THE NATIONAL REGIONAL STATE OF OROMIA
WORBATE SMALL SCALE IRRIGATION PROJECT

ENGINEERING DESIGN FINAL REPORT

CLIENT

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TABLE OF CONTENTS

CONTENTS

LIST OF TABLES

LIST OF FIGURE

SALIENT FEATURES OF THE PROJECT

- Project Name: Worbate Small Scale Irrigation Project
- River Name: Worbate River
- Head Work Type: Earth Dam
- Head Work Location:
  - Northing: N 551941
  - Easting: E 436255 zone 37
  - Elevation: 1454 m a.s.l.
  - Zone: Borena
  - Woreda: Gomole
  - Kebele: Abunu
- Net Command Area: 120 ha
- Conveyance:
  - Type: Lined Main Canal
- MC
  - Number: 1
  - Length: MC=4.4255 km
  - Cross Section type: Rectangular
  - Nature of canal: lined with masonry
  - Discharge: MC=0.315m^3/s
- SC
  - Number: 3 SCs
  - Length: 1.5959km
  - Cross Section type: Rectangular
  - Nature of canal: lined with masonry
- TC
  - Number: 18
  - Length: 6.502km
  - Cross Section type: Trapezoidal
  - Nature of canal: Unlined i.e. earthen
- TD
  - Number: 13
  - Length: 5.401km
  - Cross Section type: Trapezoidal
  - Nature of canal: Unlined i.e. earthen
- Project Cost
  - Grand Total of Project Work cost 88288660.69
  - Cost per hectare: Birr 735738.84

1 INTRODUCTION

1.1 Background

In Ethiopia, as well in Oromia under the prevalent rain-fed agricultural production system, the progressive degradation of the natural resource base, especially in highly vulnerable areas of the lowlands coupled with climate variability have aggravated the incidence of poverty and food insecurity. The major source of growth for the country as well for the region is still conceived to be the agriculture sector. Hence, this sector has to be insulated from drought shocks through enhanced utilization of the water resource potential of the country, (through development of small-scale irrigation, water harvesting, and on-farm diversification) coupled with strengthened linkages between agriculture and industry (agro-industry), thereby creating a demand for agricultural output. In line with the above, efforts have been made by the government and NOG's to improve the situation in the region in areas of domestic water supply provision, irrigation development, watershed management, etc.

In order to alleviate the situation of food insecurity in lowland areas of the region that resulted from shortage of Rainfall & recurrent drought, the region have no choice but to aggressively switch development endeavors towards effective and efficient use of the existing natural and human resources. A development strategy that could help is the use of the water and land resource endowments. As a result, development of different irrigation schemes (small- to large-scale) can play a major part not only in solving the current food insecurity, but also in enhancing the economic development of the region as well the country through the earning of more foreign currency and the creation of employment opportunities.

Accordingly, the regional government planned the feasibility study of small-scale irrigation projects that will contribute in problem alleviation due to shortage of rainfall/ drought in lowland areas of the region that affected the production and productivity of the peoples, which exposed to food insecurity. Based on these; Oromia Irrigation Development Authority (OIDA) has proposed Worbate Small Scale Irrigation project (WSSIP) to develop micro dam irrigation in Abunu kebele of Gomole district, which is financed by International Fund for Agricultural Development (IFAD). And the Clint (OIDA) has signed an agreement with consultant Oromia Water Works Design and Supervision Enterprise (OWWDSE) for detail feasibility study. Thus; the report focused on detail Dam and its related structure for the development of 120 ha of irrigable land of Worbate Small Scale Irrigation Project (WSSIP) in Borena zone, Gomole district in Abunu kebele.

1.2 Project Location and Accessibility

Worbate Small Scale irrigation project (WSSIP) is a micro dam which is located in Borena Zone, Gomole district, Abunu kebele. The project site is located at 48 km from Surupha town (district town) to the South -East direction on the asphalt road (18km) until Haro-Bake, then turn to left direction while coming from Surupha at Haro-Bake on the dry weather road which is about 30km until the project site which is the road which needs major maintenance to access the project Abunu kebele; and totally Worbate small scale irrigation (WSSIP) is located at 48km from district town (Surupha), and 52km from Yabelo zonal town; and also found at a distance of 573 km from Finfinne.

The dam site is located at a geographic location of Geographic coordinate of 436255 E and 551941

Adindan 37 zone projection. As shown in the Figure

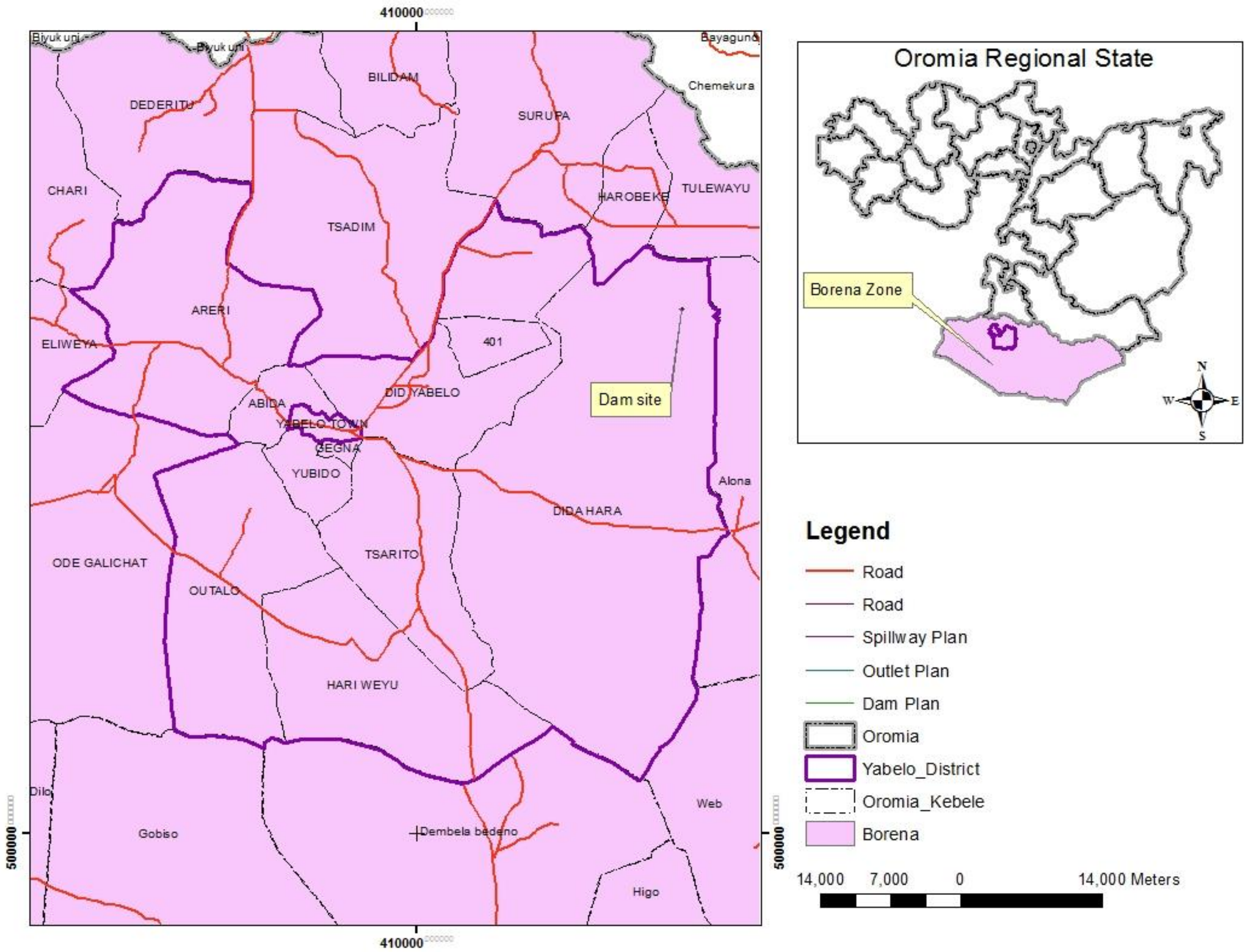

Figure 1-1 Location of the

2 DESCRIPTION OF WORKS

2.1 General Description

The Principal components of the Works which will be covered under this feasibility and detail are those stated here under but further components will be included during the detailed design.

a. Main Dam (Homogenous Earth fill Dam with Horizontal drains and Filter);
b. Spillway (incorporating Approach Channel, Chute and Stilling basin) and;
c. Irrigation intakes, conduits and outlets

2.2 Dam Body

The main dam body is designed with one option i.e. homogeneous earth embankment with drain based on the seepage developed at the downstream faces (end or horizontal drain). For there is no any shell or pervious material around the project at the nearby distance considering other option makes uneconomical except for the consumption of the report. In such condition it is practical to design homogeneous earth dam with drains at the end, Horizontal and horizontal with inclined one. However, for this specific project Drain at the End and Horizontal are compared but for safety the Horizontal in order to protect migration of soil to drain seepage water and smooth transition from fine to coarse textured drain materials Filter is Designed. Excavation is necessary to remove some areas of alluvial deposit on the left and right bank of the valley having weak shear strength

2.3 Spillway

Side channel Chute spillway is designed to discharge a maximum of 85.55m3/s corresponding to 100 years return period routed outflow. However, the crest level is fixed not to be over topped with this routed outflow. The total crest length of the spillway is 20m. The crest level (full supply level) is at 1470.5m and the maximum water level corresponding to 100 years routed outflow is 1472.4 m i.e. with flood lift height of 1.9.m. The structure comprises an approach channel, chute and terminal structure. The spilled water leaving the terminal structure will join the natural river channel.

2.4 Diversion Arrangements

The construction of Worbate Dam might take one or two years to complete. The construction will take place in the dry seasons. Therefore, it is not possible to completely block the river flow rather to create provisions for safe passage of the river flow during the rainy and dry season.

The followings are considered the main components for the diversion system during construction of the dam.

- Up and down stream cofferdams,
- Dry season conduit with an inlet and out let channel, and
- Wet season diversion i.e. the existing river course.

2.5 Irrigation outlet

The main purpose of the Worbate project is for irrigation. It is planned to develop the command area estimated to around 120ha downstream of the dam. Therefore, during dry months of the year regulated water will be released as per the downstream demand. Accordingly, intake structure is provided for this purpose.

3 HYDROLOGY

3.1 General

Water is a vital input for sustainable agriculture and hence irrigation is used as a means to provide required water for agricultural production, one of the strategic development sectors for the economy of the nation. However, irrigation investments need careful planning as water is one of the scarce resources and also multiple purposes compete for the use of water. This necessitates the need for integrated forms of management fairly share it among the competing components to satisfy the development need of the targeted community. Furthermore, the success of any irrigation project depends on the involvement of the concerned communities and a comprehensive analysis of the technical, economic, social and environmental factors. Thus, to avail and secure the required water for crop production, detention dam plays a significant role in gathering the runoff prevailing during the rainy season and use it during moisture deficit period. Thus, considering the high moisture stress affecting the availability of food for human and feed for livestock in Borena Zone of Oromia Regional State, Worbate Dam for SSIP was planned by OIDA that is specifically located in Gomole woreda, Abunu kebele. The area is characterized as arid climate region situated in Ganale-Dawa Basin, the basin with most scarce water resource basin in Oromia Region.

The main objective of the climate and hydrology studies of the project area is to investigate the availability and adequacy of water resources of the Warbate catchment at the proposed dam site. Specifically, the study estimated flood magnitudes that is useful for the design of dam and appurtenant structures, cross-drainage and field drain structures, analysis of climate data for crop water requirement, water balance of the catchments and sediment transport for estimation of dead load at the dam site, estimated water required for the possible water demanding components in the catchment and environment. In this view, the study depicted that the watershed generates 4.9 million m3 of water at the dam site for the dependability of 80% of runoff analysis carried out for the historic climate data from 1997 to 2017. In addition, the estimated sediment load constituting the dead storage of the reservoir is 1.08 m3 for the service life of 50 years.

3.2 Physiography of project Area

The general feature of the Warbate catchment is with plain in the valley with sharp rise at its upstream sides. The general slope of the catchment is categorized as gentle slope. The temperature and rainfall exhibit that of the Borena area, particularly, Ya'a Balo (Yabello) with average annual values of 20.18 °c and 627.46 mm respectively. The average annual relative humidity and potential evapotranspiration of the area are 66.42% and 1423.19 mm. The analysis made shows there is a trend of increase in annual average rainfall over the years 1997 to 2017 in the area represented by Ya'aa Ballo (Yabello) climate recording station. For detail refer the hydrology and climate study report.

3.3 Stream Flow

Daily flow data at dam site is necessary for design of dam and its appurtenant structures to estimate maximum design flood for a given return periods, dependable flow required for estimating reservoir capacity and low flow useful for estimating the base flow and decide environmental flow for the downstream environment according to the evaluation an estimation made at the dam site the dependable flow at different provided at Table 3.2. But the dependable flow of 80 % was adopted for this study for it is a common practice in the country. For detail analysis refer the Hydrology Report

Table 3-1 Dependable Flow m3/s

	Dependability					
Month	50%	70%	75%	80%	90%	95%
Jan	0.134	0.000	0.000	0.000	0.000	0.000
Feb	0.111	0.000	0.000	0.000	0.000	0.000
Mar	0.789	0.526	0.330	0.277	0.000	0.000
Apr	1.575	1.095	0.978	0.822	0.063	0.000
May	0.844	0.438	0.404	0.382	0.000	0.000
Jun	0.094	0.000	0.000	0.000	0.000	0.000
Jul	0.000	0.000	0.000	0.000	0.000	0.000
Aug	0.176	0.000	0.000	0.000	0.000	0.000
Sep	0.289	0.000	0.000	0.000	0.000	0.000
Oct	0.850	0.572	0.448	0.372	0.252	0.012
Nov	0.600	0.251	0.062	0.000	0.000	0.000
Dec	0.142	0.000	0.000	0.000	0.000	0.000

3.4 Sediment Rate

Different approach is adopted with site level observation for the analysis of sediment load at the dam site, the detail is presented in the Hydrology report. And found that, the annual sediment load estimated becomes 1.08 million m^3 for design purpose. If 50 years is proposed as design period of the dam, the overall sediment load expected becomes 1.03million tons.

3.5 Flood and Flood Routing

3.5.1 *Flood estimation*

In order to evaluate the values of design flood flows different alternative methodologies were applied. The Probable maximum flood is estimated from probable maximum precipitation and it is transformed into PMF using unit hydrograph convolution techniques. In addition to the PMF, flood design discharges for various return periods at the dam site are evaluated using different candidate probability distributions and parameter estimation techniques.

3.5.2 *Design flood selection*

The USACE (United States Army Corps of Engineers) published guidelines and USDA (United States Department of Agriculture) approaches has been consulted for the selection of the design flood. According to this approaches the spillway design flood will be between 100 and 1/2.PMF. Similarly, according to the Ethiopian Draft Dam Safety Guideline prepared by Ethiopian Commission for Large Dams (ETCOLD), the potential impact classification (PIC) falls under PIC 2.

Table 3-2 Recommended inflow design flood and safety check flood

Potential Impact Classification (PIC)	Inflow Design Flood (IDF) (Return Period, years)	Safety Check Flood (SCF) (Return Period, years)
1	500	1,000
2	1,000	10,000 or 0.5 PMF whichever greater
3	10,000 or %age of PMF whichever greater	PMF

But for Worbate dam the draft guide line proposed by USDA is adapted. Accordingly, the inflow hydrograph adapted is 100 years return period with peak discharge of 120.368 m^3/s and is shown in Figure 3.4.

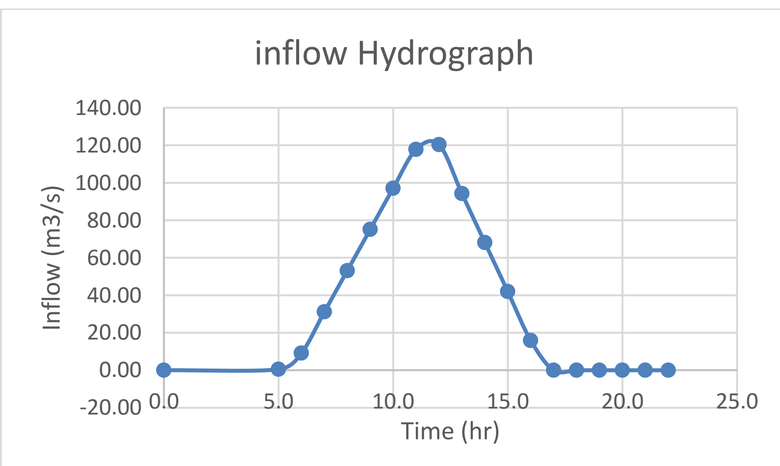

Figure 3.4: Inflow hydrograph corresponding to 100 years return period

3.5.3 *Flood Routing and spillway sizing*

The flood hydrograph developed at the dam site was then routed over the reservoir. The Full Reservoir Level (FRL) is fixed at 1470.5 m asl and a broad crested weir type spillway crest is adapted.

Accordingly using the capacity curve and SIF (storage indication factor) for uncontrolled spillway length of 20 m with C=1.7 is computed and presented in the table and graphed in figure below.

Table 3-3 Flood Routing

time (hr)	storage m^3	inflow(t) (cms)	2*S(t)/dt - O(t) (cms)	SIF (cms)	O(t) (cms)	Elevation (m)
0.0	0	0.00	0.00	0.00	0.000	1470.500
5.0	882	0.49	0.49	0.49	0.000	1470.500
6.0	18,234	9.15	10.13	10.13	0.000	1470.500
7.0	87,188	31.13	46.46	50.41	1.975	1470.650
8.0	219,797	53.12	113.51	130.71	8.601	1470.900
9.0	399,255	75.10	201.90	241.72	19.913	1471.200
10.0	607,457	97.06	300.89	374.06	36.582	1471.600
11.0	821,441	117.76	396.99	515.72	59.365	1471.950
12.0	1,001,520	120.36	477.69	635.11	78.711	1472.250
13.0	1,092,114	94.24	521.18	692.28	85.553	1472.400
14.0	1,076,363	68.12	512.43	683.53	85.553	1472.350
15.0	984,925	42.00	471.82	622.54	75.362	1472.200
16.0	846,611	15.89	410.97	529.70	59.365	1472.000
17.0	687,902	0.00	337.47	426.86	44.694	1471.700
18.0	546,252	0.00	269.47	337.47	34.000	1471.500
19.0	441,261	0.00	220.82	269.47	24.328	1471.300
20.0	365,398	0.00	185.18	220.82	17.818	1471.150
21.0	308,364	0.00	157.44	185.18	13.868	1471.050
22.0	264,926	0.00	136.92	157.44	10.264	1471.000

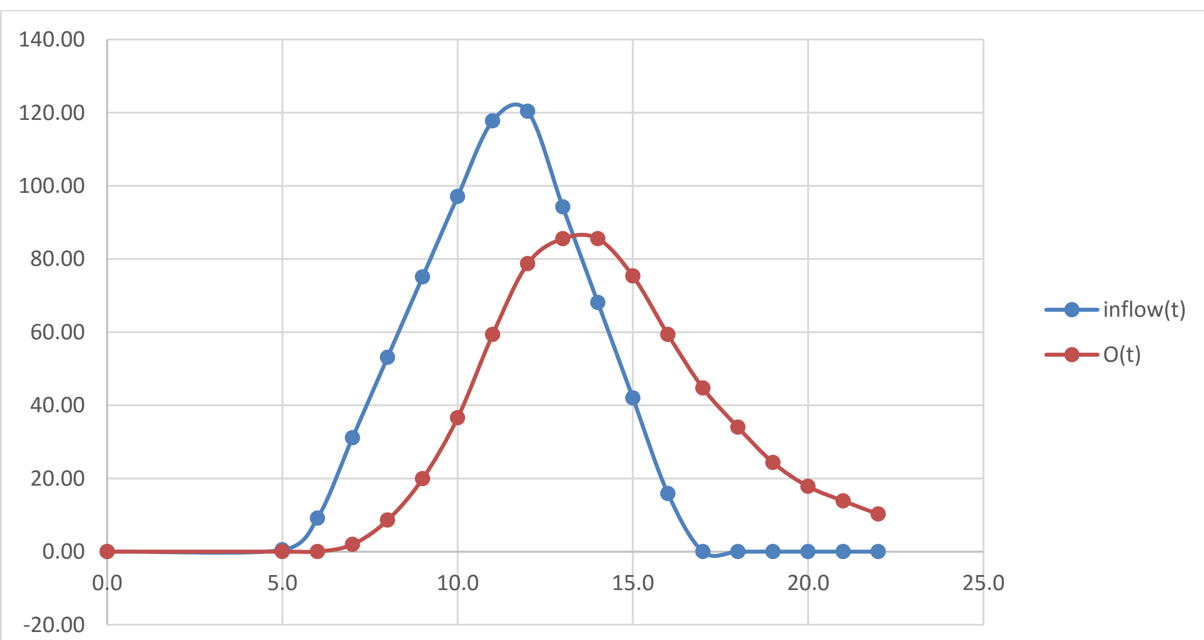

Figure 3-1 Inflow - Outflow Hydrograph 20m spill way

The routing exercise was conducted for total length of 10m, 15m and 20m width ungated spillway. Summary of the analysis result is presented in Table 3.5.

Table 3-4 Summary of Peak inflow, Outflow and Elevation for 1000 years flood

Spillway Crest Length (m)	Peak Inflow (m3/s)	Peak Outflow (m3/s)	Maximum Elevation (masl)
10	120.36	65.19	1472.95
15	120.36	77.60	1472.60
20	120.36	85.55	1472.40
25	120.36	94.20	1472.20
30	120.36	98.42	1472.05

4 GEOLOGY AND GEOTECHNICAL INVESTIGATION

4.1 Regional Geology

The Mozambique belt, first named by Holmes (1951), consists of high grade gneisses and migmatites with in folded schists, marbles and amphibolite's intruded by granites and pegmatites (Almond 1983). This orogenic belt extends from Mozambique in the south to Ethiopia in the north (Cahen et ale 1984, Berhe 1990). It is characterized by a meridional trend with a similarly oriented structural fabric and generally consists of rocks having ages between 1300 and 480 Ma (Cahen et ale 1984). The highly metamorphosed and deformed gneissic rocks of southern Ethiopia are thought to be part of the Mozambique belt (Gilboy 1970, Chater 1971, Kazmin 1978a).

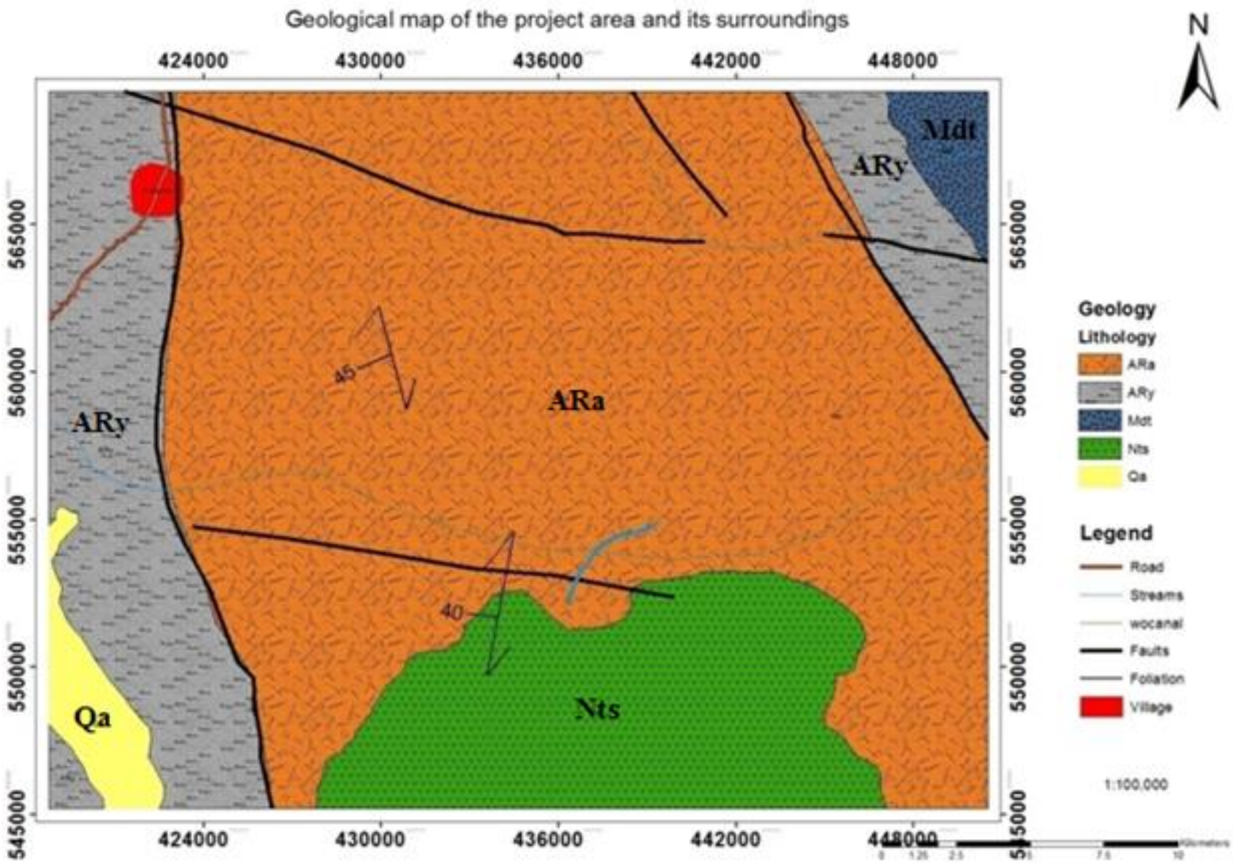

Figure 4-1 Geological map of Worbate Small Scale Irrigation area and its surrounding

4.1.1 *The Awata Group (ARa)*

The Awata Group (ARa) originally identified in Sidamo region and named as Awata Gneiss (Kazmin, 1975) consists of well layered gneiss and granulite of metasedimentary origin and occur mainly west of Negele and in the Hamar region (Kazmin,1972 &1975, Davidson 1983). The group is characterized by well-developed layering and by the presence of thick, and red- brown weathering zones.

The gneisses and granulite's contain significant amounts of interlayered biotite gneiss with or without garnet, and include the metasedimentary types such as pelitic gneiss with muscovite and /or sillimanite and garnet, calc-silicate gneiss with diopside, grossularite and scapolite, both calcite and dolomite marbles, impure quartz gneiss and quartz-rich feldspathic gneiss. In addition, many of the quartz-rich and biotite gneisses carry graphite and pyrite or pyrrhotite.

The anhydrous layered pelitic granulite contains hypersthene or sillimanite and garnet. A notable proportion of dark hornblende gneiss and amphibolite also occur in this unit. The dark bands and hornblende gneiss may represent original clayey sediments and amphibolite's may represent basic intrusions (Davidson, 1983)

The pelitic gneisses of this unit contain muscovite in a restricted region in the western Hamar plains approaching middle amphibolite facies. There and elsewhere biotite and garnet are the main mafic minerals, locally accompanied by sillimanite; kayanite is rare, and andalusite, staurolite and cordierite were not observed. Marbles and calc-silicate gneisses at low grade contain tremolite, but closer to the granulite zone of the Hamar range, they contain diopside grossularite and scapolite, and only rarely amphibole. Deformation and metamorphism have been so intense that all evidence of former sedimentary structures have been obliterated; this unit of diverse paragneiss with large aerial extent indicates a moderate to shallow marine deposition in a relatively stable tectonic regime (Davidsqn, 1983).

4.1.2 *Yabello Group (ARy)*

The Yabello Group was originally recognized in the Yabello area of Sidamo region and was named as Yabello Gneiss and is characterized by the presence of discontinuous layers of rather massive quartzo-feldspathic rocks which merge into granitic gneisses (Kazmin,.1972). Mapping carried in the Omo River area (Davidson, 1983) showed that similar quartzo-feldspathic gneiss and granulite underlie large tracts of ground between Jinka and the north end of the Hamar range, between Beto and the northern part,of the Chew Bahir rift and on the southern side of Mt. Gugu. The first two lie in broad, northwest oriented strips, separated by gray gneiss of the Alghe Group (AR1) east of Jinka, but converge in the vicinity of Lake Weyto (Lat. 5° 25' N and Long.

36° 58' E). The third is separated from the rest by tracts of units of Konso and Alghe Groups that occupy the area west of Lake Chamo.

Like the Alghe Group, the rocks of the Yabello Group vary from being relatively uniform orthogneiss to well-layered gneiss, and probably contain rocks of both plutonic and supracrustal origins. They are characterized by their light gray, pink, buff or cream colors and by low color indices. In the areas around Jinka, Beto and Balta, pink and light gray quartzo-feldspathic gneisses are foliated, biotite-bearing and commonly migmatitic with stromatitic form and pegmatitic leucosomes richer in K-feldspar. To the Southeast, these rocks merge along strike with more granular leucocratic gneisses in which magnetite is the dominant mafic mineral along with scattered garnet and rare biotite in some rocks. This change coincides with the elevation to granulite facies as indicated by the appearance of hypersthene in the rock units, and is accompanied by change in color to buff and cream hues.

Quartzo-feldspathic granulites exposed around Lake Weyto tend to be slabby or blocky on outcrops, having layering that is due to less compositional differences than to layered variation in grain size.

Many of these rocks have lenticles or rods of quartz among sugary, equant feldspars, giving a texture that is typical of a classic leucocratic granulite (leptite). The sugary quartzo-feldspathic granulites are usually anhydrous; the place of biotite has been taken by magnetite and garnet, rarely hypersthene, with additional K-feldspar.and mesoperthite is common in these rocks

4.1.3 *Teltele and Surma Basalts (Nts)*

Teltele and Surma Basalts (Nts) were mapped in southwestern Ethiopia (Davidson, 1983) and represent the youngest pre-rift food basalt volcanism in the area. The Teltele basalts lie unconformably on the crystalline basement, or are separated from the basement by variable thickness of varicolored clastic sediments. Other stratigraphically higher basalts and felsic lavas (as young as Early Pliocene) mapped in the type area are here considered analogous to Teltele Basalts. The Turkana-Teltele fissural basalts of Merla et al. (1973) which cover the plain further to the east and cross the international border with Kenya to the south are considered to be correlable with the Teltele Basalts.

The type locality of the Surma Basalts is further to the west on the international boundary with Sudan. The Surma Basalts overlie older pre-rift volcanic successions (Jima Volcanics; Pjb I Pjr). The Teltele and Surma basalts are of Early Miocene age with reported KIAr ages between 21.2 and 18.2 Ma (Davidson, 1983).

4.1.4 *Quaternary sediments (Qa)*

Alluvium (Qa) consists of sandy gravel, silt and clay, mainly occurring along river courses of very low gradient. In contrast to the eluvium, alluvial sediments are predominantly fine grained.Alluvium in the Segen basin consists of silty clays with minor detrital feldspar, quartz and rounded rock fragments. In the Gelana basin, the alluvium is buff to grey silts and clays with subordinate sands. The thickness of unit Qa varies from place to place, the maximum being 15 m in the Gelana basin.

4.2 Geology of project site

4.2.1 *Dam site: Topographic Set Up, Surface and Sub-Surface Investigation*

The intended diversion site on Worbate stream is located at a GPS location: UTM 436346mE, 552131mN and 1459m.a.s.l. elevation. The Dam site is covered with alluvial sedimentation. Topographically the location of the Dam site is flat flood plain with no slope instability problem. But its morphology has problem of thick unconsolidated alluvial sediments of silty clay, sand with some boulders. Six geological test pits have been dug on the site to get subsurface and foundation information of dam axis. Accordingly, at the center of the stream a test pit was dig up to 3.0m and thick unconsolidated brown sand mixed with dark grayish silty clay alluvial sediments were encountered

Figure 4-2 a) Partial view of reservoir area b) the head work site towards the reservoir area

At the left bank Reddish to brown silty clay soil underlain by Gray silty sandy clay soil formation was observed. At the right bank the test pits could not penetrate more than 0.2m and under the top loose silty clay and sand the out crop weathered gneiss rock was encountered.

Figure 4-3 a) Test pit at centre of Dam axis b) Outcrop gneiss rock at the right bank

4.3 Seismicity of the project area

4.3.1 *General*

Seismic tremors occur at a rate of several hundred per day around the world. Small earthquakes occur and are recorded so frequently around tectonically active areas such as the Main Ethiopian Rift. There occur earth quacks of different magnitude in several places within the rift and vicinity. Ground fissuring occurs in various parts of the Rift such as the site near Lake Ziway which is recent phenomenon. The complicated mechanisms of this fissuring are basically co seismic in Afar and a seismic in the Rift valley lakes region. The intended structures shall be designed and constructed to withstand the design seismic action without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events the stiffness of the foundations shall be adequate for transmitting the actions received from the superstructure to the ground as uniformly as possible

The construction site and the nature of the supporting ground should normally be free from risks of ground rupture, slope instability and permanent settlements caused by liquefaction or densification in the event of an earthquake. The possibility of occurrence of such phenomena shall be investigated in accordance with EBCS EN 1998-5:2013.

The Project area is situated far away from the Main Ethiopian Rift. It lies in the range between less than 0.05 ground acceleration (see Fig. 8.1.: Kebede and Asfaw, 1996). Accordingly, there will be no adverse impact associated to rifting or seismic tremor for the intended structures to be imposed on at the site

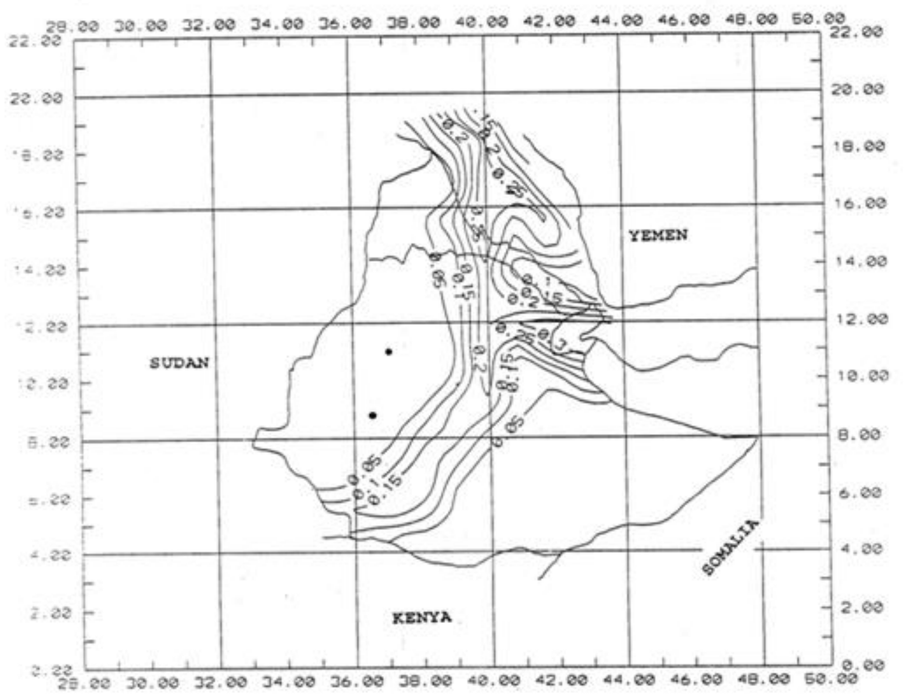

Figure 4-4 Seismic hazard zonation map of Ethiopia, (After Kebede and Asfaw,1996)

4.4 GEOTECHNICAL INVESTIGATION

4.4.1 *Preamble*

During the present study, detail site investigation works have been conducted at the proposed headwork site and main canal routes. Furthermore, assessment and investigations has been conducted to identify, evaluate and delineate potential sites of natural construction materials that will be used as source of natural construction material to the intended purpose.

Excavation of exploration holes / Pits have been conducted at representative sites. Data from valley and stream cut sections has been also used for identifying and description of prevailing geotechnical units characterizing the site considering the site geologic setting at the proposed headwork, proposed main canal routes and potential borrow sites of construction materials. Depth of excavation of exploration holes was commonly in the order of 1 to 3mts. When bed rock is encountered, excavation of holes has been imposed to terminate.

Testing of materials on selected representative soil and rock samples collected from the site has been accomplished at the central soil laboratory of Oromia Water Works Design and Supervision Enterprise (OWWDSE). This is with the objective of identification and determines important parameters to be used for the intended design.

Laboratory test results are presented in the annex and appendix of the site investigation report. Based on the site investigation data output, pertinent geological data and design parameters have been analyzed and interpreted for use to the intended purpose. Detail engineering geological longitudinal section of the proposed head work site and main canal routes have been prepared and attached in the present site investigation report for use to the intended purpose.

The geotechnical parameters for prevailing geologic units characterizing the project area have been determined from laboratory test results and by inferring / correlations from standard literatures and former works done on similar type of materials.

4.4.2 *Dam site*

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Detail engineering geological longitudinal section of the proposed Dam site had been prepared and attached in the Geological present site investigation report for use to the intended purpose.

The geotechnical parameters for prevailing geologic units characterizing the project area have been determined from laboratory test results and by inferring / correlations from standard literatures and former works done on similar type of materials

Accordingly, the site investigation indicates that, the head work site is covered with thick unconsolidated alluvial sediments of silty clay with sand and some boulders that lie on fresh to slightly weathered massive to jointed granitic gneiss bed rock unit. These alluvial sediments deposited over the weathered gneisses, meta sediment material at the center could be thicker than 5m. At the left bank Reddish to brown silty clay soil underlain by Gray silty sandy clay soil formation was observed. At the right bank the test pits could not penetrate more than 0.2m and under the top loose silty clayey sandy soil there encountered weathered gneiss rock that also outcrop around the stream bank.

These alluvial sediments deposited over the weathered gneisses, meta sediment material at the center is brown to dark grey silty clayey sandy soil material and the soil material at the left bank is reddish brown to brown sandy, silty clayey. From the field observation and description result indicated the soil profiles of the logs of the test pits at the indicated depths the center and left bank of the dam are characterized by sandy silty-clay soil materials underlain by fresh to slightly weathered massive to jointed granitic gneiss bed rock unit. Thickness of the material as inferred from excavated exploration holes / pits and valley cut sections is in the order of greater than 5mts. Based on the lab test result, the material indicates intermediate plasticity. Based on the correlation from standard literatures and previous work with similar soil nature, the angle of shearing resistance is found to be 180. Similarly, cohesion of the material is found to be 33kN/m2. Accordingly, a conservative value of angle of shearing resistance can be taken as 18° and effective cohesion (c’) of 33kN/m² can be adopted for outline design. The mean permeability of such a material indicates hydraulic conductivity which is in the order of 10-8 mm/s. The basement rock unit found at the headwork site and along the main cannel rote is dominantly GRANITE and GNEISS. Such bed rock material is commonly fresh to slightly weathered and massive. Accordingly, the Dam axis are characterized and presented as the figure below shown

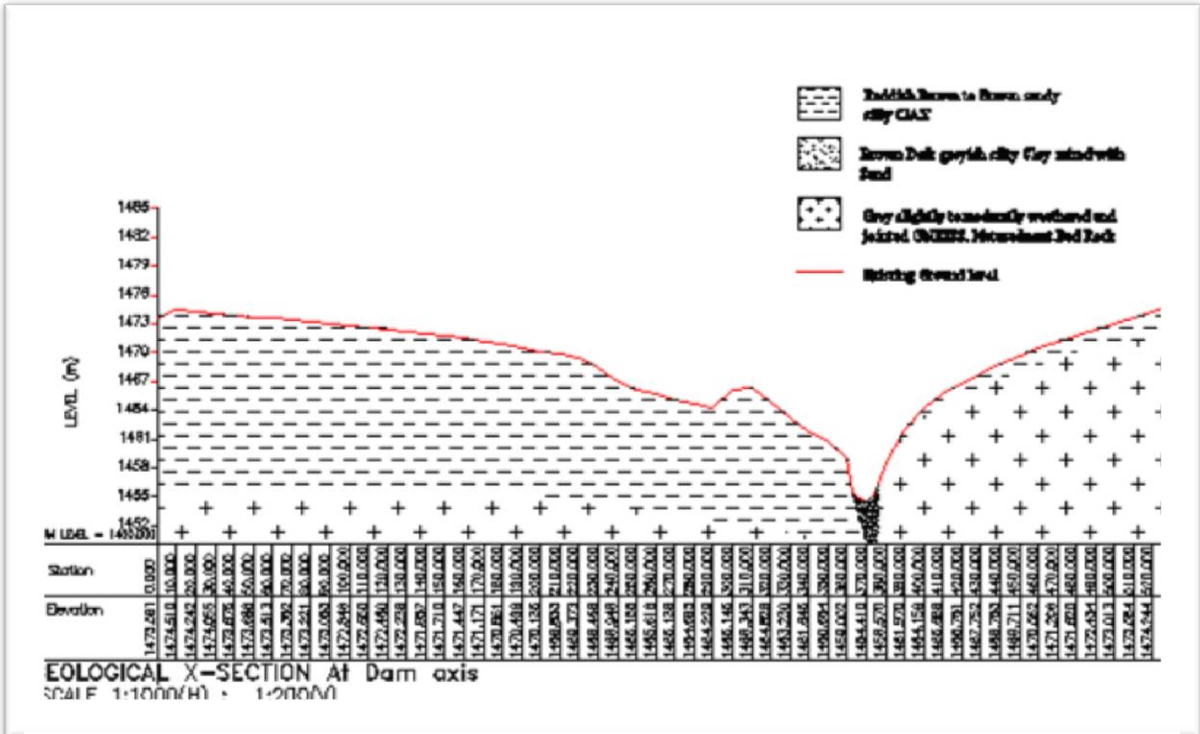


Figure 4-5 Dam site Property

4.4.3 *Spillway Site*

The foundation condition along the spill way was investigated the proposed spillway alignment is located on the right-side abatement of the proposed dam structure. The site is characterized by weathered massive to jointed granitic gneiss rock. Foundation depth is recommended to be in the underlying weathered bed rock. Such foundation material shall be properly consolidated prior to imposing the intended structure.

There is no active seismic tremor / earth quack at the site which could adversely affect sustainability of the intended structure. The old spillway which is found by the right side of the worbate stream nearby Dam axis is shown in figure below

Figure 4-6 Existing Spillway

4.5 Construction Materials Investigations

4.5.1 *Background*

An investigation has been conducted to identify, evaluate and delineate potential sites of natural construction materials in the project area. During the investigation, major emphases for selection of potential sites of construction materials have been given by considering among others

- ease of excavation and extraction
- hydrologic condition
- overburden thickness
- soundness / durability
- Quality and volume of potential reserves with respect to the intended purpose
- hauling distance
- Social and environmental factors that would hinder development of the quarry and borrow materials

Assessment for the availability of various construction materials suitable for the purpose of construction of the intended structures have been carried out in the project site and close vicinity. During the present investigation, emphases and prime consideration has been given to sites which are closer and locally available materials to the project site.

The identified Potential borrow sites was tested for the depth extent and quality upon excavation of exploration holes / pits and in situ assessments following standards (BS 5930 1999). Furthermore, representative samples have been collected for laboratory test and analysis in order to assess suitability of the material for use to the intended purpose.

The natural construction materials required for construction of the intended structures and embankment fill are Clay borrow; Sand borrow and rock / aggregate quarry sites

4.5.2 *Clay Borrow*

Clay borrow material is extracted for use as an embankment fill and if in case the Canal is open it can be used as Lining at pervious formations found characterizing the canal route. The latter is meant for use to increase water tightness. Accordingly, potential sites of Clay borrow area has been identified and delineated within the project site. Such selected clay borrow site is located centre at the geog. grid ref. 438339E and 554414N UTM.

Based on the lab test result, the hydrometer test indicates constituents as follows

Table 4-1 Borrow Area material Property

Borrow Area	Particle Percentage (%)		Liquid Limit (%)	Plastic Limit (%)	Plastic Index	Remark
Borrow Area 1	Clay	45	42.8	22.65	20.15	
	Silt	21				
	Sand	34				

According to the standard set by V.N.S Murthy the value obtained for the sample; the lab test result of the material indicates intermediate plasticity. The material is found to be moderate to low compressible. Based on the lab standard compaction test result conducted on the borrow sample, the material indicates Maximum Dry Density (MDD) 1720kg/m3, Optimum Moisture Content (OMC) being 16.7% and has a bulk density of 1.831gm/cm³ that can be considered for outline design purpose.

The proposed site is found close to the project command area. Consequently, access to the site is possible using the route to be constructed during construction of the main canal route and ancillary structures.

4.5.3 *Sand*

Sand borrow sites have been identified and delineated for use as cement mix to concrete and mason structures. Accordingly, potential sand borrows sites have been proposed for use to the intended purpose. It is found along the course and banks of prevailing Worbate River (See also table 6 below for specific site location).

Table 4-2 Location of Sand Borrow Sites

Site ID	Location (UTM)		Estimated reserve / volume (m^3)	Remark
	Easting	Northing		
ORB Sand-1	436429	552664	Excess (enough to the intended project)	Downstream of head work area, along River course and bank
ORB Sand-2	435924	551074	Excess (enough to the intended project)	Upstream of reservoir area Along River course

Figure 4-7 Worbate sand

Based on the lab gradation/sieve analysis test result, the material is found to consist SAND with trace silt and clay content. The gradation test result indicates as follows, the deposit is found in excess and estimated to be enough to the intended project

Table 4-3 Sand Particle size

Particle	Percentage of Particle size (%)	
	Sand Borrow 1	Sand Borrow 1
Clay	3	4
Silt	9	5
Sand	88	91

4.5.4 *Quarry site*

Quarry site has been identified and delineated for use as a source of Rock and aggregate for masonry works and concrete that also includes as use for canal lining. The selected quarry site is located centering at the geog. grid ref. 438397E and 552423N UTM.

Figure 4-8 Proposed stone quarry site

A soundness test has been conducted at the central soil and rock laboratory of the OWWDSE. The test done was water absorption test. Water absorption test is an excellent indicator of soundness of rock materials

Based on the lab test result, the material indicates water absorption value of 0.29%. As inferred from standards and literature, the minimum requirement of soundness for rock is 3%. The rock from the proposed quarry site records water absorption value less than 1%. According to the standard requirement criterion of soundness value of water absorption test, value below 1% is considered as an excellent sound rock that can be used for the intended purpose

4.5.5 *Water supply*

The water supply source that will be used for construction purpose is not available in the project site. However, the relatively closer water supply source recommended is from Haro Bake reservoir (pond) which is located at the geog. grid ref. 411843E and 552527N UTM near Bake village and can be used as a source of water supply to the intended construction purpose at any time throughout the year

Figure 4-9 Haro Bake water supply source

4.6 Geological conclusion and recommendations

4.6.1 *Conclusion*

The proposed Dam site is characterized by relatively thick unconsolidated soil overburden material at the centre and left bank of the worbate stream and an exposure of the slightly weathered massive to jointed granitic gneiss bed rock unit at the right bank of the stream.

These alluvial sediments deposited over the weathered gneisses, meta sediment material at the centre could be as thick as 3m that might need excavation to remove it however at left bank a homogeneous removal of 20 cm to 50cm strip top soil could need excavation to remove it.

The proposed main irrigation canal route is found being characterized dominantly by clayey silty sandy soil material. The clayey silty sandy soil material is of good hydraulic conductivity that could cause excess leakage.

An assessment and site investigation works have been conducted to identify potential sites of natural construction materials that can be used as source of borrow material, sand and quarry. Based on the investigation, potential sites of natural construction materials have been identified and delineated in the project area and close vicinity for use to the intended purpose.

4.6.2 *Recommendations*

- Foundation of the intended head work structure is recommended to be at depth in the order of 2mt to 3m removal of soil and replaced by borrow material at the centre of the stream.
- Proper compaction works on the soil materials, including the decomposed bed rock units characterising the canal route is recommended in order to increase density / stiffness and hence reduce hydraulic conductivity / achieve water tightness to avoid excess leakage.
- Proper follow up works is recommended during construction upon assigning competent geologist.

5 DAM TYPE SELECTION

5.1 Dam Type Selection

5.1.1 *General*

The geological and geotechnical investigations show that, The Dam site is covered with alluvial sedimentation. Topographically the Dam site is flat flood plain with no slope instability problem. But its morphology has problem of thick unconsolidated alluvial sediments of silty clay, sand with some boulders. From the Six geological test pits along the dam the subsurface and foundation condition of dam axis was examined. Accordingly, at the center of the stream a test pit was dig up to 3.0m and thick unconsolidated brown sand mixed with dark grayish silty clay alluvial sediments were encountered. The soil gradation of this pit is shown in the Figure 5-1, the respective grain size distribution is found to be 34, 21 and 45 for sand, silt and clay.

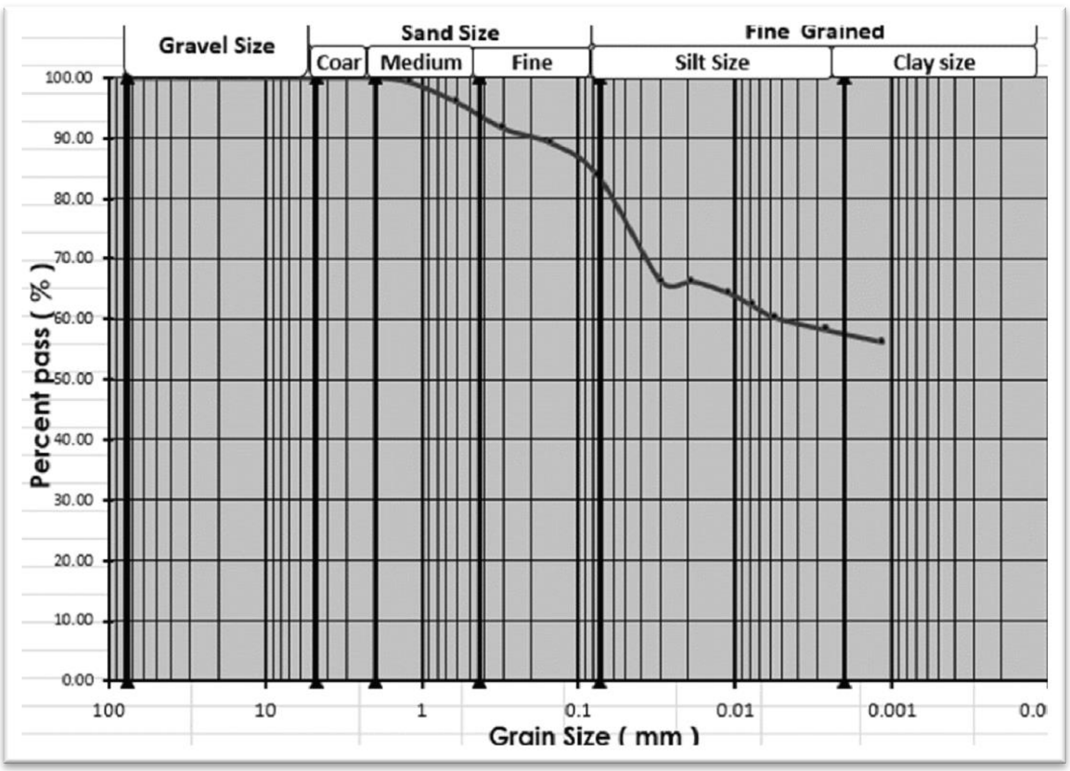


Figure 5-1Soil gradation at the center of the river (Dam)

These alluvial sediments deposited over the weathered gneisses, meta sediment material could be as thick as 6m. At the left bank Reddish to brown silty clay soil underlain by Gray silty sandy clay soil formation was observed. At the right bank the test pits could not penetrate more than 0.2m and under the top loose silty clay and sand the out crop weathered gneiss rock was encountered.

5.1.2 *Selected types of dams*

5.1.2.1 The homogeneous- type dam

This type of dam is recommended only where the lack of free-draining materials make the construction of a zoned embankment uneconomical, with the further qualification that for storage dams the homogeneous dam must be modified to include internal drain age facilities. This drainage facility is provided to perform its function of lowering the phreatic line and stabilizing the downstream portion of the dam. According to USBR this horizontal blanket is provided that the horizontal drainage blanket start at the downstream toe of the embankment and extend upstream to within a distance equal to the height of the dam plus 5 feet from the centreline of the dam. This will afford an ample blanket, yet keep the length of the path of percolation within desirable limits. The distance of height of dam plus 5 feet is selected on the basis that this will place the upstream limit of the horizontal drainage blanket at the downstream edge of minimum core that required for dams on deep pervious foundations without positive cutoff trenches.

6 DAM DESIGN

6.1 General

6.1.1 *Topographic Features*

The topographic maps produced for the present study shows that the river bed is as wide as 40m with average river bed level of around 1454masl, relatively steep hilly escarpments on the right side and flatter on the left abutments ends.

6.1.2 *Alignment*

The dam axis has been aligned nearly perpendicular to Worbate River considering the suitability of the abutments that ensure the stability of the dam. The final alignment of the dam is shown on Figure 6.1.

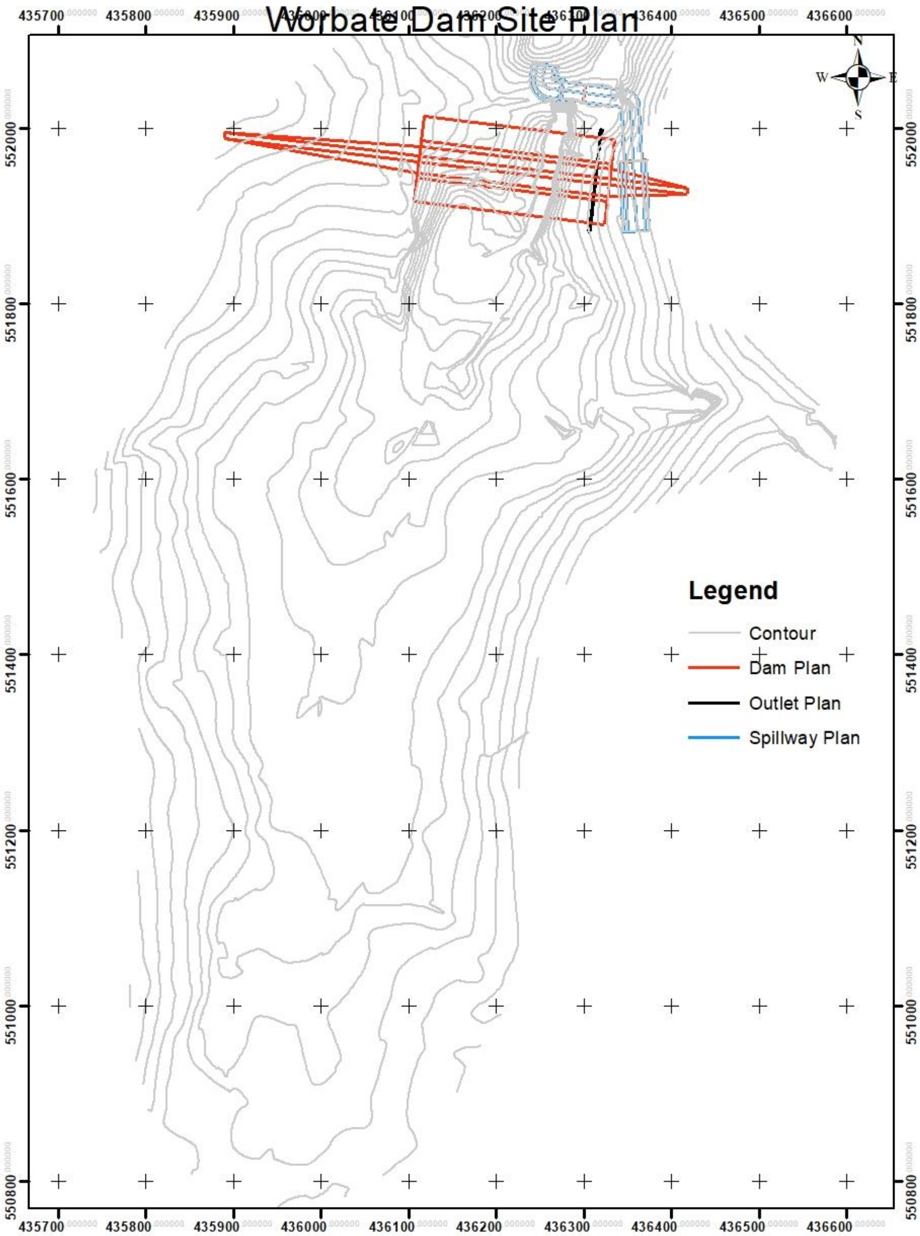


Figure 6-1 Dam layout

6.1.3 *Dam Axis and Spillway Geologic Features*

6.1.3.1 Dam axis

The left abutment is composed of top soil and weathered/ fractured basalt rocks. The topography is relatively steep and is slightly gentler compared to the right abutment.

The proposed headwork is characterized by relatively thick unconsolidated soil overburden material at the centre and left bank of the worbate stream and an exposure of the slightly weathered massive to jointed granitic gneiss bed rock unit at the right bank of the stream.

These alluvial sediments deposited over the weathered gneisses, meta sediment material at the centre could be as thick as 3m that might need excavation to remove it however at left bank a homogeneous removal of 20cm to 50cm strip top soil could need excavation to remove it.

6.1.3.2 Spillway alignment

The spillway is aligned on the right side by excavating the right abutment. The excess water passing over the spillway will be conveyed to the main river course with the necessary components of the chute spillway. Figure 6.1: Layout of the dam and appurtenant structures

6.1.4 *Available Construction Materials*

As described in Section 4.6, the available materials at Worbate project site include but not limited to the following:

6.1.4.1 Impervious material

Potential impervious material sources were identified and delineated within the project area at geographical locations of 0438339E, 0554414N. The proposed material is Silty clay, according to the laboratory result interpretation the material can be considered as slightly compressible. This is in favor of workability that may not cause problem in workability during construction of the intended core membrane and as an embankment fill. The physical property of the material is summarized as Table 4-1.

6.1.4.2 Rock materials

Potential source of Quarry site has been identified and delineated for use to the intended purpose. The identified site is located at a distance of around 2km East of the proposed dam site. It is located centering at the geog. grid ref. grid ref. 0438397E & 0552423N UTM.

The identified rock material is characterized by fresh to slightly weathered and jointed GNEISS. This type of material is among those best preferred types of sound rock material for use as source of construction material. Hence, it can be considered as a source which can produce hard, dense and durable to withstand destructive forces during placing, wave action, weathering, servicing, it is found exposed along prevailing stream course and banks.

Based on the lab test result, the material indicates water absorption value of 0.29%. As inferred from standards and literature, the minimum requirement of soundness for rock is 3%. The rock from the proposed quarry site records water absorption value less than 1%. According to the standard requirement criterion of soundness value of water absorption test, value below 1% is considered as an excellent sound rock that can be used for the intended purpose.

6.1.4.3 Sand for filter and other purposes

The source of sand for all purposes of the project is located at geographic grid ref. 0436429E, 552664N and 435924E, 551074 N at projection 37N UTM in NW direction of the dam site at a distance of around 1km.

The material is sampled and analyzed in the laboratory and found suitable for the intended purpose with a required quantity for the lab result refer the appendix of Geological investigation Report of the same project

6.1.4.4 Water

The water supply source that will be used for construction purpose is not available in the project site. However, the relatively closer water supply source recommended is from Haro Bake reservoir (pond) which is located at the geog. grid ref. 411843E and 552527N UTM near Bake village and can be used as a source of water supply to the intended construction purpose at any time throughout the year.

6.2 Foundation Condition and Preparation

The proposed Dam site is characterized by relatively thick unconsolidated soil overburden material at the center and left bank of the worbate stream and an exposure of the slightly weathered massive to jointed granitic gneiss bed rock unit at the right bank of the stream.

These alluvial sediments deposited over the weathered gneisses, meta sediment material at the centre could be as thick as 3m that might need excavation to remove it however at left bank a homogeneous removal of 20cm to 50cm strip top soil could need excavation to remove it. Moreover, this implies that the foundation material is suitable for the selected homogeneous embankment fill dam for detail refer Geotechnical report.

6.3 Seepage Control Measures through the Dam Foundation

In order to minimize seepage through the foundation, provision of seepage curtailing measures are necessary. But from the Geotechnical evaluation of the dam foundation point of view, the foundation material is overburden silty clay for over 3m, this indicate that the foundation is impervious according to USRB manual. Therefore, there is no need of controlling/ reducing the seepage but based on the computed rate the necessary seepage curtailing measures in the foundation will be considered.

6.4 Dam Design Sectional Elements

6.4.1 *General*

This section describes the dam design procedures undertaken for Worbate dam. The basic consideration in the design of Worbate Dam has been to achieve safety consistent with economy. As indicated in Chapter 5, homogenous embankment dam has been considered for this study.

A Chute spill way with appropriate approach canal, control section and energy dissipation is designed in order to protect the danger of overtopping. This spillway with chute on the right bank has been selected for this dam site. The following criteria have been considered in the Dam design:

- Under all conditions of construction, reservoir operation, and seismic activity, the embankment, foundation, and abutments must remain stable with recommended factor of safety.
- Seepage through the embankment, foundation, and abutments must be properly controlled and collected to prevent excessive uplift pressures, piping, sloughing and removal of material by solution or erosion of material by loss into cracks, joints, and cavities.

6.4.2 *Dead Storage Allocation*

According to the analysis made on the sediment load and recommendation given the annual sediment load estimated becomes 1.08Mm3 for 50 years design period. For detail refer the report of Climate and Hydrological Study.

The incoming sediment is not totally deposited in the reservoir. Some amount of suspended sediment can be passed over the spillway. Therefore, to allocate the dead storage volume from the total volume of the reservoir, estimation of the trapped sediment by the reservoir is necessary. But by taking this sediment load will be the amount to be accumulated in the reservoir will make the life period longer. Therefore, from Figure 6-2 the dead storage level found to be 1467.5m

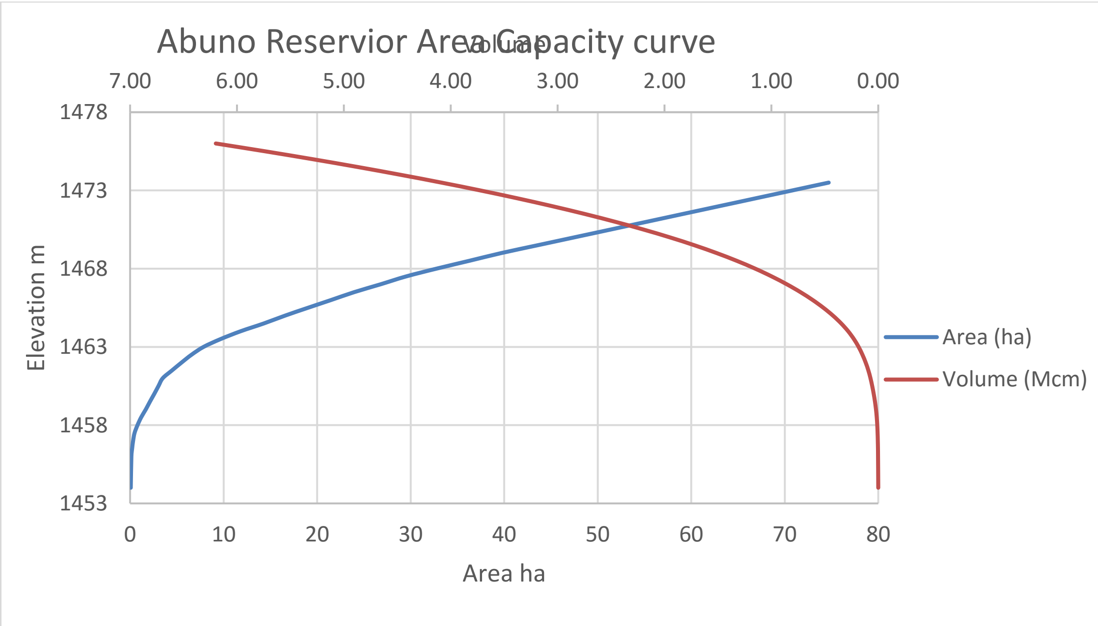


Figure 6-2 Reservoir capacity curve

6.4.3 *Water demand Analysis*

6.4.3.1 Irrigation Demand Analysis

From the Agronomy Design report the monthly Demand is collected for 2 phases with project efficiency of 50% as shown table bellow

	Parameter	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Project summary	Project efficiency	50%	50%	50%	50%	50%	50%	50%	50%	50%	50%	50%	50%
	Proposed irrigation hour	12	12	12	12	12	12	12	12	12	12	12	12
	Days in month	31	28	31	30	31	30	31	31	30	31	30	31
First Phase	Irrigated area	100	100	80	20	0	0	0	0	0	0	0	100
	Project supply (l/s/ha for 12 hours	2.16	2.56	1.24	0	0	0	0	0	0	0	0	0.88
Second Phase	Irrigated area	0	0	0	0	25	100	100	80	20	0	0	0
	Project supply (l/s/ha for 12 hour	0	0	0	0	0	1.2	1.4	0.72	0.28	0	0	0
	Project Demand (cm) per Hectare	2892.67	3096.58	1328.49	0.00	0.00	1555.20	1874.88	771.38	72.58	0.00	0.00	1178.50
	For 120 ha (Mcm)	0.35	0.37	0.16	0.00	0.00	0.19	0.22	0.09	0.01	0.00	0.00	0.14
	Cumulative Demand	0.35	0.72	0.88	0.88	0.88	1.06	1.29	1.38	1.39	1.39	1.39	1.53

6.4.3.2 Water demand for Livestock

Water is an essential nutrient for all animals. It is important for both animal welfare and business profitability that livestock have an adequate supply of good quality water. Amount and quality of water required vary between species of livestock, between classes of stock within the species, and in response to the environment in which the stock is running.

The estimated water demand for livestock considering the number and types of livestock's available surrounding the reservoir shall be estimated per each month using the table below, the number of livestock is collected from Socioeconomic Study and the demand from Dam Guideline and computed as fallows

Table 6-1 Daily Livestock Demand (litter/ m3)

No	Type of Livestock	Number Found		Demand l/day/head	Demand m3/day/head	Demand m3
		Surpha District	Abuna Local administration			
1	Cattle		12128	27.3	0.0273	331.0944
2	Goats		13516	5	0.005	67.58
3	Sheep		7279	5	0.005	36.395
4	Horses		7	28	0.028	0.196
5	Mules		51	28	0.028	1.428
6	Donkeys		496	28	0.028	13.888
7	Camels		3491	49.6	0.0496	173.1536
8	Poultry		1333	0.32	0.00032	0.42656
				Demand m3/day=		624.16156

Table 6-2 Monthly Livestock Demand (Mcm)

Item	Month											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Days in Month	31	28	31	30	31	30	31	31	30	31	30	31
Livestock Demand Mcm	0.019	0.017	0.019	0.019	0.019	0.019	0.019	0.019	0.019	0.019	0.019	0.019

6.4.3.3 Loses in the dams

Evaporation loss

For a lack of a complete set of meteorological data in the given location, it would be very difficult to estimate the actual evapotranspiration losses in an irrigation scheme. In such cases the evaporation loss from the reservoir water can be estimated using U.S geological survey method as shown below.

$E = (4.57*T+43.3)/1200$ (U.S geological survey method)

Where E = Monthly evaporation loss in m

T = Mean annual temp. (^{0}c)

Table 6-3 estimation of evaporation loss

Description	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Min Temp	12.7	13.8	15.3	15.9	15.4	14.4	13.9	14	14.6	15.4	14.3	12.7
Max Temp	28.4	29.3	28.6	24.4	24.9	24.4	23.9	24.7	26.1	25.4	25.5	26.4
Mean Temp	20.55	21.55	21.95	20.15	20.15	19.4	18.9	19.35	20.35	20.4	19.9	19.55
Evaporation (m/month)	0.114	0.118	0.120	0.113	0.113	0.110	0.108	0.110	0.114	0.114	0.112	0.111
Evaporation (Mcm)	0.06	0.06	0.06	0.06	0.06	0.05	0.05	0.05	0.06	0.06	0.06	0.06

Table 6-4 Demand Summary

	Month											
Demand	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Irrigation (120 ha)	0.35	0.37	0.16	0.00	0.00	0.19	0.22	0.09	0.01	0.00	0.00	0.14
Livestock	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02
Evaporation	0.06	0.06	0.06	0.06	0.06	0.05	0.05	0.05	0.06	0.06	0.06	0.06
Total Demand	0.42	0.45	0.24	0.08	0.08	0.26	0.30	0.17	0.08	0.08	0.07	0.22
Cumulative Demand (Mcm)	0.42	0.87	1.11	1.19	1.26	1.52	1.82	1.99	2.07	2.15	2.22	2.44

Table 6-5 Supply Summary in m^3/s

Dependability	Months Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
50%	0.13	0.11	0.79	1.57	0.84	0.09	0.00	0.18	0.29	0.85	0.60	0.14
70%	0.00	0.00	0.53	1.10	0.44	0.00	0.00	0.00	0.00	0.57	0.25	0.00
75%	0.00	0.00	0.33	0.98	0.40	0.00	0.00	0.00	0.00	0.45	0.06	0.00
80%	0.00	0.00	0.28	0.82	0.38	0.00	0.00	0.00	0.00	0.37	0.00	0.00
90%	0.00	0.00	0.00	0.06	0.00	0.00	0.00	0.00	0.00	0.25	0.00	0.00
95%	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00

Description	Month Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Days in month	31	28	31	30	31	30	31	31	30	31	30	31
80 % dependable flow (m3/s)	0.000	0.000	0.277	0.822	0.382	0.000	0.000	0.000	0.000	0.372	0.000	0.000
downstream release (20 %)	0.000	0.000	0.055	0.164	0.076	0.000	0.000	0.000	0.000	0.074	0.000	0.000
Available supply (Mcm)	0.00	0.00	0.59	1.70	0.82	0.00	0.00	0.00	0.00	0.80	0.00	0.00
Cumulative Supply (Mcm)	0.00	0.00	0.59	2.30	3.12	3.12	3.12	3.12	3.12	3.92	3.92	3.92

Table 6-6Reservoir Requirement computation

Description	Month Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Days in month	31	28	31	30	31	30	31	31	30	31	30	31
80 % dependable flow (m3/s)	0.00	0.00	0.28	0.82	0.38	0.00	0.00	0.00	0.00	0.37	0.00	0.00
downstream release (20 %)	0.00	0.00	0.06	0.16	0.08	0.00	0.00	0.00	0.00	0.07	0.00	0.00
Assumed Reservoir Capacity=			1.3	Initial storage S_i=				0.9				
Available supply (Mcm)	0.00	0.00	0.59	1.70	0.82	0.00	0.00	0.00	0.00	0.80	0.00	0.00
Total Demand (Mcm)	0.42	0.45	0.24	0.08	0.08	0.26	0.30	0.17	0.08	0.08	0.07	0.22
Reservoir Water Balance	0.48	0.03	0.38	1.30	1.30	1.04	0.74	0.57	0.49	1.21	1.14	0.92

Determination of Required storage Capacity

Reservoir operation models could be very complex. One source of this complexity is attributed to randomness or vagueness involved in specific components of water resources systems. Different kinds of methods have been used for resolving this complexity in optimizing the operation of reservoirs. Among the methods available the followings are can be adopted

1. Mass Curve Analysis
   a. Determination of reservoir capacity for a known yield
   b. Determination of yield from a known reservoir capacity
2. Sequent Peak Analysis
3. Operation Study
4. Other Approaches in Capacity Determination

For the detail and reliable analysis reliable data will be required with respect to the available water resource and demand as well as losses for a long enough time series

Therefore, the objective is to maximize the benefit of operating a reservoir over a relatively long-time horizon. The unknown variables are the reservoir volumes in different months of the year based on different states of inflows and demands. In case of this project, the supply is considered for 80 % dependability it becomes more reliable at this probability, but the demand and losses will be considered to be fixed in one year according to the computed monthly value, any change in the demand and losses result to unreliability. Keeping this limitation, the required reservoir capacity was determined according to operational study method, using the following continuity equation.

$$S_i + I_{i+1} - \text{Release}_{i+1} = S_{i+1}$$
$$i = 1, n$$
$$S_i <= \text{Maxcap}$$
$$S_i >= \text{Mincap}$$

Where:

I_i = Inflow to the reservoir in month i S_i = Volume of the reservoir in month i n = number of iterations
Maxcap = Maximum storage of the reservoir Mincap = Minimum storage of the reservoir

By trial and Error using different trial value for Si and the reservoir capacity was evaluated, with criteria of having a minimum volume of water to be available in the reservoir. finally, with this value of 0.9Mcm and 1.3 Mcm reservoir capacity will make the size satisfactory, with 0.03Mcm water balance in the reservoir as shown in Table 6-6 at the month of February. Therefore, the required live storage will be 1.3 Mcm.

For the computed live storage and dead storage. The normal pool level volume will be 2.3Mcm and this volume can be attained at level of 1470.5 from Figure 6-2 and the dam was fixed with respect to this value.

Dam height (H)

Dam height (H) = Normal pool level (NPL) above river bed + water head over the spillway crest + free board + settlement allowance

Normal pool level (NPL) above riverbed determined by

Height of normal pool above the lowest riverbed = NPL- Lowest riverbed level
1470.5-1454=16.5m

Water head over the spill way crest

As shown in the flood routing the maximum water head over the spillway crest corresponding to 100 years design flood and 20 m spill way width is found to be 1.9 m.

Free board

Free board is the margin above the estimated reservoir level at high flood, to be provided against wave splash, spray, seiches and wind set up etc. and to give margin of safety over and above these factors. And the free board is calculated as shown below.

$Fb = h_w + R + S$ The Molitor Stevenson formula

$h_w = \left(0.032\sqrt{VF}\right) + 0.76 - 0.27\sqrt[4]{F}$ for F<32km
$h_w = \left(0.032\sqrt{VF}\right)$ for F>32km

Where
V = wind velocity (km/hr) found to be 7.54 km/ hr (From Adola Station)
F = Fetch length at maximum reservoir level (km) from topography and pool level = 1.28km
R = wave Run up = 50 % of hw
$S = \frac{v^2 * F * cos\emptyset^2}{6200 * D}$

Where:
Fb = Free board (m) above spillway crest level
V = wind velocity (km/hr) found to be 7.54 km/hr (From Adola Station)
F = Fetch length at maximum reservoir level (km) from topography and pool level = 1.28km
R = wave Run up = 50 % of hw
S= wind set up
ϕ = Angle between the wind direction and the fetch =0^0
D =Average Reservoir depth in meters over the fetch distance and found to be 9.82m

Accordingly

$h_w = \left(0.032\sqrt{7.54} * 1.28\right) + 0.76 - 0.27\sqrt[4]{1.28}$ =0.572m

$$S = \frac{7.54^2 * 1.28 * cos0^2}{6200 * 9.82} = 0.001m$$

R=0.5*0.572= 0.286m.

FB=0.572+.001+0.286=0.86m

Dam height excluding settlement allowance will be

$H_{dam} = H + D_s + Fb$

Where

H_{dam} = Height of Dam excluding settlement

H = NPL-RBL= 1470.5-1454= 16.5 m

D_s= maxWL-Spillway Crest level= water depth over spillway= 1.9 m.

H_{dam} = 19.26 m

Settlement Equations for Normally Consolidated Clays

Embankment compression (st_1)

Embankment compression According to USBR Earth and Rock fill dam's manual is estimated using the following formula

$St_1 = 0.001 * (H_{dam})^{3/2}$

$St_1 = 0.001 * (19.26)^{3/2}$ =0.084m

The Total Dam Height will be 19.26+ 0.084=19.34 adopt 19.5m

Crest Width

The crest width is often governed by the working space required at top of dam for transport during construction of the dam and traffic across the dam after construction completion and requirement of future needs like public highway.

a) The width of dam at crest as per IS 8826 - 1978 "Guide lines for design of large earth and rock fill dams" should be fixed according to the working space required at top and the crest width should not be less than 6.0 m.
b) According to USBR 'Design of Small Dams' the crest width to be adopted in case of small earth fill dams is given by

$$W = \left\{ \frac{H}{5} + 10 \right\} \text{ in ft}$$

Where,

W = Width of crest in ft

H = Height of dam in feet above river bed,

In case of Worbate Dam

Dam height = 19.5 m=63.97 ft

W = 22.79ft= 6.94 m

US Army Corps of Engineers Manual on "General Design and Construction consideration for Earth and Rockfill Dams" EM 1110-2-2300 dated 30 July, 2004 provides that minimum top width of an earth and rock fill dam, depending upon the height of dam, should be between 25 ft (7.62 m) and 40 ft (12.19 m).

Crest width is commonly represented by the formula:

$$Bt = \left\{\frac{5}{3}\sqrt{H}\right\}$$

Where,

Bt = crest width in m

H = height of dam in m

Bt =7.35m

c) Robert and Jensen in their book has mentioned of providing crest width of 25.0 ft (7.6 m) for allowing for general weathering, traffic wear, cracking and damages in extreme events.
d) The Japanese code of 1957 specifies crest width as

$$B= 3.6 \times H^{1/3} - 3$$

B=6.69 m

Larger crest width is provided in seismic areas to resist the larger acceleration near the top. Keeping in view the above considerations, the top width of crest has been adopted as **7.5** m. The dam crest details are shown in drawing

6.4.4 *Berms*

Berms shall be provided for serving the following purposes:

- For providing level surface for construction and maintenance of the dam section.
- For reducing the surface erosion in case of downstream slope and breaking the continuity of the slope;
- To protect the lower edge of the riprap and from preventing it from undermining in case of the upstream slope.

Berms of 6m width have been provided at both upstream and downstream faces of the dam different elevations. The berm will be inclined at 2% slope towards the slope face at the level below

- Upstream slope (Berm elevation in m): Berm 1=1467.00m
- Downstream slope (Berm elevation in m): Berm 1=1468.00m,

6.5 Foundation of the dam site

According to the geotechnical report the foundation material is characterized by as follows

- The subsurface geophysical response obtained from Worbate dam site indicates that Relatively the abutments are strong and stable.
- Based on the studies it is concluded that the area is situated in seismically less active zone less than 0.05 seismic coefficient.

The bed rock unit is found being exposed mainly along prevailing right-side abutment bank. Such bed rock unit is fresh to slightly weathered and jointed GNEISS. This bed rock can be described fresh to slightly weathered jointed to massive GNEISS

Even though no strength test was done on such rock unit, based on field in - situ strength testing methods such as hammering, the material is found to be Very STRONG. Correlation with standard literatures on similar type of material, the Unconfined Compressive Strength (UCS) of the intact rock unit is estimated to be in the order of 150 to 200Mpa. A conservative value of UCS for the intact rock unit can be taken as 150Mpa and such a value may be adopted for an outline design purpose. However, the rock mass strength will be far less than the intact rock strength due to effect of fracturing and jointing. To this, considering degree of jointing and fracturing effect the rock mass strength of the rock unit characterizing the site can be estimated to be in the order of 75 to 100Mpa. A conservative value of the rock mass strength 75kN/m2 may be considered for an outline design purpose.

The rock unit is massive to well joint. The joints however are tight joints that are not deeply penetrative. Considering such an effect and based on correlation on standard literatures, hydraulic conductivity value for such type of material is estimated to be in the range of <10-6 cm/s. A hydraulic conductivity value of 10-6cm/s can be adopted for an outline design

Accordingly, for there is a limitation on the availability of pervious shell and gravel material, which makes the transportation relatively makes un economical, as a result, the homogeneous- type dam is recommended moreover Blanket drain was recommended

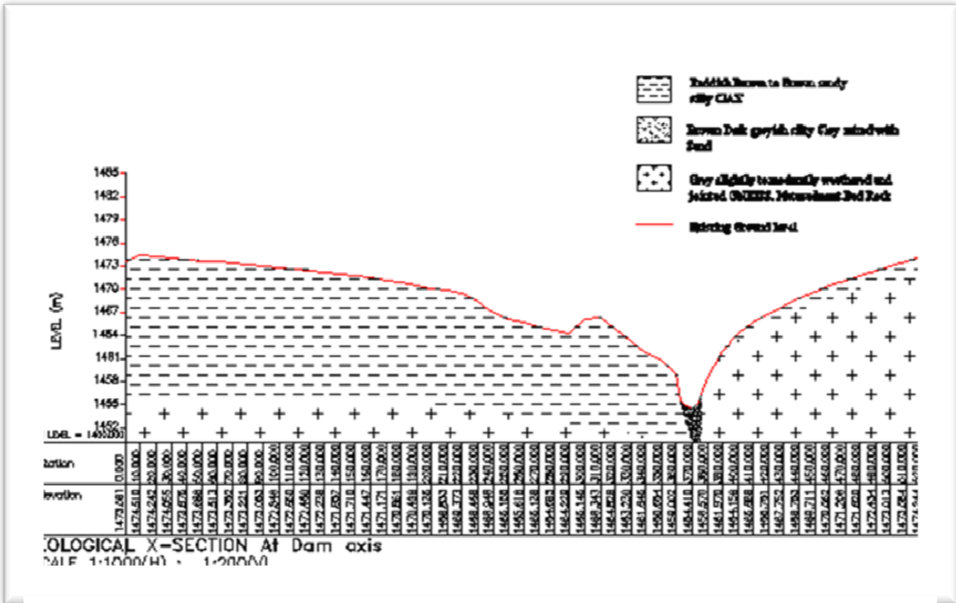


Figure 6-3 **Dam axis Geologic profile**

6.6 Embankment Slope Design

6.6.1 *General*

The design slopes of an embankment may vary significantly depending on the character of the materials available for construction, foundation conditions, and the height of the structure.
The embankment slopes, as determined in this section, are the slopes required for stability of the embankment on a stable foundation. For stability against seepage forces, pervious foundations may require the addition of upstream blankets to reduce the amount

6.6.2 *Upstream Slope*

The upstream slope may vary from 2:1 to as flat as 4:1 for stability; usually it is 2.5:1 or 3:1. Flat upstream slopes are sometimes used to eliminate expensive slope protection. A berm is often provided at an elevation slightly below the maximum drawdown of the reservoir water surface to form a base for the upstream slope protection, as a result it is provided at a level of 1467m.

The upstream slope is often steepened above the elevation where water is stored; that is, in the surcharge range. For this particular project the available borrow material is impervious material is silty clay with scarce pervious material as a result relatively a flatter upstream slope of 2:1 is recommended and the stability is checked for a storage dam subject to rapid drawdown of the reservoir should have an upstream zone with permeability sufficient to dissipate pore water pressures exerted outwardly in the upstream part of the dam. The detail analysis is presented section 7

6.6.3 *Downstream Slope*

The usual downstream slopes for small earth fill dams are 2:1 but depending the embankment material the slope will be varied. But for this particular project using the available material and staring slope of 2:1 is adopted then the stability is checked for the recommended factor of safety in all loading condition for the detail analysis refer section 7.

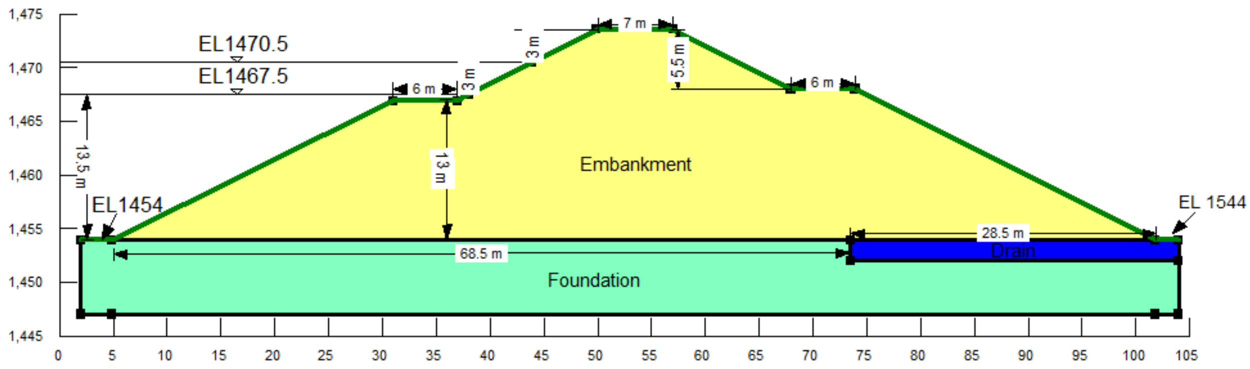


Figure 6-4 Recommended Dam section at the centre

6.6.4 *Design of Vertical (or Inclined) and Horizontal Drains*

The vertical and horizontal drains are required to control seepage through the embankment by preventing material eroded through a crack in the core impervious part from washing into the downstream shell by seepage water under reservoir head. Moreover because of the often-variable characteristics of borrow materials, it is frequently advisable to provide vertical and horizontal drains within the downstream section of the embankment, in order to ensure satisfactory seepage control.
Filter Requirements

Vertical and horizontal drains should be designed as filters. If crushed rock is used for the drain material and the material to be protected is dispersive, or material to be protected contains cracks, filter tests will be required. Well-graded materials are internally unstable and should not be used as filters when Cu > 20.

Discharge Requirements.

The drains must have sufficient discharge capacities to remove seepage quickly without inducing high seepage forces or hydrostatic pressures (Cedergren 1977). When drains are designed and constructed with ample discharge capacity, the line of seepage will not rise above the drain zone. Since drains are small compared to the overall dimensions of the earth dam, it is difficult to construct accurate flow nets within the drains themselves. The total quantity of seepage from all sources that must discharge through the drain should be evaluated from a flow net analysis in which it is assumed that the drains have an infinite permeability.

Accordingly, the seepage rate is computed using Geostudio 2012 for the assumed designed assumption and cross section of the dam at the critical stage when the reservoir is full with steady flow analysis. The embankment and foundation material permeability are considered with maximum permeability

Table 6-7 The parameter adopted for analysis foundation and Embankment material

Description	Soil Texture (%)				Strength								Remark
	Gravel	Sand	Silt	Clay	C (KN/m^2)	Ø (Deg)	S_u (KN/m^2)	$Ø_u$ (Deg)	K_{sat} (mm/hr)	θ_{sat}	Y (KN/m^3)	θ	
Foundation		17	25	58	33	18	57.11	10	1.02	0.52	16.24	26.78	
Embankment		34	21	45	25	23	67.6	13	1.24	0.47	16.36	15.65	

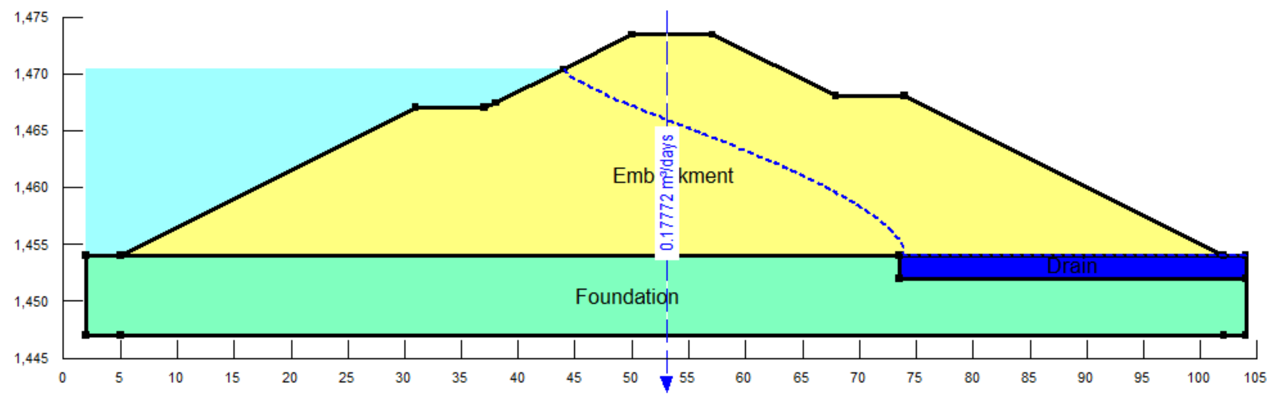


Figure 6-5Seepage amount and drain alignment

The result of evaluation indicates that the seepage before joining the drain is found to be $2.0569*10^{-6}$ m^3/s per unit length of the dam. Therefore, the horizontal drain has to be capable of discharging this amount of water. And also

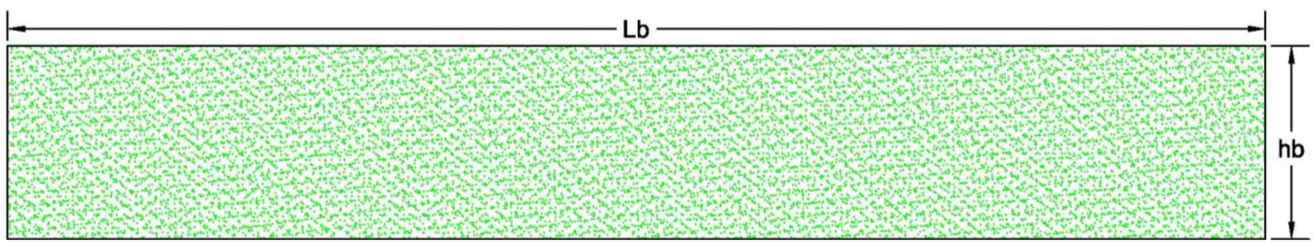


Figure 6-6 Dimensioning of Horizontal Drain

Since the drain is to be designed so that the line of seepage does not rise above the drain zone, the allowable maximum head in the horizontal drain can be no greater than its height. The required minimum permeability from Darcy's law is calculated

$$K_b = \frac{Q_d}{\frac{h_b}{L_b} A_b}$$

Where

L_b Drain length = 28.5 m
h_b Drain Depth to be determined
$A_b = L_b x h_b$
Q_d seepage amount = $2.0569*10^{-6}$ m^3/s

$$K_b = \frac{0.1777}{h_b * h_b} \text{ m/day}$$

To design the horizontal drain, select a drain height and calculate the required minimum permeability. Apply a factor of safety of 20 to the calculated permeability and select a drain material from available aggregates. Select a drain height of 1m.

The required minimum permeability becomes K_b= 0.1777 m/ day
The designed $K_{b\ design}$ will be 20* 0.1777=43.54 m/day= 0.504 mm/s

This permeability could be obtained by screened fine gravel (3/8-in. to 1/2-in. size) which has a permeability of about 30,000 ft/day or 10.6 cm/sec, (Cedergren 1977). Seepage in coarse aggregate is likely to be turbulent and a reduction factor should be applied to the permeability. The hydraulic gradient in the horizontal drain is L_b

$$i = \frac{h_b}{L_b}$$

$$i = \frac{1}{28.5} = 0.035$$

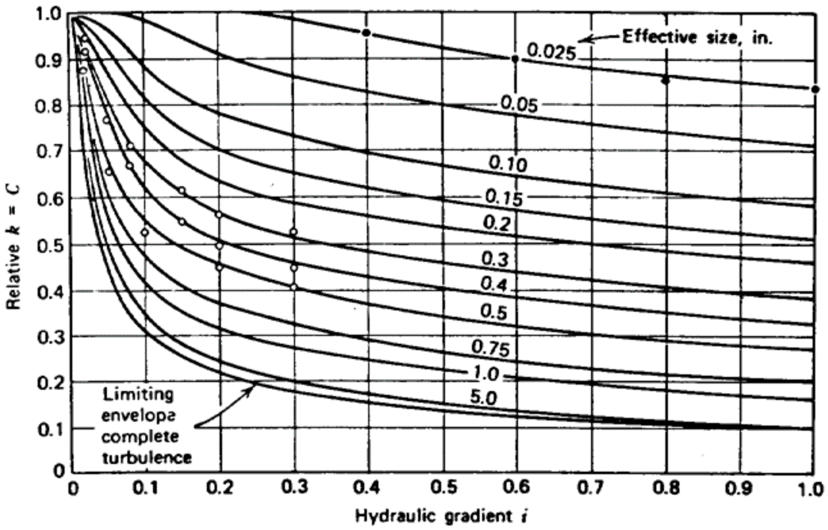


Figure 6-7Figure 8-10. Approximation for estimating reduction in permeability

The above figure shows the Approximation for estimating reduction in permeability of narrow size-range aggregate caused by turbulent flow (courtesy of John Wiley and Sons155)

From figure above, for screened fine gravel (3/8-in. to 1/2-in. size) with i = 0.035 the reduction factor for permeability is 0.65.

K = 0.65 * 10.6 cm/s= 6.9 cm/s

The permeability of the screened fine gravel (3/8-in. to 1/2-in. size) reduced for turbulence is greater than the required permeability for design:

6.9 cm/s>0.06731 cm/s

Therefore, it should be adequate when properly placed and compacted to conduct seepage water through the horizontal drain. The screened fine gravel (3/8-in. to 1/2-in. size) will be protected top and bottom with a 1-ft-thick clean washed concrete sand filter. Since the seepage from the foundation must flow across the fine filter to enter the coarse drainage layer, the fine filter must be permeable enough to allow the water to enter the coarse drainage layer freely under only a small hydraulic gradient (0.5 or less).

Clean washed concrete sand with a permeability of 10 ft/day should allow seepage from the foundation to enter the coarse drainage layer without restriction. As a result, the designed horizontal drains would have to meet the filter requirements provided under.

Design of Filter

Blanket drains were included in embankment designs both to collect seepage from the foundation and to provide an outlet for seepage collected by a chimney filter/drainage zone. Since a blanket is at the interface between the embankment and foundation, it could be classified as either an embankment or foundation element but in this project case it is to be called foundation element for the foundation material gradient is finer than the embankment. As it is observed in the figure below

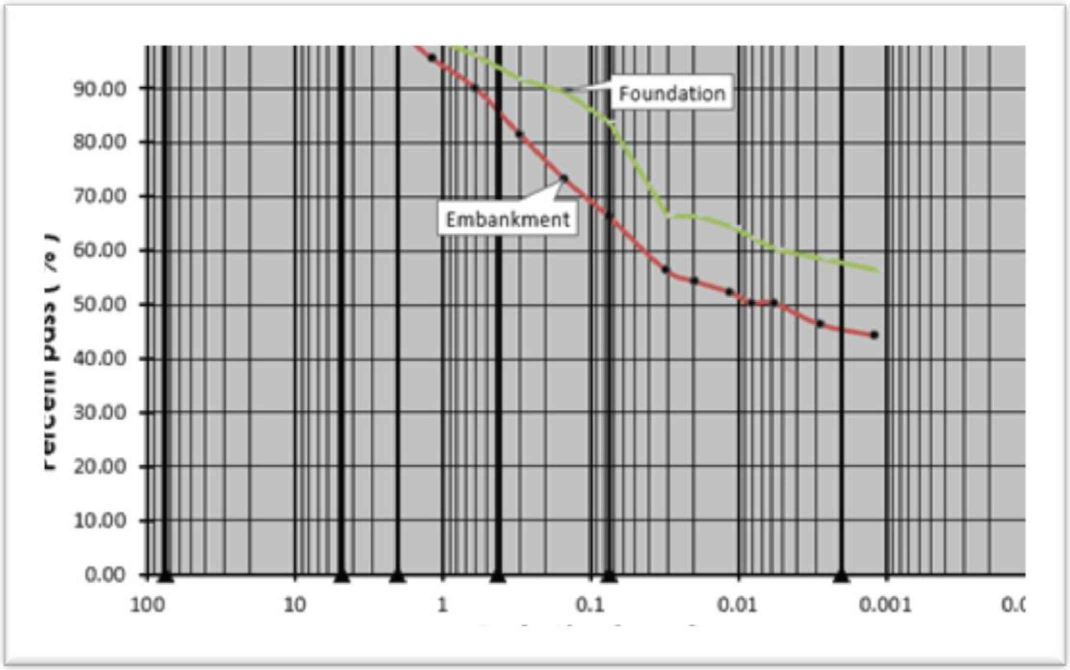


Figure 6-8 Gradation of foundation and embankment material

Therefore, for this project the base material is that of foundation, As it is observed the gradation curve the Base material have no any particles larger than the No. 4 sieve, as a result computationally re-graded is not necessary and also the according to USACE, Reclamation, and NRCS guidance the base material is categorized under Base material category 2 for which The maximum $D_{15}F$ should be $\leq$ 0.7 mm unless soil is dispersive, in which case the maximum $D_{15}F$ should be < 0.5 mm., accordingly the base soil is assumed to be not dispersive and $D_{15}F$ was taken to be 0.7mm

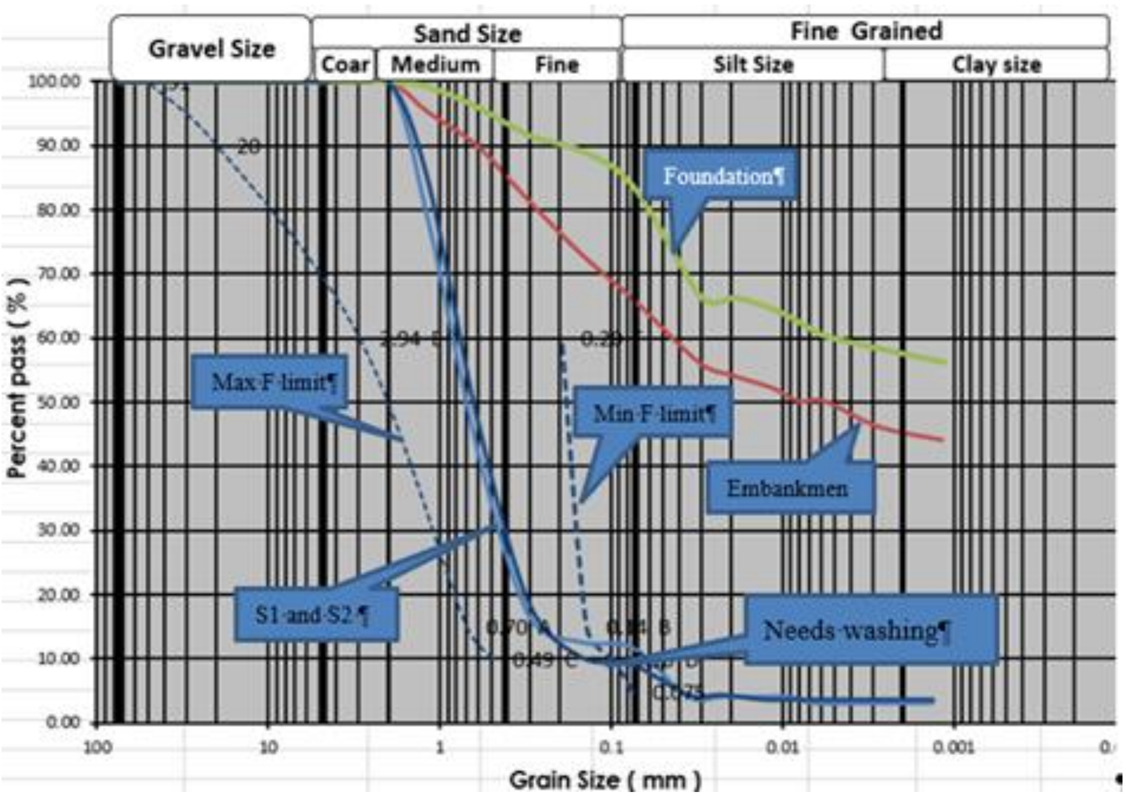
To satisfy permeability requirements, determine the minimum allowable $D_{15}F$:

- Minimum $D_{15}F \geq 5 \times$ maximum $D_{15}B$ (Reclamation)
- Minimum $D_{15}F \geq 3$ to $5 \times$ maximum $D_{15}B$ (USACE)
- Minimum $D_{15}F \geq 4$ to $5 \times$ maximum $D_{15}B$ (NRCS)
- Minimum $D_{15}F$ is computed prior to any re-grading, if any, and should not be smaller than 0.1 mm.

Since the base material have no any $D_{15}B$ as a result the minimum D15 F should be 1/5 of Max D15F but not less than 0.1mm, therefore minimum D15F will be 0.14 which is greater than 0.1mm. To limit the amount of fines and oversized material, the minimum D_5F and maximum $D_{100}F$ will be determined according to the following table:

Base Soil Category	Maximum $D_{100}F$	Minimum D_5F
ALL categories	≤ 2 in. (51 mm)	0.075 mm (No. 200 sieve)

Therefore, $D_{100}F$ and D_5F will be 51 mm and .075 mm respectively
To limit segregation potential, the maximum $D_{90}F$ is determined from the table: for a Minimum DF of 0,0.5 it is found to be $D_{90}F$=20mm.



As it is observed in the above figure the available sand fulfills the filter design with some limitation, it needs to wash the sand to remove the finer particle size 0.075mm.

7 Dam Slope Stability and Deformation Analysis

7.1 General

Variations of the loads acting on slopes, and variations of shear strengths with time, result in changes in the factors of safety of slopes. As a result, it is necessary to perform stability analyses corresponding to several different conditions reflecting different stages in the life of a slope. As conditions change, the factor of safety against slope instability may increase or decrease.

7.2 Loading Conditions

An embankment and its foundation are subject to shear stresses imposed by the weight of the embankment and by pool fluctuations, seepage, or earthquake forces.
Loading conditions vary from the commencement of construction of the embankment until the time when the embankment has been completed and has a full reservoir pool behind it. The range of loading conditions encompasses the following conditions at various stages from construction through the operational stage of the completed embankment:

- End-of-Construction
- Sudden drawdown
- Partial pool with steady seepage
- Steady seepage, normal pool
- Earthquake
- Appropriate flood surcharge pool

7.3 Static Stability Analyses

Various analytical methods for evaluating the static stability of an embankment dam exist. The method utilized should be consistent with the anticipated mode of failure, dam cross section, and soil test data.
Many methods of stability analyses exist that use the same general approach of employing the "limit equilibrium method" of slope stability analysis. In this type of approach an estimate of factor of safety can be obtained by examining the conditions of equilibrium when incipient failure is postulated, and comparing the shear stress necessary to maintain equilibrium with the available shear strength of the soil.

In analyzing both force and moment conditions of equilibrium it becomes apparent that the problem of determining the distribution of the effective normal stress or consolidation stress on the failure surface is statically indeterminate, that is, there are more unknowns than there are equations of equilibrium.

An approach to this situation is to make assumptions to reduce the number of unknowns in order that the problem is statically determinate, such as is done in the "limit equilibrium" ·analysis procedure.
Different procedures use different assumptions. Some methods do not satisfy all conditions of moment and force equilibrium. Typical slope stability analysis results of for static loading is presented at end of respective sections. The software calculates the minimum factor of safety with four method of analysis i.e. Morgenstern-Price, Spencer, Bishop and Janbu. However, for comparison and reporting purposes the Spencer method is selected which accounts both the force

and moment equilibrium.

7.4 Factors of Safety

The factor of safety provides a margin of safety to guard against ultimate failure, to avoid unacceptable deformations, to reduce the risk of progressive failure, and to cover uncertainties associated with the measurement of soil properties, the loading, or the analysis used. In selecting a minimum acceptable factor of safety an evaluation should be made on both the degree of conservatism with which assumptions were made in choosing soil strength parameters, pore water pressures, and loading conditions; the consequences and the influence of the method of analysis which is used. The latter concerns the method of calculation in which side earth forces are considered and how assumptions of directions of side earth forces affect stability analysis results. It also concerns assumptions relative to the dissipation of shear- induced pore pressures and whether anisotropic consolidation is accounted for in the determination of undrained shear strength.

Minimum factors of safety generally required by FERC are listed in Table 1. The minimum Factors of Safety given in this table are to be used in conjunction with analyses using peak shear strengths. If certain wedges involving foundation shales are analysed using residual or near residual strength, then lower Factors of Safety than those given in Table 1 for the static cases are generally acceptable. The specific values being dependent on the percentage of the failure surface assumed to be at the residual strength.

Final accepted factors of safety will also depend upon the degree of confidence in the engineering data available. In the final analysis, the consequences of a failure with respect to human life, property damage, and impairment of project functions are important considerations in establishing factors of safety for specific investigations.

Table 7-1 Recommended Factor of Safety

Loading Condition	Minimum Factor of Safety	Slope to be Analyzed	Shear Strength Envelope
End of construction condition	1.3	upstream and downstream	
Sudden drawdown from maximum pool	>1.1*	upstream	
Sudden drawdown from spillway crest or top of gates	1.2*	upstream	
Steady seepage with maximum storage pool	1.5	upstream and downstream	
Steady seepage with surcharge pool	1.4	downstream	
Earthquake (for steady seepage conditions with seismic loading using a pseudo static lateral force coefficient)	> 1.0	upstream and downstream	

7.5 Stability Analysis

7.5.1 *General*

As outlined in Section 7.2, an embankment may be subjected to several loading conditions during its life, ranging from construction to full pool operation. The loading conditions for which an embankment must be analysed and the shear strength values appropriate for use in the analyses are presented in detail in the following paragraphs.

7.5.2 *End of Construction Loading Condition*

At the end of construction, an embankment dam may still be undergoing internal consolidation under its own weight. For homogeneous dams or for zones in dams constructed of impervious materials, pore water pressures will be built up during construction if the rate of consolidation of the embankment materials is slow compared to the rate of fill placement. Low permeability, natural foundation layers which are too thick to consolidate a significant amount during construction will also develop excess pore pressures caused by the weight of the embankment.

Therefore, Slope stability during and at the end of construction is analyzed using either drained or undrained strengths, depending on the permeability of the soil. Many fine-grained soils are sufficiently impermeable that little drainage occurs during construction.

7.5.2.1 Model input

The stability analysis for this condition is analysed using GeoStudio 2012 software, using Elastic plastic model when,

- The undrained strength of the foundation varies with depth (elevation)
- The soil stiffness of the foundation varies with depth (elevation)
- The soil stiffness of the foundation material varies with the overburden stress (Y-stress)
- Elastic plastic model is used for the fill material with a constant stiffness E

The fill is placed in seven 2.5 m successive and one 2-metre lifts. And configured as it shown in the figure below.

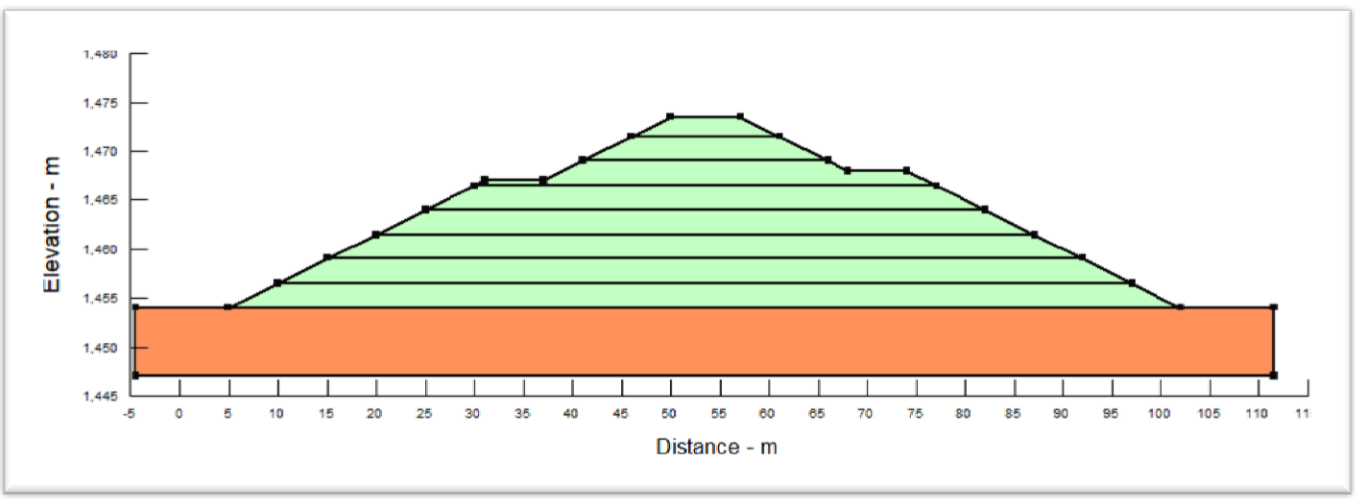


Figure 7-1 Problem configuration and setup

The initial (in situ) stresses are developed prior to the fill placement using a linear-elastic material model for the foundation soil. Prior to the fill placement the water table is assumed to be at the ground surface. Consequently, effective-drained parameters are required in the insitu analysis in order to get the correct insitu stress conditions. The effective-drained parameters are required so that pore-pressures will be considered in the insitu stress computations.

A total stress undrained behavior is assumed for the foundation; that is, the strength is specified as Cu and the E-modulus is considered to be a total stress modulus. The embankment material is assumed to be a well compacted soil with a relatively high stiffness. Figure 7-3 shows the undrained strength used for the foundation the undrained strength varies from a minimum of 57.11 kPa at the surface to a maximum of 116 kPa at depth

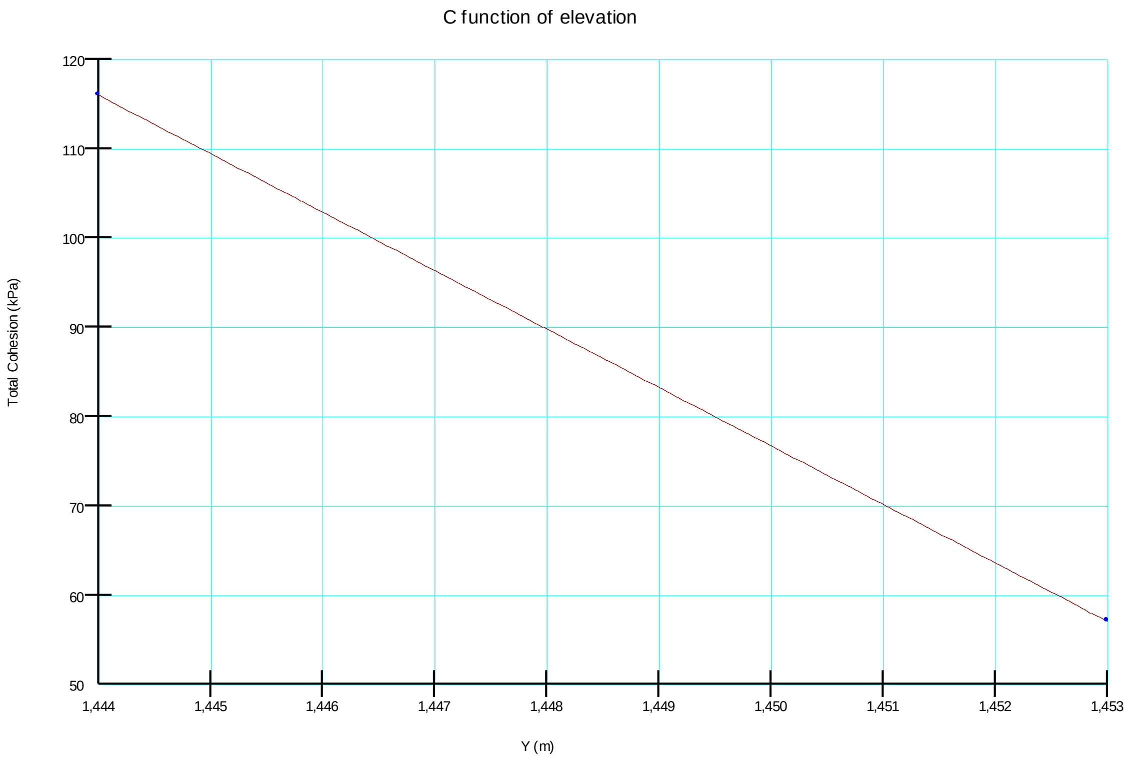


Figure 7-2 Undrained strength varying as a function of elevation

The E modulus varies from a low value of 5000 kPa near the ground surface to about 12,000 kPa at depth.

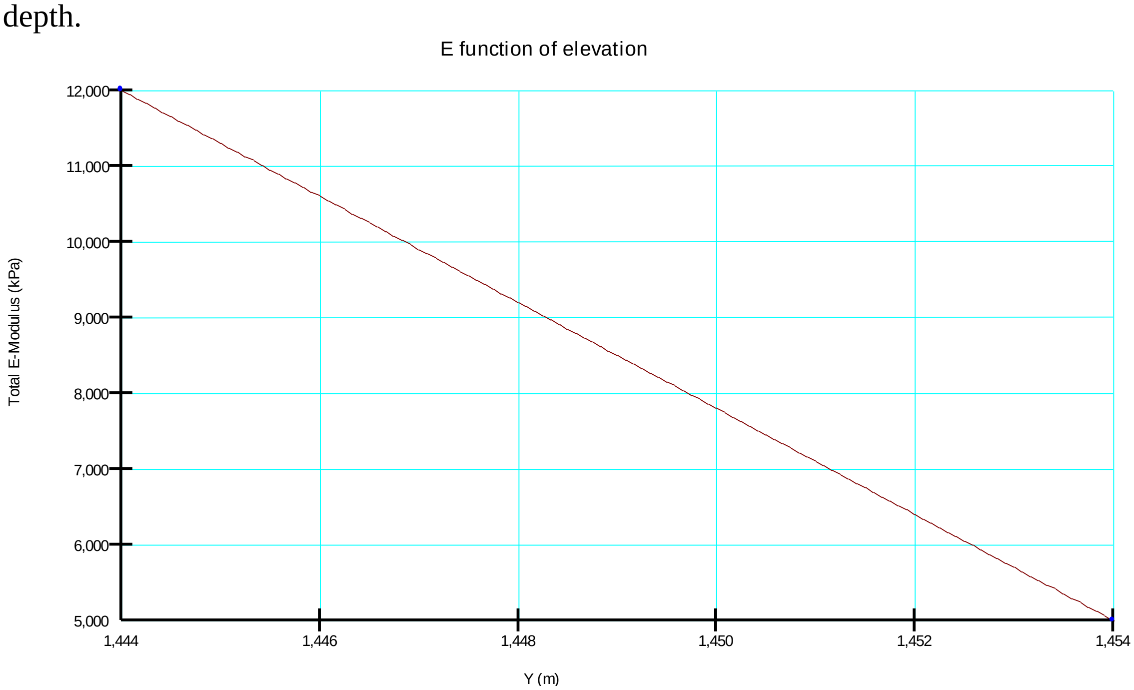


Figure 7-3 Foundation E-modulus varying as a function of elevation

The fill material is assigned a constant undrained strength of 330 kPa and the stiffness is assigned a constant E of 12000 kPa.

7.5.2.2 Result of the analysis

The following graph shows the Cu profile for the foundation for all load steps. The legend has units of seconds (sec), which is equivalent to load step number in this particular case. Notice that the profile is the same for all load steps, as it is intended to simulate the undrained behavior. This is also the case for the initial modulus E_i as shown in Figure 7.4

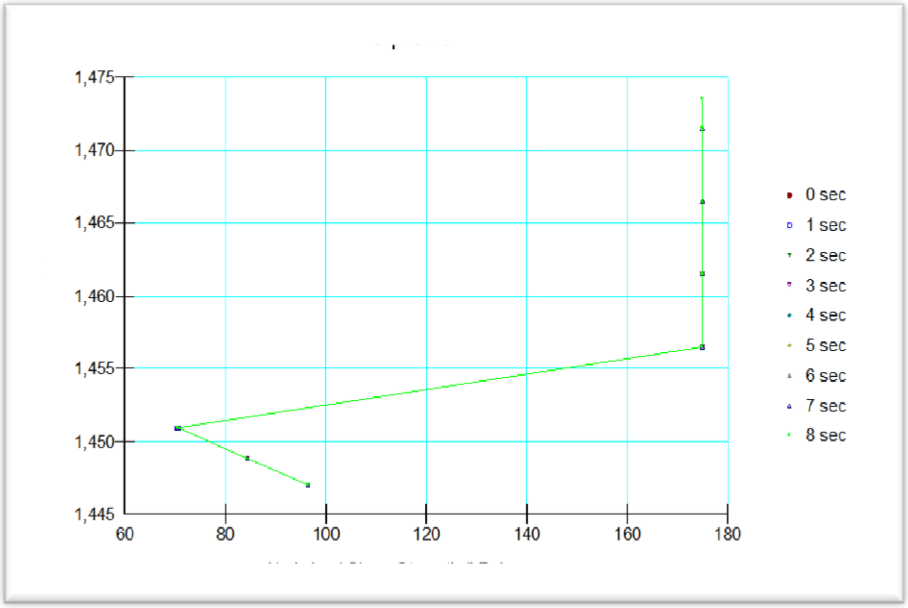


Figure 7-4 Undrained strength profile during loading

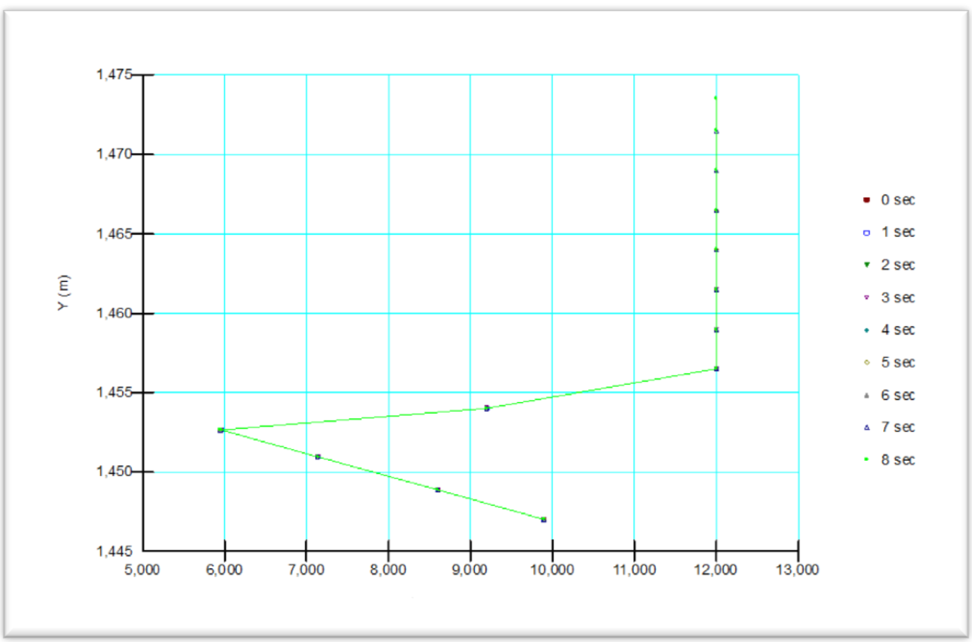


Figure 7-5 Ei profile during the embankment loading

The vertical settlements along a profile at the centre-line of the embankment are shown in Figure 7-7. Of significance is that the largest settlement is not at the dam crest.

Displacement profiles along the original ground surface are presented in Figure 7-8. Naturally, settlement occurs under the central part of the embankment and heave occurs near the toe area and outside the foundation footprint.

The foundation soil is treated as being undrained inferring that it cannot undergo any volume change (v = 0.49). Consequently, any settlement under the dam has to be reflected in heave beside the dam. This is also evident in the deformed mesh shown in Figure 7-9.

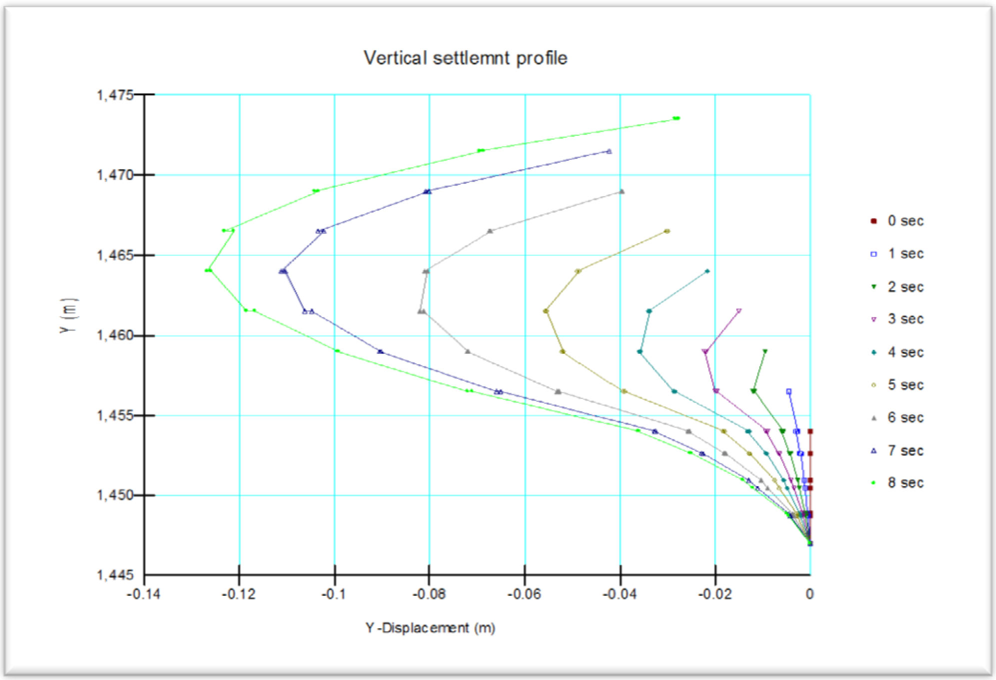


Figure 7-6 Vertical settlement profiles along centre line of structure.

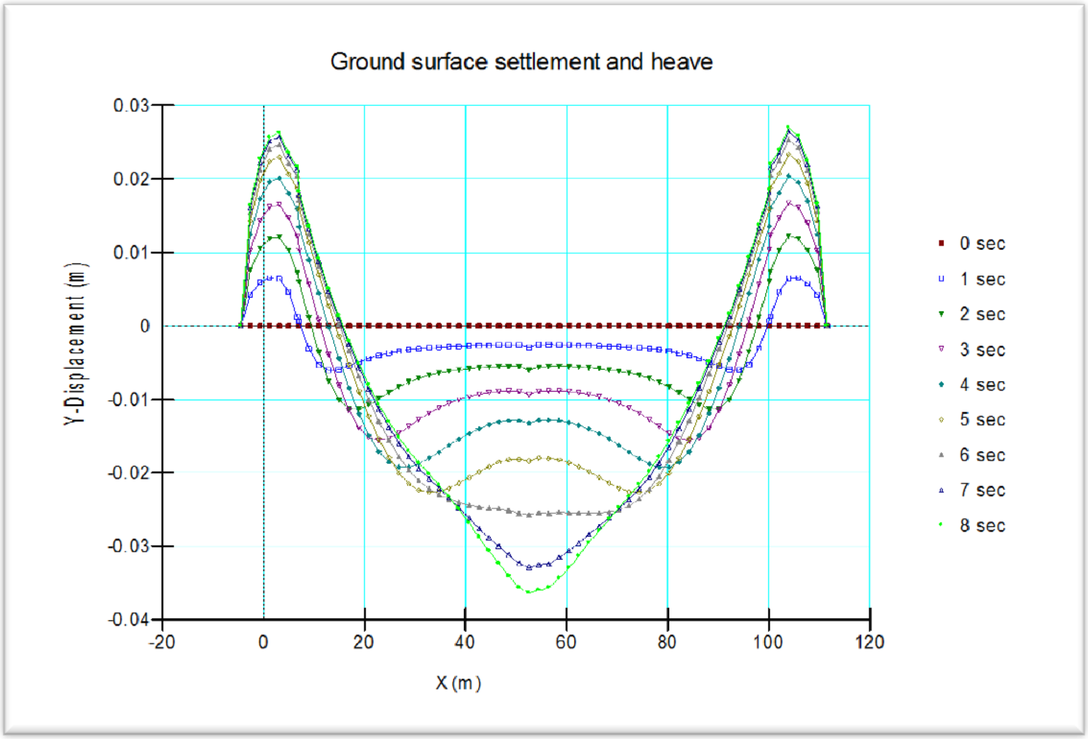


Figure 7-7 Settlement along original ground surface

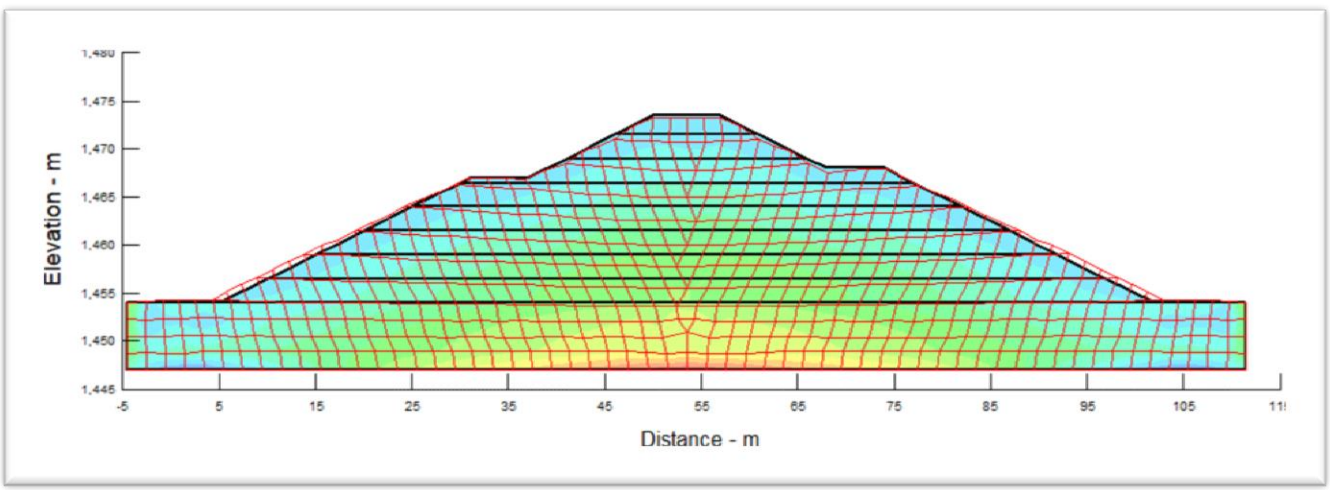


Figure 7-8 Deformed mesh

The factors of safety corresponding the end of construction was evaluated and presented as the following figure in both cases the slope factor of safety is greater than 1.3 and above the recommendation the stability result is indicated in Figure 7-9 to 11

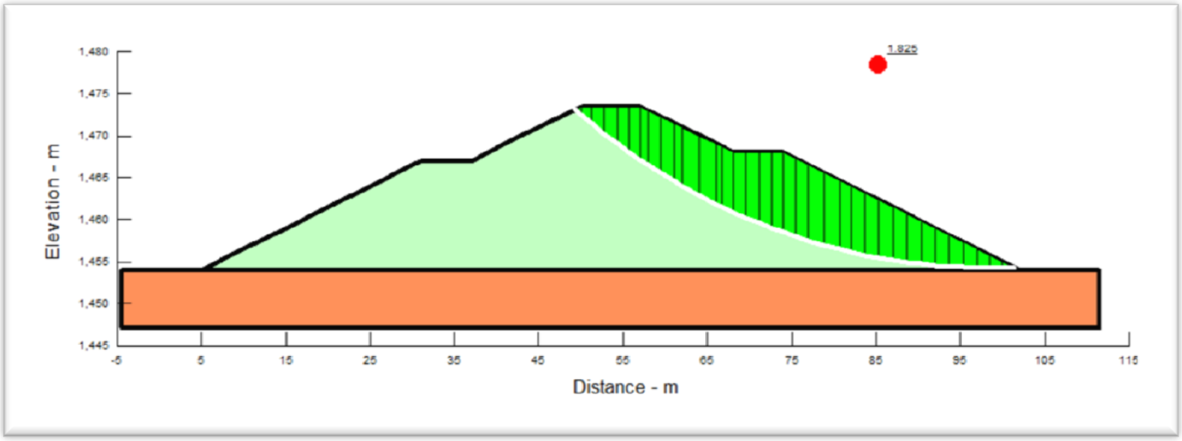


Figure 7-9 End of construction D/s slope stability

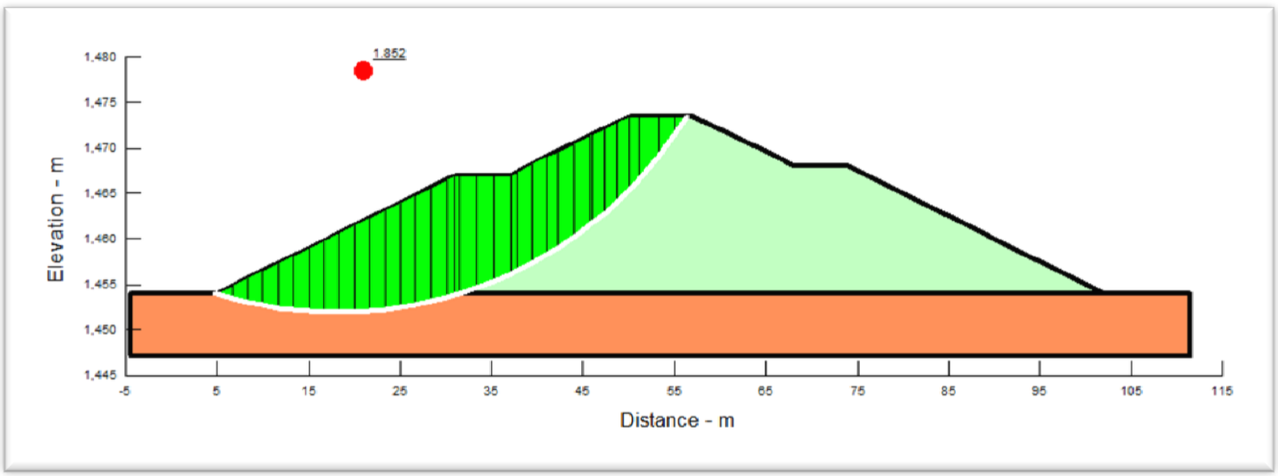


Figure 7-10End of construction U/s slope stability

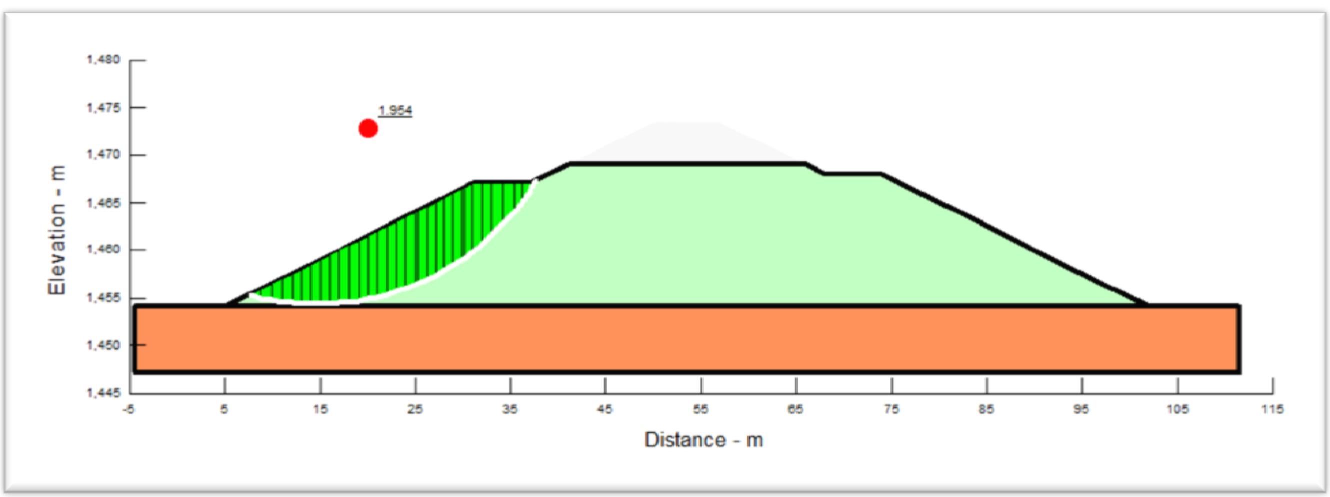


Figure 7-11 During of construction U/s slope stability

7.5.3 *Rapid Drawdown*

When an earth embankment has retained a reservoir with a fairly constant water surface elevation for a long time, the seepage conditions within the embankment will likely reached a steady-state. If it is necessary to drain the reservoir quickly, the pore-water pressures within the embankment may remain relatively high while the stabilizing effect of the reservoir's weight along the upstream side of the embankment is removed and can cause instability of the upstream face of the embankment.

A SEEP/W water transfer analysis can be used to evaluate changing pore-water pressure conditions after the reservoir has been drained. In the worst case, it is assumed that the reservoir has been drained instantaneously. More realistically, the reservoir will be drained over a period of time. This analysis presented how rapid draw down condition are modelled with a water transfer analysis, and used in SLOPE/W to determine the influence of instantaneous or gradual drawdown on slope stability.

The analysis involves an embankment that is approximately 97 m wide at the base and 19.5 m high (Figure 1). The embankment has a horizontal drain located at the toe to prevent water from exiting along the downstream slope face. The head in the reservoir before drawdown is 16.5m.

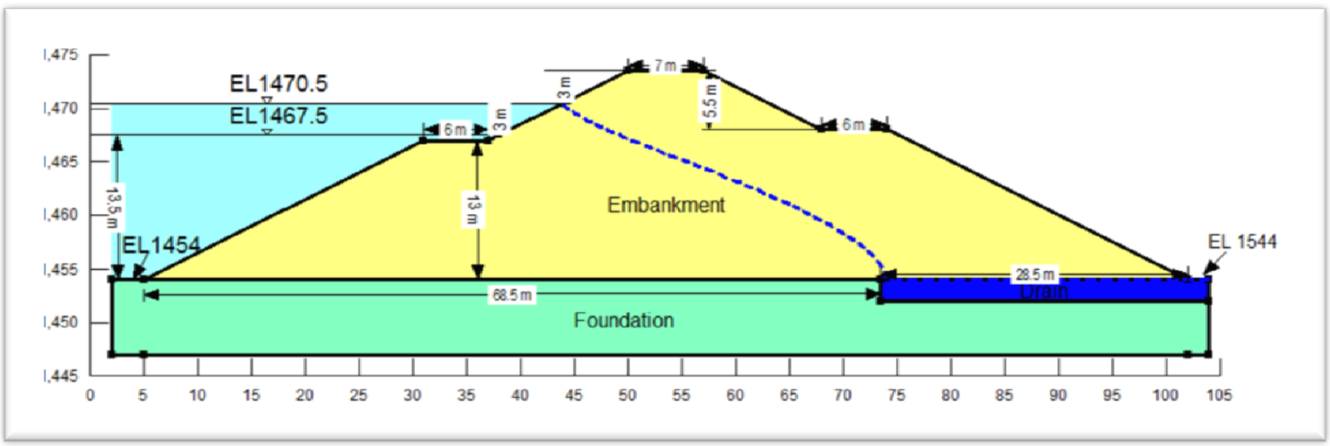


Figure 7-12 The problem configuration.

There is a total of 5 analyses in the Analysis Tree, with a steady-state water transfer analysis acting as the Parent to the remaining analyses. The first water transfer analysis represents the steady-state conditions within the embankment given the presence of the reservoir for a long period. The initial water table is drawn from the reservoir to the toe drain, and a constant head boundary condition (16.5 m) is applied to the upstream face of the embankment, representing the initial water level within the reservoir. The toe drain is simulated using a zero pressure head boundary condition applied to the toe drain region. The results from this Parent Analysis act as the initial conditions for the two scenarios: 'Instantaneous drawdown', and 'Slow drawdown.

After the water in the reservoir is drained, the relatively high porewater pressures within the embankment will cause groundwater movement out of the embankment, creating a seepage face along the upstream side of the embankment. The size and position of the seepage face are not known. Moreover, the seepage face will change with time after the drawdown occurs. A SEEP/W analysis can incorporate the seepage face along the upstream side of the embankment using the 'Potential Seepage Face Review' water rate boundary condition.

The second water transfer analysis is a transient simulation of the reservoir undergoing instantaneous drawdown, with the level of the reservoir changing from 16.5 m to 0 m. The instantaneous drawdown is simulated using a zero-pressure head boundary condition at the toe of the upstream slope and a potential seepage face boundary condition along the upstream slope to the height of the original reservoir level.

The slope stability analysis child of this transient water transfer simulation helps determine the influence of this instantaneous change in the pore-water pressure conditions within the embankment on the factor of safety over a period of 30 days following drawdown. The pore-water pressure results from all time steps in this analysis are used in SLOPE/.

The final water transfer analysis simulates gradual drawdown of the reservoir over a period of 90 days. with the level of the reservoir changing from 16.5 m to 13.5 m. The changing water level of the reservoir is defined using a head function boundary condition (Figure 4). This boundary condition is applied to the upstream slope of the embankment to the point representing the highest level of the reservoir. The potential seepage face review option is also activated to allow flow to leave the upstream side of the domain. As with the rapid drawdown scenario, the pore water pressures from the gradual drawdown water transfer analysis are used in SLOPE/W to determine the resulting factor of safety.

Both of the slope stability analyses use the Spencer analysis type, with the slip surface defined using the entry and exit method in the right to left direction. The potential entry location of the slip surface is defined at the top of the embankment core, with the exit defined along the lower toe of the slope (Figure 5).

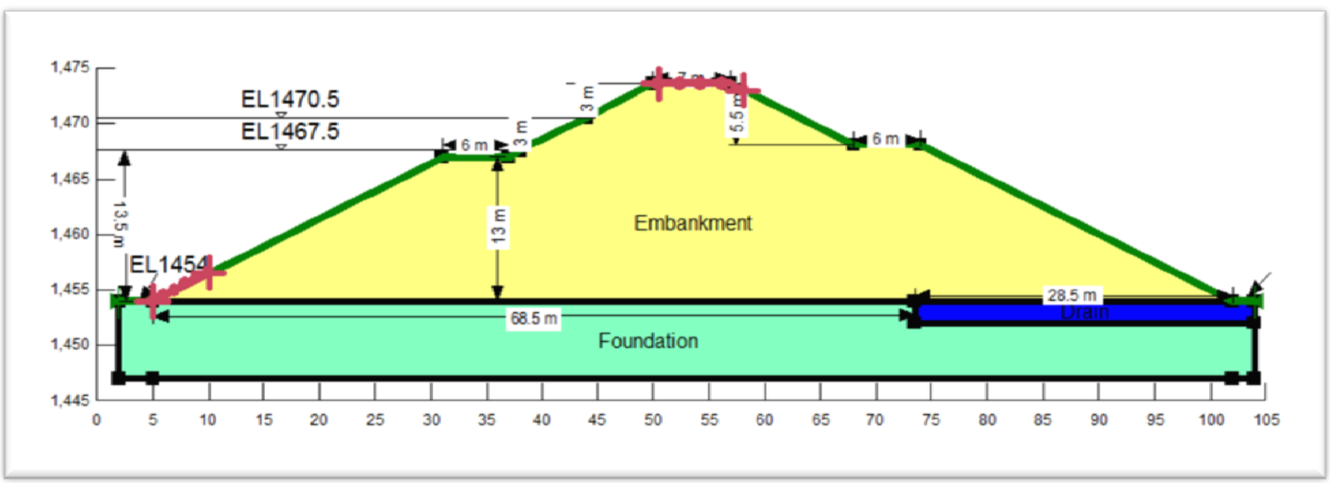


Figure 7-13Entry and exit locations for the slope stability analyses

The global element size has been set to 1 m. But the mesh size was refined to 0.5 m for the water transfer analyses. The transient analyses have a total duration of 90 days, with 10-time steps increasing exponentially from an initial time step of 6 hours.

7.5.4 *Model Output*

The long-term, steady-state conditions established using the Parent analysis are shown in Figure 7-14. After instantaneous drawdown, water stored within the embankment gradually drains from areas of high pore-water pressure. Thus, the piezometric line changes position over time, as illustrated Figure 7-15. This means that the seepage face also changes over time, gradually decreasing in size.

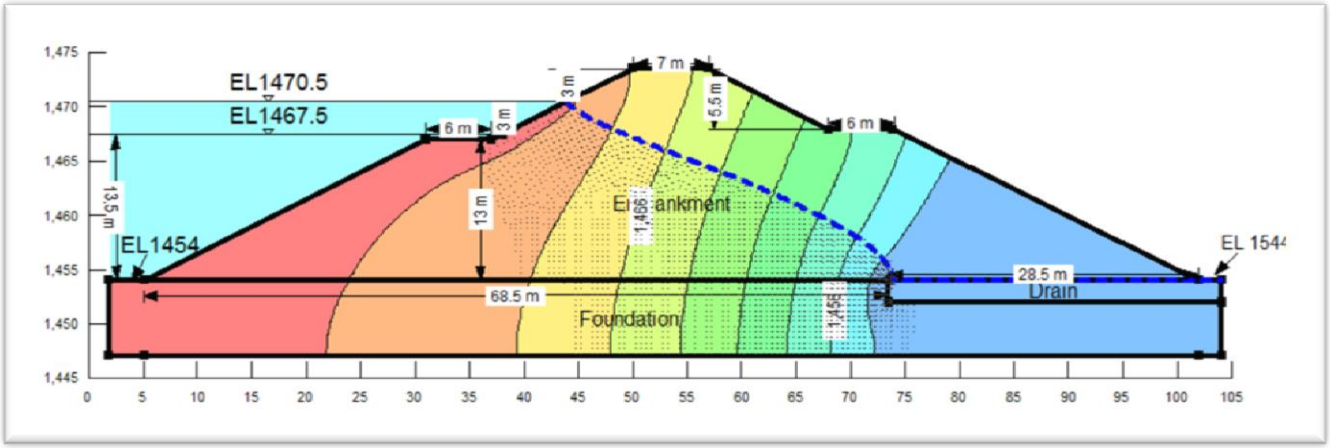


Figure 7-14

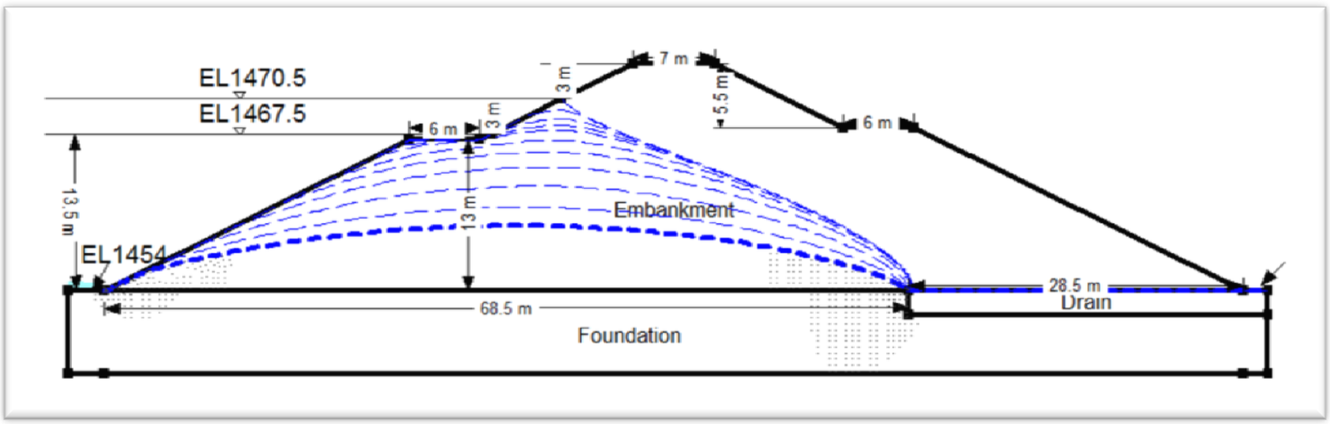


Figure 7-15 Changing positions of the piezometric line after instantaneous drawdown

Figure 8 shows the location of the piezometric line and velocity vectors at the end of day 1 from the instantaneous drawdown water transfer analysis. The colored circles on the upstream face show which nodes are H nodes (red) and which are Q nodes (blue). Water is flowing out of the bottom portion of the upstream face at the H nods (red). These will change with each time step.

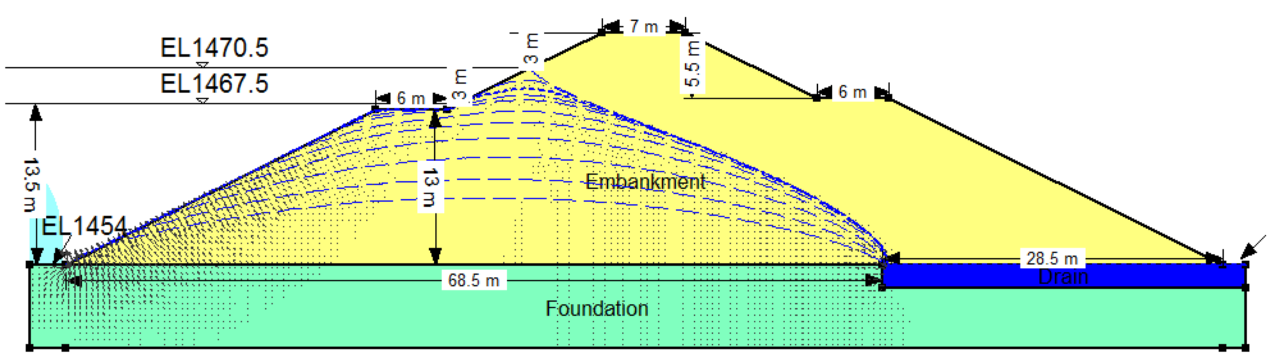


Figure 7-16 Piezometric line and velocity vectors at the end of day 1 from the instantaneous drawdown analysis.

The Factor of Safety versus time given instantaneous drainage of the reservoir was shown in Figure 7-17. The factor of safety drops below 1.0 immediately following drawdown. However, the factor of safety recovers over time as the excess pore-water pressure within the embankment dissipates.

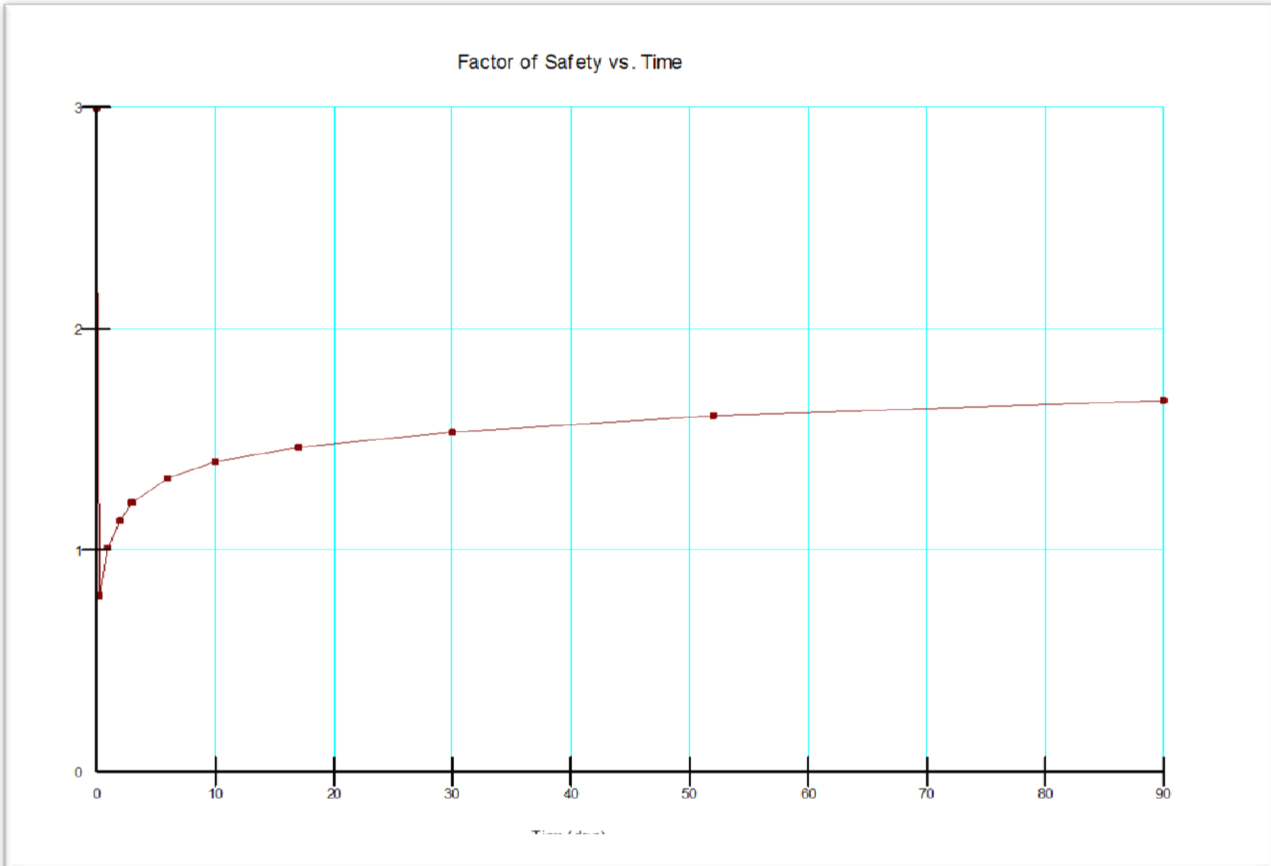


Figure 7-17 Instantaneous drawdown Factor of Safety vs time

Figure 7-18 shows the piezometric line and velocity vectors at the end of day 1 from the slow drawdown water transfer analysis. Here the velocity vectors show that less water is leaving the domain via the upstream slope, and more of the water is flowing out of the toe drain, due to the continued presence of water in the reservoir. As the reservoir level decreases over time, flow from the upstream face increases.

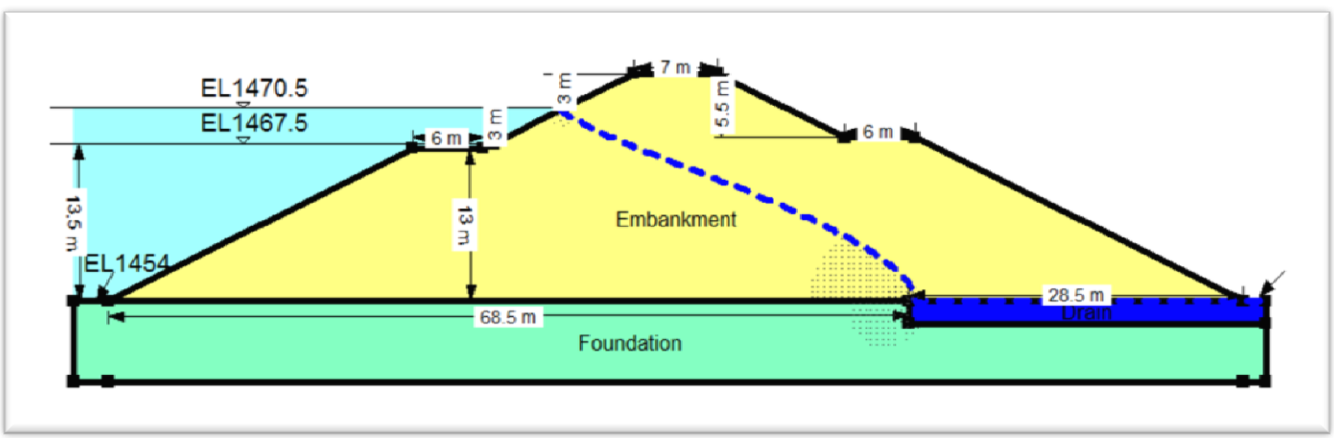


Figure 7-18 Piezometric line and velocity vectors at the end of day 1 from the slow drawdown analysis

When the reservoir level is drawn down slowly instead of instantaneously, the stability of the embankment is substantially different Figure 7-19. As with the instantaneous drawdown analysis, the factor of safety decreases when the reservoir is drawn down. However, the factor of safety for the slow drawdown analysis does not drop below 1.0.

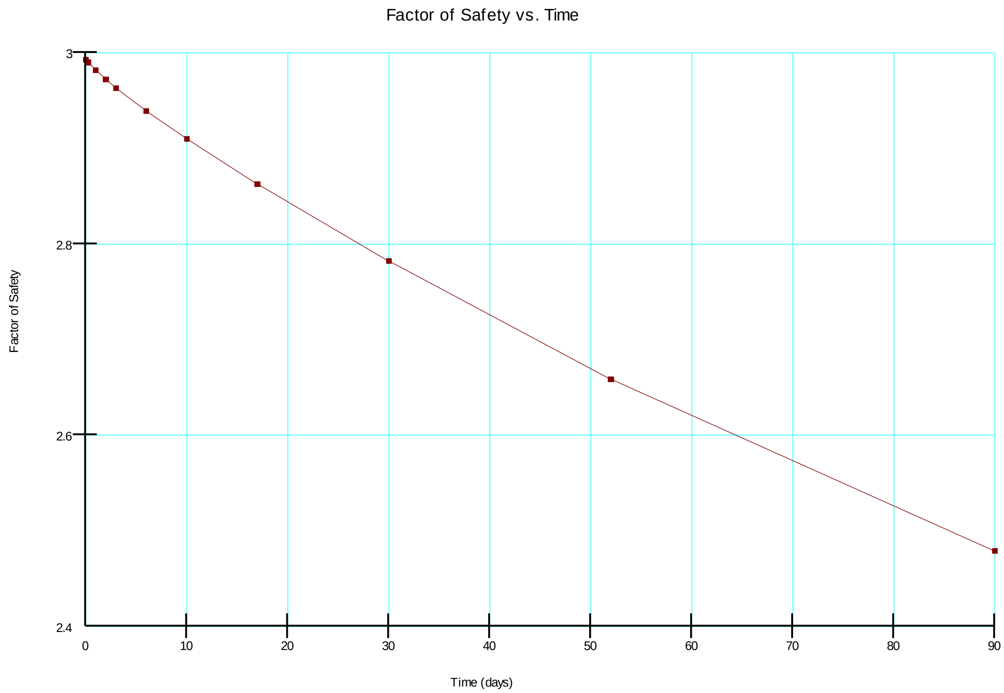


Figure 7-19 Slow Drawdown Factor of safety vs Time

Generally, the instantaneous drawdown scenario is a conservative approach for assessing stability of an embankment during reservoir drawdown; however, it likely represents unrealistic conditions as it is difficult to drain a reservoir over a very short period. The second scenario includes a gradual decline in the reservoir's water level so it is more realistic, and therefore, provides a more reasonable evaluation of the embankment's factor of safety during drawdown and satisfies the recommended FOS for this kind of loading condition.

7.5.5 *Pseudo-static Analysis*

Analyses for earthquake loading should begin with simplified procedures and proceed to more rigorous methods of analyses as a particular situation may warrant. Projects with well compacted embankments and dense foundation soils located in all confirmed low hazard potential zones, projects, may be evaluated by the pseudo static method using the seismic coefficient assigned to the seismic zone in which the project is located. Therefore, this project was evaluated with Pseudo static analysis for it is found in hazard potential zone with 0.05 seismic coefficient refer Geotechnical investigation report.

It is generally assumed that during a seismic event the loading is so rapid that there will be no change in the shearing resistance along a potential slips surface. In other word the soil will behave in an undrained manner. The seismic loading may increase the total stress at the base of a slice and there may be an equivalent change in the pore-pressure, but the effective stress will remain unchanged, and consequently the strength will remain unchanged. A staged pseudo-static analysis can be adopted to consider this scenario. Two options are available for calculating the undrained strengths prior to the application of the seismic forces.

7.5.5.1 Staged Pseudo-Static Analysis

A staged pseudo-static analysis involves the completion of two stability analyses for every single slip surface. The first stage is completed without applying the seismic forces so that the effective stresses can be obtained at the base of each slice. The effective stresses can then be used to calculate the strengths at the base of each slice, which are subsequently used in the second stage when the seismic forces are included.

SLOPE/W has the above stated two options for calculating the strengths from the effective stresses obtained in Stage 1. One of the options calculates the strength according to the Mohr-Coulomb strength law and the effective stress strength properties. The resulting effective stress strengths are treated as equivalent undrained strengths in the second stage of the analysis. The other option uses the undrained strength calculations proposed by Duncan et al. (1990). The undrained strength calculations of Duncan et al. (1990) require both the effective stress strength properties and those corresponding to the undrained R-envelop. These two strengths envelop are used to compute an equivalent undrained strength at the base of each slice. The “None” option results in a conventional, single stage, pseudo- static analysis.

The analysis was defined with 19.5m high water retention embankment with 2:1 side slopes. The material is defined according to the Table 6-7

One trial slip surface is considered for illustrative purposes. The specified pseudo-static seismic coefficient is 0.05. Four analyses were completed: 1) a conventional analysis excluding seismic forces; 2) a conventional pseudo-static analysis; 3) staged pseudo-static analysis using effective stress strengths; and, 4) staged pseudo-static analysis using undrained strength calculated according to Duncan et al. (1990).

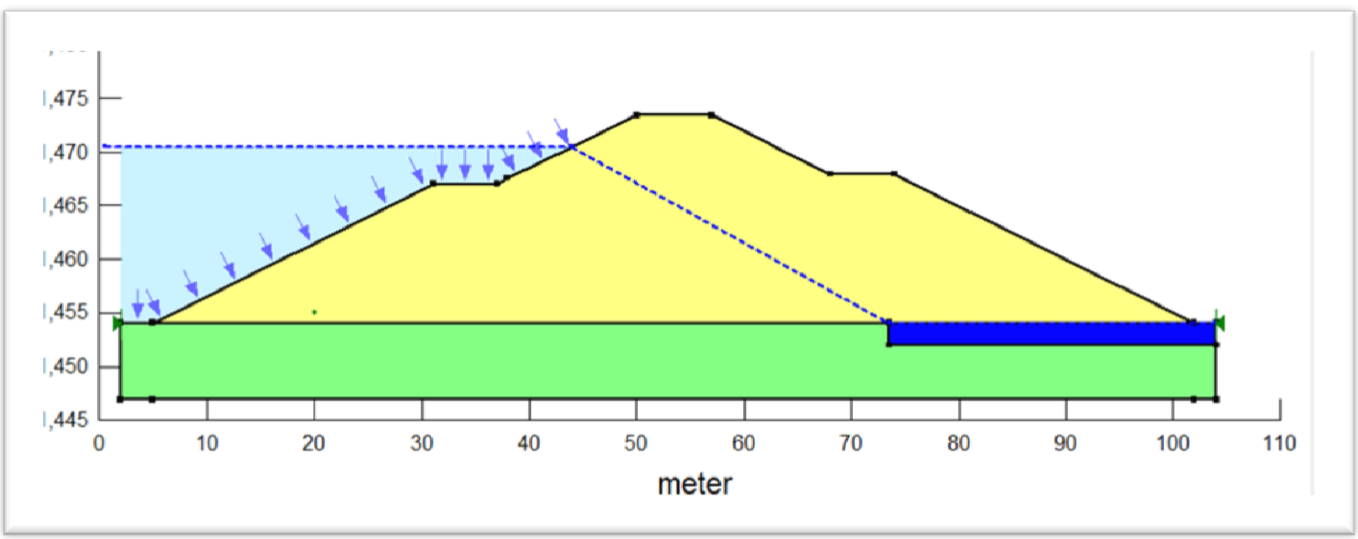


Figure 7-20 Pseudo Problem configuration

7.5.5.2 Results of the analysis

The factors of safety corresponding to each analysis is shown in the figures below

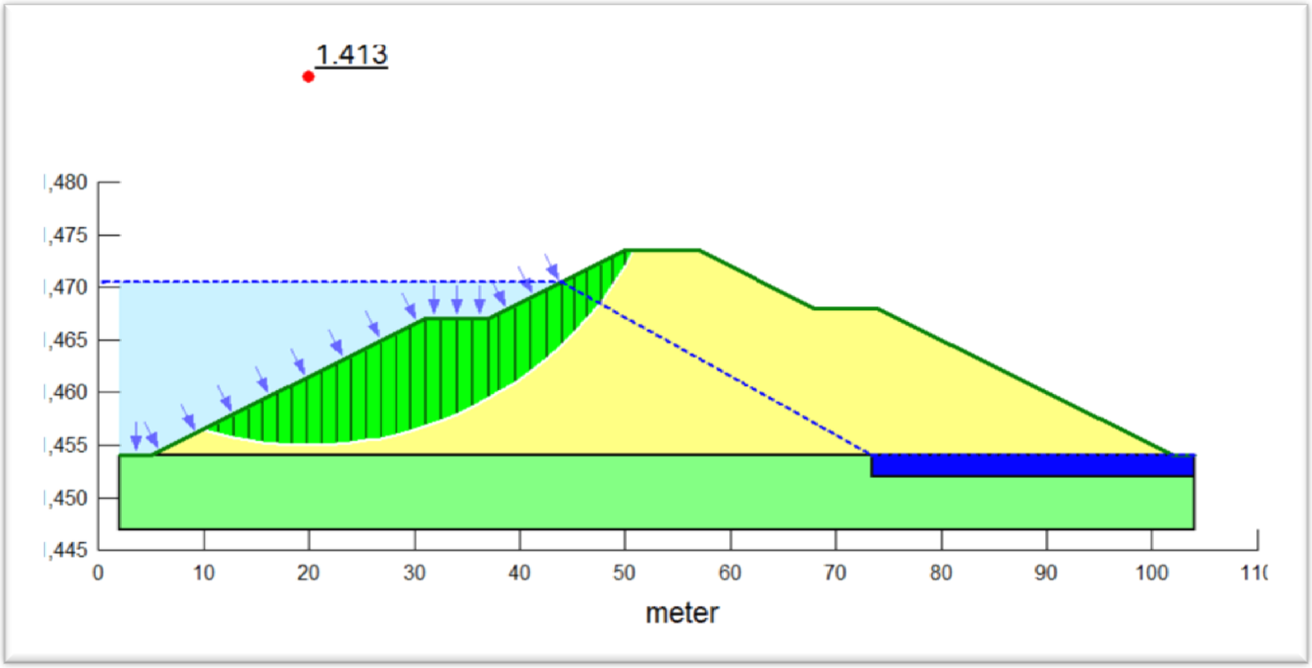


Figure 7-21 Excluding seismic forces

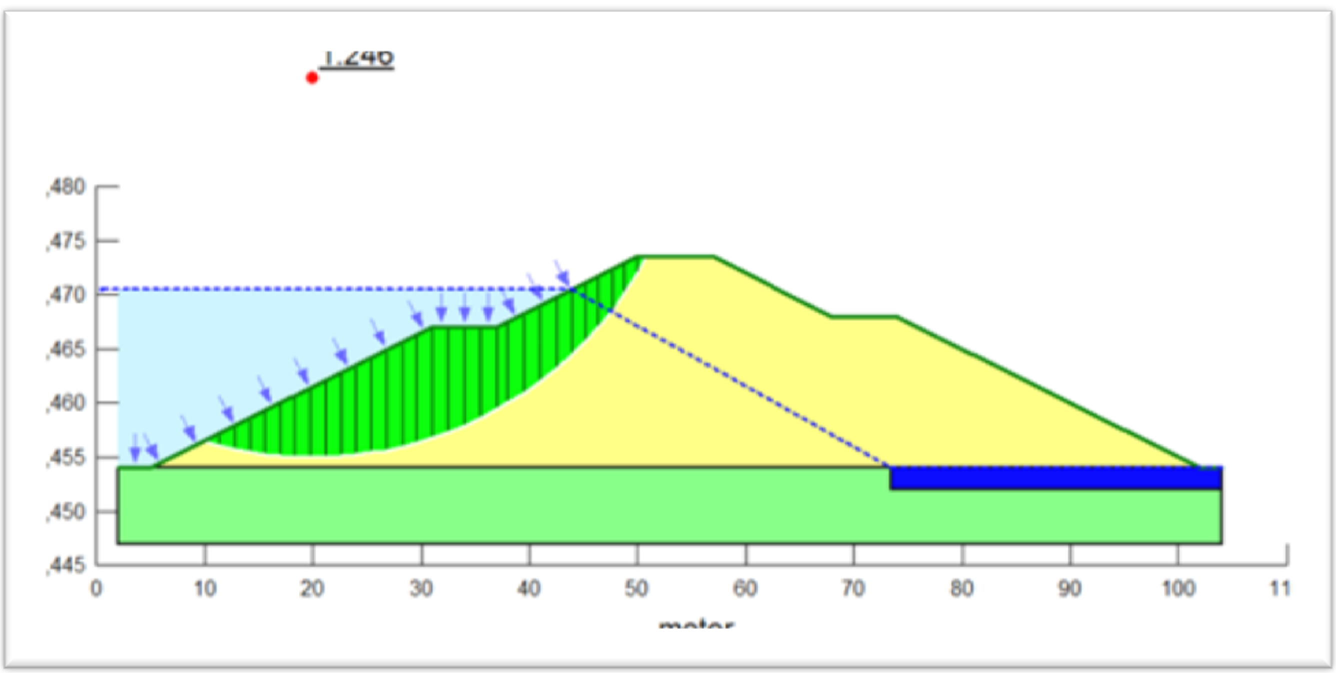


Figure 7-22 None (conventional pseudo-static)

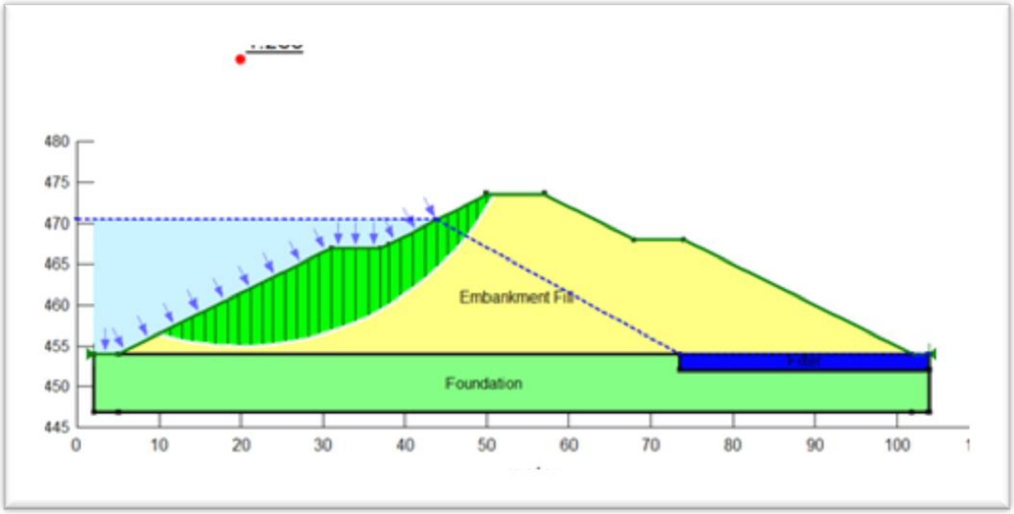


Figure 7-23 Effective stress (equivalent undrained) strengths

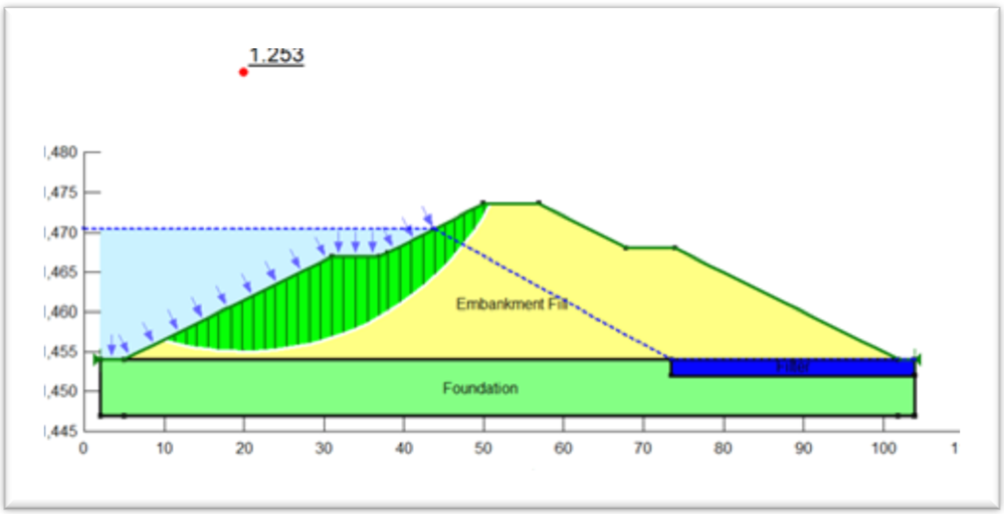


Figure 7-24 Duncan et al. (1990) undrained strengths

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8 SPILLWAY WORKS

8.1 General Design Considerations

The proposed Dam is homogenous embankment dam with horizontal drain. This type of dam needs sufficient spillway capacity. The safety of the dam cannot be secured only with provision of sufficient discharging capacity of the spillway; the hydraulic design of the spillway must also be adequate to convey and dissipate the energy associated with water passing over the spillway.

The spillway crest elevation is at 1470.5 masl and the following main design criteria have been considered for the design of the spillway:

- There should be complete dissipation of energy of spillway flow before it enters the river;
- The dimension of the control section (chute), have been fixed such that it could pass the maximum routed flood with a design discharge head (surcharge);
- The spillway must be hydraulically and structurally safe;
- The spillway and chute in general should be erosion resistant to withstand the high scouring velocities; and

The spillway design is accomplished in a manner that minimizes cost subject to the following: -

- Sufficient crest length to convey the design discharge.
- Acceptable minimum pressures acting on the crest boundary.
- Acceptable maximum energy head on the spillway crest.
- Acceptable velocities and flow characteristics through the spillway system.

8.2 Spillway inflow and outflow design flood

As indicated in section 3.4.2, the design discharge for the spillway is fixed based on the recommendation given in the Dam Safety Draft Guide of the Ethiopian Commission of Large Dams. Accordingly, the inflow hydrograph considered is 100 years return period and its peak discharge is 120.386m3/s. For the selected total crest length of 20m, the routed outflow discharge is 85.55m^3/s.

8.3 Type of spillway and selected scheme

Spillway type selection can be governed mainly on type of dam, site condition (i.e. topography), and flood discharge, simplicity for construction and adaptability for foundation. Chute and side channel spillway have been considered. For the topography favoured. Similarly, in order to create flow access towards the control section of the chute spillway it needs longer approach channel excavation with relatively smaller depth of excavation is considered. To minimize the length of an approach, channel a curvature was provided at the upstream side of the approach channel so that an optimal length and depth of excavation is achieved. Therefore, by considering all factors listed above, un-gated chute spillway is selected to convey maximum outflow discharge i.e. 85.55m³/s.

8.4 Location and alignment of spillway

Primarily, location of the spillway is dictated by the safety of the dam, economic considerations, construction difficulties, and the possible interference with the layout of outlet and canal structures of the project. In addition to this, topography and geology of the area are among the main considerations guiding the choice of spillway location.
For Worbate dam the spillway site is selected or proposed on the right abutment by excavating the top end of the abutment. Naturally, the spillway route has an excavated approach channel, relatively mild slope before the start of the chute and steeper slope before ending at the energy dissipater. The dissipated flow will join the Worbate River.

8.5 Hydraulic design of spillway

The main components of the spillway include, approach channel, control section, chute section, terminal structure and exit channel. In this section the hydraulics aspects of the mentioned components will be presented.

Table 8-1 Hydraulic design of Chute

Chute design			Chute	Chute
Basic Input Data				
Chainage at Start of chute	Ch. Ch.St	m	120.000	185.000
Chainage at End of chute	Ch. Ch.En	m	150.000	210.000
Design Discharge	Q	m^3/s	86.000	86.000
Manning Roughness	n		0.035	0.035
U/s Normal Water depth	d_o	m	1.80	1.80
D/s Normal Water depth	d3	m	1.80	1.80
Velocity at section 3	V3	m/s	2.43	2.73
Velocity head at Section 3	$h_{v3} = V_3^2/2g$	m	0.301	0.379
elevation at Chute Start	EL. Ch. Str	m	1,470.14	1,465.57
Elevation at Chute End	EL. Ch. End	m	1,465.04	1,453.07
Width control	L	m		
Drop height	Z	m	5.10	12.50
Length	L	m	30.00	25.00

1. Critical flow hydraulics				
Dscharge	Q	m^3/s	86.000	86.000
Width of notch	$bc = 0.734Q/d_0^{3/2}$	m	26.139	26.139
Adopt		adopt	20.00	20.00
Unit discharge	$q = Q/bc$	$m^3/s/m$	4.300	4.300
Critical depth	$dc = (q^2/g)^{1/3}$	m	1.235	1.235
Critical Velocity	$V_c = q/dc$	m/s	3.481	3.481
Velocity head	$hvc = V_c^2/2g$	m	0.618	0.618

Water area	$Ac = bc*dc$	m^2		24.705	24.705
Wetted Perimeter	$Pc = bc + 2*dc$	m		22.471	22.471
Hydraulic radius	$RC = Ac/Pc$	m		1.099	1.099
Water surface slope	$Ic = (nVc/Rc^{2/3})^2$			0.013	0.013

2. Energy at Section (C)	Z = EL.A - EL.B	m	-	5.10	12.50
	$E_C = dc + hvc + Z$	m	-	6.95	14.35

3. Energy at Section (1)

d_1	m	0.535	0.382
b1=bc	m	20.000	20.000
A1=b1d1	m^2	10.700	7.642
V1=Q/A1	m/s	8.037	11.254
$hv1 = V1^2/2g$	m	3.292	6.455
P1=b1+2d1	m	21.070	20.764
R1=A1/P1	m	0.508	0.368
$I1 = (nV1/R^{2/3})^2$	m	0.195	0.588
$I_m = (Ic + I1)/2$	m	0.104	0.301
$hf1 = I_m.L$	m	3.126	7.516
E1 = d1 + hv1+hf1		6.953	14.353
E1 = Ec =		6.953	14.353

4. Conjugate depth after jump

Froude number	F = V1/(gd1)1/2	m	3.508	5.813
Conjugate depth	d2=d1/2((1+8Fr2)1/2-1)	m	2.400	2.956

5. Stilling basin

a. Length for Fr<5		L =3.5*d1Fr11.5	m	12.30	18.74
b. Length for Fr>5		L =8*d1Fr1	m	15.02	17.77
			Adopt	10.00	20.00
c. Width			m	1.78	1.78
			Adopt	20.00	20.00
d. Bottom elevation (EL.C)	Lip height, a= dc/2	m		0.62	0.62
		Adopt		0.60	0.80
	El.C = EL.B - a	m		1,464.44	1,452.27

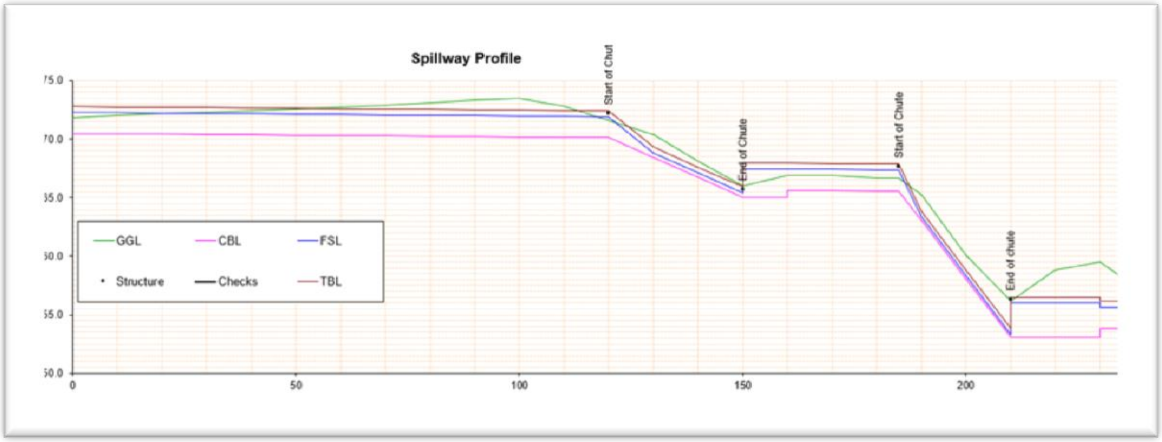

Figure 8-1 Longitudinal profile of spillway

8.6 Spillway exit channel

The downstream exit channel will convey the spilled water for about 50m before it joins the natural river course. The spilled water is made to join small stream which joins the Worbate River. The size of the stream is small and hence its existing cross section will be modified to convey the spillway discharge at least the flood corresponding to 1:100 routed outflow discharge. Accordingly, a modified cross section is proposed for the whole reach with an appropriate treatment measure.

9 DESIGN OF OUTLET WORKS

9.1 Lay out of outlet conduit and related works

The outlet work system covers mainly the irrigation outlet and provisions made to pass the dry season during construction. As it was discussed in section 2.3, the construction is planned to be finished in two years and hence during the first year of construction the wet season flow will pass with the existing river course; however, foundation excavation of the main river course will be finished during the dry months of the first-year construction. Therefore, simple temporary dry season outlet is proposed to pass the minimum flows during the dry months.

The main purpose of this outlet system is for irrigation. However, during emergency cases it will be used to evacuate the volume of water above it. The control mechanism will be downstream (valve). The water entering the small intake will be conveyed towards upstream and downstream with a conduit crossing the dam body. The current dam design concept did not encourage provision of conduits within the dam. Though locating conduit outside the dam is the safest approach, there are also several dams in which their outlet system crosses the dam body. For this project special consideration has been given in most part of the dam to be embedded in excavated trench along the rocky staff and also provision of filter thus minimizes potential failure. Water leaving the dam will be dissipated before it joins the main canal. The centre line of the irrigation conduit is maintained to be at 1466.5 m.

9.2 Irrigation Outlet

The proposed net command area for Worbate project is about 120ha. The required design discharge considering including the demand for livestock is summarized in the table below

Table 9-1 Peak Discharge Estimation

	Month												
Demand	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	
Irrigation (120 ha)	0.35	0.37	0.16	0.00	0.00	0.19	0.22	0.09	0.01	0.00	0.00	0.14	
Livestock	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	
Evaporation	0.06	0.06	0.06	0.06	0.06	0.05	0.05	0.05	0.06	0.06	0.06	0.06	
Total Demand	0.42	0.45	0.24	0.08	0.08	0.26	0.30	0.17	0.08	0.08	0.07	0.22	
Cumulative Demand (Mcm)	0.42	0.87	1.11	1.19	1.26	1.52	1.82	1.99	2.07	2.15	2.22	2.44	
Analysis of Peak Demand													
Days in the month	31	28	31	30	31	30	31	31	30	31	30	31	hr/day
Livestock (m3/s)	0.087	0.087	0.087	0.087	0.087	0.087	0.087	0.087	0.087	0.087	0.087	0.087	2
Irrigation (m3/s)	0.259	0.307	0.119	0	0	0.144	0.168	0.069	0.007	0	0	0.106	12
Daily Demand (m3/s)	0.346	0.394	0.206	0.087	0.087	0.231	0.255	0.156	0.093	0.087	0.087	0.192	
	Max=	0.394											

Accordingly, for the calculated peak Demand for 12 hour and 2-hour supply for the daily Demand found to be 0.4m3/s however considering extra 20 % downstream release the outlet capacity becomes 0.5m3/s. The main purpose of this outlet is for irrigation. However, where there is a need it will be used to evacuate the reservoir water above 1454m which accounts to be about 1.0MCM. The Irrigation outlet is designed to have a small inlet, trash rack, circular conduit, flow regulation valves at the downstream end and an energy dissipater.

9.2.1 *Hydraulic Design of Intake/Inlet*

9.2.1.1 i. Intake/Inlet opening

The proposed design discharge is 0.5m3/s and an opening pipe diameter of 500mm ductile iron pipe is provided.

9.2.2 *Irrigation Conduit Hydraulics*

A. General

The conduit is designed to pass the irrigation flow and environmental release. Further the outlet will evacuate the reservoir water above irrigation outlet which is about 1.3MCM in volume.

The size of an outlet conduit for a required discharge varies according to an inverse relationship with the available head for producing the discharge. This relationship is expressed by the following equation:

$$H_T = k_T \frac{Q^2}{2gA^2}$$

Where:

H_T= total available head needed to overcome the various head losses to produce the discharge (m);

Q = required outlet works discharge (m^3/s);

K_L = loss coefficients;

A = required area of the conduit (m^2); and
g = acceleration due to gravity 9.81(m/s^2). Head loss computation for various parts of the structure is discussed below.

B. Pressure flow in the irrigation conduit

The valves fitted downstream will basically control the flows and hence the conduit is always under pressure. The flow in a closed conduit pipe system follows Bernoulli's equation which is written as:

$$H_T = h_L + h_{v^2}$$

Where:

H_T = total head needed to overcome the various head losses to produce discharge,
h_L= Cumulative losses of the systems, and
h_{v2} = Velocity head at the valve.

The above equation is expanded to include the system losses of the outlet structure as follows:

$$H_T = (h_t + h_e + h_f + h_g) + h_{v^2}$$

Where:

h_t= Trash rack losses,
he= entrance losses,
h_f= friction losses and
h_g= gate or valve losses.
For the discharge at the outlet,
H_T is measured from the reservoir water surface to the centreline of the outlet gate.

C. Head loss computation

i. Trash rack Losses

The trash rack consists of 16mm thick mounted at the inlet and its head loss is almost negligible because the trash rack is placed on the top of the box horizontally.

ii. The other loss computation is shown in table 1 below (Entrance, Friction and Valve losses) in pipe size calculation

Table 9-2 Intake /Outlet Pipe size calculation

Item	Symbols	Formulae	Units	*Main Outlet*
Diameter	D		mm	500.00
Number of pipes	N		Nr	1.00
Design discharge	Q_d		m^3/s	0.40
chez	C			100.00
Pipe roughness coefficient	f			0.01
Inlet loss coefficient	ξin			0.50
Outlet loss coefficient	ξout			1.00
Pipe length	L		m	18.00
Acceleration	g		m/s^2	9.81
Hydraulic radius	R	D/(4*1000)	m	0.13
Flow section area	W	PI*((D/1000) ^2)/4	m^2	0.20
Pipe velocity	v_1	Q_d/(N*W)	m/s	2.04
Length loss coefficient	ξL	f*v2/2g*(L/D)		0.06
Discharge coefficient	Cd	1/Sqrt(ξin+ξout+ξL)		0.80
NPL				1470.50
Minimum water Level				1467.50
Water level difference	Z		m	0.50
Water level in intake tower				1467.00
Entrance loss		$0.5* V^2/2g$		0.11
Head loss through Gate		$0.2* V^2/2g$		0.04
Trash rack loss			m	0.10
H_{eff}				0.25
$Q=C_{d*}A*(2*9.81*H)^{1/2}$				0.49
$Q=A*(2*9.81*H_{eff)})^{1/2}$				0.44
Calculated discharge	Q_c	N*Cd*W*Sqrt(2*g*Z)	m^3/s	0.49
Difference	Qd - Qc			-0.09
V_1				2.51

Partial Flow Hydraulics

Assumed Ø			m	0.50
H	For	0.80		0.40
H/r	Ratio of D		m	1.60
$\beta=A/r^2$	From Table		m^2	2.69
$\alpha=R/r$	From Table		m/s	0.61
V_2			m	2.92
R			m	0.15
Pipe Length			m	61.00
n				0.010
$I_2 = (nV_1/R^{2/3})^{\wedge 2}$			m	0.01
$h_{f2} = I_2.\boldsymbol{L}$			m	0.64
H_T			m	1.17
Water level in the stilling basin			m	1465.83
Pipe invert level in stilling Basin			m	1465.43
Discharge Capacity at MWL			m^3/s	0.49

Maximum Discharge Capacity

Length loss coefficient	ξL	f*v2/2g*(L/D)		0.26
Discharge coefficient	Cd	1/Sqrt(ξin+ξout+ξL)		0.75
Maximum Water level difference	Z		m	4.57
Calculated discharge	Q_{max}	N*Cd*W*Sqrt(2*g*Z)	m^3/s	1.40

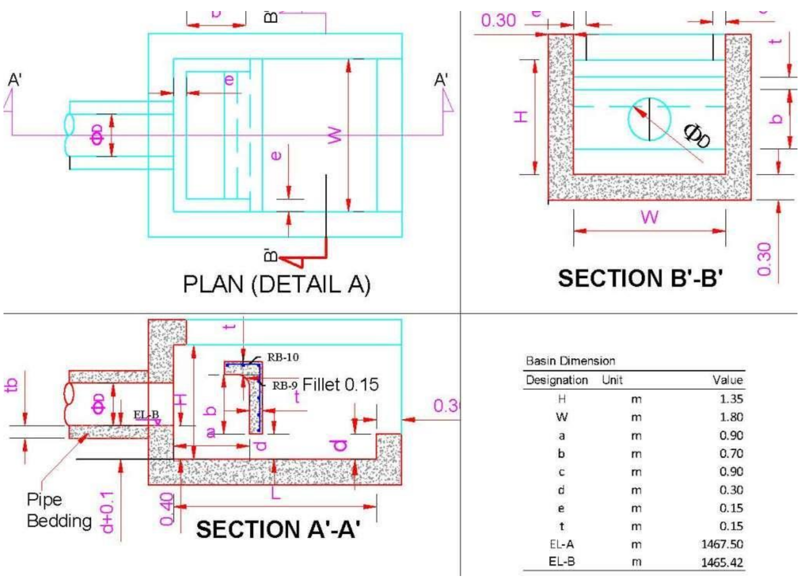

Figure 9-1 Outlet cross section

From the above Table-9.2 the main outlet was designed based on the peak demand discussed in table 9.1 for the right-side command and livestock. This capacity was designed based on the minimum water level but the outlet can discharge a maximum capacity of 1.4 m3/s when the reservoir is full which can help to reduce the flood intensity

9.2.3 *Type and sizes of Regulating Valves*

The selected types of controlling mechanism were sliding gate, which was provided at the upstream and downstream. The upstream was provided in order to regulate the development of pressure at the downstream and regulate the flow. But the downstream was provided for control of seepage or leakage as well as for easily access to the regulation of flow.

9.2.4 *Energy Dissipater Design*

Before water is conveyed to the main canal from the outlet, the flow energy should be dissipated so that downstream erosion will be avoided. Therefore, at the end of the outlet pipe a stilling basin is needed for energy dissipation. It is designed to safely dissipate the maximum energy i.e. when the reservoir is full and discharging the maximum flow. The Selected Basin type is Impact type one which was commonly adopted basin type.

10 Irrigation System Layout

10.1 Topography

In order to prepare a realistic system layout, it would be inevitable to scrutinize the local topographic condition of the area. For this purpose and as part of the present feasibility study, detailed topographic survey was done. topographic data was used to extract the topography of the irrigable area. This was augmented by field level surveys alone.

10.2 Project Layout

The irrigation system layout for the project was made based on the 1:10,000 scale topographic maps (with 1m contour interval) of the area prepared by the assigned crew team for this project.

The layout of irrigation canals, furrows are made to run more or less parallel to contours and field canals are aligned perpendicular to contour lines, subsequent higher-level canals were made to run perpendicular and/or near perpendicular to lower level canals that they discharge into.

The alignment procedure used for drainage canals is such that the positioning of higher-level drainage channels would more or less be along depressions. Accordingly, field ditches are placed at the foot of furrows and are aligned down the contour lines. Similarly, tertiary and secondary drains are made to run perpendicular to field ditches and to each other.

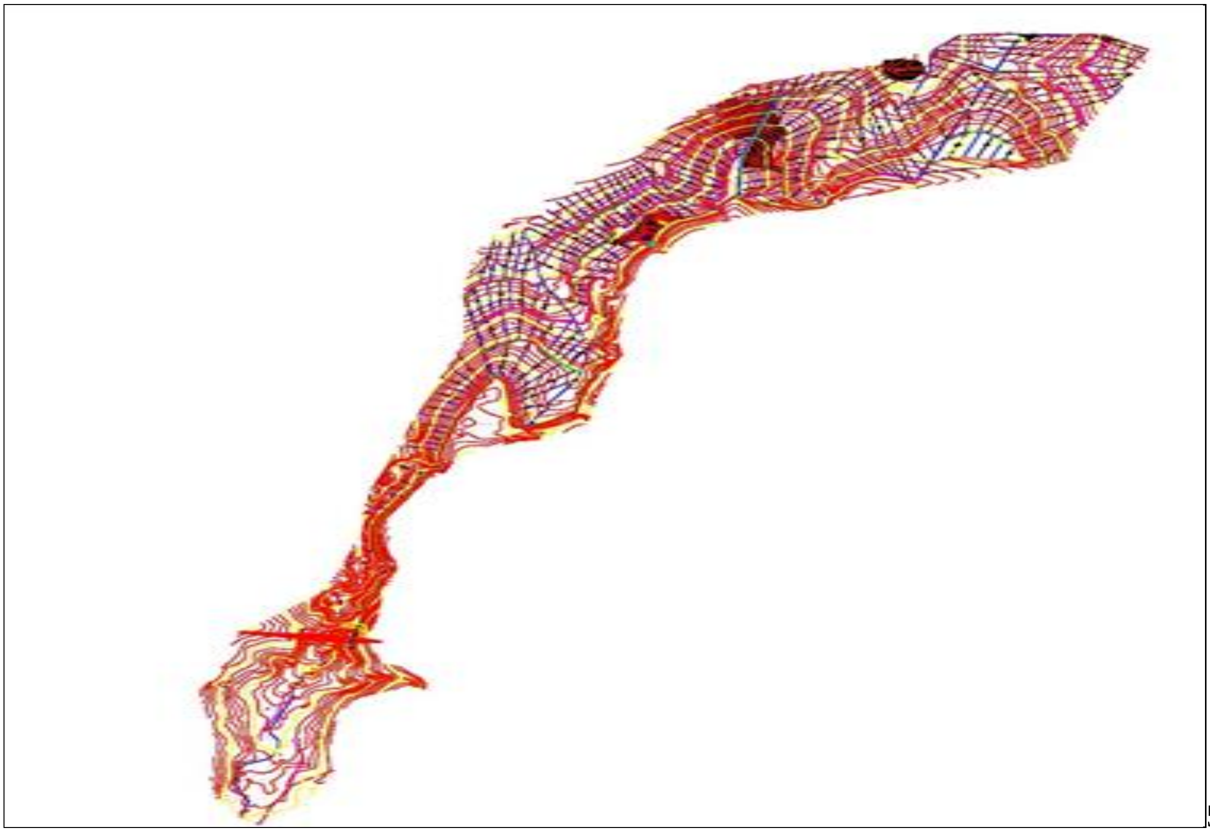
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Figure 10-1 General System layout of Worbate Small scale Irrigation project

10.3 System Nomenclature

10.3.1 *General*

The multitude irrigation and drainage canals including their hydraulic structures in the present irrigation system will call for proper designation and naming of each canal and drain their accompanying structures in a systematic way. Ultimately every drawing will be prepared in relation to the designation and naming of each canal and drain their accompanying structures. For this purpose, it will be inevitable to designate and name every canal and drain in the irrigation system in a systematic and standard manner.

10.3.2 *Irrigation Canals*

Main Canal: The main canal in the irrigation system layout of was abbreviated as MC.

Secondary Canals: Secondary canals are given the name of the area the canal conveys water to suffixed by the main canal number it takes off from hyphenated by sequential numbers designating the number of the secondary canal. Thus, the first secondary canal in the system taking off from the main canal is designated as SC-1-1.

Tertiary Canals: Tertiary canals are given the name of the area the canal conveys water to prefixed by "G" and suffixed by the main canal and secondary canal number it takes off from hyphenated by sequential numbers indicating the secondary canal it takes-off from hyphenated by sequential numbers to indicate the number of the tertiary canal. Thus, the first tertiary canal taking off from the first secondary canal that takes off from the main canal is designated as TC-1-1-1.

Filed Canals: Field canals are named by an abbreviated field canal name "FC" prefixed by “G” and suffixed by a sequence number which indicates the main canal unit they are in followed by the secondary and tertiary canal number that it takes off and the farm plot number. Thus, the first filed canal supplying water to the first farm plot in the first tertiary, secondary and main canal is designated as FC-1-1-1-1.

10.3.3 *Drainage Canals*

Tertiary Drain: Tertiary drains are given the name of the secondary drain they dewater in to from followed by sequence number to indicate the number of the tertiary. Thus, TD-1-1 indicates the first tertiary drain that dewaters in to the first collector drain that again outfalls in to the first natural drain in Worbate area.

Field Drains: Field drains are named by an abbreviated field name which is FD and the farm number from which they dewater. Thus FD-1-1-1 designates the field drain which drains the first farm field that drains in to tertiary drain number one in the first secondary unit.

10.3.4 *System Description*

10.3.4.1 General

Based on the geographic setting of the irrigable area with respect to the Worbate River, the command area for this project one main canal, three secondary canal and eighteen tertiary canals.

10.3.4.2 Irrigation sub-system

Secondary Command Units: -The entire command area at Worbate Irrigation Projects supplied with irrigation water by a main conveyor canal designated as MC and starts at the intake. The main canal is designed for **12 hrs** contentious supply.

Based on the geographic setting of the command area, is divided into three secondary command units and field command units. Each of the secondary canals and field canals will take off from the main canal to convey water to subsequent tertiary canals and command area.

Tertiary Command Units; -To facilitate a systematic classification for design and management purposes each secondary command unit is sub divided into tertiary command units. Each tertiary command unit will be a self-contained irrigation sub-system within the secondary command unit it is found in. Based on social aspects, topography, occurrence of natural drains and size of irrigable area one secondary command units and field command units are defined.

10.3.4.3 Drainage Sub-System

Drainage System; - Drainage system protect the Irrigation system such as secondary, tertiary, field canals, and irrigation land from damage, which would result from uncontrolled excess flow of irrigation water and surface runoff caused due to rainfall. Rainwater and excess irrigation water must be controlled to prevent erosion and damage of the irrigation system and the land. Rainwater and excess irrigation water should be removed safely from the irrigation land by different drainage systems. Lastly, the collected drainage water should enter the natural drainage system

To evacuate excess irrigation water and rainfall runoff a network of drainage channels are provided. At field and tertiary level, the designation, arrangement and layout of the present drainage system more or less follows the designation, arrangement and layout of the irrigation system. As each secondary command unit is a self-contained system, the higher-level drainage canal that exists in the system in this project is collector drain. In addition, the system comprises tertiary drain and field drains.

10.3.5 *Road/Access Sub-System*

10.3.5.1 Road System

The present irrigation area is provided with a network of road systems. While the main road which runs parallel to the main canal and serves as a major artery to connect the project area with the nearest Road. In this study the various types of roads within the project areas is dealt with.

The following types of roads are defined for the present system:

Secondary Roads: Secondary roads will run parallel to secondary irrigation canals and will connect tertiary Roads with the main road. Connecting roads will have 3m (2m c/c) width and 0.4m thickness.

Main Road: Main roads run alongside the main canal and are meant to link the connecting roads with the nearest national highway around 6m width. While their entire width is formed from compacted earth fill sub-base, 6m of their width will be surfaced with road base material (gravel).

10.4 Irrigation System Design

10.4.1 *Hydraulic design of supply canals*

10.4.1.1 Main Canal

Worbate small scale irrigation command area leis right side of Warbate River. The design discharge of the main canal is 315.80 l/s with free board of 30cm was provided for safety purpose and for farmers to use extra flows. The main canal is rectangular masonry canal with bed slope ranging from 0.0005 to 0.003 along its overall canal length. General formula adopted for designing of the distribution canals including main canal is Manning's formula and the procedures followed is as described below.

Table 10-1 Summary of hydraulic design of main canals

Reach Name	Station	D	B	Hyd. rad	n	S	FB	Velocity	Q	Remark
		m	m	m		m/m	m	m/s	m3/s	
MC										
Reach-1	0 .00 up to 398.58	0.45	0.90	0.225	0.018	0.0018	0.3	0.87	0.3158016	CD-1
Reach-2	398.58 up to 468.78	0.45	0.85	0.22	0.02	0.004	0.30	1.19	0.3158	FC-0-0-1
Reach-3	468.78 up to 577.61	0.45	0.85	0.22	0.02	0.003	0.30	1.10	0.3132	FC-0-0-2
Reach-4	577.61 up to 667.78	0.45	0.85	0.22	0.02	0.003	0.30	1.10	0.3122	FC-0-0-3
Reach-5	667.78 up to 700.00	0.45	0.85	0.22	0.02	0.003	0.30	1.10	0.3108	FC-0-0-4
Reach-6	700.00 up to 874.04	0.45	0.85	0.22	0.02	0.003	0.30	1.10	0.3085	FC-0-0-5
Reach-7	874.04 up to 974.91	0.45	0.85	0.22	0.02	0.003	0.30	1.10	0.3056	FC-0-0-6
Reach-8	974.91 up to 1095.96	0.45	0.85	0.22	0.02	0.003	0.30	1.10	0.3020	FC-0-0-7
Reach-9	1095.96 up to 1185.09	0.45	0.85	0.22	0.02	0.003	0.30	1.10	0.2951	CD-2
Reach-10	1185.09 up to 1319.61	0.45	0.85	0.22	0.02	0.002	0.30	0.78	0.2951	FC-0-0-8
Reach-11	1319.61 up to 1395.03	0.45	0.85	0.22	0.02	0.002	0.30	0.78	0.2913	FC-0-0-9
Reach-12	1395.03 up to 1479.48	0.45	0.85	0.22	0.02	0.002	0.30	0.78	0.2888	FC-0-0-10
Reach-13	1479.48 up to 1861.80	0.45	0.85	0.22	0.02	0.002	0.30	0.78	0.2854	SC-1
Reach-14	1861.80 up to 2288.54	0.45	0.85	0.22	0.02	0.001	0.30	0.70	0.1857	CD-3
Reach-15	2288.54 up to 2633.84	0.45	0.85	0.22	0.02	0.001	0.30	0.49	0.1857	CD-4
Reach-16	2633.84 up to 3033.90	0.40	0.70	0.19	0.02	0.001	0.30	0.68	0.1857	CD-5
Reach-17	3033.90 up to 3257.42	0.40	0.70	0.19	0.02	0.001	0.30	0.68	0.1857	CD-6
Reach-18	3257.42 up to 3822.41	0.40	0.70	0.19	0.02	0.001	0.30	0.68	0.1857	CD-7
Reach-19	3822.41 up to 3828.36	0.35	0.70	0.18	0.02	0.003	0.30	0.95	0.1857	SC-2
Reach-20	3828.36 up to 4175.63	0.35	0.60	0.16	0.02	0.003	0.30	0.90	0.1155	CD-8
Reach-21	4175.63 up to 4328.86	0.35	0.60	0.16	0.02	0.001	0.30	0.57	0.1155	SC-3
Reach-22	4328.86 up to 4425.50	0.15	0.30	0.08	0.02	0.014	0.30	1.17	0.0040	FC-0-0-11

10.4.1.2 Secondary Canals

Generally, secondary canals will flow for 12hr/day and flow will be distributed down each tertiary canal in proportion to the area they irrigate. Secondary canals run down the natural slope. There are three secondary canals are designed for Worbate small scale irrigation project. All secondary canals are lined canals aligned across the contours. The canal geometric design results for irrigation canals including related hydraulic particulars such as design discharge, flow depth, velocity etc. are given in the standard longitudinal profile drawings of each irrigation canal. Table below gives hydraulic design of secondary canals

Table 10-2 Summary of hydraulic design of secondary canals

S.No	Reach Name	Station	D	B	Hyd. rad	n	S	FB	Velocity	Q	Remark
			m	m	m		m/m	m	m/s	m3/s	
1	SC-1										
	Reach-1	14.30	0.35	0.60	0.13	0.03	0.00	0.25	0.52	0.0997	TC-1-1 & TC-1-2
	Reach-2	173.06	0.35	0.50	0.12	0.03	0.0022	0.25	0.45	0.0577	TC-1-3
	Reach-3	355.94	0.20	0.35	0.08	0.03	0.0048	0.25	0.50	0.0343	TC-1-4
	Reach-4	554.67	0.20	0.35	0.08	0.03	0.0024	0.25	0.35	0.0156	TC-1-5
2	SC-2										
	Reach-1	18.11	0.25	0.50	0.10	0.03	0.0060	0.25	0.68	0.0702	TC-2-1 & TC-2-2
	Reach-2	172.81	0.25	0.50	0.10	0.03	0.0060	0.25	0.68	0.0502	TC-2-3 & TC-2-4
	Reach-3	320.53	0.25	0.50	0.10	0.03	0.0040	0.25	0.56	0.0212	TC-2-5 & TC-2-6
3	SC-3										
	Reach-1	49.83	0.40	0.70	0.15	0.03	0.0020	0.25	0.51	0.1115	FC-3-0-1
	Reach-2	166.30	0.40	0.70	0.15	0.03	0.0012	0.25	0.40	0.1068	FC-3-0-2
	Reach-3	279.30	0.40	0.70	0.15	0.03	0.0012	0.25	0.40	0.1033	TC-3-1
	Reach-4	410.07	0.30	0.50	0.11	0.03	0.0050	0.25	0.65	0.0934	TC-3-2
	Reach-5	477.77	0.25	0.45	0.10	0.03	0.0100	0.25	0.85	0.0799	TC-3-3
	Reach-6	532.90	0.25	0.45	0.10	0.03	0.0100	0.25	0.85	0.0602	TC-3-4
	Reach-7	585.37	0.25	0.45	0.10	0.03	0.0025	0.25	0.42	0.0464	TC-3-5
	Reach-8	685.33	0.20	0.35	0.08	0.03	0.0080	0.25	0.64	0.0218	TC-3-6
	Reach-9	720.73	0.20	0.35	0.08	0.03	0.0080	0.25	0.64	0.0166	TC-3-7

Flow Regime

Secondary canals comparatively run-down steep slopes or along the ridge and in most cases the flow regime in secondary canals is supercritical. To avoid the prevalence of super-critical flow in secondary canals it was inevitable to provide drop structures where conditions permit.

10.4.1.3 Tertiary Canals

Tertiary canals will also flow for 12hr/day and flow will be distributed down each off-taking field canals in proportion to the area they irrigate. The command area of each tertiary canal is in the order of blocked hectares, on average and discharging capacity is also depending on the area of the block. The canals are earthen open channels, which are generally laid along the contours. The canal geometric design results for irrigation canals including related hydraulic particulars such as design discharge, flow depth, velocity etc. are given in the standard longitudinal profile drawings of each irrigation canal. Table below gives hydraulic design of tertiary canal.

Table 10-3 Summary of hydraulic design of tertiary canal

S.No	Reach Name	Station	D	B	Hyd. rad	n	S	FB	Velocity	Q	Remark
			m	m	m		m/m	m	m/s	m3/s	
1	TC-1-1										
	Reach-1	74	0.20	0.30	0.12	0.03	0.0020	0.15	0.42	0.0154	FC-1-1-1
	Reach-2	132	0.20	0.35	0.12	0.03	0.0010	0.15	0.31	0.0089	FC-1-1-2
	Reach-3	211	0.20	0.35	0.12	0.03	0.0012	0.15	0.34	0.0057	FC-1-1-3
	Reach-4	294	0.20	0.35	0.12	0.03	0.0010	0.15	0.31	0.0029	FC-1-1-4
2	TC-1-2										
	Reach-1	32	0.25	0.40	0.15	0.03	0.00	0.15	0.30	0.0267	FC-1-2-1
	Reach-2	118	0.20	0.35	0.12	0.03	0.00	0.15	0.41	0.0200	FC-1-2-2
	Reach-3	193	0.20	0.35	0.12	0.03	0.00	0.15	0.31	0.0125	FC-1-2-3
	Reach-4	280	0.15	0.30	0.09	0.03	0.01	0.15	0.60	0.0059	FC-1-2-4
3	TC-1-3										
	Reach-1	69	0.20	0.35	0.12	0.03	0.00	0.15	0.44	0.0233	FC-1-3-1
	Reach-2	200	0.15	0.30	0.09	0.03	0.00	0.15	0.40	0.0182	FC-1-3-2
	Reach-3	313	0.15	0.30	0.09	0.03	0.00	0.15	0.49	0.0126	FC-1-3-3
	Reach-4	417	0.15	0.30	0.09	0.03	0.00	0.15	0.45	0.0079	FC-1-3-4
	Reach-5	497	0.15	0.30	0.09	0.03	0.00	0.15	0.49	0.0033	FC-1-3-5
4	TC-1-4										
	Reach-1	51	0.20	0.35	0.12	0.03	0.00	0.15	0.31	0.0187	FC-1-4-1
	Reach-2	139	0.20	0.35	0.12	0.03	0.00	0.15	0.31	0.0140	FC-1-4-2
	Reach-3	218	0.15	0.30	0.09	0.03	0.01	0.15	0.60	0.0100	FC-1-4-3
	Reach-4	346	0.20	0.35	0.12	0.03	0.00	0.15	0.52	0.0054	FC-1-4-4
5	TC-1-5										
	Reach-1	95	0.20	0.35	0.12	0.03	0.00	0.15	0.47	0.0156	FC-1-5-1
	Reach-2	193	0.15	0.30	0.09	0.03	0.00	0.15	0.41	0.0111	FC-1-5-2
	Reach-3	544	0.15	0.30	0.09	0.03	0.00	0.15	0.41	0.0050	FC-1-5-3

Table 10-4 Summary of hydraulic design of tertiary canal

S.No	Reach Name	Station	D	B	Hyd. rad	n	S	FB	Velocity	Q	Remark
			m	m	m		m/m	m	m/s	m3/s	
6	TC-2-1										
	Reach-1	97	0.20	0.35	0.12	0.03	0.00	0.15	0.43	0.0155	FC-2-1-1
	Reach-2	205	0.15	0.30	0.09	0.03	0.00	0.15	0.42	0.0127	FC-2-1-2
	Reach-3	488	0.15	0.30	0.09	0.03	0.00	0.15	0.32	0.0083	FC-2-1-3
	Reach-4	583	0.15	0.30	0.09	0.03	0.00	0.15	0.42	0.0044	FC-2-1-4
	Reach-5	686	0.20	0.30	0.12	0.03	0.00	0.15	0.30	0.0034	FC-2-1-5
7	TC-2-2										
	Reach-1	173	0.20	0.35	0.12	0.03	0.00	0.15	0.31	0.0045	FC-2-2-1
	Reach-2	263	0.20	0.35	0.12	0.03	0.00	0.15	0.31	0.0031	FC-2-2-2
8	TC-2-3										
	Reach-1	32	0.20	0.35	0.12	0.03	0.00	0.15	0.43	0.0229	FC-2-3-1
	Reach-2	117	0.20	0.35	0.12	0.03	0.00	0.15	0.44	0.0201	FC-2-3-2
	Reach-3	189	0.20	0.35	0.12	0.03	0.00	0.15	0.52	0.0179	FC-2-3-3
	Reach-4	264	0.20	0.35	0.12	0.03	0.00	0.15	0.44	0.0149	FC-2-3-4
	Reach-5	343	0.20	0.35	0.12	0.03	0.00	0.15	0.44	0.0115	FC-2-3-5
	Reach-6	473	0.20	0.35	0.12	0.03	0.00	0.15	0.59	0.0079	FC-2-3-6
9	TC-2-4										
	Reach-1	60	0.20	0.35	0.12	0.03	0.00	0.15	0.42	0.0061	FC-2-4-1
	Reach-2	129	0.15	0.30	0.09	0.03	0.01	0.15	0.59	0.0041	FC-2-4-2
	Reach-3	212	0.15	0.30	0.09	0.03	0.01	0.15	0.59	0.0023	FC-2-4-3
10	TC-2-5										
	Reach-1	47	0.20	0.35	0.12	0.03	0.00	0.15	0.48	0.0145	FC-2-5-1
	Reach-2	125	0.20	0.35	0.12	0.03	0.00	0.15	0.48	0.0115	FC-2-5-2
	Reach-3	215	0.20	0.35	0.12	0.03	0.00	0.15	0.31	0.0084	FC-2-5-3
	Reach-4	300	0.15	0.30	0.09	0.03	0.01	0.15	0.59	0.0054	FC-2-5-4
	Reach-5	386	0.20	0.35	0.12	0.03	0.00	0.15	0.53	0.0030	FC-2-5-5
11	TC-2-6										
	Reach-1	38	0.20	0.35	0.12	0.03	0.00	0.15	0.38	0.0067	FC-2-6-1
	Reach-2	114	0.15	0.30	0.09	0.03	0.01	0.15	0.58	0.0040	FC-2-6-2
	Reach-3	187	0.15	0.30	0.09	0.03	0.00	0.15	0.45	0.0020	FC-2-6-3

Table 10-5 Summary of hydraulic design of tertiary canal

S.No	Reach Name	Station	D	B	Hyd. rad	n	S	FB	Velocity	Q	Remark
			m	m	m		m/m	m	m/s	m3/s	
12	TC-3-1										
	Reach-1	22	0.20	0.35	0.12	0.03	0.00	0.15	0.31	0.0099	FC-3-1-1
	Reach-2	100	0.20	0.35	0.12	0.03	0.00	0.15	0.31	0.0077	FC-3-1-2
	Reach-3	174	0.20	0.35	0.12	0.03	0.00	0.15	0.31	0.0058	FC-3-1-3
	Reach-4	251	0.20	0.35	0.12	0.03	0.00	0.15	0.53	0.0044	FC-3-1-4
	Reach-5	339	0.20	0.35	0.12	0.03	0.00	0.15	0.31	0.0024	FC-3-1-5
13	TC-3-2										
	Reach-1	98	0.15	0.30	0.09	0.03	0.00	0.15	0.52	0.0135	FC-3-2-1
	Reach-2	178	0.15	0.30	0.09	0.03	0.00	0.15	0.49	0.0121	FC-3-2-2
	Reach-3	282	0.20	0.35	0.12	0.03	0.00	0.15	0.34	0.0098	FC-3-2-3
	Reach-4	379	0.20	0.35	0.12	0.03	0.00	0.15	0.31	0.0071	FC-3-2-4
	Reach-5	464	0.15	0.30	0.09	0.03	0.01	0.15	0.58	0.0044	FC-3-2-5
	Reach-6	536	0.15	0.30	0.09	0.03	0.01	0.15	0.58	0.0021	FC-3-2-6
14	TC-3-3										
	Reach-1	39	0.20	0.35	0.12	0.03	0.00	0.15	0.34	0.0197	FC-3-3-1
	Reach-2	127	0.20	0.35	0.12	0.03	0.00	0.15	0.34	0.0177	FC-3-3-2
	Reach-3	215	0.15	0.30	0.09	0.03	0.00	0.15	0.37	0.0138	FC-3-3-3
	Reach-4	302	0.15	0.30	0.09	0.03	0.01	0.15	0.58	0.0109	FC-3-3-4
	Reach-5	377	0.15	0.30	0.09	0.03	0.01	0.15	0.58	0.0061	FC-3-3-5
	Reach-6	435	0.15	0.30	0.09	0.03	0.01	0.15	0.59	0.0047	FC-3-3-6
	Reach-7	562	0.15	0.30	0.09	0.03	0.00	0.15	0.32	0.0035	FC-3-3-7
15	TC-3-4										
	Reach-1	190	0.20	0.35	0.12	0.03	0.00	0.15	0.36	0.0138	FC-3-4-1
	Reach-2	265	0.20	0.35	0.12	0.03	0.00	0.15	0.36	0.0113	FC-3-4-2
	Reach-3	368	0.15	0.30	0.09	0.03	0.01	0.15	0.60	0.0085	FC-3-4-3
	Reach-4	480	0.20	0.30	0.12	0.03	0.00	0.15	0.37	0.0047	FC-3-4-4
16	TC-3-5										
	Reach-1	45	0.20	0.35	0.12	0.03	0.00	0.15	0.44	0.0246	FC-3-5-1
	Reach-2	130	0.15	0.30	0.09	0.03	0.00	0.15	0.37	0.0218	FC-3-5-2
	Reach-3	198	0.20	0.35	0.12	0.03	0.00	0.15	0.53	0.0179	FC-3-5-3
	Reach-4	270	0.15	0.30	0.09	0.03	0.01	0.15	0.58	0.0131	FC-3-5-4
	Reach-5	366	0.15	0.30	0.09	0.03	0.00	0.15	0.49	0.0088	FC-3-5-5
	Reach-6	485	0.15	0.30	0.09	0.03	0.00	0.15	0.44	0.0050	FC-3-5-6
17	TC-3-6										
	Reach-1	33	0.20	0.35	0.12	0.03	0.00	0.15	0.50	0.0052	FC-3-6-1
	Reach-2	117	0.15	0.30	0.09	0.03	0.00	0.15	0.44	0.0030	FC-3-6-2
	Reach-3	234	0.15	0.30	0.09	0.03	0.00	0.15	0.56	0.0020	FC-3-6-3
18	TC-3-7										
	Reach-1	40	0.15	0.30	0.09	0.03	0.00	0.15	0.52	0.0166	FC-3-7-1
	Reach-2	120	0.15	0.30	0.09	0.03	0.00	0.15	0.52	0.0147	FC-3-7-2
	Reach-3	209	0.15	0.30	0.09	0.03	0.00	0.15	0.45	0.0124	FC-3-7-3
	Reach-4	309	0.20	0.35	0.12	0.03	0.00	0.15	0.44	0.0097	FC-3-7-4
	Reach-5	372	0.15	0.30	0.09	0.03	0.01	0.15	0.59	0.0065	FC-3-7-5
	Reach-6	476	0.20	0.30	0.12	0.03	0.00	0.15	0.51	0.0021	FC-3-7-6

Flow Regimes

As tertiary canals are aligned nearly along contours in most cases the flow regime in tertiary canals will be sub-critical and therefore the number of drops are decreased.

1.7.2.4 Field Canals

The command area of tertiary canal will be divided in to sub blocks/fields depend on the topography. The maximum length of field canals are 200 m and the maximum furrow length is about 100m.The field channels which are earthen open channels will takeoff water from tertiary canal (incase main canal). The alignment of the field canals are across the contour on the ridge so that it can supply furrows on both sides.

10.4.2 *Drainage system*

The natural drainage available is used as the collective drains in the layout of drainage system. The design of tertiary drains is included in the project, but the construction of field drains is totally left for users since it needs to be made frequently at the beginning of seasons. Manning equation was used to design the dimension of drains. The capacities of drains are fixed using drainage module estimated in hydrology study. The cross sectional design of drainage canal is similar to that of irrigation canal hydraulic design discussed above. All drains are designed as trapezoidal earthen open ditch as shown below:

Table 10-6 Summary of hydraulic design of tertiary drain

S.No	Reach Name	Station	D	B	Hyd. rad	n	S	Velocity	FB	Q	Remark
			m	m	m		m/m	m/s	m	m3/s	
1	TD-1-1										
	Reach-1	0.00	0.15	0.30	0.09	0.03	0.003	0.36	0.25	0.0043	FD-1-2-1
	Reach-2	200.98	0.20	0.30	0.12	0.03	0.003	0.40	0.25	0.0160	FD-1-2-1/2
	Reach-3	387.57	0.20	0.35	0.12	0.03	0.002	0.36	0.25	0.0255	FD-1-2-2/3
2	TD-1-2										
	Reach-1	0.00	0.15	0.30	0.09	0.03	0.002	0.31	0.25	0.0035	FD-1-3-1
	Reach-2	132.09	0.15	0.30	0.09	0.03	0.00	0.31	0.25	0.0117	FD-1-3-1/2
	Reach-3	306.37	0.20	0.35	0.12	0.03	0.00	0.31	0.25	0.0185	FD-1-3-2/3
	Reach-4	428.68	0.20	0.35	0.12	0.03	0.00	0.36	0.25	0.0266	FD-1-3-3
3	TD-1-3										
	Reach-1	0.00	0.15	0.30	0.09	0.03	0.00	0.31	0.25	0.0031	FD-1-4-1
	Reach-2	127.01	0.15	0.30	0.09	0.03	0.00	0.31	0.25	0.0100	FD-1-4-1/2
	Reach-3	214.37	0.20	0.35	0.12	0.03	0.00	0.36	0.25	0.0159	FD-1-4-2/3
	Reach-4	352.38	0.20	0.35	0.12	0.03	0.00	0.41	0.25	0.0228	FD-1-4-3
4	TD-2-1										
	Reach-1	0.00	0.20	0.35	0.12	0.03	0.00	0.30	0.25	0.0016	FD-2-1-1
	Reach-2	223.09	0.20	0.35	0.12	0.03	0.00	0.30	0.25	0.0072	FD-2-1-1/2
	Reach-3	403.40	0.20	0.35	0.12	0.03	0.00	0.41	0.25	0.0151	FD-2-1-2/3
	Reach-4	530.54	0.25	0.45	0.15	0.03	0.00	0.31	0.25	0.0175	FD-2-1-3/4
	Reach-5	587.80	0.25	0.45	0.15	0.03	0.00	0.31	0.25	0.0195	FD-2-1-4/5
5	TD-2-2										
	Reach-1	0.00	0.200	0.350	0.120	0.030	0.002	0.363	0.250	0.0002	FD-2-2-1
	Reach-2	172.28	0.250	0.500	0.155	0.030	0.002	0.409	0.250	0.0041	FD-2-2-1/2
6	TD-2-3										
	Reach-1	0.00	0.20	0.30	0.12	0.03	0.00	0.31	0.25	0.0019	FD-2-3-1
	Reach-2	97.55	0.20	0.35	0.12	0.03	0.00	0.31	0.25	0.0056	FD-2-3-1/2
	Reach-3	175.84	0.20	0.35	0.12	0.03	0.00	0.31	0.25	0.0094	FD-2-3-2/3
	Reach-4	277.53	0.25	0.50	0.16	0.03	0.00	0.30	0.25	0.0138	FD-2-3-3/4
	Reach-5	374.51	0.25	0.50	0.16	0.03	0.00	0.41	0.25	0.0190	FD-2-3-4/5
	Reach-6	463.83	0.25	0.50	0.16	0.03	0.00	0.37	0.25	0.0264	FD-2-3-5
7	TD-2-4										
	Reach-1	0.00	0.25	0.45	0.15	0.03	0.00	0.31	0.25	0.0015	FD-2-4-1
	Reach-2	44.32	0.25	0.50	0.16	0.03	0.00	0.30	0.25	0.0044	FD-2-4-1/2
	Reach-3	111.02	0.25	0.50	0.16	0.03	0.00	0.30	0.25	0.0079	FD-2-4-2/3

Table 10-7 Summary of hydraulic design of tertiary drain

S.No	Reach Name	Station	D	B	Hyd. rad	n	S	Velocity	FB	Q	Remark
			m	m	m		m/m	m/s	m	m3/s	
8	TD-3-1										
	Reach-1	0.00	0.20	0.30	0.12	0.03	0.00	0.31	0.25	0.0015	FD-3-1-1
	Reach-2	132.40	0.20	0.35	0.12	0.03	0.00	0.30	0.25	0.0050	FD-3-1-1/2
	Reach-3	259.75	0.25	0.40	0.15	0.03	0.00	0.32	0.25	0.0070	FD-3-1-2/3
	Reach-4	365.13	0.25	0.45	0.15	0.03	0.00	0.33	0.25	0.0097	FD-3-1-3/4
	Reach-5	473.09	0.25	0.45	0.15	0.03	0.00	0.33	0.25	0.0128	FD-3-1-4/5
9	TD-3-2*										
	Reach-1	0.00	0.20	0.35	0.12	0.03	0.00	0.30	0.25	0.0037	FD-3-2*-1
	Reach-2	418.01	0.25	0.50	0.16	0.03	0.00	0.30	0.25	0.0104	FD-3-2*-1/2
	Reach-3	514.55	0.25	0.50	0.16	0.03	0.00	0.30	0.25	0.0168	FD-3-2*-2/3
10	TD-3-2										
	Reach-1	0.00	0.20	0.30	0.12	0.03	0.00	0.40	0.25	0.0009	FD-3-2-1
	Reach-2	227.60	0.20	0.30	0.12	0.03	0.00	0.31	1.25	0.0037	FD-3-2-1/2
	Reach-3	337.58	0.20	0.35	0.12	0.03	0.00	0.31	0.25	0.0076	FD-3-2-2/3
	Reach-4	423.45	0.25	0.50	0.16	0.03	0.00	0.43	0.25	0.0114	FD-3-2-3/4
	Reach-5	503.16	0.25	0.50	0.16	0.03	0.00	0.53	0.25	0.0154	FD-3-2-4/5
	Reach-6	570.04	0.25	0.50	0.16	0.03	0.00	0.53	0.25	0.0189	FD-3-2-5
11	TD-3-3										
	Reach-1	0.00	0.15	0.30	0.09	0.03	0.00	0.31	0.25	0.0004	FD-3-3-1
	Reach-2	150.57	0.20	0.30	0.12	0.03	0.00	0.42	0.25	0.0059	FD-3-3-1/2
	Reach-3	275.93	0.20	0.35	0.12	0.03	0.00	0.36	0.25	0.0116	FD-3-3-2/3
	Reach-4	425.66	0.20	0.35	0.12	0.03	0.00	0.31	0.25	0.0148	FD-3-3-3/4
	Reach-5	520.13	0.25	0.45	0.15	0.03	0.00	0.50	0.25	0.0174	FD-3-3-4/5
	Reach-6	583.21	0.25	0.45	0.15	0.03	0.00	0.56	0.25	0.0192	FD-3-3-5/6
12	TD-3-4										
	Reach-1	0.00	0.20	0.35	0.12	0.03	0.00	0.30	0.25	0.0019	FD-3-4-1
	Reach-2	358.73	0.20	0.35	0.12	0.03	0.00	0.30	0.25	0.0048	FD-3-4-1/2
13	TD-3-5										
	Reach-1	0.00	0.15	0.30	0.09	0.03	0.00	0.31	0.25	0.0017	FD-3-5-1
	Reach-2	113.30	0.20	0.35	0.12	0.03	0.00	0.41	0.25	0.0054	FD-3-5-1/2
	Reach-3	266.63	0.20	0.35	0.12	0.03	0.00	0.44	0.25	0.0129	FD-3-5-2/3
	Reach-4	397.83	0.20	0.35	0.12	0.03	0.00	0.41	0.25	0.0192	FD-3-5-3/4

11 Irrigation Infrastructures

11.1 General

The selection and design of irrigation infrastructures is primarily dictated by various factors among which are economy, safety and ease of operation and maintenance. By and large the following factors were considered to select the required structure in the present system:

- Operation and maintenance of the structures should be simple
- Experience obtained from on-going similar construction work in the area was considered
- The structures were evaluated from safety perspectives, such that the system be safe from unforeseen damages
- Availability of construction material in the vicinity of the project site was considered

11.2 Types of Irrigation Infrastructures

The type and nature of irrigation infrastructures that are adopted in the present system are varied. In general, the irrigation infrastructures that are found in the present system are off takes, division box, drops and cross drainage (CD) structure.

The detail designs of these structures are described in the following section and their typical drawings described in the drawing album and dimensions provided in appendix table.

Table 11-1 Summary of infrastructure structures

S.No	Canal name	Drop	Cross drainage	Off take	Division box
1	MC	7	8	14	
2	SC	42		12	4
3	TC			81	
4	TD				
TC TOTAL		49	8	107	4

11.2.1 *Off take Structures*

The offtake structures are made simple pipe off takes with a suppressed rectangular inlet part followed by a rectangular bay at the pipe inlet and a masonry guide walls at the pipe outlet. All pipes are circular reinforced concrete pipes with standard dimensions. At the upstream end gates are provide to control the flow. All the dimensions are shown on typical drawing and Table below shows the hydraulic design. See detail table of dimension and quantity on excels sheet.

Table 11-2 Summary table of dimension Main canal off take

S.No	Parameters	Unit	FC-0-0-1	FC-0-0-2	FC-0-0-3	FC-0-0-4	FC-0-0-5	FC-0-0-6	FC-0-0-7	FC-0-0-8	FC-0-0-9	FC-0-0-10	SC-1	SC-2	SC-3	FC-0-0-11
	Supply canal		469	578	668	700	874	975	1096	1320	1395	1479	1862	3828	4329	4426
1	Full Supply Level	m	1459.90	1459.57	1459.30	1459.20	1458.68	1458.38	1457.02	1455.05	1454.94	1454.81	1454.24	1442.02	1440.79	1437.72
2	Full Supply Depth	m	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.35	0.35	0.15
3	Free Board	m	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30
4	Side Slope	m/m	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	Top Bank Level	m	1460.20	1459.87	1459.60	1459.50	1458.98	1458.68	1457.32	1455.35	1455.24	1455.11	1454.54	1442.32	1441.09	1438.02
6	Top Bank Width	m	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.70	0.60	0.30
7	Minimum Earth Cover Over Precast Non Pressure Pipe	m														
	Offtaking Canal															
8	Design discharge of Turnout	m^3/s	0.0026	0.0010	0.0013	0.0023	0.0029	0.0036	0.0069	0.0038	0.0025	0.0034	0.1089	0.0854	0.1432	0.0040
9	Full Supply Level	m	1459.21	1459.26	1458.71	1458.62	1458.53	1457.90	1456.55	1454.90	1454.79	1454.68	1454.10	1440.26	1440.61	1436.79
10	Full Supply Depth	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	13.80	0.25	0.40	0.15
11	Free Board	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.25	0.25	0.25	0.15
12	Side Slope	m/m	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	0.00	0.00	1.00
13	Top Bank Level	m	1459.36	1459.41	1458.86	1458.77	1458.68	1458.05	1456.70	1455.05	1454.94	1454.83	1454.35	1440.51	1440.86	1436.94
14	Top Bank Width	m	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.60	0.50	0.70	0.30
11	Wall thickness of precast chamber	m	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
12	Width of Cast in Situ Concrete Head Chamber	m	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
13	Internal Height of Cast in Situ Concrete Head Chamber	m	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	14.25	0.70	0.85	0.50
14	Internal Length of Cast in Situ Caoncrete Head Chamber	m	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	21.38	1.05	1.28	0.75
15	Width of Cast in Situ Concrete D/S Chamber	m	0.86	0.87	0.88	0.89	0.90	0.91	0.92	0.94	0.95	0.96	0.97	1.03	1.05	1.06
16	Internal Height of Cast in Situ Concrete D/S Chamber	m	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	14.25	0.70	0.85	0.50
17	Internal Length of Cast in Situ Caoncrete D/S Chamber	m	0.86	0.87	0.88	0.89	0.90	0.91	0.92	0.94	0.95	0.96	0.97	1.03	1.05	1.06
18	Bed Width of delivery Channel	m	25.85	26.85	27.85	28.85	29.85	30.85	31.85	33.85	34.85	35.85	36.85	42.85	44.85	45.85
19	Exit Width (Length of Weir)	m	25.85	26.85	27.85	28.85	29.85	30.85	31.85	33.85	34.85	35.85	36.85	42.85	44.85	45.85
	Orifice turnout															
20	Length of Pipes	m	25.90	26.90	27.90	28.90	29.90	30.90	31.90	33.90	34.90	35.90	36.90	42.90	44.90	45.90
21	Internal diameter of pipe	m	0.26	0.27	0.28	0.29	0.30	0.31	0.32	0.34	0.35	0.36	0.37	0.43	0.45	0.46
23	Width of Orifice gate	m	0.46	0.47	0.48	0.49	0.50	0.51	0.52	0.54	0.55	0.56	0.57	0.63	0.65	0.66
24	Height of orifice gate	m	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	14.15	0.60	0.75	0.40
25	Elevation of top pipe	m	1459.61	1459.29	1459.03	1458.94	1458.43	1458.14	1456.79	1454.84	1454.74	1454.62	1454.06	1442.00	1440.79	1437.93
26	Elevation of bottom pipe	m	1459.35	1459.02	1458.75	1458.65	1458.13	1457.83	1456.47	1454.50	1454.39	1454.26	1453.69	1441.57	1440.34	1437.47
27	Elevation of turnout floor	m	1458.86	1458.91	1458.36	1458.27	1458.18	1457.55	1456.20	1454.55	1454.44	1454.33	1440.10	1439.81	1440.01	1436.44
28	Maximum Elevation at Top D/S Vertical Wall of D/S Chamber	m	1459.36	1459.41	1458.86	1458.77	1458.68	1458.05	1456.70	1455.05	1454.94	1454.83	1454.35	1440.51	1440.86	1436.94
29	Elevation top of Pipe at Outlet Head Wall	m	1459.11	1459.16	1458.61	1458.52	1458.43	1457.80	1456.45	1454.80	1454.69	1454.58	1454.00	1440.16	1440.51	1436.69
30	Elevation Pipe Invert at Outlet	m	1458.85	1458.89	1458.33	1458.23	1458.13	1457.49	1456.13	1454.46	1454.34	1454.22	1453.63	1439.73	1440.06	1436.23

Table 11-3 Summary table of dimension secondary canal off take

S.No	Parameters	Symbol		TC-1-3	TC-1-4	TC-1-5	FC-3-0-1	FC-3-0-2	TC-3-1	TC-3-2	TC-3-3	TC-3-4	TC-3-5	TC-3-6	TC-3-7
	Supply canal	**Chainage**		173.06	355.94	554.67	49.83	166.30	279.30	410.07	477.77	532.90	585.37	685.33	720.73
1	Full Supply Level	FSL	m	1446.91	1438.89	1432.41	1440.51	1440.37	1439.49	1435.49	1431.76	1428.21	1425.48	1420.38	1418.24
2	Full Supply Depth	FSD	m	0.35	0.20	0.20	0.40	0.40	0.40	0.30	0.25	0.25	0.25	0.20	0.20
3	Free Board	FB	m	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
4	Side Slope	SS	m/m	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	Top Bank Level	TBL	m	1447.16	1439.14	1432.66	1440.76	1440.62	1439.74	1435.74	1432.01	1428.46	1425.73	1420.63	1418.49
6	Top Bank Width	TBW	m	0.50	0.35	0.35	0.70	0.70	0.70	0.50	0.45	0.45	0.45	0.35	0.35
7	Minimum Earth Cover Over Precast Non Pressure Pipe	Cmin	m												
	Offtaking Canal														
8	Design discharge of Turnout	Q_{ot}	m^3/s	0.05	0.03	0.05	0.004736	0.0034816	0.03	0.04	0.037	0.04	0.048	0.055	0.036
9	Full Supply Level	FSL	m	1446.68	1438.57	1432.25	1440.05	1440.23	1439.37	1435.30	1431.60	1428.00	1425.35	1420.23	1418.08
10	Full Supply Depth	FSD	m	0.20	0.20	0.20	0.15	0.15	0.20	0.15	0.20	0.20	0.20	0.20	0.15
11	Free Board	FB	m	0.25	0.25	0.25	0.15	0.15	0.25	0.25	0.25	0.25	0.25	0.25	0.25
12	Side Slope	SS	m/m	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
13	Top Bank Level	TBL	m	1446.93	1438.82	1432.50	1440.20	1440.38	1439.62	1435.55	1431.85	1428.25	1425.60	1420.48	1418.33
14	Top Bank Width	TBW	m	0.35	0.35	0.35	0.30	0.30	0.35	0.30	0.35	0.35	0.35	0.35	0.30
11	Wall thickness of precast chamber	0.2	m	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
12	Width of Cast in Situ Concrete Head Chamber	0.7	m	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
13	Internal Height of Cast in Situ Concrete Head Chamber	H_Cham_U/S	m	0.65	0.65	0.65	0.50	0.50	0.65	0.60	0.65	0.65	0.65	0.65	0.60
14	Internal Length of Cast in Situ Caoncrete Head Chamber	L_Cham_U/S	m	0.98	0.98	0.98	0.75	0.75	0.98	0.90	0.98	0.98	0.98	0.98	0.90
15	Width of Cast in Situ Concrete D/S Chamber	W_Cham_D/S	m	0.80	0.83	0.90	0.75	0.75	0.75	0.75	0.76	0.77	0.78	0.79	0.80
16	Internal Height of Cast in Situ Concrete D/S Chamber	H_Cham_D/S	m	0.65	0.65	0.65	0.50	0.50	0.65	0.60	0.65	0.65	0.65	0.65	0.60
17	Internal Length of Cast in Situ Caoncrete D/S Chamber	L_Cham_D/S	m	0.80	0.83	0.90	0.75	0.75	0.75	0.75	0.76	0.77	0.78	0.79	0.80
18	Bed Width of delivery Channel	BW_{DC}	m	0.94	0.94	0.94	0.94	0.94	0.94	0.94	1.94	2.94	3.94	4.94	5.94
19	Exit Width (Length of Weir)	EW	m	0.94	0.94	0.94	0.94	0.94	0.94	0.94	1.94	2.94	3.94	4.94	5.94
	Orifice turnout														
20	Length of Pipes	L_pipe	m	0.90	0.90	0.90	0.90	0.90	0.90	0.90	1.90	2.90	3.90	4.90	5.90
21	Internal diameter of pipe	PID	m	0.2	0.225	0.3	0.15	0.15	0.15	0.15	0.16	0.17	0.18	0.19	0.20
23	Width of Orifice gate	WOG	m	0.40	0.43	0.50	0.35	0.35	0.35	0.35	0.36	0.37	0.38	0.39	0.40
24	Height of orifice gate	HOG	m	0.55	0.55	0.55	0.40	0.40	0.55	0.50	0.55	0.55	0.55	0.55	0.50
25	Elevation of top pipe	RL_entry_top	m	1446.66	1438.81	1432.41	1440.16	1440.02	1439.14	1435.24	1431.57	1428.03	1425.31	1420.27	1418.14
26	Elevation of bottom pipe	RL_entry_bot	m	1446.46	1438.59	1432.11	1440.01	1439.87	1438.99	1435.09	1431.41	1427.86	1425.13	1420.08	1417.94
27	Elevation of turnout floor	RL_entry_tof	m	1446.28	1438.17	1431.85	1439.70	1439.88	1438.97	1434.95	1431.20	1427.60	1424.95	1419.83	1417.73
28	Maximum Elevation at Top D/S Vertical Wall of D/S Chamber	RL_exist_crest	m	1446.93	1438.82	1432.50	1440.20	1440.38	1439.62	1435.55	1431.85	1428.25	1425.60	1420.48	1418.33
29	Elevation top of Pipe at Outlet Head Wall	RL_exit_top	m	1446.58	1438.47	1432.15	1439.95	1440.13	1439.27	1435.20	1431.50	1427.90	1425.25	1420.13	1417.98
30	Elevation Pipe Invert at Outlet	RL_exit_bot	m	1446.38	1438.24	1431.85	1439.80	1439.98	1439.12	1435.05	1431.34	1427.73	1425.07	1419.94	1417.78

Table 11-4 Summary table of dimension tertiary canal off take

S.No	Parameters	Off take canal		FC-1-1-1	FC-1-1-2	FC-1-1-3	FC-1-1-4	FC-1-2-1	FC-1-2-2	FC-1-2-3	FC-1-2-4	FC-1-3-1	FC-1-3-2	FC-1-3-3	FC-1-3-4	FC-1-3-5	FC-1-4-1	FC-1-4-2	FC-1-4-3	FC-1-4-4	FC-1-5-1	FC-1-5-2	FC-1-5-3
	Supply canal	**Chainage**		74.28	131.66	210.70	294.19	31.92	118.10	192.94	279.77	68.73	200.25	313.25	417.46	496.75	50.83	139.36	218.42	346.04	95.07	192.68	544.11
1	Full Supply Level	FSL	m	1453.48	1453.42	1453.33	1453.24	1453.90	1453.69	1453.62	1453.10	1446.54	1446.17	1445.78	1445.47	1445.19	1438.52	1438.43	1437.96	1437.65	1432.03	1431.74	1430.86
2	Full Supply Depth	FSD	m	0.20	0.20	0.20	0.20	0.25	0.20	0.20	0.15	0.20	0.15	0.15	0.15	0.15	0.20	0.20	0.15	0.20	0.20	0.15	0.15
3	Free Board	FB	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
4	Side Slope	SS	m/m	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
5	Top Bank Level	TBL	m	1453.63	1453.57	1453.48	1453.39	1454.05	1453.84	1453.77	1453.25	1446.69	1446.32	1445.93	1445.62	1445.34	1438.67	1438.58	1438.11	1437.80	1432.18	1431.89	1431.01
6	Top Bank Width	TBW	m	0.30	0.35	0.35	0.35	0.40	0.35	0.35	0.30	0.35	0.30	0.30	0.30	0.30	0.35	0.35	0.30	0.35	0.35	0.30	0.30
7	Minimum Earth Cover Over Precast Non Pressure Pipe	Cmin	m																				
	Offtaking Canal																						
8	Design discharge of Turnout	Q_{ot}	m^3/s	0.02	0.00	0.01	0.01	0.00	0.02	0.00	0.01	0.03	0.01	0.01	0.01	0.01	0.01	0.01	0.00	0.01	0.01	0.03	0.01
9	Full Supply Level	FSL	m	1453.33	1453.27	1453.18	1453.09	1453.75	1453.54	1453.47	1452.95	1446.39	1446.02	1445.63	1445.32	1445.04	1438.37	1438.28	1437.81	1437.50	1431.88	1431.59	1430.71
10	Full Supply Depth	FSD	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
11	Free Board	FB	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
12	Side Slope	SS	m/m	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
13	Top Bank Level	TBL	m	1453.48	1453.42	1453.33	1453.24	1453.90	1453.69	1453.62	1453.10	1446.54	1446.17	1445.78	1445.47	1445.19	1438.52	1438.43	1437.96	1437.65	1432.03	1431.74	1430.86
14	Top Bank Width	TBW	m	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30
11	Wall thickness of precast chamber	0.20	m	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
12	Width of Cast in Situ Concrete Head Chamber	0.70	m	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
13	Internal Height of Cast in Situ Concrete Head Chamber	H_Cham_U/S	m	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
14	Internal Length of Cast in Situ Caoncrete Head Chamber	L_Cham_U/S	m	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
15	Width of Cast in Situ Concrete D/S Chamber	W_Cham_D/S	m	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
16	Internal Height of Cast in Situ Concrete D/S Chamber	H_Cham_D/S	m	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
17	Internal Length of Cast in Situ Caoncrete D/S Chamber	L_Cham_D/S	m	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
18	Bed Width of delivery Channel	BW_{DC}	m	0.30	1.30	2.30	3.30	4.30	5.30	6.30	7.30	8.30	9.30	10.30	11.30	12.30	13.30	14.30	15.30	16.30	17.30	18.30	19.30
19	Exit Width (Length of Weir)	EW	m	0.30	1.30	2.30	3.30	4.30	5.30	6.30	7.30	8.30	9.30	10.30	11.30	12.30	13.30	14.30	15.30	16.30	17.30	18.30	19.30
	Orifice turnout																						
20	Length of Pipes	L_pipe	m	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
21	Internal diameter of pipe	PID	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
23	Width of Orifice gate	WOG	m	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35
24	Height of orifice gate	HOG	m	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40
25	Elevation of top pipe	RL_entry_top	m	1453.33	1453.27	1453.18	1453.10	1453.70	1453.54	1453.47	1453.01	1446.39	1446.08	1445.68	1445.37	1445.09	1438.37	1438.28	1437.86	1437.50	1431.88	1431.64	1430.76
26	Elevation of bottom pipe	RL_entry_bot	m	1453.18	1453.12	1453.03	1452.94	1453.55	1453.39	1453.32	1452.85	1446.24	1445.92	1445.53	1445.22	1444.94	1438.22	1438.13	1437.71	1437.35	1431.73	1431.49	1430.61
27	Elevation of turnout floor	RL_entry_tof	m	1452.98	1452.92	1452.83	1452.74	1453.40	1453.19	1453.12	1452.60	1446.04	1445.67	1445.28	1444.97	1444.69	1438.02	1437.93	1437.46	1437.15	1431.53	1431.24	1430.36
28	Maximum Elevation at Top D/S Vertical Wall of D/S Chamber	RL_exist_crest	m	1453.48	1453.42	1453.33	1453.24	1453.90	1453.69	1453.62	1453.10	1446.54	1446.17	1445.78	1445.47	1445.19	1438.52	1438.43	1437.96	1437.65	1432.03	1431.74	1430.86
29	Elevation top of Pipe at Outlet Head Wall	RL_exit_top	m	1453.23	1453.17	1453.08	1452.99	1453.65	1453.44	1453.37	1452.85	1446.29	1445.92	1445.53	1445.22	1444.94	1438.27	1438.18	1437.71	1437.40	1431.78	1431.49	1430.61
30	Elevation Pipe Invert at Outlet	RL_exit_bot	m	1453.08	1453.02	1452.93	1452.84	1453.49	1453.29	1453.21	1452.70	1446.14	1445.77	1445.38	1445.06	1444.79	1438.12	1438.03	1437.56	1437.25	1431.63	1431.34	1430.46

Table 11-5 Summary table of dimension tertiary canal off take

S.No	Parameters	Off take canal		FC-2-1-1	FC-2-1-2	FC-2-1-3	FC-2-1-4	FC-2-1-5	FC-2-2-1	FC-2-2-2	FC-2-3-1	FC-2-3-2	FC-2-3-3	FC-2-3-4	FC-2-3-5	FC-2-3-6	FC-2-4-1	FC-2-4-2	FC-2-4-3	FC-2-5-1	FC-2-5-2	FC-2-5-3	FC-2-5-4
	Supply canal	**Chainage**		97.22	204.99	487.83	583.28	686.06	173.21	262.51	32.01	117.19	189.30	263.99	343.47	473.25	59.88	128.60	212.18	46.93	125.44	214.95	299.80
1	Full Supply Level	FSL	m	1439.55	1439.22	1438.80	1438.55	1438.50	1439.83	1439.74	1434.93	1434.76	1434.56	1434.41	1434.25	1433.77	1434.88	1434.48	1434.05	1428.82	1428.63	1428.54	1428.05
2	Full Supply Depth	FSD	m	0.20	0.15	0.15	0.15	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.15	0.15	0.20	0.20	0.20	0.15
3	Free Board	FB	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
4	Side Slope	SS	m/m	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
5	Top Bank Level	TBL	m	1439.70	1439.37	1438.95	1438.70	1438.65	1439.98	1439.89	1435.08	1434.91	1434.71	1434.56	1434.40	1433.92	1435.03	1434.63	1434.20	1428.97	1428.78	1428.69	1428.20
6	Top Bank Width	TBW	m	0.35	0.30	0.30	0.30	0.30	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.30	0.30	0.35	0.35	0.35	0.30
7	Minimum Earth Cover Over Precast Non Pressure Pipe	Cmin	m																				
	Offtaking Canal																						
8	Design discharge of Turnout	Q_{ot}	m^3/s	0.01	0.04	0.01	0.01	0.01	0.03	0.01	0.01	0.01	0.01	0.06	0.01	0.00	0.02	0.02	0.02	0.03	0.01	0.01	0.03
9	Full Supply Level	FSL	m	1439.40	1439.07	1438.65	1438.40	1438.35	1439.68	1439.59	1434.78	1434.61	1434.41	1434.26	1434.10	1433.62	1434.73	1434.33	1433.90	1428.67	1428.48	1428.39	1427.90
10	Full Supply Depth	FSD	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
11	Free Board	FB	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
12	Side Slope	SS	m/m	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
13	Top Bank Level	TBL	m	1439.55	1439.22	1438.80	1438.55	1438.50	1439.83	1439.74	1434.93	1434.76	1434.56	1434.41	1434.25	1433.77	1434.88	1434.48	1434.05	1428.82	1428.63	1428.54	1428.05
14	Top Bank Width	TBW	m	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30
11	Wall thickness of precast chamber	0.20	m	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
12	Width of Cast in Situ Concrete Head Chamber	0.70	m	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
13	Internal Height of Cast in Situ Concrete Head Chamber	H_Cham_U/S	m	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
14	Internal Length of Cast in Situ Caoncrete Head Chamber	L_Cham_U/S	m	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
15	Width of Cast in Situ Concrete D/S Chamber	W_Cham_D/S	m	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
16	Internal Height of Cast in Situ Concrete D/S Chamber	H_Cham_D/S	m	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
17	Internal Length of Cast in Situ Caoncrete D/S Chamber	L_Cham_D/S	m	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
18	Bed Width of delivery Channel	BW_{DC}	m	20.30	21.30	22.30	23.30	24.30	25.30	26.30	27.30	28.30	29.30	30.30	31.30	32.30	33.30	34.30	35.30	36.30	37.30	38.30	39.30
19	Exit Width (Length of Weir)	EW	m	20.30	21.30	22.30	23.30	24.30	25.30	26.30	27.30	28.30	29.30	30.30	31.30	32.30	33.30	34.30	35.30	36.30	37.30	38.30	39.30
	Orifice turnout																						
20	Length of Pipes	L_pipe	m	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
21	Internal diameter of pipe	PID	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
23	Width of Orifice gate	WOG	m	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35
24	Height of orifice gate	HOG	m	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40
25	Elevation of top pipe	RL_entry_top	m	1439.40	1439.12	1438.70	1438.45	1438.35	1439.68	1439.59	1434.78	1434.61	1434.41	1434.26	1434.10	1433.62	1434.73	1434.38	1433.96	1428.67	1428.48	1428.39	1427.95
26	Elevation of bottom pipe	RL_entry_bot	m	1439.25	1438.97	1438.55	1438.30	1438.20	1439.53	1439.44	1434.63	1434.46	1434.26	1434.11	1433.95	1433.47	1434.58	1434.23	1433.80	1428.52	1428.33	1428.24	1427.80
27	Elevation of turnout floor	RL_entry_tof	m	1439.05	1438.72	1438.30	1438.05	1438.00	1439.33	1439.24	1434.43	1434.26	1434.06	1433.91	1433.75	1433.27	1434.38	1433.98	1433.55	1428.32	1428.13	1428.04	1427.55
28	Maximum Elevation at Top D/S Vertical Wall of D/S Chamber	RL_exist_crest	m	1439.55	1439.22	1438.80	1438.55	1438.50	1439.83	1439.74	1434.93	1434.76	1434.56	1434.41	1434.25	1433.77	1434.88	1434.48	1434.05	1428.82	1428.63	1428.54	1428.05
29	Elevation top of Pipe at Outlet Head Wall	RL_exit_top	m	1439.30	1438.97	1438.55	1438.30	1438.25	1439.58	1439.49	1434.68	1434.51	1434.31	1434.16	1434.00	1433.52	1434.63	1434.23	1433.80	1428.57	1428.38	1428.29	1427.80
30	Elevation Pipe Invert at Outlet	RL_exit_bot	m	1439.15	1438.82	1438.39	1438.15	1438.09	1439.43	1439.34	1434.53	1434.36	1434.16	1434.01	1433.85	1433.37	1434.48	1434.08	1433.65	1428.42	1428.23	1428.14	1427.65

Table 11-6 Summary table of dimension tertiary canal off take

S.No	Parameters	Off take canal		FC-2-5-5	FC-2-6-1	FC-2-6-2	FC-2-6-3	FC-3-1-1	FC-3-1-2	FC-3-1-3	FC-3-1-4	FC-3-1-5	FC-3-2-1	FC-3-2-2	FC-3-2-3	FC-3-2-4	FC-3-2-5	FC-3-2-6	FC-3-3-1	FC-3-3-2	FC-3-3-3	FC-3-3-4	FC-3-3-5
	Supply canal	**Chainage**		386.49	37.69	113.82	186.55	21.83	99.86	174.11	251.07	339.17	97.78	178.12	281.80	379.47	463.93	535.80	38.60	127.09	215.34	301.82	376.83
1	Full Supply Level	FSL	m	1427.84	1428.39	1427.96	1427.75	1439.35	1439.27	1439.20	1438.97	1438.88	1434.91	1434.63	1434.56	1434.46	1433.99	1433.63	1431.56	1431.45	1431.23	1430.79	1430.40
2	Full Supply Depth	FSD	m	0.20	0.20	0.15	0.15	0.20	0.20	0.20	0.20	0.20	0.15	0.15	0.20	0.20	0.15	0.15	0.20	0.20	0.15	0.15	0.15
3	Free Board	FB	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
4	Side Slope	SS	m/m	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
5	Top Bank Level	TBL	m	1427.99	1428.54	1428.11	1427.90	1439.50	1439.42	1439.35	1439.12	1439.03	1435.06	1434.78	1434.71	1434.61	1434.14	1433.78	1431.71	1431.60	1431.38	1430.94	1430.55
6	Top Bank Width	TBW	m	0.35	0.35	0.30	0.30	0.35	0.35	0.35	0.35	0.35	0.30	0.30	0.35	0.35	0.30	0.30	0.35	0.35	0.30	0.30	0.30
7	Minimum Earth Cover Over Precast Non Pressure Pipe	Cmin	m																				
	Offtaking Canal																						
8	Design discharge of Turnout	Q_{ot}	m^3/s	0.00	0.01	0.03	0.01	0.01	0.01	0.03	0.01	0.01	0.00	0.02	0.01	0.01	0.04	0.02	0.02	0.01	0.01	0.01	0.00
9	Full Supply Level	FSL	m	1427.69	1428.24	1427.81	1427.60	1439.20	1439.12	1439.05	1438.82	1438.73	1434.76	1434.48	1434.41	1434.31	1433.84	1433.48	1431.41	1431.30	1431.08	1430.64	1430.25
10	Full Supply Depth	FSD	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
11	Free Board	FB	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
12	Side Slope	SS	m/m	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
13	Top Bank Level	TBL	m	1427.84	1428.39	1427.96	1427.75	1439.35	1439.27	1439.20	1438.97	1438.88	1434.91	1434.63	1434.56	1434.46	1433.99	1433.63	1431.56	1431.45	1431.23	1430.79	1430.40
14	Top Bank Width	TBW	m	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30
11	Wall thickness of precast chamber	0.20	m	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
12	Width of Cast in Situ Concrete Head Chamber	0.70	m	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
13	Internal Height of Cast in Situ Concrete Head Chamber	H_Cham_U/S	m	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
14	Internal Length of Cast in Situ Caoncrete Head Chamber	L_Cham_U/S	m	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
15	Width of Cast in Situ Concrete D/S Chamber	W_Cham_D/S	m	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
16	Internal Height of Cast in Situ Concrete D/S Chamber	H_Cham_D/S	m	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
17	Internal Length of Cast in Situ Caoncrete D/S Chamber	L_Cham_D/S	m	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
18	Bed Width of delivery Channel	BW_{DC}	m	40.30	41.30	42.30	43.30	44.30	45.30	46.30	47.30	48.30	49.30	50.30	51.30	52.30	53.30	54.30	55.30	56.30	57.30	58.30	59.30
19	Exit Width (Length of Weir)	EW	m	40.30	41.30	42.30	43.30	44.30	45.30	46.30	47.30	48.30	49.30	50.30	51.30	52.30	53.30	54.30	55.30	56.30	57.30	58.30	59.30
	Orifice turnout																						
20	Length of Pipes	L_pipe	m	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
21	Internal diameter of pipe	PID	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
23	Width of Orifice gate	WOG	m	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35
24	Height of orifice gate	HOG	m	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40
25	Elevation of top pipe	RL_entry_top	m	1427.69	1428.25	1427.86	1427.65	1439.20	1439.12	1439.05	1438.82	1438.73	1434.81	1434.53	1434.41	1434.31	1433.89	1433.53	1431.41	1431.30	1431.13	1430.69	1430.30
26	Elevation of bottom pipe	RL_entry_bot	m	1427.54	1428.09	1427.71	1427.50	1439.05	1438.97	1438.90	1438.67	1438.58	1434.66	1434.38	1434.26	1434.16	1433.74	1433.38	1431.26	1431.15	1430.98	1430.54	1430.15
27	Elevation of turnout floor	RL_entry_tof	m	1427.34	1427.89	1427.46	1427.25	1438.85	1438.77	1438.70	1438.47	1438.38	1434.41	1434.13	1434.06	1433.96	1433.49	1433.13	1431.06	1430.95	1430.73	1430.29	1429.90
28	Maximum Elevation at Top D/S Vertical Wall of D/S Chamber	RL_exist_crest	m	1427.84	1428.39	1427.96	1427.75	1439.35	1439.27	1439.20	1438.97	1438.88	1434.91	1434.63	1434.56	1434.46	1433.99	1433.63	1431.56	1431.45	1431.23	1430.79	1430.40
29	Elevation top of Pipe at Outlet Head Wall	RL_exit_top	m	1427.59	1428.14	1427.71	1427.50	1439.10	1439.02	1438.95	1438.72	1438.63	1434.66	1434.38	1434.31	1434.21	1433.74	1433.38	1431.31	1431.20	1430.98	1430.54	1430.15
30	Elevation Pipe Invert at Outlet	RL_exit_bot	m	1427.44	1427.99	1427.56	1427.34	1438.95	1438.87	1438.79	1438.56	1438.48	1434.51	1434.23	1434.15	1434.06	1433.58	1433.23	1431.16	1431.05	1430.82	1430.39	1430.00

Table 11-7 Summary table of dimension tertiary canal off take

S.No	Parameters	Off take canal		FC-3-3-6	FC-3-3-7	FC-3-4-1	FC-3-4-2	FC-3-4-3	FC-3-4-4	FC-3-5-1	FC-3-5-2	FC-3-5-3	FC-3-5-4	FC-3-5-5	FC-3-5-6	FC-3-6-1	FC-3-6-2	FC-3-6-3	FC-3-7-1	FC-3-7-2	FC-3-7-3	FC-3-7-4	FC-3-7-5	FC-3-7-6
	Supply canal	**Chainage**		434.67	561.66	190.32	265.37	367.81	479.93	45.00	129.99	198.30	269.97	366.22	485.49	32.70	116.90	233.84	39.82	119.78	209.49	309.15	372.34	476.41
1	Full Supply Level	FSL	m	1430.32	1430.13	1427.73	1427.63	1427.04	1426.92	1425.26	1425.03	1424.88	1424.47	1424.13	1423.79	1420.14	1419.85	1419.31	1417.92	1417.60	1417.33	1417.18	1416.81	1416.56
2	Full Supply Depth	FSD	m	0.15	0.15	0.20	0.20	0.15	0.20	0.20	0.15	0.20	0.15	0.15	0.15	0.20	0.15	0.15	0.15	0.15	0.15	0.20	0.15	0.20
3	Free Board	FB	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
4	Side Slope	SS	m/m	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
5	Top Bank Level	TBL	m	1430.47	1430.28	1427.88	1427.78	1427.19	1427.07	1425.41	1425.18	1425.03	1424.62	1424.28	1423.94	1420.29	1420.00	1419.46	1418.07	1417.75	1417.48	1417.33	1416.96	1416.71
6	Top Bank Width	TBW	m	0.30	0.30	0.35	0.35	0.30	0.30	0.35	0.30	0.35	0.30	0.30	0.30	0.35	0.30	0.30	0.30	0.30	0.30	0.35	0.30	0.30
7	Minimum Earth Cover Over Precast Non Pressure Pipe	Cmin	m																					
	Offtaking Canal																							
8	Design discharge of Turnout	Q_{ot}	m^3/s	0.02	0.01	0.00	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.00	0.03	0.00	0.01	0.01	0.02	0.01	0.01	0.03	0.01	0.01
9	Full Supply Level	FSL	m	1430.17	1429.98	1427.58	1427.48	1426.89	1426.77	1425.11	1424.88	1424.73	1424.32	1423.98	1423.64	1419.99	1419.70	1419.16	1417.77	1417.45	1417.18	1417.03	1416.66	1416.41
10	Full Supply Depth	FSD	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
11	Free Board	FB	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
12	Side Slope	SS	m/m	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
13	Top Bank Level	TBL	m	1430.32	1430.13	1427.73	1427.63	1427.04	1426.92	1425.26	1425.03	1424.88	1424.47	1424.13	1423.79	1420.14	1419.85	1419.31	1417.92	1417.60	1417.33	1417.18	1416.81	1416.56
14	Top Bank Width	TBW	m	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30
11	Wall thickness of precast chamber	0.20	m	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
12	Width of Cast in Situ Concrete Head Chamber	0.70	m	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
13	Internal Height of Cast in Situ Concrete Head Chamber	H_Cham_U/S	m	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
14	Internal Length of Cast in Situ Caoncrete Head Chamber	L_Cham_U/S	m	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
15	Width of Cast in Situ Concrete D/S Chamber	W_Cham_D/S	m	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
16	Internal Height of Cast in Situ Concrete D/S Chamber	H_Cham_D/S	m	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
17	Internal Length of Cast in Situ Caoncrete D/S Chamber	L_Cham_D/S	m	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
18	Bed Width of delivery Channel	BW_{DC}	m	60.30	61.30	62.30	63.30	64.30	65.30	66.30	67.30	68.30	69.30	70.30	71.30	72.30	73.30	74.30	75.30	76.30	77.30	78.30	79.30	80.30
19	Exit Width (Length of Weir)	EW	m	60.30	61.30	62.30	63.30	64.30	65.30	66.30	67.30	68.30	69.30	70.30	71.30	72.30	73.30	74.30	75.30	76.30	77.30	78.30	79.30	80.30
	Orifice turnout																							
20	Length of Pipes	L_pipe	m	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
21	Internal diameter of pipe	PID	m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
23	Width of Orifice gate	WOG	m	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35
24	Height of orifice gate	HOG	m	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40
25	Elevation of top pipe	RL_entry_top	m	1430.22	1430.03	1427.59	1427.48	1426.94	1426.77	1425.11	1424.93	1424.73	1424.37	1424.03	1423.69	1419.99	1419.75	1419.21	1417.82	1417.50	1417.23	1417.03	1416.71	1416.41
26	Elevation of bottom pipe	RL_entry_bot	m	1430.07	1429.88	1427.43	1427.33	1426.79	1426.62	1424.96	1424.78	1424.58	1424.22	1423.88	1423.54	1419.84	1419.60	1419.06	1417.67	1417.35	1417.08	1416.88	1416.56	1416.26
27	Elevation of turnout floor	RL_entry_tof	m	1429.82	1429.63	1427.23	1427.13	1426.54	1426.42	1424.76	1424.53	1424.38	1423.97	1423.63	1423.29	1419.64	1419.35	1418.81	1417.42	1417.10	1416.83	1416.68	1416.31	1416.06
28	Maximum Elevation at Top D/S Vertical Wall of D/S Chamber	RL_exist_crest	m	1430.32	1430.13	1427.73	1427.63	1427.04	1426.92	1425.26	1425.03	1424.88	1424.47	1424.13	1423.79	1420.14	1419.85	1419.31	1417.92	1417.60	1417.33	1417.18	1416.81	1416.56
29	Elevation top of Pipe at Outlet Head Wall	RL_exit_top	m	1430.07	1429.88	1427.48	1427.38	1426.79	1426.67	1425.01	1424.78	1424.63	1424.22	1423.88	1423.54	1419.89	1419.60	1419.06	1417.67	1417.35	1417.08	1416.93	1416.56	1416.31
30	Elevation Pipe Invert at Outlet	RL_exit_bot	m	1429.92	1429.72	1427.33	1427.23	1426.64	1426.52	1424.86	1424.63	1424.48	1424.07	1423.73	1423.38	1419.74	1419.44	1418.91	1417.52	1417.20	1416.93	1416.78	1416.41	1416.16

11.2.2 *Drop Structures*

The economical drop size is designed on Worbate small scale irrigation project. Since secondary canals are aligned across the contour, the numbers of drops are increased. Typical drawing is prepared with table of dimensions. Table below shows the hydraulic design of all drops.

Table 11-8 Summary table of dimension for vertical drop on secondary canal

S.no	Canal	chainage	Drop height(m)	$Q(m^3/sec.)$	B(canal)	d(Flow)	D(m)	L. of stilling basin(m)	B(m)	sill height(m)	L1(U/S protection)	L2(D/S protection)
1	**SC-1**	20.00	1.00	0.079	0.50	0.35	0.45	1.90	0.60	0.20	1.50	1.50
	"	60.00	1.50	0.079	0.50	0.35	0.45	2.30	0.60	0.20	1.50	1.50
	"	80.00	1.25	0.079	0.50	0.35	0.45	2.10	0.60	0.20	1.50	1.50
	"	100.00	0.75	0.079	0.50	0.35	0.45	1.60	0.60	0.20	1.50	1.50
	"	120.00	1.50	0.079	0.50	0.35	0.45	2.30	0.60	0.20	1.50	1.50
	"	140.00	0.80	0.079	0.50	0.35	0.45	1.70	0.60	0.20	1.50	1.50
	"	180.00	1.25	0.035	0.35	0.20	0.30	1.60	0.40	0.20	1.50	1.50
	"	200.00	1.50	0.035	0.35	0.20	0.30	1.70	0.40	0.20	1.50	1.50
	"	240.00	1.50	0.035	0.35	0.20	0.30	1.70	0.40	0.20	1.50	1.50
	"	280.00	1.25	0.035	0.35	0.20	0.30	1.60	0.40	0.20	1.50	1.50
	"	300.00	1.50	0.035	0.35	0.20	0.30	1.70	0.40	0.20	1.50	1.50
	"	355.94	1.50	0.025	0.35	0.20	0.30	1.70	0.30	0.20	1.50	1.50
	"	400.00	1.50	0.025	0.35	0.20	0.30	1.70	0.30	0.20	1.50	1.50
	"	460.00	1.50	0.025	0.35	0.20	0.30	1.70	0.30	0.20	1.50	1.50
	"	500.00	1.50	0.025	0.35	0.20	0.30	1.70	0.30	0.20	1.50	1.50
2	**SC-2**	18.11	1.75	0.085	0.50	0.25	0.35	2.10	0.60	0.20	1.50	1.50
	"	40.00	0.50	0.085	0.50	0.25	0.35	1.20	0.60	0.10	1.50	1.50
	"	60.00	0.75	0.085	0.50	0.25	0.35	1.40	0.60	0.10	1.50	1.50
	"	140.00	1.05	0.085	0.50	0.25	0.35	1.70	0.60	0.20	1.50	1.50
	"	180.00	1.25	0.070	0.50	0.25	0.35	1.80	0.50	0.20	1.50	1.50
	"	200.00	1.25	0.070	0.50	0.25	0.35	1.80	0.50	0.20	1.50	1.50
	"	220.00	1.25	0.070	0.50	0.25	0.35	1.80	0.50	0.20	1.50	1.50
	"	240.00	0.50	0.070	0.50	0.25	0.35	1.10	0.50	0.10	1.50	1.50
	"	280.00	0.50	0.070	0.50	0.25	0.35	1.10	0.50	0.10	1.50	1.50
	"	300.00	0.75	0.070	0.50	0.25	0.35	1.40	0.50	0.10	1.50	1.50
3	**SC-3**	180.00	0.75	0.111	0.70	0.40	0.50	1.70	0.70	0.20	1.50	1.50
	"	300.00	1.50	0.098	0.50	0.30	0.40	2.10	0.60	0.20	1.50	1.50
	"	360.00	1.00	0.098	0.50	0.30	0.40	1.80	0.60	0.20	1.50	1.50
	"	380.00	0.75	0.098	0.50	0.30	0.40	1.50	0.60	0.10	1.50	1.50
	"	410.07	0.50	0.002	0.50	0.30	0.40	1.20	0.10	0.10	1.50	1.50
	"	420.00	1.50	0.095	0.45	0.25	0.35	2.00	0.60	0.20	1.50	1.50
	"	440.00	1.00	0.095	0.45	0.25	0.35	1.70	0.60	0.20	1.50	1.50
	"	480.00	1.50	0.095	0.45	0.25	0.35	2.00	0.60	0.20	1.50	1.50
	"	500.00	1.50	0.095	0.45	0.25	0.35	2.00	0.60	0.20	1.50	1.50
	"	540.00	1.50	0.048	0.45	0.25	0.35	1.90	0.50	0.20	1.50	1.50
	"	560.00	1.10	0.048	0.45	0.25	0.35	1.70	0.50	0.20	1.50	1.50
	"	585.37	0.50	0.001	0.45	0.25	0.35	1.10	0.10	0.10	1.50	1.50
	"	600.00	1.50	0.045	0.35	0.20	0.30	1.80	0.40	0.20	1.50	1.50
	"	640.00	1.00	0.045	0.35	0.20	0.30	1.50	0.40	0.10	1.50	1.50
	"	660.00	1.25	0.045	0.35	0.20	0.30	1.60	0.40	0.20	1.50	1.50
	"	685.33	0.75	0.045	0.35	0.20	0.30	1.30	0.40	0.10	1.50	1.50
	"	700.00	1.10	0.045	0.35	0.20	0.30	1.50	0.40	0.10	1.50	1.50

Table 11-9 Summary table of dimension for vertical drop on main canal

Canal Name	Chainage	Drop height(m)	Q(m^3/sec.)	B(canal)	d(Flow)	D(m)	L. of stilling basin(m)	B(m)	sill height(m)	L1(U/S protection)	L2(D/S protection)
MC	0+126.14	1.50	0.0108	0.90	0.450	0.55	2.50	0.20	0.20	1.50	1.50
"	0+980.00	1.00	0.0133	0.85	0.450	0.55	2.10	0.30	0.20	1.50	1.50
"	1+158.47	1.50	0.0133	0.85	0.450	0.55	2.50	0.30	0.20	1.50	1.50
"	2+274.61	1.00	0.0084	0.85	0.450	0.55	2.10	0.20	0.20	1.50	1.50
"	3+033.90	0.75	0.0063	0.70	0.400	0.50	1.70	0.20	0.20	1.50	1.50
"	3+822.41	0.65	0.0063	0.70	0.400	0.50	1.60	0.20	0.20	1.50	1.50
"	4+341.00	1.50	0.0016	0.30	0.150	0.25	1.50	0.10	0.10	1.50	1.50

11.2.3 *Drainage crossing circular culvert*

Culverts carry storm runoff or drainage water under the canal. Thorough consideration should be given to the culvert alignment, profile, conduit, inlet, and outlet, with special attention given to the hydraulic design. In this project drainage crossing was provided at eight points along the main canal. In all the eight case FSL of the main canal is greater than FSL of drainage canal (run off) and the difference is less than 3m. Therefore, for such cases culvert structure is more economical. Furthermore, the design discharge obtained for all crossing point is below 2.5m^3/s which recommended to use of circular culver. The table dimensions of all drainage crossing culverts were presented in the following table.

Table 11-10 Table of dimension of a crossing drainage

No	Structure name	Chainage	Q	D	L	Lu1	Lu2	Ld1	Ld2	B	bd	bc	hc	T	t	h	H	m	(1)	(2)	(3)	(4)
			m^3/s	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm		m	m	m	m
1	CD-1	398.58	0.31	450	7621.75	2700	2700	3600	1800	900	1350	550	320	4104	613	850	1650	1.50	1458.04	1458.04	1457.96	1459.69
2	CD-2	1185.09	0.37	450	11399.14	2550	2550	3400	1700	850	1275	550	320	4054	613	850	1650	1.50	1451.89	1451.89	1451.86	1454.80
3	CD-3	2288.54	0.23	450	5767.88	2550	2550	3400	1700	850	1275	550	320	4054	613	850	1650	1.50	1451.25	1451.25	1451.21	1452.28
4	CD-4	2633.84	0.00	450	5287.26	2550	2550	3400	1700	850	1275	550	320	4054	613	850	1650	1.50	1447.80	1447.80	1447.73	1448.68
5	CD-5	3033.90	0.17	400	7552.85	2100	2100	2800	1400	700	1050	550	320	3763	600	800	1600	1.50	1442.61	1442.61	1442.57	1444.19
6	CD-6	3257.42	0.17	400	6300.00	2100	2100	2800	1400	700	1050	550	320	3763	600	800	1600	2.50	1442.45	1442.35	1441.76	1443.12
7	CD-7	3822.41	0.17	400	9070.03	2100	2100	2800	1400	700	1050	550	320	3763	600	800	1600	3.50	1441.34	1441.34	1441.34	1442.33
8	CD-8	4175.63	0.17	350	11474.10	1800	1800	2400	1200	600	900	550	320	3521	588	750	1550	4.50	1439.59	1439.59	1439.59	1440.62

11.2.4 *Division box*

For safe and sufficient quantity of withdrawal from the parent canal to the receiving canals, provision of appropriate structure is necessary. For this project, division boxes are proposed to divide water from the secondary canal to the tertiary canals. Off takes are also provided to divide the flow from secondary to tertiary canals and tertiary canals to field canals. Since the flow in all canals are open channel, the division boxes are designed using broad crest flow formula by assuming the same equal discharge coefficient and sill height for all direction. The detail of dimension table of all division boxes are described as below.

Table 11-11 Table dimension of a division box

Division Box	Parent and originating canals	Chainage(m)	L_1(m)	L_2(m)	L_3(m) min 0.25	D(m)	B(m)	B_0(m)
DB -01	**SC-1 to (TC-1-1 & TC-1-2)**	0+14	0.20	0.10	0.25	0.47	0.30	1.30
DB -02	**SC-2 to (TC-2-1 & TC-2-2)**	0+18	0.20	0.20	0.25	0.45	0.30	1.20
DB -03	**SC-2 to (TC-2-3 & TC-2-4)**	0+173	0.20	0.20	0.25	0.57	0.30	1.50
DB -04	**SC-2 to (TC-2-5 & TC-2-6)**	0+321	0.20	0.20	0.25	0.65	0.30	1.60

12 Cost Estimates and option Comparison

Cost estimation

The estimate of quantities presented at this draft feasibility design report considers the permanent works as set out in the feasibility level studies.

The unit rate adapted for cost estimation is taken from other similar projects with some adjustments. All rates are presented in Ethiopian Birr (ETB) only. Accordingly, the investment cost of the dam and its appurtenant structures and development of the canal system is about 98814972.62 Birr after VAT, the cost per hectare for this particular project becomes 823458.11 with respect to the development of the project. The summary and Detail breakdown of the estimated cost items are presented in Table 12.1 and Table 12-2.

Table 12-1 Summary of Worbate SSIP Cost Estimate

SN	Description	Total Cost
Bill Nr. 1	Camping & General Preparatory Works	654000.00
Bill Nr. 2	Gravel access road of 7 m wide	4,270,225.40
Bill Nr. 3	Head Works (Dam) Works	44,795,587.45
Bill Nr. 4	Main Canals	7,631,307.68
Bill Nr. 5	Secondary Canals	13,462,161.84
Bill Nr. 6	Tertiary Canal	1,295,507.42
Bill Nr. 7	Tertiary Drains	4,663,958.64
	Total of carried to summary	76,772,748.42
	VAT	11,515,912.26
	Grand Total of Project Work	**88,288,660.69**
	Net Area	**120.00**
	Cost Per Hectare	**735,738.84**

Table 12-2: Worbate SSIP Detail BoQ and Cost Estimate

SN	Description	Unit	Qty	Rate	Total Cost
Bill Nr. 1	**Camping & General Preparatory Works**				
1.1	Allow for Mobilization (Machineries, material, labor, etc.)	Ls	1.00	50000.00	50000.00
1.2	Allow for Demobilization after finalizing the whole project activities	Ls	1.00	50000.00	50000.00
1.3	Construction of consultants residence and/or office of size 4.4m*4.4m from G-32 CIS for roof & external wall, internally partitioned with chip wood wall, & ceiling founded on a 25cm thick hardcore, 8cm of C-10 (1:3:6) lean concrete and 2cm thick screed floor. The room should be well ventilated thus equipped with window and door of same material as shown on the drawing	Ls	1.00	72000.00	72000.00
1.4	Construction of contractors residence and/or office of size 9.65m*3m from G-32 CIS for roof & external walls and internally partitioned with chip wood wall,& ceiling founded on a 25cm thick hardcore, 8cm of C-10 (1:3:6) lean concrete and 2cm thick screed floor. Two rooms each has size of 3m*3m and are well ventilated equipping with windows and doors of same material as per the drawing.	Ls	1.00	86400.00	86400.00
1.5	Construction of 5m*5m store which is constructed from G-32 CIS wall and roof with door and window, ceiling founded on a 25cm thick hardcore, 8cm C10 (1:3:6) lean concrete and 2cm screed floor as per drawing	Ls	1.00	80000.00	80000.00
1.6	Construction of Cafeteria and kitchen facility size 6m*4m, constructed from G-32 CIS wall and roof with door and window, ceilings on a 25cm thick hardcore filled cement, 8cm of C-10 (1:3:6) lean concrete and 2cm screed floor as per drawing	Ls	1.00	86400.00	86400.00
1.7	Construction of shower and toilet rooms of total size 4m*2m, constructed from G-32 CIS wall, and roof; ventilated with separate door and window, on a 10cm thick hardcore filled cement, 5cm of C-10 (1:3:6) lean concrete as per drawing	Ls	1.00	28800.00	28800.00
1.8	Construction of guard house facility of size 2m*2m, constructed from G-32 CIS wall and roof with door and window, ceiling founded on a 25cm thick hardcore filled cement, 8cm C-10 (1:3:6) lean concrete and 2cm screed floor as shown on the drawing	Ls	1.00	14400.00	14400.00
1.9	Fence works all around the camp of area 50m*30m, 2.0m height and 15cm diameter treated timber post/eucalyptus poles with barbed wire at 20cm vertical interval & posted in a minimum of 0.6m depth backfilled with lean concrete, C-10 (1:3:6)	Ls	1.00	50000.00	50000.00

SN	Description	Unit	Qty	Rate	Total Cost
1.1	Survey works, preparation of as-built drawings and site plan including operation and maintenance manual	Ls	1.00	56000.00	56000.00
1.11	Sign Post at junction and Camp Office, with dimension of 1.0m*1.5m of 3mm thick with 0.3m*0.3m of 2.5m angle Iron pole, buried in C-10 (1:3:6) mass concrete of 0.5m minimum depth	Nr	2.00	40000.00	80000.00
	Total of Bill Nr.1 carried to summary				**654000.00**
Bill Nr. 2	**Gravel access road of 7 m wide**	**LS**	**5.00**		
2.1	Clearing and grabbing works in areas including interceptor drain & working areas as specified by the engineer	m^2	46925.00	7.00	328475.00
2.2	Excavation/cut earth works of the road section in normal soil	m^3	23463.00	70.00	1642410.00
2.3	Fill and compaction work in excavated/cut earth works of the road	m^3	32847.72	70.00	2299340.40
	Total of Bill Nr.2 carried to summary				**4270225.40**
Bill Nr. 3	**Head Works (Dam) Works**				
3.1	Dam body Earth				
3.1.1	Site Clearing and grubbing works	m^2	167968.3	5.00	838416.47
3.1.2	Soil Excavation and cart away surplus material	m^3	49818.96	70.00	3487327.20
3.1.3	Soft rock excavation	m^3	9963.79	400.00	3985516.80
3.1.4	Selected material production, loading & hauling as per drawing & specification	m3	171234.96	70.00	11986447.20
3.1.5	Back fill with selected material & compactions embankment as shown in the drawing and specification, Includes production of fill material, transportation and compaction complete in all aspect	m^3	171234.96	100.00	17123496.00
3.1.6	Construction of Horizontal drain as shown in the drawing and specification, Includes supply of filter material, transportation and compaction complete in all aspect	m^3	14436.00	150.00	2165400.00
3.2	**Spillway works**				**0.00**
3.2.1	Site Clearing and grubbing works	m2	52.00	7.00	364.00
3.2.2	Soil Excavation and cart away surplus material	m^3	9480.04	70.00	663602.58
3.2.3	Soft rock excavation	m^3	948.00	400.00	379201.47
3.2.4	Back fill with selected material & compaction on outer face of walls	m^3	1821.67	100.00	182166.57
3.2.5	Reinforced concrete in cement sand mortar C25. The thickness of concrete as indicated in the drawing. Price should include scaffolding.	m^3	503.93	3000.00	1511784.00
3.2.6	10 cm thick lean concrete under the structure	m^2	187.34	250.00	46835.00

SN	Description	Unit	Qty	Rate	Total Cost
3.2.7	Formwork supply, fixing as per the drawing	m^2	728.00	100.00	72800.00
3.2.8	Supply, cut, bending and fix binding Tor steel Grade 400 reinforcement, as per the drawing & specification complete in all respect for chute section	kg	12644	90.00	1137960
3.2.9	Dry Stone pitching	m^2	92.4	278.00	25687.2
3.3	**Outlet works**				**0.00**
3.3.1	Site Clearing and grubbing works	m2	244.00	7.00	1708.00
3.3.2	Soil Excavation and cart away surplus material	m^3	207.40	70.00	14518.00
3.3.3	Soft rock excavation	m^3	51.85	400.00	20740.00
3.3.4	Back fill with selected material & compaction on outer face of walls	m^3	130.90	100.00	13090.00
3.3.5	Reinforced concrete in cement sand mortar C25. The thickness of concrete as indicated in the drawing. Price should include scaffolding.	m^3	22.63	3000.00	67893.75
3.3.6	50mm thick lean concrete under the structure	m^2	4.50	250.00	1125.00
3.3.7	Formwork for canal side walls	m^2	221.60	100.00	22160.00
3.3.8	Supply, cut, bending and fix binding Tor steel Grade 400 reinforcement, as per the drawing & specification complete in all respect for intake section	kg	1039.40	90.00	93546.00
3.4	Φ 500 GIS Pipe providing and installation according to Drawing and Technical specification complete in all respect		78.00	3000.00	234000.00
3.5	gate as per drawing and specification				
3.5.1	Sliding gate 700 x 700 with spindle height of 8000 and screw hoist capacity of 3 tones	No	1.00	76000.00	76000.00
3.5.2	Sliding gate 700 x 700 with spindle height of 2000 and screw hoist capacity of 3 tones	No	1.00	75000.00	75000.00
5.6	Bars for trash rack dia.16mm @150mm C/C spacing (bending, cutting, placing and welding) on the top of the intake	kg	53.20	90.00	
	Total of Bill Nr.3 carried to summary				**44795587.45**
Bill Nr. 4	**Main Canals**				
4.1	Site clearing				
4.1.1	Clearing and grubbing bushes trees and shrubs on the alignment of main canal to the width of canal and embankment and depth of 0.2m.	m2	9306.00	11.89	110648.40
4.2	Earth Work				
4.2.1	Channel Excavation	m3	813.72	50.00	40685.93
4.2.2	Canal Embankment fill and compaction Spread & compact 20 cm thick layer by layer	m3	4531.80	48.00	217526.23
4.2.3	Masonry work with Cement Mortar (1:4) ratio	m3	3783.45	1577.99	5970219.83
4.3	**Main Canal Structures**				

SN	Description	Unit	Qty	Rate	Total Cost
4.3.1	Off take				
4.3.1.1	Excavation for foundation in all sorts of soil including depositing the excavated material as directed.	m3	37.14	50.00	1857.00
4.3.1.2	RCC Concrete CC20	m3	27.11	1878.23	50918.87
4.3.1.3	Lean concrete CC10 (t=75mm)	m2	35.08	1850.30	64908.52
4.3.1.4	Cemented Stone pitching	m2	15.40	166.46	2563.48
4.3.1.5	selected material Backfill with selected material for structure support	m3	8.31	48.00	398.88
4.3.1.6	Reinforcement Bar	kg	1067.94	49.32	52670.80
4.3.2	Drop				
4.3.2.1	Excavation for foundation in all sorts of soil including depositing the excavated material as directed.	m3	566.91	50.00	28345.50
4.3.2..2	selected material Backfill with selected material for structure support	m3	334.84	48.00	16072.32
4.3.2.3	Cemented Stone Pitching	m3	52.09	166.46	8670.90
4.3.2.4	Masonry work with Cement Mortar (1:4) ratio	m3	96.10	1577.99	151644.36
4.3.2.5	Plastering with Cement Mortar (1:4) mix thick complete for water exposed faces of masonry walls	m2	105.86	187.93	19894.27
4.3.3	Crossing drainage				
4.3.3.1	Excavation of normal soil	m3	499.20	50.00	24960.00
4.3.3.2	Back fill and compaction	m3	299.52	48.00	14376.96
4.3.3.3	Masonry work with 1:3 mortar work	m3	53.30	1150.00	61295.00
4.3.3.4	Concrete(C-20)	m3	16.08	2314.88	37223.31
4.3.3.5	formwork	m2	39.45	100.00	3945.00
4.3.3.6	Cemented Stone Pitching (1:3 ratio)	m3	109.28	166.46	18190.75
4.3.4	Cattle Trough	No	3.00		
4.3.5.1	Earthworks				
4.3.5.2	Site clearance	m2	134.45	7.00	941.15
4.3.5.3	Foundation excavation for structure in soft material	m3	10.04	60.00	602.40
4.3.5.4	Backfill to structure with granular/ selected material including compaction as per drawing	m3	7.20	80.00	576.00
4.3.5.5	Graded gravel bed with average depth of 300mm and particle size ranging from 3 to 6mm	m3	26.40	125.00	3300.00
4.3.5.6	50 mm thick C10 lean concrete under the cattle trough	m2	111.36	145.00	16147.20
4.3.5.7	Cement screening of internal and external faces of troughs with a cement mortar mix of 1:3 to 20mm thickness	m2	218.08	150.00	32712.00
4.3.5.8	Supply and Install 110 mm PVC pipe and required fittings	m	48.00	10.45	501.60
4.3.5.9	Masonry works of mix ratio 1:3	m3	72.00	1450.00	104400.00

SN	Description	Unit	Qty	Rate	Total Cost
4.3.5	Washing Basin	No	3.00		
4.3.5.1	Earthworks				
4.3.5.2	Site clearance	m2	64.00	7.00	448.00
4.3.5.3	Foundation excavation for structure in soft material	m3	48.00	60.00	2880.00
4.3.5.4	Backfill to structure with granular/ selected material including compaction as specified on drawing	m3	6.00	80.00	480.00
4.3.5.5	Concrete and Stonework				
4.3.5.6	Class C15/ Blinding concrete, 50mm thick	m2	16.80	160.00	2688.00
4.3.5.7	Masonry walls washing basin as shown on the drawings	m3	12.30	1450.00	17835.00
4.3.5.8	Plastering 1:2 mix to the top and internal surface of masonry wall of the canal	m2	6.00	130.00	780.00
4.3.6	Cross structure (Slab)	No	11.00	50000.00	550000.00
	Total of Bill Nr.4 carried to summary				**7631307.68**
Bill Nr. 5	**Secondary Canals**				
5.1	Site clearing				
5.1.1	Clearing and grubbing bushes trees and shrubs on the alignment of main canal to the width of canal and embankment and depth of 0.2m.	m2	5967.90	11.89	70958.33
5.2	Earth Work				
5.2.1	Channel Excavation	m3	933.70	50.00	46685.00
5.2.2	Canal Embankment fill and compaction Spread & compact 20 cm thick layer by layer	m3	1323.11	48.00	63509.28
5.2.3	Masonry work with Cement Mortar (1:4) ratio	m3	1020.67	1577.99	1610594.86
5.3	Secondary Canal Structures				
5.3.1	Off take				
5.3.1.1	Excavation for foundation in all sorts of soil including depositing the excavated material as directed.	m3	9.07	50.00	453.50
5.3.1.2	RCC Concrete CC20	m3	20.44	1878.23	38391.06
5.3.1.3	Lean concrete CC10 (t=75mm)	m2	28.24	1850.30	52252.47
5.3.1.4	Cemented Stone pitching	m2	13.20	166.46	2197.27
5.3.1.5	selected material Backfill with selected material for structure support	m3	4.72	48.00	226.56
5.3.1.6	Reinforcement bar	kg	805.53	49.32	39728.74
5.3.2	Division box				
5.3.2.1	Excavation for foundation in all sorts of soil including depositing the excavated material as directed.	m3	7.56	50.00	378.00
5.3.2.2	selected material Backfill with selected material for structure support	m3	3.12	48.00	149.76
5.3.2.3	Cemented Stone Pitching	m3	2.47	166.46	411.16

SN	Description	Unit	Qty	Rate	Total Cost
5.3.2.4	Masonry work with Cement Mortar (1:4) ratio	m3	8.78	1577.99	13854.71
5.3.2.5	Plastering with Cement Mortar (1:4) mix thick complete for water exposed faces of masonry walls	m2	20.12	187.93	3781.15
5.3.2	Drop				
5.3.2.1	Excavation for foundation in all sorts of soil including depositing the excavated material as directed.	m3	31109.05	50.00	1555452.50
5.3.2.2	selected material Backfill with selected material for structure support	m3	17368.40	48.00	833683.20
5.3.2.3	Cemented Stone Pitching	m3	2512.92	166.46	418300.66
5.3.2.4	Masonry work with Cement Mortar (1:4) ratio	m3	4855.78	1577.99	7662348.24
5.3.2.5	Plastering with Cement Mortar (1:4) mix thick complete for water exposed faces of masonry walls	m2	5580.83	187.93	1048805.38
	Total of Bill Nr.5 carried to summary				**13462161.84**
Bill Nr. 6	**Tertiary Canal**				
6.1	Site clearing				
6.1.1	Clearing and grubbing bushes trees and shrubs on the alignment of main canal to the width of canal and embankment and depth of 0.2m.	m2	20900.10	11.89	248502.14
6.2	Earth Work				
6.2.1	Channel Excavation	m3	479.89	50.00	23994.26
6.2.2	Canal Embankment fill and compaction Spread & compact 20 cm thick layer by layer	m3	4722.66	48.00	226687.67
6.3	Tertiary Canal Structures				
6.3.1	Off take				
6.3.1.1	Excavation for foundation in all sorts of soil including depositing the excavated material as directed.	m3	113.54	50.00	5677.00
6.3.1.2	RCC Concrete CC20	m3	122.24	1878.23	229595.09
6.3.1.3	Lean concrete CC10(t=75mm)	m2	190.90	1850.30	353222.27
6.3.1.4	selected material Backfill with selected material for structure support	m3	11.32	48.00	543.36
6.3.1.5	Cemented Stone Pitching	m2	89.10	166.46	14831.59
6.3.1.6	Reinforcement bar	kg	3902.15	49.32	192454.04
	Total of Bill Nr.6 carried to summary				**1295507.42**
Bill Nr. 7	**Tertiary Drains**				
7.1	Site clearing				
7.1.1	Clearing and grubbing bushes trees and shrubs on the alignment of main canal to the width of canal and embankment and depth of 0.2m.	m2	190970.86	11.89	2270643.52
7.2	**Earth Work**				
7.2.1	Channel Excavation	m3	1572.28	50.00	78614.00

SN	Description	Unit	Qty	Rate	Total Cost
	Canal Embankment fill and compaction Spread & compact 20 cm thick layer by layer	m3	48222.94	48.00	2314701.12
	Total of Bill Nr.7 carried to summary				**4663958.64**
	Total of carried to summary				**76772748.42**

13 CONCLUSIONS AND RECOMMENDATIONS

The catchment area at the dam site is about as 48.3km2. According to the reservoir simulation exercise the Worbate reservoir can irrigate about 120 ha with full storage capacity of 2.2MCM. The proposed dam will have maximum dam height of 19.5m above river bed and 535m crest length.

The dam design is carried out with considering the geological and geotechnical investigation made at the dam axis, spillway and partly the reservoir area. The geological features derived in this study phase are basically based on surface observation, Pit sampling, geophysical survey along selected investigation routes and few laboratory test results with regards to top foundation condition and construction materials. An attempt was made to extract as much as possible information required for design purposes from the limited investigation indicated above.

For a successful detailed design, the detailed investigation of geology and geotechnical investigation makes the design more reliable. The estimated loss due to seepage as it is shown in the Figure 6-5 was found to be $0.177m^3$/day per meter length of the dam with a total 8522m3 with in the operation period of 3 months, this may vary due to the borrow area material property variability. Therefore, it is essential to check the assumed soil property in the design prevails the existing condition during construction.

Therefore, the detailed dam and appurtenant structures design will be done immediately after the acceptance of the draft feasibility and detail design report.

The design duty (maximum duty) of the command area for 12 hours per day irrigation with overall project efficiency of 50 % is 2.56l/s/ha. The method of irrigation of the project area is furrow surface irrigation in which the main, secondary and tertiary canals are working continuously.

The reason in which the main canal is lining up to the 4426.52 m is to avoid the siltation problem, time saving to reach at the tail part, and to reduce maintenance cost. The main and secondary canals of the irrigation systems are associated with drops. They are designed as far as possible to be partially filled and cut. The tertiary canal layout is designed as far as possible to avoid cross-structures within them.

The following recommendations are drown:

- For better performance and long service year of the project, regular inspection and maintenance is highly required.
- Farmers training how to operate and maintain the project as a whole and available land water resources has a paramount important
- The irrigation hours per day and per week should be flexible based on the demand
- Detail supervision and follow up is critical and basic necessity during the construction
- During the construction the design parameter have to be verified and adjusted accordingly

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