EASTERN NILE IRRIGATION AND DRAINAGE STUDY/FEASIBILITY STUDY
DINGER BEREHA IRRIGATION PROJECT

ANNEX 5: GEOPHYSICAL AND GEOTECHNICAL INVESTIGATIONS

Table of contents

List of Tables

List of Figures

1. GEOTECHNICAL INVESTIGATIONS

1.1 INTRODUCTION

1.1.1 Existing information

The main source for information has been the Abbay River Basin Master Plan report, and the report on geology of Dinger, Dega, and parts of Arjo sub sheets by Tadesse Alemu and Yonas Hageresalam. In addition to the mentioned reports, the following maps and areal photographs were available and used for data processing purposes:

- Topographic map 1:50,000;
- 1:2,000,000 scale geological map of Ethiopia compiled by Kazmin (1973) and Mengesha et al. (1996);
- Geological map of Gore sheet, 1:250,000, 1997;
- Aireal photographs;
- Scope and Objective of the Study.

The aim of the geological and geotechnical study was to obtain sufficient information about the ground conditions for the Project optimization and cost assessment at the current study level. A general geologic map, covering the weir site and canal route provides general geological information, such as location and elevation for boundaries between the main geological formations and soil variation. Detailed geological and geotechnical mapping at the weir site shall give information on ground conditions for project components, including assessment of:

- Soil cover above bed rock;
- Rock quality in regard to foundation;
- Information on possible sources for construction materials for use as concrete aggregate as well as for sand;
- Information on possible sources of clay materials to be used in potential fill materials;
- Information on deposits of alluvial materials with potential of being used as concrete aggregate.

1.1.2 Location and accessibility

The Project area is situated in Western Ethiopia, 250km (air distance) West of Addis Ababa in Oromiya Regional State, Illubabor zone. The Project area can be reached by an all weather asphalt (490km), gravel (70km) and dry weather (14km) road from Addis Ababa. It is 490km to Bedele Town while Village no 1 and Chewaka towns from where Kobo Berha (weir site) is accessed are located at 54km and 60km respectively from Bedele Town. Seasonal track, negotiable 4-WD drive track, 14 km long has been constructed to weir site (Bereha Kobo) from Village no1.

The Dinger Bereha Irrigation Project command area is located along about 18 km long section of the Didessa River between points defined by the co-ordinates UTM (Zone 37) (N 983 437m, E 203 702m) to (N 991 423m, E 19 467). The topography of the area is undulating and incised by gullies, and the right bank was accessed by wooden local boat. The average elevation of the site is about 1248m.The land in the vicinity of river is extensively covered with thick forest and savanna grass.

1.2 REGIONAL AND SITE GEOLOGY

1.2.1 Regional Geology

The Ethiopian plateau is underlain at depth by Precambrian crystalline basement successions of the Northeast African-Arabian Shield covered in part by Paleozoic and Mesozoic sedimentary sequences and tertiary to quaternary volcanic rocks. The Main Ethiopian Rift valley and the Afar Depression of Ethiopia, divide the Plateau into the Northeastern and Southwestern Plateaux. The Precambrian Basement rocks of the Ethiopian Shield crop out in four areas, that is Northern, Western, Southern and Eastern parts of the country, near the plateau margins. The Western Ethiopia Shield where the Project area is located is one of these exposures.

The western Ethiopia shield is a mosaic of raised metamorphic gneissic domains separated by metamorphic geniuses domains separated by North-South trending belt of low grade metamorphic rocks that contain recognizable volcanic and sedimentary sequences, many of which are highly sheared and intruded by diverse suite of plutonic intrusions. In the Gore Map Sheet Mengesha and Seife (1987) recognized three major domains based on structural style, lithology, and metamorphic grade and referred to the Baro, Birbir, Alghe and Geba Domains. The project is located in the Alghe Group. Early plutonic bodies that are lenticular are recognized and are internally foliated and concordant with their host rocks. There are also late more equi-dimensional and discordant intrusions.

Around the Project area the Precambrian basement terrain is directly overlain above 1,400m meter elevation by flat lying, fine grained, aphanatic, Tertiary Basalt Flows that have a preserved thickness of at least 3 to 4 meters. There is no evidence of Mesozoic Sedimentary rock deposits between the basalts and the underlying Precambrian basement rocks. Thus the marine transgression and regression that led to the deposition of sedimentary rocks elsewhere (mainly in the North and East) of the country presumably did not reach here or were eroded away. Thus the Tertiary basalt lava flows overlie the Precambrian rocks resting directly on them.

1.2.2 Site geology

1.2.2.1 Soil formation (Quaternary covers)

Alluvial deposits, sand, silt and clay with a thickness up to five meters have been found following the Didessa river flood plan and along its tributaries. These soil formations were deposited during the rainy seasons, when these areas are flooded. Most part of the Project area is covered by residual soil derived from the underlying crystalline basement rocks and the tertiary basalt flows. Along the canal route almost all of the areas except the creaks are covered by gravelly silty sand. The thicknesses of this residual soil vary from place to place. In the test pits excavated along the route, its thickness ranges from 0 at the creeks to 2.50m in test pit DCTP 52.

1.2.2.2 Bed rocks

The bedrock of the area underlie by Precambrian rocks and Tertiary volcanic (Fig. 6.1). The Precambrian rocks cover the largest part of the Dinger irrigation Project area. Lithologically, they are represented by high-grade gneiss and migmatites, deformed granitoids.(Tadesse and Yonas, 2005).

The Tertiary volcanics exposed as a cape at high altitude and comprised of volcanic succession of dominantly basaltic composition with minor pyroclastic deposits. They cover few parts of the Project area and it is designated as Lower basalt (TV_1).

1.2.2.3 Lower basalt (TV_1)

This unit represents the largest part of the Tertiary volcanics and crop out in high elevated parts of Dinger irrigation project area. It is forming gentle slope and steep cliffs and unconformably overlies the Precambrian rocks. The flow is attaining an average thickness of 2 to 4m in the Project area. The rock is grayish black to black and commonly aphanitic to locally porphyritic and amygdaloidal, with amygdules filled by calcite and probably zeolite. The bottom part of the flow is largely porphyritic with phenocrysts of dominant olivine and rarely pyroxene, which lie within aphanitic to fine-grained groundmass. Petrographically, the Lower basalt is composed of 10-30% olivine, and 30-50% plagioclase, and 10-20% opaque minerals, which lie within 30-40% cryptocrystalline to glassy groundmass. Olivine grains are anhedral to subhedral and are fractured and show zoning. They are variably altered to iddingsite and chlorite. The plagioclase grains show zoning and observed as phenocrysts and as laths dominate the groundmass. The texture of the rock is varying from intergranular seriatic to ophitic and subophitic.

1.2.2.4 Precambrian Rocks

This unit generally forms low-lying topography and good exposures are found within stream, riverbeds, and creeks and at the mountain Chuta. This unit occurs surrounded by the granitoid orthogneiss (unit $Pgne_2$), see Fig. 6.1. Quartzo-feldspathic gneiss and banded gneiss are the dominant rock types, which accounts 60 to 70% of the unit. Amphibolite, biotite gneiss and biotite-hornblende represent few parts of the area. Massive to deformed intrusive rocks ranging in composition from gabbro to granite variably intrude the unit. Compared to unit $Pgne_2$ these rocks are less migmatized.

Quartzo-feldspathic gneiss

They are dominantly outcrops within streambeds and creeks but at places they form isolated hills and peaks with blocky and slabby disturbed outcrop. The rock is gray to grayish white and pale pink, fine to medium-grained and shows weakly to strongly developed gneissic banding. The gneissic banding is defined by alternation of quartz and feldspar rich and biotite and/or magnetite rich layers ranging in thickness from 1 to 5 mm. At places the rock is coarse-grained and looks pegmatitic. Along shear zone the rocks are strongly foliated and lineated and crop out as quartzo-feldspathic mylonite.

Banded gneiss

The rock is characteristically banded which consists of alternating felsic and mafic bands/layers ranging in thickness from 0.5 cm to 1 m and dips gently to moderately. The boundaries between the felsic and mafic layers are sharp to gradational. The felsic layers are gray to pinkish gray, medium-grained and are of granitic to granodioritic and rarely tonalitic in composition. In thin sections the felsic layers are composed of quartz, K-feldspar (perthitic microcline), biotite.

Epidote, opaques, represents the accessory minerals. The mafic bands are dark gray, medium-grained and composed of plagioclase, quartz, biotite, hornblende and trace amounts of opaque minerals, epidote, sphene and apatite.

Granitoid orthogneiss (Pgne$_2$)

This unit is characteristically forming domical hills and ridges, and the outstanding Chuta Mountain. It has sub-circular and elliptical shape in area that is widest in the center and narrows at ends. Its contact with unit Pgne$_1$ is tectonic, which is marked by NE-trending ductile D$_3$ shear zones and by NW-trending Didessa Shear zone.

Lithologically, it is represented by varying proportions of migmatized granite gneiss, granodiorite gneiss, tonalite gneiss and rarely dioritic gneiss with subordinate hypersthene-quartz-feldspar (charnokitic) gneiss, biotite gneiss, hornblende gneiss and quartzo-feldspathic gneiss. Minor screens of metasediments and metavolcanics tectonically interleaved with the granitoid gneiss are common.

1.2.2.5 Tectonic of the area

Geological maping report of the Project area by Tadesse Alemu and Yonas Hageresalam has confirmed that the existence of geological structures such as foliation, lineation, folds and fault/shear zones. On the basis of their geometric characteristics, principles of superposition and crosscutting relationships the structures are belived to be formed in four phases of deformation designated as D$_1$, D$_2$, D$_3$ and D$_{4.}$

D$_1$ deformations are represented by foliation (S$_1$) and fold (F$_1$). The S$_1$ of penetrative planar structure found throughout the high-grade gneissic units (unit Pgne$_1$ and Pgne$_2$), which is represented by amphibolite to locally granulite facies foliation. D$_2$ is the other phase of the deformation structer which has characterized by folding of gneissic layering (S') and S$_1$ foliation into open to tight upright F$_2$ folds. F$_2$ folds are N- and NE-trending and plunging at low to moderate angle either to the NE and SW. N- and NNE-trending dominantly ductile shear zones and by brittle to ductile conjugate NW-trending sinistral and ENE- and NE-trending dextral strike-slip faults are another group of geological structures which are resulted in the deformational phase designated as D$_3$, and the structures this group are recognized in the field as steep to subvertical mylonitic foliation. The final phase (D$_4$) of the geological structures in the area is represented by folding and shearing events. The structures orginated in this phase are characterized by NW-SE trending ductile to brittle-ductile shear zones and moderately northwest and southeast plunging and stretching lineations. However, no clear geological structures has been observed or noted close to the site where the weir axis, either during the current field obsevation as well as in the report and structural map of the previous geological mapping.

1.3 GEOTECHNICAL INVESTIGATIONS

1.3.1 Planning and procedure used for site investigations

The general initiative is to deal with important geotechnical issues necessary to establish, to a feasibility study level, the technical and economic feasibility of the scheme, with sufficient level of investigations. Relevant information was collected from the previous study and analysis of sufficiency was made related to the actual structures in considerations. Based on the existing information, an expectation model was developed, taking into account how the planned structures are affected by the actual ground condition. In this development, important site related problems were identified; furthermore gap of information in relation to complementary study evaluations and design of the Project. As result, ground condition parameters considered appropriate for technical optimization are defined.

1.3.2 Site investigation program

Geological and geotechnical site investigation for the weir site and canal route is planned considering obtaining appropriate design parameters for these structures. Following analysis of available data and site assessment, a site investigation was concluded which comprised geophysical investigation, core drillings, test pitting and laboratory testing of rock and soil samples. In the course of implementation of the investigation activities, the conceptual expectation model of the ground is progressively improved and the remaining investigations were adjusted following the results. Results of the site investigations are summarized and interpreted in next sub topic. Reports with detailed investigation results are enclosed as appendices in Volume III, Annex C of the Field Investigations Report.

1.3.2.1 Geophysical investigations

Geophysical investigation is intended for verifying the ground conditions of weir site for further investigations and as a supporting information source for the decision of the possible weir axis from different alternatives. This was conducted by senior geophysicist with a close supervision of the Project Geotechnical Engineer. A total of eight Vertical Electrical Soundings (VES) were made, with the aim of determining the thickness of the overburden material, depth of the bed rock and anticipating the quality of the rock (degree of weathering) which of course supplement the geotechnical investigations.

UPSTREAM OF WEIR SITE

At the investigated site (upstream weir axis) the traverse line/weir axis along which the section is constructed is orienting in a N-S direction. Along the profile four VESs namely VES1, VES2 (right bank) and VES3, VES4 (left bank) were carried out. The total length of the surveyed line is 275 m.

The geo-electric section clearly delineates three resistivity layers. The first layer resistivity value is about 11 Ohm-m and the thickness varies between 4m and 1.8m. It is relatively thin (1.8m) at VES4 (left bank), it gets progressively thicker (4m) towards VES1 -VES2. This horizon represents the alluvial soil that covers the river flood area. The low resistivity value of this layer along the whole traverse indicates that it is composed of uniform and fine material, i.e. clay and/ or silty clay.

The second layer has an average thickness of 2m, and the computed true resistivity is almost 90 Ohm-m. This layer may correspond to moderately weathered granitic gneiss which outcrops in the gullies and river bed. The bottom electrical substratum is represented by a high resistivity values ranging between 330 and 380 Ohm-m, it is related to a fresh and massive basement rock / granitic gneiss.

DOWNSTREAM OF WEIR SITE

The resistivity profile was laid down on the weir axis and its length is 270 meters. Similar to the up stream site, four VESs were conducted, VES5, VES6 (right bank) and VES7, VES8 (left bank). The geo-electric section along VES5-VES8 (see Fig.2, Appendix Geophysical Investigations) is marked by three distinct resistivity layers. The resistivity of the uppermost layer ranges between 5 and 19 ohm-m. The layer has an average thickness of 3m. It possibly represents the top loose sediment along the banks of Didessa River.The second layer is dominantly marked by moderate resistivities (74-90 Ohm-m), its average thickness is about 2.2 meters. This horizon is related to moderately weathered metamorphic rocks (granitic gneiss). The underlying third layer along the section exhibits high resistivities ranging between 335-424 Ohm-m and lies relatively at shallow depth 4.2m, beneath VES7-VES8 (left bank) while under VES5 and VES6 (right bank) the inferred depth to the top of this resistive layer is about 5.5m. This stratum is interpreted as representing fresh massive basement rock (granitic gneiss). The geophysical investigation report submitted by the geophysicist is enclosed as appendix. Drawings showing the profiles of the weir axes with detail interpretations and conclusions are also presented in the report.

1.3.2.2 Test Pitting

Test pits for characterization of the Project site as well as for collection of samples for laboratory tests have been excavated at the weir axis, canal route, pump stations and borrow sites of Dinger Bereha Irrigation project area. Excavation of test pits at the main canal was conducted from the beginning and end point of the route. The pitting was always accompanied by logging, sampling and conducting in-situ permeability tests of falling head type, when it becomes relevant, see plate 6.1. The interval of pitting was 250m unless it was interrupted by creeks, rock exposures and very dense forest which make GPS reading impossible.

1.3.2.3 Exploratory core drilling

Because of the necessity for evaluation of ground conditions in line with the infrastructure under consideration, rotary core drillings were carried out at the weir site. The objective of core drillings was to collect samples of the soil and rock materials in a particular continuous profile of under ground formation. Assessment of the core samples and laboratory testing on a selected core samples gave information on the ground condition of a particular location of the borehole. Results of laboratory testing as well as the classification are crucial information for determination the engineering properties of the materials underneath. The planned program for core drillings was totally 30m for both boreholes, which are located on both side of the river. Because of the available ground condition, the drilling program was revised in order to obtain optimum out put from the drillings and avoid unnecessary waste of time and resources. Rotary core drilling was sub contracted to Addis Geosystems Co. Ltd after evaluation of the proposals, which the consultant had requested from local drilling contractors. This rotary core drilling was performed on both sides by using a rotary core drilling rig (see plate 6.2)

1.3.3 Laboratory testing

1.3.3.1 Testing on soil and sand samples

Soil samples collected from the test pits excavated at the canal route, borrow areas and sand quarries of the Project were tested in the soil mechanics laboratory of Construction Design Share Company in Addis Ababa. The soil samples were collected from all parts the canal route and the areas identified as borrow materials for fill of the canal. Moreover, the samples are representative of the materials along the canal route and that of the selected fill materials. The sand samples were also collected from the rivers along the road Chewaka to Village no 5. The types of laboratory tests include:

- Classification tests
- Analysis of grain size distribution
- Plasticity index
- Determining of shrinkage or swelling properties
- Determination of engineering properties, which comprises
    - Compaction tests
    - Compressive strength tests
    - Triaxial shear strength tests
    - Compressibility tests
    - Organic content
    - Mortar making property

1.3.3.2 Testing on rock samples

Rock samples were also collected from the boreholes and the quarry area; and were tested in the rock mechanical laboratory mentioned for the soil sample test and in Addis Ababa University, Department of Earth Science.

The samples were collected from both boreholes and the quarry sites in order to determine engineering properties of the rock and classify and naming. The laboratory tests comprised:

- Tests for determination of engineering properties
    - Point load
    - Unconfined compression tests
    - Sodium sulphate soundness test
    - Mortar bar method of potential alkali silica reactivity tests
- Tests conducted for classifying and naming is
    - Thin section petrographic analysis

1.4 FINDINGS OF THE INVESTIGATIONS

1.4.1 General

This chapter deals with the results of geophysical investigations, exploratory borehole drillings, test pitting and laboratory testing of the Project which include weir axis, canal route, pump stations and borrow areas as well as the quarry sites. The detailed investigation results are compiled and annexed separately in Volume II, Annex C as:

- Appendix A: Vertical Electrical Sounding (report compiled by Ato Tibeba, Freelance Senior Geophysist)
- Appendix B: Rotary core drilling (from Addis Geosystem P.L.C)
- Appendix C: Test pit logs
- Appendix D: Permeability tests
- Appendix E: Laboratory test results (Construction Design Share Company)

Furthermore, both summaries of the investigation results (factual results) as well as interpretation and discussion of the results from all investigations are included. Discussions and interpretations of the result in terms of the infrastructures under considerations are presented in sub topic 6.5.

1.4.2 Geophysical investigation

1.4.2.1 Vertical Electrical Sounding (VES)

Alluvial deposits, sands and gravels from the weathering of the rock underneath are found on the top of the Precambrian gneiss formation of the weir axis. This has been well thought-out in the interpretation of the electrical soundings conducted in both, the upper weir axis option and the lower as well, see appendix A.

Clear increase in resistivity was observed, from less than 30 ohm-m in the upper 2 - 4 m; then to 90 ohm-m (from the depth 2.7 - 5.6m) and to greater than 300 ohm-m.

The low resistivity, topmost layer is expected to be alluvial sediments and that of the medium resistivity layer is sandy gravel resulting from weathering of bed rock and/or fractured, slightly weathered strong gneiss rock.

The high resistivity values below are believed to represent fresh Precambrian bedrock. The result of resistivity measurement interpretation shows that the top of fractured and weathered rocks bed vary from 4.75 to 6m on the right and 4.2 to 5m on the left side of the river. Interpretation of Vertical Electrical Soundings is an interpretation of indirect measurement based on difference in resistivity conditions of the ground. The precision of the interpretation depends on how well the considered interpretation model fits the actual ground conditions. The interpretation presented on the geophysical investigation is convincing and this has also been checked by drilling two exploratory boreholes on each side of the river.

1.4.3 Exploratory core drilling

Weir site ground condition information from the exploratory boreholes drilling includes depth to bed rock, type of the overburden and the rock type under the overburden, degree of weathering and fracturing of the rock mass. All detail information including core log, core photographs and field in situ testing is enclosed in Appendix E. Core drillings conducted for the weir axis are presented in table 6.1 and their location is shown in Figure 6.2.

Table 1: Rotary Core drilling at the Projects weir site

Borehole	Easting (UTM)	Northing (UTM)	Elevation	Drilled Depth (m)	In-situ Test (SPT)
DWBH - 1	203652	983381	1247	12.25	2
DWBH - 1	203635	983545	1247	10.65	2
Total				22.9	4

1.4.4 Ground condition

Not much variation of ground condition was observed in the boreholes which were drilled on both sides of the river, see fig 6.4. In table 6.2 the information on overburden thickness, fractured rocks thickness and in situ SPT tests is presented. The first 4.45 – 4.75m is characterized by dark grey reddish brown silty clay, see plate 6.3, and all the materials were found to be well sorted alluvial deposits. Standard penetration tests were conducted in this overburden layer at the depths 2m and 4m in both boreholes and their N-values vary from 23 to 39 showing that the consistency of this alluvial formation is very firm. The core drillings also show that the rock below the overburden is feldspar quartz gneiss. The composition is summarized in the petro-graphic thin section analysis report. Regarding the degree of fracturing some variation is noticed between the two boreholes. In borehole DWBH – 1, in the left side of the river, the RQD value is higher than in borehole DWBH – 2, on the right side of the river, see plate 4. In the borehole DWBH – 2, however, the RQD values increases towards the depth. The dominant area between both boreholes is the riverbed, which is characterised by exposures of massive strong sparsely jointed gneiss. Visual observation of the variation in rock is not possible in this area.

Table 2: Summary of information from drilled boreholes

Borehole name	Overburden thickness (m)	In-situ test Depth(m)/N-value	Rock Quality Designation	
			RQD (average)	class
DWBH - 1	4.45	2.0/23 4.0/39	4.45 - 5.25, 37.5 5.25 - 12.25, 88	Poor good
DWBH - 2	4.75	2.0/32 4.0/36	4.75 - 9.05, 37 9.05 - 10.65, 57	Poor fair

1.4.5 Test pitting

A total of 71 test pits were excavated along the canal route, at the borrow areas and at the two pump station sites for characterization of the foundation of the canal alignment and pump houses, for collection of soil samples for laboratory tests for the determination of engineering properties of the soil with in the canal alignment as well as for determination of engineering properties for use of soil as canal fill materials. Moreover, two test pits were dug on the banks at the diversion site for more visual observations and descriptions of the overburden along the weir axis. The locations of the test pits are shown on Figure 6.1.

All the test pits have been described and logged following the BS soil description and logging procedures. Moreover, permeability conditions of the soil along the canal route have been measured by using the falling head method. A summary of permeability test results is presented in table 6.3 below. The results are presented according to Darcy's law, where coefficient of permeability = k in m/s. Details on each and every test are presented in appendix D in Volume II, Annex C.

Dispersivity

Double hydrometer tests were conducted for evaluation of the dispersivity properties of borrow materials (see table 6.4). In all cases, the specimens behaved as non-dispersive.

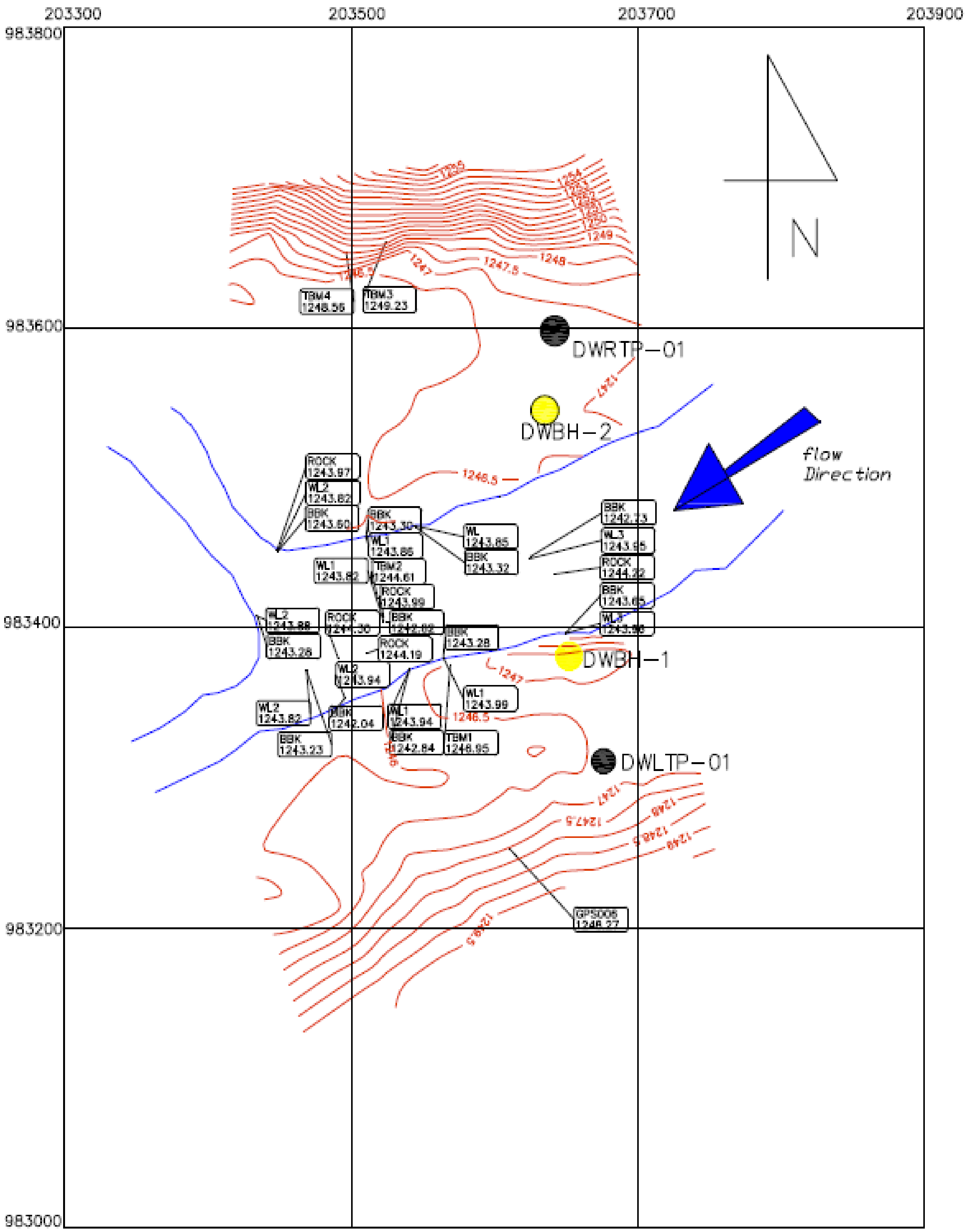


Figure 1: Location of boreholes and testpits

Table 3: Summary of field permeability test results

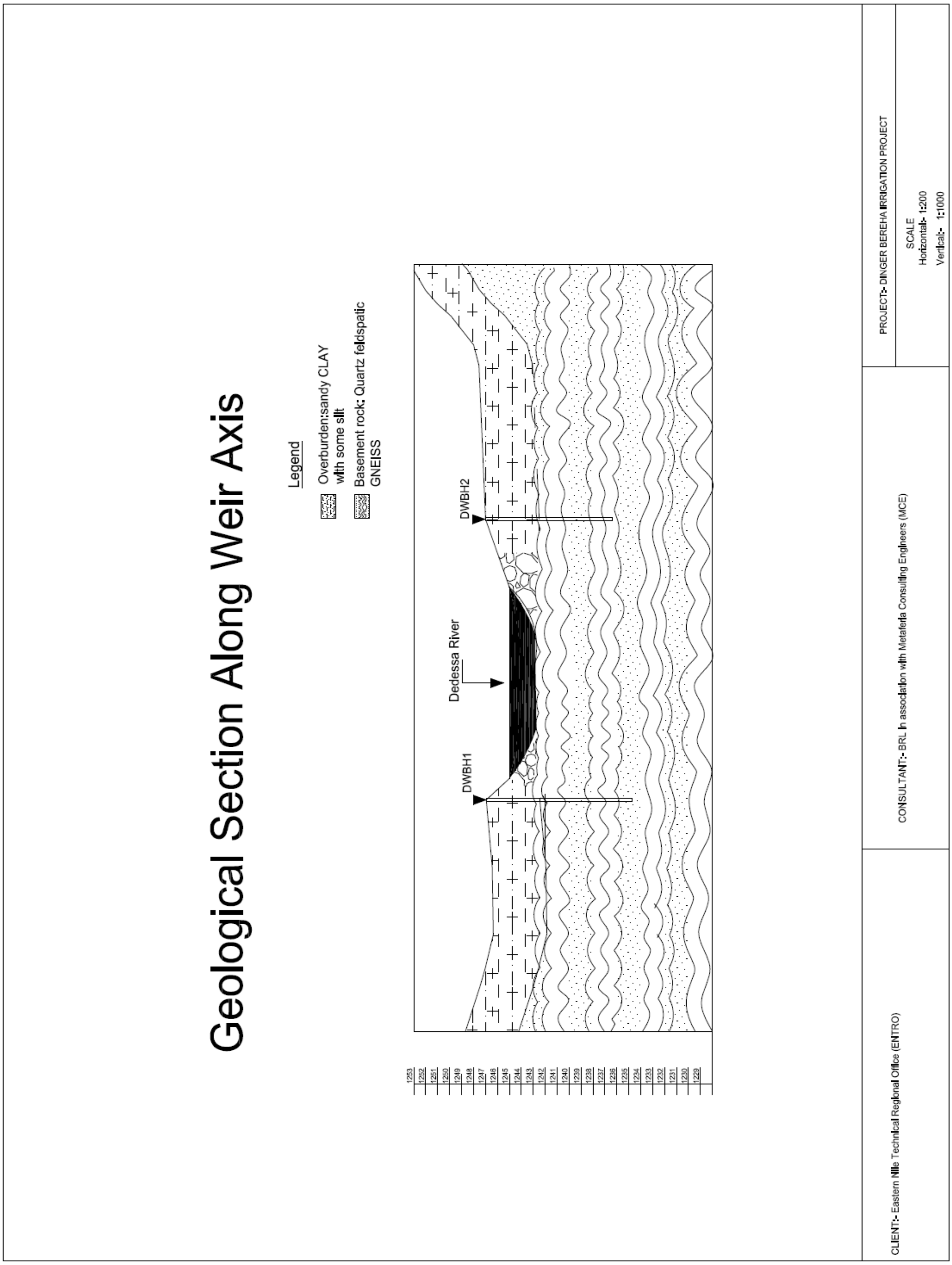
Test pit no	Tested section	Soil type	Coefficient of permeability, K (m/sec)	Comments on permeability condition
DCTP – 1	0.85 –1.15	Gravelly sand with some clay	3.03E-04	Very High
DCTP – 2	0.50 – 0.70	Black cotton soil	6.41E-03	Very High
DCTP – 5	0.95 – 1.25	Black cotton soil	8.50E-04	Very High
DCTP – 8	0.60 – 1.20	Gravelly sandy clay with silt	2.09E-04	Very High
DCTP – 13	0.30 – 1.00	Black cotton soil	4.27E-03	Very High
DCTP – 23	0.50 – 1.80	Sandy silty Clay	4.00E-05	Low to moderate
DCTP – 28	0.85 – 1.15	Gravelly silty Sand	3.38E-04	Very High
DCTP – 45	0.40 – 1.40	Silty Clay	2.96E-05	Low to moderate
DCTP – 48	0.50 – 1.80	Silty Clay	2.62E-05	Low to moderate
DCTP – 52	0.50 – 2.20	Silty Clay	5.30E-05	Low to moderate

Table 4: Summary of dispersion test

Sample no	Dispersiveness (%)	Dispersivity
DCoTP – 1	5.56	ND
DCoTP – 2	6.20	ND
DCTP – 23	6.80	ND
DCTP – 48	5.58	ND
DCTP – 52	6.02	ND
DCTP39	45.14	ND

ND= Not Dispersive D = Dispersive

Figure 2: Geological cross section along the weir axis



1.4.5.1 Variation of materials along the canal route

The length of the conveyor/feeder/main canal is more than 16 km. Due to its long distance coverage and variations in depth, the materials/soil varies considerably and the following four materials types have been identified.

- *Black cotton soil:* this soil type is exposed at the first 6 to 7 km in the areas where the topography is flat and slightly gently sloping. The thickness of this soil group varies from 0.30m to 0.95m. The soil is silty clay material with a high content of clay. Cracking when there is no water is very common for the formation in all locations. Three falling head permeability tests had been conducted in different test pits to characterize the soil conductivity and the soil has been found very permeable (table 6.3).
- *Silty gravelly sand:* this material group was found below the other top soil types and it is the product of weathering and decomposition of the gneiss rocks underneath. The thickness of this material group varies from point to point along the canal alignment. The in-situ falling head tests show that the soil group is very permeable. Detail description of the test pits is presented in Appendix C, Volume II, Annex C.
- *Silty clay*: the soil in this group, in most cases, originates from the decomposition of basalt rocks. The material in this category starts to appear at mid way and the end point of the canal route; however, it is not continuous all the way. The thickness of this soil group along the mentioned canal area is in the range of 1.0 to 1.5m; with interruption to nil at the creeks where weathered gneiss has been exposed in rock exposures. In general the soil is red, moist and firm silty clay. In-situ permeability test results show in general that this soil group exhibits moderate to low permeability.
- *Rock exposures:* the type of rock formation which covers the Project area as a whole is gneiss. Fresh exposures of the formation are common on the elevated areas, in the riverbed and in creeks which are known as temporary and permanent water ways. More over, fresh, strong and massive gneissic rocks exposure, which covers about 1.5km of the canal route is identified between the test pits DCTP31 to DTP 28, see location map of the canal root test pits. This rock is been exposed following the main ridge of the area. In this location, the rock is exposed continuously up to the bed of river Didessa.
- *Organic humus: t*his soil group are observed in the place where the land is covered by dense forest, covering the top of the surface. It is grey to brown in colour and very loose. The thickness of the soil varies from 0.10 to 0.50m.

1.4.6 Laboratory test results

Soil and rock samples collected from the test pits of the canal route; borrow areas, boreholes and the aggregate quarry sites have been tested in the soil and rock mechanics laboratory of Construction Design Share Company, for determination of engineering properties and for classification. Laboratory test result reports for all of the tests are presented in Appendix E. Most of the test pits are excavated along the canal route for characterization of the canal routes and possible checking for the use as the fill materials. The samples from these test pits are representative of the soil types observed during excavation for investigation of the route.

1.4.7 Soil Samples

Summaries of results from laboratory testing of soil samples are presented in the table 6.7. All soils are collected from the canal route and borrow sites.

1.4.7.1 Samples from black cotton soil

The soil samples from this area are highly plastic, Black to dark grey silty clayey soil with clay content greater than 85%. The liquid limit is varying from 66 – 87%. The plastic index is also increase up to 29.49%; and it swell up to 120% when in contact with water.

1.4.7.2 Samples of borrow areas

The moisture content at an optimum compaction density is in the normal range of 22.6 -22.9%, with an average OMC 22.7%. The results of compaction test show that the materials are suitable to use as a fill material. In addition to this, the shear strength tests conducted by using tri-axial test on the samples shows that the test results of the material is in the normal limit. The detail results from the tri-axial tests for the determination of the shear strength of the material are shown in Appendix E.

1.4.8 Rock samples

All laboratory test results on rock core samples and block rock samples from quarry area for aggregate are summarized and presented in Table 6.5.

1.4.8.1 Rock strength

Uni-axial compressive strength (UCS) and point load testing were conducted on the core samples collected from the bore holes drilled along the weir axis for evaluation of the strength.

a) Point load strength: The point load index is an expression for the induced tensile strength of the rock. Variations in point load indexes indicate the variation in tensile strength.

In the geo-mechanics classification system (Bieniawski system), Point Load Indexes (Is) are classified as shown in the table below:

Point load index, Is(50) in Mpa	Strength class
< 1	Very low
1 – 2	Low
2- 4	Medium
4 – 8	High
> 8	Very High

Point load testing has been conducted on drilled core samples collected from both boreholes. The test results are presented in the table 6.5, and the rocks are generally categorized in the range of medium to high strength class.

b) UCS: International Society for Rock Mechanic (ISRM) classifies Uni-axial Compressive Strength (UCS) strength as:

UCS (Mpa)	Strength class
5 – 25	Low
25 – 50	Medium
50 – 100	High
100 – 250	Very High

Only the basement rock core samples from the boreholes drilled at the weir axis were tested. The results of the testing and their strength class are presented in table 6.5 with those of the point load tests.

Table 5: Laboratory testing of core samples strength

BH	Depth	UCS		Point load		Rock porosity (%)	Unit weight	Specific gravity	Sodium sulphate soundness
		Mpa	**Class**	**Mpa**	**Class**				
DWBH -1	5.0- 5.15			5.74	High	0.85			
DWBH -1	5.57- 5.95	51.60	High				2449	2.72	
DWBH -1	7.0-7.5			8.70	V. High		2722		5.25
DWBH -2	5.68- 5.85			4.62	High		2688		
DWBH -2	6.65-6.87	36.37	Medium			0.72	2610		
DWBH -2	9.75-9.96			7.32	High			2.7	

1.4.8.2 Petro graphic analyses

Petrographic thin section analyses have been done on two samples, one from each borehole, at Addis Ababa University, Department of Earth Sciences. The results of petrographic thin section analyses are summarized and presented in table 6.6 showing percentage of minerals and the corresponding rock name and types. Details of the test results are attached in Appendix E.

Table 6: Petrographic thin section analyses

Borehole / Depth	Minerals present in % by volume							Rock name
	Feldspars	Quartz	Biotite	Epidote	Hornblende	Muscovite	Opaque	
DWBH-1 / 5.57 -5.95	55*	12.5	22	Trace	10	-	-	Biotite feldspar Gneiss
DWBH-2 / 6.65 -6.87	80*	11.5	4	1	-	3**	trace	Feldspar Gneiss
Remarks	*K-feldspar/ perthite			**Alteration product				

Table 7: Summary of laboratory test results on collected soil samples , classification tests

No	Code	Depth	Sieve Analysis			Hydrometer Analysis							Atterberg Limits		
			% passing (mm)			Smaller than			Particle size mm						
			2	0.425	0.075	0.02	0.002	0.001	>2mm	Sand (%) 2.0 - 0.075 mm	Silt (%) 0.075-0.002 mm	Clay (%) <0.002mm	LL	PI	Soil Class
1	DCoTP-1	0.30-2.50	100	85	75.6	65.86	49	47	0	25.24	71.62	3.14	57.33	23.67	MH
2	DCoTP-2	0.40-2.10	100	87	75.62	64.84	43.8	42.15	0	22.68	74.08	3.24	55.45	25.37	MH
3	DCTP-16	0.30-0.80	100	56.82	47.94	43.33	36.67	36.67	0	52	11	37	66.10	29.49	CL
4	DCTP-20	0.40-1.60	100	74.7	57.46	50.27	40.22	40.22	0	43	17	40	56.25	23.96	CL
5	DCTP-23	0.50-1.80	100	79	69	50.27	36.86	36.86	0	31	33	36	48.25	22.69	CI
6	DCTP-48	0.80-1.80	100	92	79	66.7	53	47.65	0	19.14	77.72	3.18	61.50	25.58	MH
7	DCTP-52	0.50-2.20	100	89	76.6	63.50	52	50.13	0	25.46	71.2	3.34	47.50	22.25	MI
8	DCTP39	0.6 – 3.77	100	75	67	29	8	0	0	33	59	8	38.50	12.98	CI
9	TPRes01	0.50 – 3.00	100	78	65	26	11	0	0	35	54	11	47.80	14.61	ml
10	DCTP37	0.60 – 3.00	-	-	-	-	-	-	-	-	-	-	46.20	17.21	CI
11	DCTP35	0.65 – 3.40	100	82	58	33	13	0	0	42	45	13	-	-	m
12	DTP28	0.70 – 2.20	100	86	63	49	33	0	0	37	30	33	-	-	c

1.4.8.3 Construction materials

Rocks for aggregate

Rock samples have been collected from drilled cores of Gneiss rock as well as blocks of basaltic rock samples from potential quarry sites for testing for evaluation of suitability for the use of as concrete aggregates. The samples were tasted for Alkali silica reactivity by using OPC type cement made of Pakistan, Rock porosity and sodium sulphate soundness. The results of the laboratory test, for both rock types show that there is no alkali silica reactivity sensitivity. Sodium soundness test and porosity of the mentioned rock types are also in the normal allowable range. Detailed results are also presented in Annex C, Appendix E.

Sand

The conducted test results on sand include grain size distribution, shear strength, organic content and mortar making property. The grain size distribution showed that the sand is well graded and; no fine percentage particles which affect the quality of the structure. The organic content observed in the sand quarries is also minimal especially considering that the sampling had been done not prior to the major rainy season. The friction angle found from the test results is quite high. That is not however expected to be a major influence in the design of the structure. Summary of the laboratory tests conducted and interpretative charts are provided in Appendix E.

Soils

The description and notes for borrow materials have been presented in section 6.4.7.2.

Water

Water samples have been collected from Didessa River on June 17, 2009 in order to evaluate the suitability for construction. The tests include sulphate, chloride, total alkali and TDS contents. In addition to this PH value of the water has been tested. The result of all the tests has showed that the content of all elements which are suspected to cause damage on concrete structures are below their maximum allowable limits.

1.5 ANTICIPATED ENGINEERING CONDITIONS, CONCLUSIONS AND RECOMMENDATIONS

1.5.1 General

Elevation of the riverbed at the weir axis is 1244m. Total length of the canal route is more than 16 km with pump station sites # 1 and 2 (Res01) at E 0192 467m, N 0991 423m and E 188 976, N 0989 689 respectively.

1.5.2 Weir axis

At the weir axis, the river is in an approximately 200m wide flat bottom of a U shaped valley. The bedrock in the weir axis, on both sides, out of the river channel is covered by soil. On the basis of the field investigations, the soil overburden is found to be shallow with a thickness nil at the riverbed to 4.75m for both banks of the river.

The soil at the flat flood plain, along the river consists a layer of alluvial deposited sand silt and clay with a thin layer of organic maters overlaying this alluvial deposit on the parts close to the river. Under a thin layer of top soil the overburden, a bit away from the river bank, consists of light grey silty gravelly sand, which is resulted from the decomposing of basement rock underneath. This sand layer has also contains clay and gravel size particles with gravel is being the dominant particle size next to sand. The thickness of the layer and gravel content of the residual soil seem to increase with the increase of distance from the river.

The river bed at the weir axis consists of massive, very strong and competent Precambrian rock. This formation covers the riverbed through out the Project area. The rock in the boreholes of weir site is found solid and competent, with some jointing near to the surface. The strength test shows that the rock is in the class of high to very high strength. The riverbed through out the Project area is characterized by out crops of apparently best quality Precambrian gneiss formation. Spacing of joint are also in the order of one to two meters indicating the bed rock is massive, solid, fresh and strong. The combination of the basement rock competency and shallow overburden of the weir site leads to the conclusion that the weir structure will be founded on the competent bedrock.

1.5.3 Canal route

The concluded canal alignment is approximately 16 to 17 km in length and its tentative elevation lies between 1248m at the intake and 1237m at the final point where the pump station#1 has to be constructed. Extensive field traversing and excavation of test pits has been undertaken to characterize the ground condition of canal route from the beginning to the end point of the route. Based on the assessment, a considerable variation of soil type, which in turn varies with the properties like permeability, stability and the so on, is identified. According to the available soil type and their engineering properties, the route has been categorized in to three classes.

Canal sections that need replacement by fill material

The soil in this category covers more than 9km. It has estimated to be 90% of the total length. The types of soil in the category are black cotton and residual silty sand resulted from the weathering of bedrock. The engineering properties of both soil types have found not suitable, because of high permeability noticed during the field in situ permeability tests.

Canal sections crossing suitable soils

The soil with good engineering properties is found in the location where test pits no DCTP 23, 52, 48, 44, 39, 38, 37 and Dtp 28 & 29 have been excavated. The exact length of the route of the soil in this category will be well estimated after the surveying of the route completed. From the field observation the coverage of this soil group is estimated to be 7 to 8%. The soil in this group is identified by a characteristic of being red to reddish brown, firm, not permeable to slightly permeable silty clay with some sand. The dispersiveness tests also show that the soil in this category is not dispersive.

Canal sections in cut and crossing creeks and the ridge

More than 20 creeks and a ridge with rock exposure have been identified upon the field traversing and visualizing of topographic map of the route. Moreover, there are also some areas with hard, solid and fresh rock exposures. Slightly to moderately weathered basement rocks is exposed in almost all of the creeks which show that the foundation condition for the crossing structures is found excellent. On the other hand, the rocks exposed at the ridge are fresh and very hard; as the result, excavating of this rock locality may need blasting due to its hardness. The estimated coverage of the route by this category is about 1.5 km.

1.5.4 Pump stations

The location of pump station site #1 is at a geographic location E 192 467m, N 991 423m. One test pit has been excavated to a depth of 0.70m to evaluate the ground condition of the site. The soil type is loose soil resulting from the decomposition of the basement rock underneath. It is reddish grey, sandy silty clay. The layer below, up to the depth 0.70m is gravelly sand, being residual soil of the bedrock of the Project area. Below this depth, fresh and solid rock with slightly to moderately weathering has been noted. Moreover the creeks in the near the alignment are characterized by exposures of the same rocks as mentioned. From the general observation of the area and the information from the test pit, it can be concluded that the soil in the area has proved shallow and loose. On the other hand, the bedrock observed in the test pit and the creeks are which favour founding the pump station and its equipment on this rock.

The location of pump station#2 has been investigated by excavating one test pit up to the depth 3m. Its location is at the geographic coordinates E 188 976m, N 989 689 and the symbol of the test pit is TPReso1. The soil is firm to very compact clayey silt with no or low plasticity. The result of strength test also shows that the soil has excellent shear strength which indirectly implies that the bearing capacity is very good.

1.5.5 Construction material assessment/ borrow sites

Rock materials

Rock samples have been collected from drilled cores of Gneiss rock as well as blocks of basaltic rock samples from potential quarry sites for testing for evaluation of suitability for the use of as concrete aggregates. The samples were tested on alkali silica reactivity by using OPC type cement, Pakistan, Rock porosity and sodium sulphate soundness. The results of the laboratory tests for both rock types show that there is no alkali silica reactivity sensitivity. Sodium soundness test and porosity of the mentioned rock types in the above paragraph are also in the normal allowable range. These rock materials are found on the hilly areas on the left and right side of the river. Since the material is available in every hilly area, the volume is in excess and the maximum distance from the weir site not more than 15km. Detailed results are also presented in Appendix E.

Gravel for concrete/coarse aggregates

There are no borrow areas for gravel and coarse aggregates, so this material has to come from crushed rock.

Sand

Fante River and the river between the villages no 4 and 5 have been identified as sources for sand. The sand is to be used for concrete works of the weir as well as cross drainage structures. Both sites are located along the road from Chewaka town to Village no 5.

The approximate maximum distance of both sites from the weir axis is in the range of 22 to 24 km. Representative sand samples have been collected and tested from the sites in laboratory and the test result shows that the sand is suitable for the desired propose. The volume of sand observed on the sites during the study is estimated to be more than 30,000m^3.

A summary of the laboratory test results conducted and interpretation charts are provided in Volume II, Annex C, Appendix E of the Field Investigations Report.

Water

Didessa River would be the main source of water for the construction of the main canal and structures. During the wet season floodwater will carry a lot of silt, but then compaction will be halted anyway because of high rainfall. There are many streams in the Project area that carry relatively clean water of good quality throughout the dry season.

Masonry stone for structures

There are no borrow areas for masonry stone in the Project area or in the near surroundings.

Borrow (Clay) materials for canal fill

Fine clay material suitable for use of fill materials had been identified at three different locations of the Project area. Sources of red clay have been identified at locations close to the beginning, middle way and end point of the canal route. The potential borrow areas are located on high grounds and two of them are currently being used as farm areas. The three borrow areas identified are shown on the location map in Volume 3: Annex Geotechnics. A total of 7 test pits were excavated manually within the identified clay borrow sources. The depths of the tests pits excavated varied from 1.6m to 2.5m. Based on the test pits excavated, the total estimated volume of material available from the three borrows is in excess of 1 million m^3. The clay borrows identified in the Project area are similar in origin (mainly residual), are brownish to reddish in colour and also exhibit in general similar properties regarding plasticity. All of them plot above the A line signifying property of low plasticity silt clay.

The results of the Atterberg limit tests showed that the PI varied between 22 and 29% with the average in the order of 24%. The average LL was in general greater than 56% signifying high plasticity properties. Test results for dispersion property using double hydrometer testing apparatus indicated that the clay material is not dispersive. The coefficient of permeability is quite low (an average of 10^{-6} cm/s) making it quite usable as a fill material. The natural moisture content of the clay sources varied between 11 and 15%. Moisture application will in general be required to get it close to the Optimum moisture content to attain maximum strength. The variations in the optimum moisture content between the various borrow material is insignificant with the average for all the clay borrows being around 22.7%. The tri-axial shear strength tests conducted gave the values reported in Appendix D. The effective cohesion value reported is well higher than could be expected for such material.

1.6 SEISMIC HAZARDS

The geophysical observatory of the Addis Ababa University has specifically assessed the seismic hazard for the Arjo Didessa Irrigation and Hydropower dam site according to the methods advised in the global seismic hazard assessment program (ref: Main Report Feasibility Study Arjo Didessa Irrigation Project, Febr 2007). The proposed weir and other infrastructure sites for the Dinger Irrigation project are very close to the Arjo Didessa Dam site and it has a similar hazard probability zone.

Consequently the analysis and assessment made by the above organization works for the current project. Usually the horizontal acceleration on the seismic hazard map is presented by contouring as a fraction of g. The nearest contour values taken for the Arjo Didessa Dam is 0.05g for the peak horizontal acceleration and 0.025g as a vertical seismic coefficient, which has been also considered for the stability analysis of the structures at Dinger Irrigation project site. According to the ICOLD recommendations, the site with a peak ground acceleration of 0.05g is ranked to hazard class I, which is low hazard. Most structures in this category will not experience damage under maximum design earthquake. Design Earth quake has to be selected at a return period of 300 years, which corresponds to a design peak ground acceleration of 0.05g, i.e., $0.5 m/s^2$ for the structures founded on rock. The consequences for the design will be only slightly stronger concrete. All large structures related with Dinger Irrigation project will be founded on hard rock, and no significant deposit of fine sand or silt, which will have a tendency for liquefaction..