not result in settlement due to consolidation. The sound basalt and trachyte are overlain by alluvial and residual sediments that are susceptible to immediate settlement. Immediate and consolidation settlement of both the embankment and foundation material was analyzed. There is also settlement expected from the weight of the water to be impound by the reservoir but since the whole reservoir area settle at the same amount this settlement is not considered critical. The results of the analysis indicate that settlement for Gerbi dam has been found to be with in an acceptable range.

II. Seepage analysis

Seepage analysis has been carried out to determine the seepage loss through and underneath the Gerbi dam. An insitu permeability test was conducted to determine the amount of seepage quantity through the dam body and foundation material by using single and double Packer tests in nine boreholes.

Laboratory permeability test has also been conducted using Constant and Falling Head methods on representative samples from the construction material site (borrow areas).

Values of permeability coefficients for the different samples located at the borrow area for the dam has also been calculated from consolidation tests using appropriate relationships. The permeability test results indicate that the bed rock materials have low permeability value. This condition cannot cause piping problems in the long run and the foundation material needs no grouting.

The proposed borrow areas are found with in the reservoir area and the laboratory permeability tests conducted on the construction materials can represent the permeability of the reservoir and hence its water tightness. The laboratory results indicate there is no problem related to excessive loss from the reservoir. The clay materials used in the construction of the dam and as a foundation have acceptable values of permeability.

III. Stability analysis

A. Stability analysis of the reservoir rim

During the present study a reconnance survey has been carried out in the reservoir area and it has been found out that as such there is no unstable slopes in the reservoir area as the topography, in general is relatively undulating and gentle. However the slope at the quarry site was identified to have stability problems. The stability analysis has been carried out for this slope and it was found that it is kinematically stable for plane or wedge mode of failure. Thus, the results of the kinematic check indicate that this portion will provide stable slope even if it is cut at steeper angles during the construction stage for the construction material.

B. Stability analysis of the abutment slopes

Slope stability analysis has also been carried out on the abutment slopes. The analysis was carried out utilizing a software known as SARC. This software utilizes the limit equilibrium method to compute the factor of safety.

The results indicates that the left abutment slope is critically stable (F_s=1.08) for the existing conditions i.e dry slope under static condition. However, as the results indicates that the left abutment slope is unstable for possible worst conditions in static and dynamic conditions. The stability analysis for Right abutment indicates that the slope is stable for existing and the possible worst conditions. During the construction stage the clayey material will be removed from both the abutment slopes and the dam will be founded over the sound aphanitic basalt. At the excavation stage proper care has to be taken to remove this material, as the cohesion for this clayey material is zero, means it has got a very poor strength and also the stability analysis results indicates that the left abutment slope is critically stable. Therefore, the excavation must be planned from top to the bottom, so that removal of toe support does not trigger the failure of the slopes.

IV. Construction materials

The clay to be used as a construction material comes from the proposed borrow areas and the various tests conducted indicate that the clay soil is with in the range of medium to high activity. The plasticity is also high, most clay samples fall above A-line of the plasticity chart and the soils are described as CH/MH clays. The clay soils are highly expansive. The shearing strength of the soil is low so great care should be taken in using the clay from the borrow areas as a clay core.

The investigations carried out in the quarry site indicate that there is sufficient amount of rock fill material (trachyte). The abrasion test results (50%, grading A) indicate that this rock provides good strength. From slake durability test results (98.8%) it can be concluded that the trachyte is suitable for use as rock fill. Selected rocks (larger in size) from the trachyte can be used as a rip-rap since they are capable of resisting wave action without damage. Basaltic boulders and colluvial deposits consisting of basaltic rock fragments can be used as concrete aggregate.

V. Water quality

Results from water quality analysis are also analyzed to determine the quality of water for drinking purpose and the effect of water on the engineering structure. Some results (TDS, total alkalinity, total hardness, Nitrate and Ammonia, Chloride, Fluoride, Copper, Sodium) are below the WHO's guidelines and some results (Iron, Manganese, Phosphate) are above the WHO's guidlines. It is therefore important to treat the water (physically and chemically) to make the result match with the WHO's quality requirement.

The sulphate content of the water is negligible and hence do not cause significant damage on the concrete structure of the dam. The value of CO_2 is 2.5 mg/l which is with in the neutral aggressiveness range indicating that it may not cause deterioration.

The foundation can be rendered stable and competent to with stand the loads of the dam structure. The foundation materials have acceptable limits of permeability and hence excessive leakage is not expected.

The reservoir slopes characterized by undulating and gentle slopes would provide stable reservoir slope. Hence, failure of the dam due to unstable reservoir slope is not a threat on Gerbi dam.

The results of the stability analysis utilizing different softwares indicates that the right abutment is stable even for the worst dynamic-saturated condition and the left abutment is critically stable for the existing dry-static condition. Hence great care should be taken during the excavation and by no means should the spill way rest on the critically stable portion (left abutment) unless the entire unstable mass is excavated and removed.

The project if completed in time will help to solve shortage of drinking water in Addis.

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Annexure

Annex A

Climate data of Addis Ababa bole, Chancho and Sululta stations

Element: Monthly Rainfall (in mm)												
Region: Shoa, Station: Addis Ababa Bole												
Latitude 9:02:00 N, Longitude 38:45:00 E Altitude 2408m												
Year	January	February	March	April	May	June	July	August	September	October	November	December
1993	11.7	52.1	11.6	168.3	91.5	157.2	209.5	291.7	190.1	24.1	0	0
1994	0	0	52.9	70	31.7	112.9	242.2	199.3	100.9	0.5	11	0
1995	0	81.3	73.3	140.3	95.9	78.2	165.1	256.9	97	0	0	28.6
1996	20.5	5.8	126.2	95.4	128.1	289.7	346.3	312.7	211.4	0.2	0.4	0
1997	29.1	0	22.1	66.8	44.8	128	257	160.7	94.7	58.6	15.3	0
1998	66.6	40	43.8	99.8	197.7	153.4	270.7	236.8	173.4	139.4	0	0
1999	4.4	0	35	17.8	30.5	104.6	294	270.5	62.8	127.1	0	0
2000	0	0	17.6	109.7	95.2	102.1	192.9	221.9	157.5	19.6	7.5	0
2001	0	10.3	165.3	14.8	106.7	163	274.4	179.1	107.3	10.6	0	0
2002	30.6	25.9	79.4	36.6	49.6	109	213.9	233.6	72.6	0.5	0	32.8
2003	4.8	34.1	48.9	121.9	33	128	226.4					

Station: Addis Ababa Bole , Element: Monthly Piche Evaporation (in mm)												
Year	January	February	March	April	May	June	July	August	September	October	November	December
1992	163.5	128.7	229.2	211.8	204.2	136.5	71.1	62.2	96.7	199.1	169.7	150.1
1993	183.9	152.7	286.4	154.8	136.2	68.6	67.4	62.5	64.1	162.9	236.6	x
1994	260.1	262.8	211.2	249.9	200.5	86.3	64.8	58	117	255.6	164	239.7
1995	272.1	232.4	216.7	140.9	232.9	152.4	60.4	49.5	76.2	219.6	225.1	209.8
1996	174.4	272.7	198.4	191.9	190.3	58.7	62.7	59.1	84.6	223	239.7	265.3
1997	189.5	285.1	291.6	228	365	209.7	61.2	x	133.8	197.5	x	138.9
1998	202.1	180.6	x	230.6	166.2	134.2	66.4	49	68.6	144.9	207.3	174.9
1999	195.2	270.1	166.5	223.4	209.8	128.9	71	80.4	176.7	83.1	x	163.7
2000	209	223.4	247.1	202.4	165.9	108.9	64.7	66.5	70.4	86.8	135.8	172.3
2001	116.1	93.1	90.9	139.2	93.6	64.1	57.2	55.5	74.7	117.6	147.4	x
2002	108.5	145.6	141.9	185.8	181.3	108.9	59.2	53.9	88.2	87.7	176.5	126.8
2003	x	121.1	98.6	137	233.7	124.2						

Element: Monthly Sunshine Duration, Station; Addis Ababa												
Year	Jan	Feb	Mar	April	May	June	July	Aug	Sep	Octr	Nov	Dec
1990	9.6	5.8	7.5	7.5	8	6.2	4	4.6	5.2	9	9.3	10.5
1991	x	x	x	x	x	x	x	x	x	x	x	x
1992	6.9	x	7.9	7.1	7.7	5.8	3.9	2.6	4.7	7.8	8.5	8.7
1993	8	6.8	9.7	4.8	7	6	4.4	5.4	4.6	8.6	10.3	10.3
1994	10.5	9.8	7.6	7.7	8.7	4.4	3.1	3.8	6.4	10.2	9.3	10.2
1995	13.3	9	8.1	5.5	8.3	7.4	3.8	4.1	6.2	9.4	10.3	9.6
1996	7.8	10	7.2	7.5	7.2	4.5	3.7	4.5	5.8	9.6	9.1	9.8
1997	7.2	10.7	8.1	6.5	9.2	6.9	4.4	4.8	8.6	7.7	7.9	9.9
1998	7.9	7.5	7	8.5	7.2	6.7	3.6	3.9	5.7	5.9	10.3	10.4
1999	8.7	x	x	x	x	x	x	x	x	x	x	x

Element: Monthly Minimum Temperature, Region: Shoa, Station: Sululta

Year	Jan	Feb	Mar	April	May	June	July	Aug	Sep	Oct	Nov	Dec
1992	3.5	2.6	3	3.8	2.7	3.3	x	x	x	x	x	x
1993	3.8	3.6	3.5	3.5	3.5	x	x	x	x	x	x	x
1994	x	x	0.2	0.3	x	x	x	x	x	x	2.7	2.9
1995	3.4	3.8	3.6	3.7	3.5	3.5	2.1	2.9	3.1	2.4	1.6	1.4
1996	0.7	1.6	2.6	3.2	x	2	3	2.9	2.5	1.5	0.8	0.9
1997	2.1	3.2	3.1	3.9	3.8	2.9	3.3	3.1	2.7	1.5	1.6	1.4
1998	2.5	3.2	3.8	3.6	3.5	2.7	2.2	x	x	2.5	0.7	2.4
1999	3	3.6	4.3	3.9	4.3	2.6	2.3	1.5	1.4	1.5	-0.1	2.7
2000	3.2	3.7	4.3	3.7	3.1	2.9	1.1	1.2	2.1	1.1	1.2	1.9
2001	1.9	2.6	2	3.3	3.6	2.6	1.8	1.1	2.4	2.3	2.7	3.1
2002	2.3	3.3	3.6	4.1	3.8	3.4	2.6	3.1	3.3	1.6	3	3.6
2003	3	4.1	4.7	3.7	4.7	3.8						

Element: Monthly Maximum Temperature, Region: Shoa, Station: Sululta

Year	Jan	Feb	Mar	April	May	June	July	Aug	Sep	Octr	Nov	Dec
1992	21.5	21.7	23	21.4	23.8	21.4	x	x	x	x	x	x
1993	25.8	24.8	28.9	28.7	x	x	x	x	x	x	x	x
1994	x	24.7	24.4	22.6	25.4	19.7	17.4	16.5	20.1	22.3	23.7	24.8
1995	24.3	25.6	25.4	25.1	25.1	24.6	22.4	22.8	22.8	22.2	21.6	21.1
1996	21.7	24	24.4	24.9	x	20.4	20.6	19.9	20.7	21.4	22.5	22.2
1997	23	23.8	23.3	24.4	24.9	24	21.3	21.6	20.6	20.2	20.6	21
1998	22.4	24.2	24.6	24.6	24.2	23	21	x	x	21.9	22.9	24.3
1999	24.1	23.8	24.4	24.5	24.6	22.9	21.9	20.3	20.5	21.4	21.3	22.7
2000	22.6	23	22.7	22.7	23	22.7	20.6	20.3	21.3	21.1	21.2	21.9
2001	21.8	22.5	21.6	23	22.8	21.9	20.4	20.3	21.5	21.8	21.7	22.4
2002	22.2	23.2	23.4	24.2	24.2	23.5	21.9	22.8	23.1	22.2	23.1	23.4
2003	23.1	24.3	24.3	24.2	24.6	24						

Element: Monthly Rainfall (in mm), Region: Shoa

Station: Sululta, Latitude9:11:00 N, Longitude 38:46:00 E Altitude 2604m

Year	Jan	Feb	Mar	Ap	May	June	July	Aug	Sep	Octr	Nov	Decr
1986	0.0	59.6	96.5	53.8	47.7	120.5	466.0	231.3	233.9	46.7	0.0	0.0
1987	0.0	47.9	159.5	0.0	31.0	176.4	289.4	386.2	220.9	14.5	x	x
1988	x	x	13.8	611.2	x	220.4	9.5	130.8	151.7	x	0.0	0.0
1989	0.0	0.0	223.5	172.9	0.0	x	x	x	0.0	x	x	x
1990	x	x	x	x	x	124.6	364.6	528.2	319.5	8.0	2.4	0.0
1991	12.3	31.0	274.0	18.0	4.3	371.5	608.6	218.2	92.9	0.0	0.0	12.7
1992	33.3	35.8	63.6	11.9	53.1	48.2	63.4	74.8	63.9	0.9	4.2	0.2
1993	17.4	68.4	22	102.9	41.3	x	80.7	234.5	129.1	6.8	0	x
1994	0	4.4	27.7	28.6	49.6	174.9	295.5	322.9	43.3	0	10	0
1995	6.4	17.9	37.3	169.9	112	174.2	383.3	249.1	48	0	7.2	12.7
1996	58.9	19.3	83.8	52.8	x	221.5	225.9	509.7	100.7	4.1	0	0
1997	8.7	0	18.4	53.5	56.2	64.4	291.8	290.1	48.8	55.1	63.6	0
1998	68.5	20.1	6	76.9	120.3	190.2	301.6	x	x	69.8	0	0
1999	24.2	12.9	28.5	16	46.9	189.4	400.7	443	146.8	78.7	0	0
2000	0	0	7.5	95.8	80.9	164.4	469.9	422.3	138	7	23.6	13.3
2001	3.6	0.5	164.2	45.3	107.3	166.7	336.8	215.4	79.3	5.1	0	0
2002	35.6	24.6	97.3	54.5	25.2	153.5	302.3	289.1	81.2	2.5	0	35.6
2003	4.1	14	32.7	87	6.5	162.9						

Ground water level data

Ground water level data (Taken in May,1997)						
Gerbi dam site						
Geographic position- UTM	Bore hole	E(m)	N(m)	Ground elevation	SWL (m)	Elevation Water level
	GB-96-5	464052	1012312	2651	20.14	2631
	GB-96-8	463961	1012295	2645	1.96	2643
	GB-96-6	463929	1012277	2645	1.66	2643
	GB-96-9	463888	1012321	2648	4.5	2644
	GB-96-7	463878	1012253	2645	1.11	2644
	GB-96-12	463840	1012372	2666	20.74	2645
	GB-96-11	463881	1012377	2668	21.75	26.46
	GB-96-02	463727	1012192	2655	6.9	2648
	GB-96-10	463846	1012376	2677	27.52	2649

Annes B

Table B.1 WHO's guidline for quality requirements for drinking water

Parameters	Recommended limits
P^H	6.5-8.5
Total Dissolved solids	1000 mg/l
Total alkalinity	500 mg/l
Nitrate, Nitrite and Ammonia	10 mg/l NO_3
Iron	0.3 mg/l
Manganese	0.1 mg/l
Chloride	250 mg/l
Fluoride	1.4 mg/l(26.3-32.5°C) to 2.4 mg/l (T<12°C)
Chromium Hexavalent	0.05 mg/l
Copper	1 mg/l
Sodium	200 mg/l
Sediment load	kl

TableB.2 physical characteristics (recommended limits for drinking water)

Criteria	Recomonded Limits
Color	20
Odour and taste	Non-objectionable
Turbidity	5
Temperature	20^0C-25^0C

Table B.3 National interim drinking water standards

Contaminant	Level(mg/l)
Fluoride	2.2
Chloride	250
Color	15 units
Copper	1
Corrosivity	Non-corrosive
Iron	0.3
Manganese	0.05
Odor	3(Threshold no)
Sulfate	250
Total residue	500
Hydrogen Sulfide	Not detectable
Nitrate as N	10
Zinc	5

Annex C

A Brief outline of SARC computer program

The computer program SARC is prepared by Prof. Bhawani Singh, Department of Engineering, Indian Institute of Techonology. The program (x) is written in Fortran 77 and EXE files work in DOS environment. The users' manual is also indexed as IX.NEW for preparation of input data files. Further, typical input data files are also given as IX.DAT beginning with I. The corresponding out put files OX.DAT are added, beginning with O.

The typical computer commands are:

NE IX.DAT- To open input file

NE OX.DAT- To open out put file

X- Name of computer program

IX.DAT- Input file name

OX.DAT- Output file name

2- For execution

1- For help menu

NE OX.DAT- To see the output file OX.DAT

SARC

This program facilitates to compute the factor of safety with circular failure surface emerging at the toe. It analyses any general profile of the slope surface and for various forces that is pore water pressure, depth of tension crack at the top of the slope, depth of water in tension crack and earthquake force. In the first step it draws the various slip surfaces along which failure can take place. Then it calculates the radius and center of each slip surface.

In the next step, the factor of safety is computed using Bishop,s equation for various slip surfaces until a minimum factor of safety is obtained. The analysis evaluates critical acceleration for slopes with factor of safety less than unity and compute dynamic displacement utilizing coorelation developed by Lavania et al. (1987).

Annex D

I. Terms used in the description of soils

Table D.1 Plasticity of fine soils from range of LL

Range of LL (%)	Description
<35	Low plasticity
35- 50	Intermediate plasticity
50- 70	High plasticity
70- 90	Very high plasticity
>90	Extremely high plasticity

Table D.2 Description of compressibility based on coefficient of volume compressibility

Description of compressibility	Coefficient of Volume compressibility (M_v)	Clay soil type
Very high	>0.15	Very organic alluvial clays and peat
High	0.03- 0.15	Normally consolidated alluvial clay
Medium	0.01- 0.03	Lake clays fluvio-glacial clays
Low	0.005- 0.01	Very stiff or hard boulder clays
Very low	<0.005	Stiff over consolidated weathered rock

Table E.3 Engineering properties of soil groups

Soil group		Permeability	Compressibility	Shear strength	Workability
Gravels	GW	Pervious	Negligible	Excellent	Excellent
	GP	Very pervious	Negligible	Good	Good
	GM	Semi pervious to impervious	Negligible	Good	Good
	GC	Impervious	Very low	Good to fair	Good
Sand	SW	Pervious	Negligible	Excellent	Excellent
	SP	Pervious	Very low	Fair	Fair
	SM	Semi pervious to impervious	Low	Low	Low
	SC	Imperviious	Low	Good to fair	Good
Low to medium plastic silt and clay	ML,MI	Semi pervious to impervious	Medium	Fair	Fair
	CL.CI	Impervious	Medium	Fair	Good to Fair
	OL, OI	Semi pervious to impervious	Medium	Fair	Fair
Low to medium plastic silt and clay	MH	Semi pervious to impervious	High	Fair to poor	Poor
	CH	Impervious	High	Poor	Poor
	OH	Impervious	High	Poor	Poor

II. Terms used in the description of rocks

Table E.4 Weathering grade for rock mass by Irfan and Dearman, 1978

Term	Description	Weathering grade
Soil	All the rock is converted to soil, no recognizable rock texture	VI
Extremely weathered	The rock is completely weathered but the texture can be recognized.	V
Highly weathered	More than 35% of the rock material is decomposed to soil. Fresh od discolored can be found.	IV
Moderately weathered	Less tan 35% of the rock material is decomposed to soil. Fresh or discolored rovk is found. 50 to 90% rock.	III
Slightly weathered	There is only slight weathering .Discoloration is observed on discontinuity surfaces	II
Fresh rock	No visible signs of rock material weathering, slight discoloration may be observed.	I

Table E.5 Description of block sizes in a rock mass

Average size	Term
>2m	Very large
600mm- 2m	Large
200mm- 600mm	Medium
60mm- 200mm	Small
<60mm	Very small

Table E.6 Terms used in the description grain shape for rocks and soils

	Terms
Form	Equidimensional Flat Elongated Irregular
Angularity	Angular Sub- angular Sub-rounded Rounded
Surface character	Rough Smooth

Table E.7 Construction materials and their engineering use

Zone	Description	Construction material
1	Earth fill	Clay, sandy clay, clayey sand, silty sand, possibly with some gravel. usually 15% passing 75μm. Weathered silt stone, shale and sandstone can be compacted in thin layers to give sufficient fine material.
2A	Fine filter	Sand or gravelly sand, with 5% (preferably <2%)fines passing 75μm. Fines should be non-plastic. Manufactured by crushing, washing, screening and recombing sand, gravel deposits and/or quarried rock.
2B	Coarse filter	Gravelly sand or sandy gravel. They are required to be hard, dense durable aggregates.
2C	Up stream filter	Sandy gravel/gravelly sand, well graded, 100% passing75mm, not >8% passing 75μm, fines not plastic. Obtained as crusher run or gravel pit run with a minimum of washing, screening and regarding. Relaxed durability and filter requirement compared to zone 2A and 2B.
3A	Rock fill	Quarry run rock fill, preferably dense, strong, free draining after compaction. Compacted in 0.5 to 1mm layers with maximum particle size equal to compacted layer thickness.
3B	Coarse rock fill	Quarry run rock fill, preferably dense, strong, free draining after compaction. Compacted in 0.5 to 1mm layers with maximum particle size equal to compacted layer thickness.
4	Riprap	Selected dense,durable rock fill sized to prevent erosion by wave action.In earth and rock fill dams often constructed by sorting larger rocks from zones 3A and 3B.In earth fill dams either selected rock fill or a wider zone of quarry run rock fill.

Annex E

Table E.1 Slope class and Slope gradient classification (UNESCO-FAO)

Slope class	Slope gradient classification
0-2%	Flat
2-8%	Gentle to undulating
8-15%	Rolling
>15%	Moderately steep to very steep

The thesis is my original work and has not been presented for a degree in any other university.

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Signature: ____________.

Date of Submission: July 15, 2004.

The thesis has been submitted for examination with my approval as a university advisor.

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Ato. Kebede Tsehayu