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Feasibility Study and Detail Design of Omo Valley Farm Irrigation Project

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Feasibility Study and Detail Design of Omo Valley Farm Irrigation Project

Pump Station Design Report

Final Report

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1. Introduction

1.1. General

The general bed slope of the Omo River and particularly at the project site is very gentle. It signifies the importance of carefully comprehensive study and investigates on the physical and hydrological settings especially pertinent to make decision on the type of headwork structure selected and design of the various appurtenant structures. For such type of river in highly meandering region dominated by alluvial deposit in river course as well as flood plain poor foundation condition is inevitable. Most importantly, the high variation in annual peak and minimum flow and associated water level of the river is problematic with regards to river diversion as well as direct pump intake.

In this regards the design followed comprehensive study and design process in order to come up with realistic and functioning system as much as possible. The hydrological study provide the river morphological behavior in relation to flood magnitude with expected level of probability and the anticipated level of water surface under the given flood magnitude. Furthermore, it provides the dependable and minimum water level in the river course. The geological and geotechnical investigation provided the necessary input information on foundation materials property and immunity from or level of seismic hazards. The design of the headwork will use the hydrological and investigation report as an input and prepare the planning of the headwork structure and fix the different dimensions of the civil works. Once the civil work is setup the structural safety and the electromechanical component has been performed.

There are two pumping stations namely the main pump station and the boosting pump station. The main pump station is located on the Omo river side and irrigates command area which are below 420 m elevation above mean sea level. Command which is above 420 m and below 490 m elevation above mean sea level will be irrigated by the boosting pump station.

This report aims to provide the approaches, description, considerations; assumptions used to carry out the design work. Furthermore, it covers the different parts of work used in civil engineering design of the headwork (river intake culvert, main pumping station) and its appurtenant structures (pipe network, settling basin, buster pump) to supply water to the main conveyance canal for irrigation development of net 5,600 ha of land.

1.2 Scope of the Report

This report summarizes the principles and methods of design used on the Omo valley irrigation project head work. Considerations, assumptions, formulas and sample calculation (where ever relevant) is presented. The report should be read in conjunction with the Drawing Albums. As far as possible all reference material has been cited properly and acknowledged in appropriate places. The emphasis of this report is the design for the finally agreed and adopted pump station, intake structures and associated electromechanical works.

2. Topography

The head work site is selected near Kircho village specifically around the currently serving pump station site. It lies at an elevation of 370m a.s.l at the river and range 389 - 390 m a.s.l. on the river bank. The survey of the river cross-section indicates that the river is very wide (about 172m wide) and very deep (almost 19m depth) (refer drawing album). The project command area is relatively flat land with an average altitude of about 400 m asl and land on its periphery rise gently to an elevation of over 490 m asl. The Omo River flows on the right of command area from north to south direction. Very few intermittent flushing tributaries of the Omo River and gulley dissect the command area into interfluves significantly wide. These interfluves become more incised to the south of the Turmi/Kircho River, a majority tributary which passes in northern part of the project area from east to west, bordering the 5,600 ha command area in the north.

3. Hydrology

3.1 Design Flood

Different recurrence interval flood analysis for the design of the project has been carried out in the hydrology study including consideration on the regulating effect of Gibe-III dam. Table 3.1 shows the flood estimation using measured river cross section at the intake location and simulated in HEC-RAS. A 50 years return period flood which is equivalent to 5,149.3 m^3/s that corresponds to high flood level (HFL) of 384.05 meter is adopted for design.

Table 3-1 Flood Frequency

Return periods	Maximum flow m^3/s	Water surface elevation m.a.s.l	Energy grade line elevation m.a.s.l
25 yrs	3973.7	382.85	383.6
50 yrs	5149.3	384.05	384.99
100 yrs	6447.0	385.23	386.37

3.2 Water Availability

The water balance study for this project has a specific purpose that of checking the adequacy of Omo river for irrigating the proposed Omo valley farm command area, release sufficient amount of water for downstream user and Omo river channel as well. The amount of water available at Karadus pump scheme was estimated by considering three scenarios, viz. natural flow condition, and developing command area (175,000ha) of Kuraz sugar irrigation project and fully operational of Gibe III HP.

The mean monthly inflow of Omo river to Karadus pump site is presented on Table 3-2. The annual generated flow at Karadus pump site is 15.1Bm^3. The mean monthly flow ranges from 1202 to 197 m^3/s. The minimum dependable flow of 197m^3/s and the corresponding minimum water level 373meter is taken from the hydrology study.

Table 3-2 Summarized Monthly Flow (m^3/s) at Karadus Pump Site

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Mean	249.6	197.0	224.1	299.7	289.1	301.8	729.7	1201.9	948.7	625.6	370.3	298.2	478.0
75%	248.4	189.9	223.3	297.5	289.5	266.2	648.0	932.6	595.2	421.7	342.5	289.8	395.4
80%	245.3	187.8	222.3	294.0	272.1	219.7	643.0	927.8	588.8	420.6	341.1	288.6	387.6
85%	241.9	187.5	221.3	286.1	243.7	187.1	622.5	908.8	581.2	420.4	339.5	288.0	377.3

3.3 River Cross Section

The river cross section at the axis of river intake location is surveyed and used for the design.

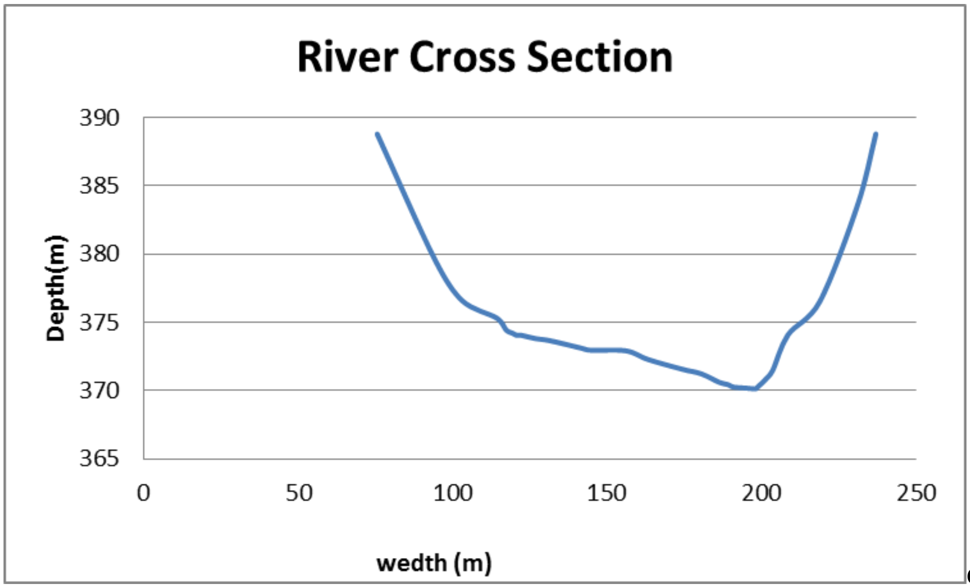

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Figure 3-1 River cross-section

3.5 Stage Discharge Rating Curve

Stage discharge curve at the proposed pump intake axis has been developed after conducting topographical survey on the river cross-sections and bed gradient using Manning equation as;

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

And

$$Q = VA$$

Where *Q* is the discharge in m^3/s, *V* is the mean flow velocity in m/s, *n* is Manning roughness coefficient (dimensionless), *R* is the hydraulic radius in m, *A* is flow cross sectional area in m^2, *S* is the bed slope of the river (assuming uniform flow prevails) in m/m.

The stage discharge curve for different stages and their corresponding discharges is developed using the above river cross section and bed gradient. The stage discharge relationship is presented for both minimum and maximum river flows as shown in Figure 3-2 and Figure 3-3.

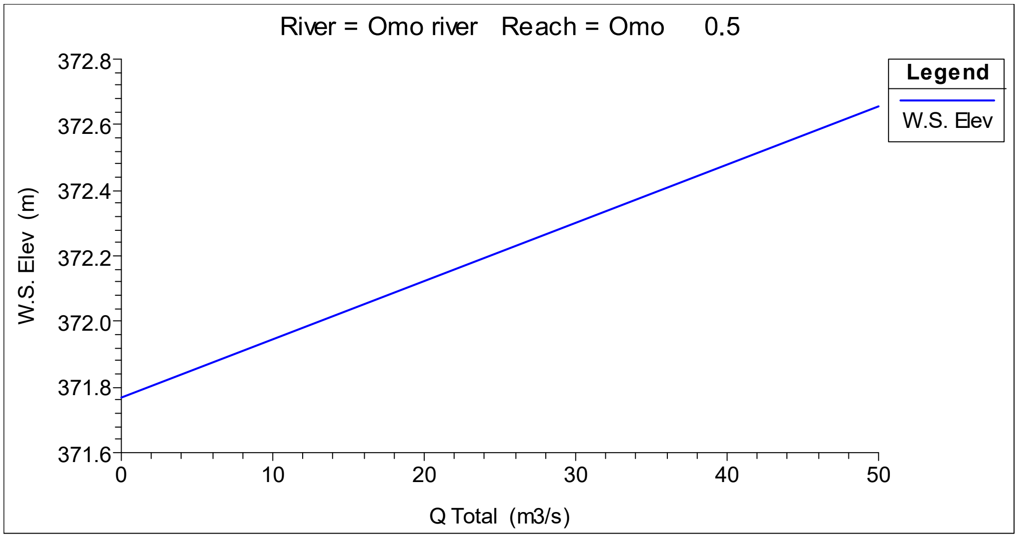

Figure 3-2 Rating Curve for Minimum Flow

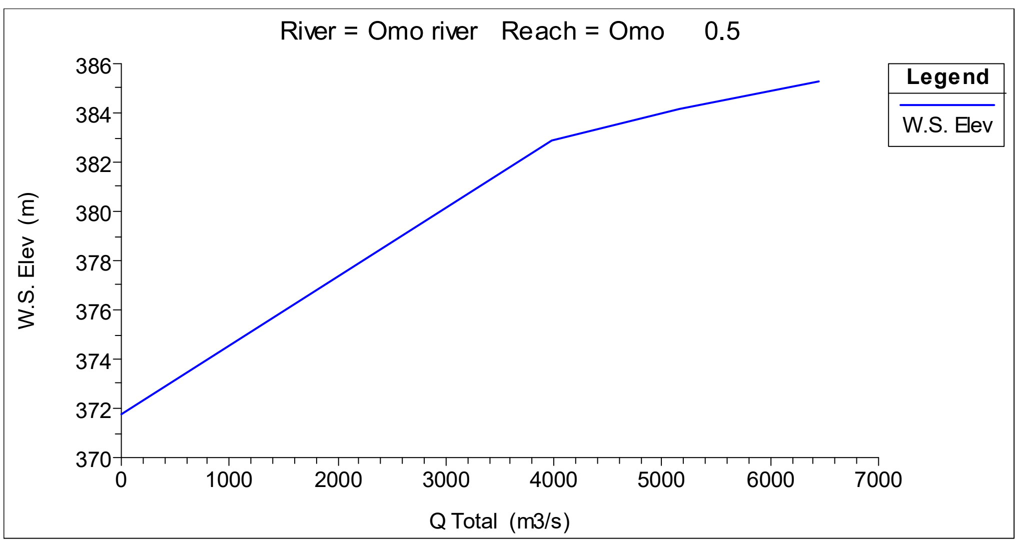

Figure 3-3 Rating Curve for maximum flow

3.5.1 High Flood Level

High flood level (HFL) corresponding to 50 years return period is taken from the developed curve and its value is 384m above sea level (refer Table 3-1). Furthermore, the design of the dyke and different appurtenant structures considered the HFL mark observed during site visit.

3.5.2 Minimum Water Level

Minimum water level (MWL) in the river corresponding to 80% dependable flow is also taken from developed rating curve and its value is 373 m above mean sea level.

Table 3-4 Water levels at the pump location for low flow condition

Q Total (m^3/s)	**Minimum Channel elevation m a.s.l**	**Water surface elevation m a.s.l**	**Energy grade line elevation m a.s.l**
50	371.45	373.07	373.33

4. Geotechnics

Planning and execution of irrigation project schemes require detailed investigation of the geological and geotechnical conditions of the proposed site. Furthermore, availability of suitable construction materials in the vicinity of the project site is mandatory.

The detailed investigation concentrates mainly on the bearing capacity of the foundation material, water tightness, liquefaction, settlements and stability of the foundations of the structure sites. In addition, the geological and geotechnical conditions of the overall has been evaluated and used for this design.

At the main pump station site three layers are identified these are: inorganic silt of high plasticity (MH), Organic clay of high plasticity (OH) and inorganic Clayey silty sand with low plasticity (ML) whereas at the Pump Station-2 one geotechnical layer, i.e. silty sand (SM) is identified.

According to the investigation result, from the visual description three soil layers at main pump station site and one soil layer at the booster pump site were found to occur As per the result of field and laboratory activities carried out and the analysis of the available data and test results and assuming that the data obtained from the excavated soil formation will have similarity also up to foundation depth, the following engineering recommendations can be made for the pump station site (Note that the final recommendation will be improved after we obtain the necessary data of foundation at the required depth during in-situ test).

4.1 Allowable Bearing Capacity

According to the bearing capacity analysis for foundation of main pump station the third layer would provide at least 794.1 - 994.1Kpa at depth of 1 to 4m and rectangular mat foundation width of 12m and length of 60m; and for PS-2 bearing capacity of 914.5 to 976 kPa at the depth of 6.0m depending on the mat pad width B=6m to B=2 which is considered adequate to accommodate the proposed pump with rather uniform load distribution.

4.2 Selection of Foundation

According to the nature and characteristics of the materials encountered in the Test pits, it is recommended to use stiffened mat foundation if the footing is designed on the alluvial sandy clay soil.

It is known that a mat foundation is commonly used where the base soil has a low bearing capacity and/or the column loads are so large that more than 50 percent of the area is covered by conventional spread footings. It is also used for deep basement foundation with both spread the column loads with a more uniform pressure distribution. Mat foundation can also be the floor slab for the basement and used to bridge over horizontal variation of the soil layer on the ground. Accordingly, stiffed/reinforced mat foundation is recommended for the pump house of this project. The decision whether to use mat foundation or pile foundation depends on the nature of the encountered formation at the foundation depth, structure load distribution, subsurface drainage efficiency and obviously based on the cost analysis. This will further be improved after investigating the formation by observing, conducting field in-situ test and laboratory tests.

4.3 Drainage of Site

It is recommended to design an effective surface water drainage system as well as proper subsurface drainage facility to get rid of the consequences of the surface and infiltrated water into the foundation layers. Mainly if the foundation footing is on the alluvial soil and the closeness of the structure to river, the site should be graded so as to direct surface water and lateral water flow if encountered during construction away from all planned structures.

4.4 Materials for Replacement, Backfill and Compaction Criteria

Replacing and back filling could be employed to improve the foundation conditions for the foundation footing on the alluvial soil. In general, materials for the backfilling is granular, not containing rocks or lumps over 15 cm in greatest dimension, free from organic matter, with plasticity index (PI) not more than 10. The backfill material is proposed to be laid in lifts not exceeding 25 cm in loose thickness and compacted to at least 95 percent of the maximum dry density at optimum moisture content as determined by modified compaction test (Proctor) (ASTM D-1557).

4.5 Seismic Condition

According to seismicity hazard map of Ethiopia the site located in Zone 4; therefore, being at zone 4 horizontal acceleration of 0.15g can be considered for design.

5 Crop Water Requirement and Design Discharge

5.1 Crop Water Requirement

The annual water requirement of cotton, and other proposed crops on monthly bases has been taken from irrigation water demand analysis (refer crop water requirement report) and summarized in Table 5.1.

Table 5-1 Crop water requirement

Month	NIR*		Duty @ 24hr at field	GIR*		Duty@ M.C.	
						24hr	20hr
	mm/day	mm/month	l/s/ha	mm/day	mm/month	l/s/ha	l/s/ha
January	6.6	206.0	0.78	13.2	412.0	1.56	1.87
February	5.3	147.4	0.65	10.6	294.8	1.30	1.56
March	1.3	39.8	0.17	2.6	79.6	0.34	0.41
April	0.1	1.5	0.30	0.2	3.0	0.60	0.72
May	0.3	10.5	0.04	0.6	21.0	0.08	0.10
June	3.5	106.3	0.41	7.0	212.6	0.82	0.98
July	5.7	177.1	0.66	11.4	354.2	1.32	1.58
August	6.1	190.2	0.71	12.2	380.4	1.42	1.70
September	4.4	131.8	0.51	8.8	263.6	1.02	1.22
October	0.7	20.4	0.04	1.4	40.8	0.08	0.10
November	0.7	22.2	0.10	1.4	44.4	0.20	0.24
December	4.8	148.9	0.56	9.6	297.8	1.12	1.34
Maximum/Total	6.6	1202.1	0.78	13.2	2404.2	1.56	1.87

*NIR: net irrigation requirement, GIR: gross irrigation requirement.

The maximum monthly gross irrigation requirement is 412 mm in the month of January. Hence the corresponding 24 hour irrigation design duty at the head of the main canal is 1.56 lit/s/ha and becomes 1.87 l/sec/ha for 20hr irrigation.

5.2 Design Discharge

For the irrigation development of 5000 ha net irrigable area, 20 hour design discharge becomes 9.35m^3/s. Considering the flexibility factor of 7%, the required discharge becomes:

Q_{20} = 1.1x Q_{20} = 10.2m^3/sec.

Where: Q_{20} = 20 hours design discharge m^3/sec.

Since the discharge obtained refers to the worst case that may happen for few days, the design discharge can be taken 10 m^3/sec, which is the main canal design discharge.

6. Design of Head Work Structures

6.1 Planning of Intake and Main Pumping Station

Three reinforced concrete pipe having 1220 mm internal diameter will divert and lead the river flow to the sump. A trash rack and stop log will be provided at the inlet to concrete pipe. Manually operated double sliding gate (one operating and another emergency) is provided at the exit of concrete pipe and inlet to the sump. The sump is made of reinforced concrete structure and its wall extends up to the top level of the embankment.

The pumping station facility is protected from flood by providing an embankment/dyke with side slope of similar to the original river bank slope on the river side. The other side of the dyke depends on the shear wall provided at the sump, access road and permissible side slope provided in cut on the type of the soil material. All surface runoff towards the pumping station (either from pump house roof or ground surface is collected in a drain pond constructed at the west corner of the pump house. A drain network will lead the runoff towards drain pond. Another small pump house is required to drain out the drain water.

The type, number and arrangement of pumps in a pump station are based on price, power consumption, ease of operation, installation and maintenance. Taking the above factors in to account instead of providing one big capacity pump a number of smaller capacity pumps are advisable. Hence, nine on duty/operating and one stand by pumps with discharging capacity of 1 m^3/s have been proposed. Connection of pumps in parallel in order to avoid inconvenient delivery pipe arrangement and reduce head loss and hence for energy saving couldn't be feasible because the client has already purchased 800mm steel delivery pipe with PN10 specification. Therefore, the design might not be economical but tried to accommodate the use of existing pipes for the design without compromising the functioning of its purpose properly. That means the delivery pipe of each pump is aligned parallel to each other. Site plan showing the layout of intake, pumps arrangement and delivery is given in figure 6.1.

The delivery and suction pipes are used to convey irrigation water from the sump at the main pump station to de-silting basin at the exit of the delivery pipe. The delivery pipe is 500 m long. These pipes can be installed over or under ground by considering factors such as the nature of the ground, the pipe material, the ambient temperatures and the environmental requirements. The pipes are aligned in straight lines and provide with concrete anchor blocks at each bend and on joint between two pipes. The anchor blocks will be designed based on the required resistance to thrust inside the pipe plus the frictional forces caused by its expansion and contraction.

The delivery pipe discharge in to a de-settling basin located at the beginning of the main canal. The main canal irrigates the command area which is below 420m elevation by gravity flow. The remaining part of the command which is higher in elevation compared to the main canal require addition pumping station. Therefore, a Boosting pump station is planned on the main canal in order to lift the water in to a new elevation (489m) so that the reaming command can be irrigated by gravity flow.

In general, the design of the headwork consists of the river intake (pipe culvert), the main pumping station (sump, pump and pipes), the de-settling basin and the Boosting pump station components which will be described in subsequent sections,

6.2 Consideration in River Intake Design

For highly meandering river with alluvial deposit similar to this project site, provision of intake with sufficient sediment transport capacity during the dominant intake discharge regardless of the variation in the river water level is very important. Furthermore, existence of good foundation soil in order to support the stress developed by the intake structure (like concrete wall, gates, etc) is important.

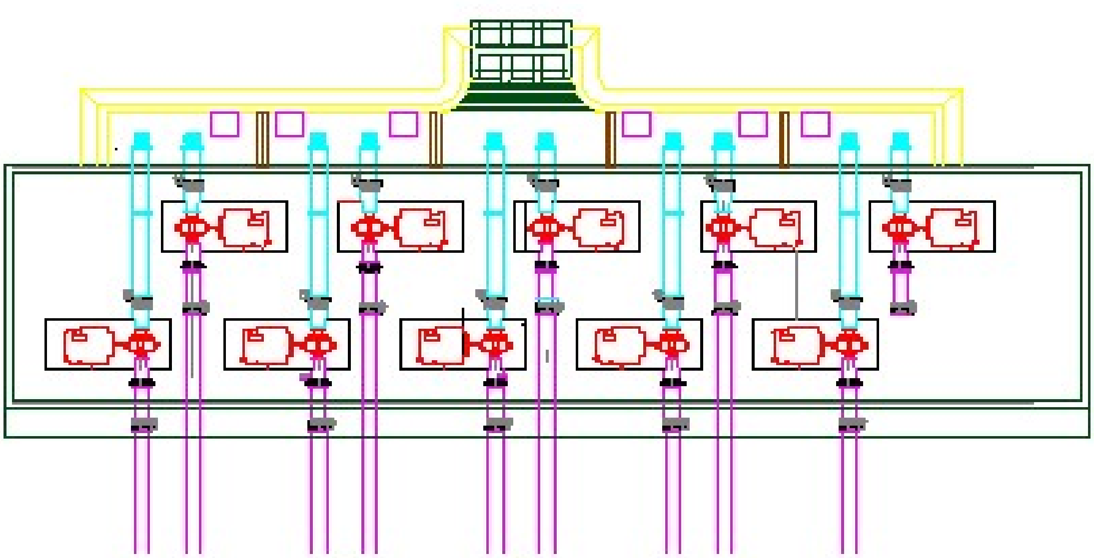

Figure 6.1 Proposed pump station arrangement

In this regards, a river bank intake structure with concrete pipe culvert is proposed. In order to make the intake economical and safe very long height is avoided. Only a stop log is providing at this site which can only be accessed by divers. The concrete pipe will be constructed in cut and cover provide with collar at each pipe connection. An emergency and operation sliding vertical gate (dual gate) is provided at the exit of the pipe culvert or at the inlet to the sump. The gate is mounted on a reinforced concrete abutment which is connected (constructed in monolithic with shear wall which will be discussed later). The foundation of the soil have sufficient bearing capacity (refer geotechnical report) for the proposed structure.

Omo river is one of the river in Ethiopia with high sediment transport (refer Hydrology report). Therefore, it is important to make sure that sediment entering in to the pipe culvert leaves the culver so that there is no risk of clogging as a result of sediment deposition.

6.3 Design of River Intake

According to the irrigation water demand analysis, the peak water demand is $10.2m^3/s$. The three concrete pipes with 1220mm internal diameter are found sufficient for the design flow rate. According to culvert flow principles the flow in the culvert is pipe flow with downstream control (because according to this arrangement the water depth to pipe diameter ratio at inlet exceed 1 and the tail water depth is greater than pipe diameter or submerged under the Minimum Water Level condition depicted according to Novak *et al.* (2004)). Therefore, the culvert slope can be any type under such circumstance and the control is at outlet. Furthermore as long as the depth at MWL is greater than barrel opening, full pipe flow will prevail. In order to make sure that hydraulic assumption prevails two barrel slope (one 1:10 and another 1:40) in the form of drop culvert is provided. The joint of each pipe is provided with collar of sufficient height and width. Table 6.1 shows the hydraulic calculation of a single pipe flow. The expected rating (discharge versus head relationship) during rising water level in the river is also analyzed and summarized in Table 6.2. In this regards, a pipe flow with water depth to pipe diameter ratio in excess of 1.2 is assumed and the energy equation is used from Novak *et al.* (2004) as follows;

$$H + S_o L = D + H_L$$

Where H is the energy level (simply depth of water) above culvert invert at inlet in m, S_o is the bed slope of culvert in m/m, L is the length of culvert in m, D is the culvert diameter in m, H_L is the total head loss in m.

The total head loss can be expressed as

$$H_L = K_e \frac{V^2}{2g} + \left(\frac{Vn}{R^{2/3}}\right)^2 L + K_{ex} \frac{V^2}{2g} \text{ at}$$

Where K_e is the head loss coefficient at inlet (0.5 is assumed), V is the mean flow velocity in the barrel in m/s, g is gravitational acceleration in m/s^2, n is the manning roughness coefficient for barrel (0.013 is assumed), R is the hydraulic radius in m, K_{ex} is the head loss coefficient at barrel exit (1 is assumed).

Table 6.1 Hydraulic calculation of concrete pipe flow

Roughness coefficient	Design discharge	Pipe diameter	Flow area	Flow velocity	Wetted perimeter	Hydraulic radius	Slope of energy grade line	Length of pipe
n	Q	d	A	V	P	R	Sf	L
-	m^3/s	m	m^2	m/s	m	m	m/m	m
0.013	3.12	1.22	1.17	2.67	3.83	0.31	0.006	30

Table 6.2 Stage discharge relationship per barrel

H (m)	1.5	2	2.5	3	3.5	4	4.5	5	5.5
Q (m^3/s)	2.6	3.6	4.4	5.1	5.7	6.3	6.8	7.3	7.7

6.3.1 Head loss in concrete pipe

The head loss in concrete pipe includes head loss due to trash rack, entrance, pipe wall friction and gate at exit. The general equation can be expressed as

$$H_T = h_t + h_e + +h_g + h_{ex} + h_f$$

Where h_t is trash rack head loss, h_e is entrance head loss, h_f is friction head loss, h_g is gate head loss, h_{ex} is head loss at exit and h_f is head loss due to pipe wall friction.

These losses can be expressed in terms of a loss coefficient and velocity head as follows.

$$H_T = K_t\left(\frac{V^2}{2g}\right) + K_e\left(\frac{V^2}{2g}\right) + K_g\left(\frac{V^2}{2g}\right) + K_{ex}\left(\frac{V^2}{2g}\right) + h_f$$

Where K_t is trash rack head loss coefficient, K_e is entrance head loss coefficient, K_g is gate head loss coefficient, K_{ex} is exit head loss coefficient.

Table 6.3 Culvert pipe head loss in m

h_e	h_g	h_{ex}	h_t	h_f	H_T
0.18	0.36	0.36	0	0.2	**0.74**

Table 6.4 Culvert flow hydraulic condition

Parameters	**Symbol**	**Unit**	**Quantity**
Input			
Diameter of culvert	D	m	1.22
Culvert length	L	m	33
Culvert Invert level at Inlet		m	371.5
Minimum water level	MWL	m	373
High Flood level	HFL		384
Height of water above Culvert Invert	H	m	1.5
Ratio	H/D	-	1.23
Roughness coefficient (assumed 0.013 according to USBR)	n	-	0.013
Design discharge	Q	m3/s	10
No of barrel		Nr	3
Culvert bed slope	So_1	m/m	0.1
	So_2	m/m	0.025
Computation			
Single Culvert discharge	q	m3/s	3.333
Flow area	A	m^2	1.17
Wetted Perimeter	P	m	3.83
Hydraulic radius	R	m	0.305
Culvert Flow Velocity	V	m/s2	2.85

Slope of Energy grade line	Sf	m/m	0.0067
Minimum friction headloss	hf	m	0.22
Velocity head in culvert	V2/2g	m	0.41
Headloss at culvert inlet	hf1	m	0.21
Headloss at culvert outlet	hf2	m	0.41
Total headloss required	Hf	m	0.84
Culvert Invert level at outlet		m	370.30
Water level in Sump		m	372.16

Where maximum loss values are desired, it is assumed that 50 percent of the trash rack area is clogged. This will result in velocity which is twice the designed velocity through the trash rack. For minimum trash rack losses, assume no clogging of the openings when computing the loss coefficient, or neglect the loss entirely. In order to avoid excessive head loss it is assumed that there is no clogging.

A square edged entrance and exit is assumed so that a head loss coefficient of 0.5 and 1 is used for K_e and K_{ex}, respectively.

Where a gate is mounted in a conduit so that the floor, sides, and roof, both upstream and downstream, are continuous with the gate opening, only the losses caused by the slot must be considered; for this a value of K_g not exceeding 0.1 should be assumed. For partly open gates, the loss coefficient depends on the top contraction; for smaller openings, it approaches a higher value of unity which is most likely in this project.

The head loss is calculated and tabulated as shown in Table 6.3. It is almost similar to the result presented in Table 6.4.

6.3.2 Sediment Transport in Concrete Pipe

Irrigation canals are generally designed based upon an assumption of uniform and steady flow of water. And this assumption implies that water and sediments entering in to canal will be transported to the fields. However, uniform and steady flow is seldom found in reality and as a consequence the sediment transportation and deposition in a canal needs to be evaluated.

Sediment transport in open channel is governed by equations of motion and continuity both water and sediment (Depeweg and Mendez, 2007). Analytical descriptions of all the physical processes involved in those equations are not clearly understood. Therefore, sediment transport still relies in field and experimental data and on dimensional analysis.

The most widely used semi-empirical approach to defining the threshold of sediment motion was proposed in the early 1900s by the German physicist Albert F. Shields (1936). Shields plotted the dimensionless particle mobility parameter or simply shear stress (θ) against the dimensionless particle Reynolds number (R_{e*}) and this plot is called *Shield diagram* or *Shields stress* or *Shields parameter* and used to decide whether a sediment particles of interest is in motion or not. Later on several researchers including van Rjin (1993), as cited by Depeweg and Mendez (2007) used similar principles. Figure 8.2 shows the shield diagram.

Therefore, any process related to sediment transport can be expressed as a function of independent dimensionless parameters like particle parameter, particle mobility parameter, shear Reynolds number. These parameters are a function of the hydraulic property of the canal. The following dimensionless parameters are widely used for describing the sediment movement in channels according to use of shield diagram (Depeweg and Mendez, 2007).

Particle parameter: reflects the influence of gravity, density and viscosity on the sediment transport and is given by:

$$D_* = \left(\frac{(s-1)g}{v^2}\right)^{1/3} d_{50}$$

Where s is relative density of sediment particle ($\frac{\rho_s}{\rho_w}$ where ρ_s is the particle density of sediment in kg/m^3 and ρ_w is density of water in kg/m^3), g is acceleration due to gravity in m/s^2, d_{50} is sediment median diameter in m, υ is kinematic viscosity of water.

Particle mobility parameter: is the ratio between the drag force and the submerged particle weight.

$$\theta = \frac{u_*^2}{(s-1)gd_{50}} = \frac{\tau}{(s-1)\rho g d_{50}}$$

Where τ is shear stress on particles in N/m^2, u_* is shear velocity of sediment particles in m/s, $\rho$is density of water in kg/m^3.

And the shear velocity of sediment particles can be estimated by

$$u_* = \sqrt{hgS_o}$$

Where h is the depth of flow in m and S_o is the bed slope in m/m.

The particle mobility parameter under critical condition will describe whether the sediment is in motion or not and is given by:

$$\theta_{cr} = \frac{u_{*cr}^2}{(s-1)gd_{50}} = \frac{\tau_{cr}}{(s-1)\rho g d_{50}}$$

Where τ_{cr} is critical shear stress for initiation of motion in N/m^2, u_{*cr} is critical shear velocity of sediment particles in m/s, $\rho$is density of water in kg/m^3.

Excess bed shear stress parameter, T is defined as:

$$T = \frac{\tau` - \tau_{cr}}{\tau_{cr}}$$

Where τcr is critical shear stress in N/m^2 according to Shields, τ`is grain shear stress in N/m^2.

Shear Reynolds number parameter: is represented by

$$R_{e*} = \frac{u_* d_{50}}{\nu}$$

Where R_{e*} is Shear Reynolds number.

Several aspects of the Shields diagram are particularly important for instance;

1. The lowest Shields stress occurs in the sand range (0.06 - 2.00 mm). Sand is small enough to have small mass but too large for adhesion forces to come into play. Shields diagram confirms that sand is the most easily worked and eroded sediment.
2. Silt/clay, in spite of the smaller size, requires a higher shear stress for motion than sand. Here adhesion forces become overwhelmingly large and bind the sediment together into a mass that is very resistant to erosion.

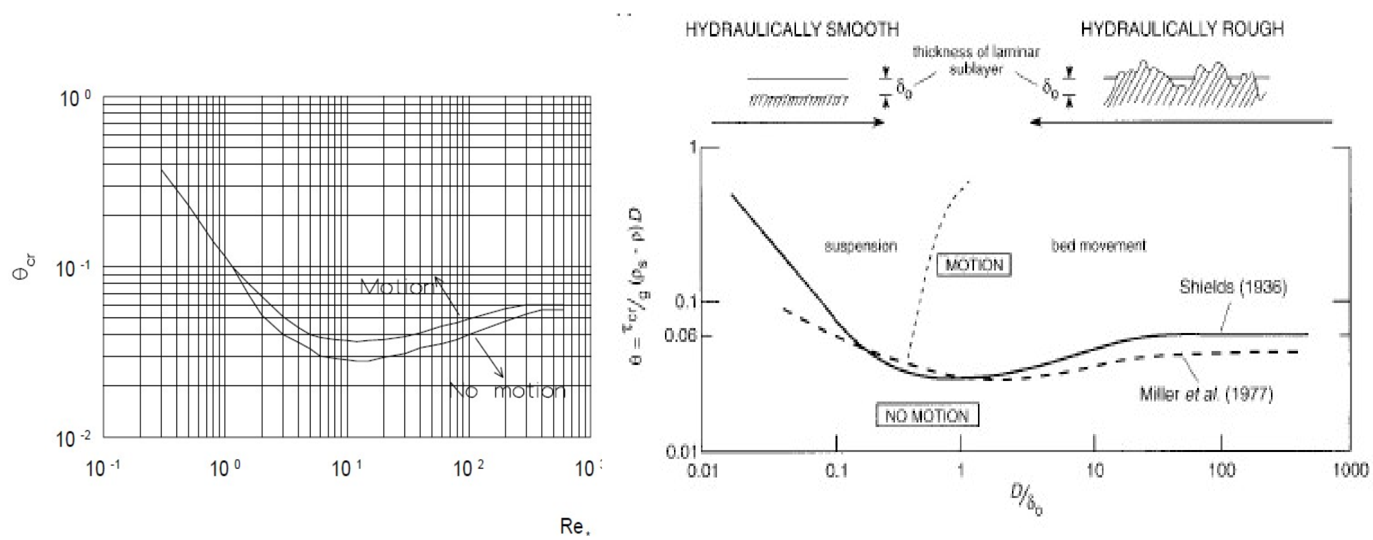

Figure 6.2 Shields' diagram for initiation of motion

Van Rijn in 1993 (Depeweg and Mendez, 2007) expressed the critical particle mobility parameter as a function of particle parameter as follows in order to make the use of the shield diagram easier for the determination of the initiation of motion.

$$\theta_{cr} = 0.24 D_*^{-1} \qquad for \quad 1 < D_* \le 4$$

$$\theta_{cr} = 0.14 D_*^{-0.64} \qquad for \quad 4 < D_* \le 10$$

$$\theta_{cr} = 0.04 D_*^{-0.1} \qquad for \quad 10 < D_* \le 20$$

$$\theta_{cr} = 0.013 D_*^{0.29} \qquad for \quad 20 < D_* \le 150$$

Also the initiation of suspension is expressed as follows (van Rijn, 1984)

$$\theta_{cr} = \frac{16}{D_*^2} \frac{w_s^2}{(s-1) g d_{50}} \qquad for \quad 1 < D_* \le 10$$

$$\theta_{cr} = 16 \frac{w_s^2}{(s-1) g d_{50}} \qquad for \quad D_* > 10$$

6.3.2.1 Analysis of Sediment transport

Coarse sand is highly concentrated near the bed and declines with height at a faster rate than fine sand. Fine silt is so easily suspended that it is far more uniformly distributed in a vertical section than is the coarser material. Similarly, the grain-size distribution within a sample of sand displays far more vertical variation than does the vertical distribution of grain size within the silt range. The former is too large for the flow to move much of it into the upper water column and the latter is so small and easily suspended that it is well represented at all levels thus giving rise to a more uniform grain-size profile.

Following the above explanation, the sediment transport has been analyzed for range of expected sediments particle size in Omo river (from silt to fine sand) and the result is presented in Table 6.5. Accordingly, the result revealed that sediment in incoming discharge is in motion and no risk of sedimentation.

Table 6.5 Sediment transport

Parameters	**Symbol**	**Unit**	**Silt**		**sand**	
Median diameter	d_{50}	mm	0.04	0.062	0.125	0.2
Density of water	ρw	Kg/m^3	1000		1000	
Density of particle	ρs	Kg/m^3	2550	2650	2650	2650
Acceleration due to gravity	g	m/s^2	9.81		9.81	
Specific density of sediment	s	-	2.55	2.65	2.65	2.65
Kinematics viscosity of water	ν		0.000001004		0.000001004	
Shear velocity	u_*	m/s	0.283	0.283	0.283	0.283
Particle parameter	D_*	-	0.99	1.56	3.15	5.05
Critical Particle mobility parameter	θ_{cr}	-	0.2	0.2	0.1	0.05
Particle Reynolds number	$Re*$	-	11.28	17.49	35.26	56.41
Remark			*Sediment are in motion zone according to Shield diagram*			

6.4 Considerations in Design of Sump

According to the American National Standard for Pump Intake Design and guideline of the Hydraulic Institute Standard the performance of sump is largely determined by characteristics of the approach flow i.e. the direction and distribution of the flow at the entrance to the sump. The ideal condition exists when the structure draws flow with no cross flow in the vicinity of the intake structure (suction pipe inlet mouth) that creates asymmetric flow pattern approaching any of the pump. As a general guide cross flow velocity are significant if they exceed 50% of the pump bay entrance velocity. It is also important to provide adequate depth of flow to limit velocities in the sump bay, reduce potential for formation of surface vortices and adequate pump bay width to limit the maximum pump approach velocity to 0.5 m/s but narrow and long enough to channel flow uniformly towards the pump are the basic design requirement for satisfactory hydraulic performance of sump. Water should enter the sump horizontally with no free fall (Butterworth-Heinemann, 1998).

Performance of pump is best when all suction intakes bell mouth clear the floor of sump by half bell mouth diameter. The trench of sump should be at least be twice the bell mouth diameter wide and extend up to a level at least twice the bell mouth diameter above the pump intake mouth. Intakes may be spaced at least 2.5 timed the bell mouth diameter (centre to centre), but the spacing must leave enough room around for machinery and other equipments for ease in construction as well as maintenance. In general a clear spacing of at least 1 m on each of three sides is a minimum for a single pump. The last pump intake clears the end wall by at least one quarter of the bell mouth diameter to inhibit surface vortices (Butterworth-Heinemann, 1998). An anti-rotation baffle can be provided if deemed necessary.

6.4.1 Suction Pipe

In order to make the flow velocity below the maximum limit a 900mm suction pipe is recommended. The minimum submergence, *S* required to prevent strong air core vortices in to the bell mouth entrance at suction inlet is designed using several alternatives (Butterworth-Heinemann, 1998).

Option 1: 1.6 times the bell mouth diameter
Option 2: twice the bell mouth diameter
Option 3: based on Froude number according to Hecker (1987) as cited by Butterworth-Heinemann (1998) *S* should be calculated as

$$S = D(1 + 2.3Fr)$$

and

$$Fr = \frac{V}{\sqrt{gD}}$$

Where *V* is the velocity at suction inlet in m/s, *D* is the outside diameter of the bell mouth shaped suction pipe inlet in m, *g* is the gravitational acceleration (i.e. 9.81 m/s^2).

Table 6.6 Suction pipe spacing used

Description	Minimum Recommended* (m)	Designed
Spacing between two suction pipe inlet mouth	1.8 – 2.25	2.95 (7.26)
Submergence of suction pipe inlet mouth	1.5 – 2	1.6
Suction inlet mouth clearance from sump wall	0.25 – 0.5	1.5 - 2.05
Suction inlet mouth clearance from floor of sump	0.25 – 0.5	0.5

*according to Butterworth-Heinemann (1998) it is a function of suction pipe inlet mouth diameter.

6.5 Selection of Pump

Two types of pump have been proposed as an option; horizontal shaft centrifugal surface pump and vertical shaft turbine pump. The vertical shaft turbine pump is proposed with zero suction head and operating submerged while the motor is placed above high flood level. After discussion on feasibility, performance, efficiency, etc and with suggestion, opinion and recommendation by the client finally a horizontal shaft double suction centrifugal pump is selected.

The crop water demand study report for the planned irrigated area indicates that the peak discharge of 10 m^3/s. Therefore, the selected pump and its arrangement are required to provide the required discharge in addition to the lift in head requirement. However, having a pump type that full fill the demand requirement will not met by a single pump. It is common to encounter a pump that fulfill the design discharge requirement but not the head (energy) or vice versa. Therefore, it is quite often to select a certain pumps that satisfy the head requirement with possibility of having the required discharge requirement with operating point near the best efficiency point. In this regards, the maximum pump discharge capacity that can be obtained in the market is assumed to be $1m^3/s$. Furthermore, the pump specification shall fulfill that the type of liquid pumped have solid with a maximum solid diameter equivalent to sand particles (i.e. 0.2mm size).

6.5.1 Total Dynamic Head

Total dynamic head, H_d is the potential energy imparted in to the water in order to lift the water from minimum water level of the river (or minimum sump water level) to a level granting full supply level (FSL) of the main canal at the de-silting basin. It is the sum of static head (H_S) and headloss (H_L) in the system.

The main pumping station is designed to lift the required irrigation water from the minimum water level of the river (373m) to full supply level (FSL) of the main canal at the beginning (423m). This indicates a static head lift of 50m. The remaining part of command area located above the FSL level will be irrigated by using boosting pump station located at the end of the proposed main canal.

6.6 Pipe Size

The optimum pipe diameter is usually selected based on economic and head loss analysis by making a trade-off between pipe cost and power requirement. The suction pipe diameter is 900mm in order to limit the flow velocity below 1.6m/s (Table 6.7). The client has already purchased/ordered an 800mm diameter steel pipe which is going to be used for the delivery section. The pump type is also selected (i.e. 1 m^3/s). Therefore, the viability of the ordered pipe diameter to comply with the recommended pipe flow velocity (i.e. below 1.6m/s) especially to avoid water hammer effect and its consequence on maximum pressure developed inside pipe is required to be evaluated and should be confirmed not to cause any danger. The delivery pipe flow velocity under the existing circumstance is in excess of recommended maximum limit (Table 6.8).

Table 6.7 Pipe Friction Loss on suction pipe

Design discharge	Roughness Coefficient	Pipe Diameter	Pipe Length	Friction Loss	Velocity
Q_d	C	D	L	h_f	V
m^3/s	-	m	m	m	m/s
1.0	130	0.9	18	0.04	1.57

6.6.1 Head Loss in Steel Pipe

The flow in a pipe entails energy or head loss as a result of friction, bends and joints. These losses are required to be considered during the design of the pumping system. Head loss in pipe can be classified as major loss (which is the fiction loss) and minor losses (associated with bend, fittings, valves, expansion/contraction, etc). The pumping system energy should be sufficient enough to overcome the cumulative head loss and the lift (static head) in order to produce the desired discharge In general,

$$H_L = h_f + h_m$$

Where H_L is total head loss in m, h_f is frictional head loss in m, and h_m is minor headloss associated with fittings, valves, bends, etc in m.

6.6.1.1 Pipe Friction Loss

When the water flows through the pipe the pressure decreases due to friction against the wall of the pipes. The magnitude of friction loss is calculated using Hazen William formula;

In SI unit

$$h_f = 10.7 \frac{L}{D^{4.87}} \left(\frac{Q}{C}\right)^{1.852}$$

Where h_f is the pipe friction headloss in m, L is the length of pipe in m, Q is design discharge in m^3/s, D is the pipe diameter in m, C is the roughness coefficient of pipe material ($C = 130$ for steel pipe).

Table 6.8 Pipe friction loss on delivery pipe

Design discharge	Roughness Coefficient	Pipe Diameter	Pipe Length	Friction Loss	Velocity
Q_d	C	D	L	h_f	V
m^3/s	-	m	m	m	m/s
1.0	130	0.8	500	1.93	1.99

6.6.1.2 Head Loss in Pipe Transition and Appurtenance

Pumping stations contain so many pipe transitions (bends, contractions) and appurtenances (valves, meters) that head losses due to form resistance (turbulence at discontinuities) are usually greater than the frictional resistance of the pipe. The simplest approach to design is to express the head losses in terms of the velocity head, $\frac{V^2}{2g}$, usually immediately upstream of the transition or appurtenances. The equation for these losses can be expressed as cumulative formula;

$$h_m = \sum K \frac{V^2}{2g}$$

Where *K* is the head loss coefficient (dimensionless) and *g* is acceleration due to gravity (i.e. g = 9.81m/s^2).

The value of *K* for different fittings, bends, contractions and valves is given in Table 6.9.

It is required to be understood that there might be more than one transition in the pipe system. Therefore, the type of transitions and their numbers should be understood before carrying out the computation so that the total minor losses can be properly estimated.

Table 6.9 Minor head losses in pipe

Transitions and appurtenant	Headloss coefficient, K	Number	Headloss, hm
1. Suction Pipe			
Single flanged strainer with foot valve DN 900	0.7	1	0.09
Single flanged Elbow (90^o) DN 900	0.75	1	0.09
Double flanged reducer DN 900	0.08	1	0.01
Double flanged butterfly valve DN 900	0.5	1	0.06
Total suction side head loss			**0.26**
2. Delivery Pipe			
Double flanged butterfly valve DN 800	0.3	2	0.12
Double flanged Elbow (45^o) standard SR DN 800	0.3	6	0.36
Double flanged Elbow (22.5^o) standard SR DN 800	0.15	6	0.18
Double flanged Elbow (45^o) standard LR DN 800	0.2	6	0.24
Double flanged Elbow (22.5^o) standard LR DN 800	0.1	6	0.12
Double flanged reducer DN 800	0.08	1	0.02
Swing check	2.5	1	0.50
All type	1	1	0.20
Total delivery side head loss			**1.75**

6.7 Net Positive Suction Head

The absolute pressure plus the velocity head at the eye of the impeller converted to absolute total dynamic head is called the net positive suction head, NPSH. Pump performance declines rapidly as the NPSH becomes less than the NPSH required, NPSHR.

The NPSH available (NPSHA) in the actual installation is calculated using the following equation

$$NPSHA = H_{bar} + h_S - H_{vap} - h_{fs} - \sum h_m - h_{vol} - F_S$$

Where H_{bar} is the barometric pressure in m of water column corrected for elevation above mean sea level, h_S is the static head of the intake water surface above the eye of the impeller (if the water surface is below the eye, hs becomes minus), H_{vap} is vapor pressure of the fluid at the maximum expected temperature, h_{fs} is pipe friction in m between the suction intake and the pump, Σh_m is the sum of minor pipe friction losses such as entrance, bend, reducer, and valve losses, h_{vol} is the partial pressure of dissolved gases such as air in water (customarily ignored), F_S is a factor of safety used to account for uncertainty in hydraulic calculations and for the possibility of swirling or uneven velocity distribution in the intake (this is usually ignored in most design).

The NPSHA is calculated and presented in Table 6.10. In general, always make sure NPSHA is at least 1.35 times NPSHR, and should never be less than 1.5 m from NPSHR. But be aware that to eliminate cavitations and its effects on TDH entirely, the NPSHA must, depending on the operating flow rate relative to the best efficiency point (*bep*), be 2 to 5 times the NPSHR (Butterworth-Heinemann, 1998).

Table 6. 10 NPSHA in meters

H_{bar}	h_S	H_{vap}	h_{fs}	$\sum h_m$	h_{vol}	**Total**
9.74	-2	0.904	0.04	0.26	0	6.55

6.8 Power Requirement

After the preliminary selections of the main pumping equipment have been made, estimates of total station power requirements can be prepared. The electrical engineer should produce a preliminary estimate of station service requirements and discuss the alternative availability of power for the project with the local electric utility. Therefore, the adequacy and reliability of the power supply is required to be determined. The power required by the pump determines the power which the electric or diesel motor supplies.

6.8.1 Output Power

$$P_o = \gamma Q H_d$$

Where P_o is the output power in watt, γ is unit weight of water (N/m^3), Q is design discharge (m^3/s) and H_d is total dynamic head (m).

6.8.2 Input Power

$$P_i = \frac{P_o}{\eta_o}$$

Where P_i is the input power and η_o is overall efficiency.

Table 6.11 Summary of Power Calculation

Design Discharge	Total Dynamic Head	Unit Weight of Water	Overall Pump Efficiency*	Output Power	Input Power
Q	**H_d**	γ	η_o	P_o	P_i
m^3/s	m	N/m^3	%	KW	KW
1	56	9810	63.8	549.4	861.7

* overall pump efficiency assumes pump efficiency (η_p= 75%) and motor efficiency (η_m = 85%)

6.8.3 Power Source

There is on hydro-electric power supply grid in the vicinity of the project area. Therefore, diesel engine is one of the options to provide the required power to the pumps. The selection of the type of the engine is described in the Electro-mechanical and associated work section.

6.9 Energy Requirement

The energy requirement is directly proportional to discharge, total dynamic head, efficiency of the pumping and irrigation system. The monthly energy requirement per hectare has been calculated on the bases of the project total crop water requirement from planting up to harvesting for cotton and other proposed crops. The amount of fuel required to produce this energy also has been quantified and given in Table 6.12.

Table 6.12 Monthly energy requirement per unit land

Month	Duty, l/s/ha	Rating Head, m	Power	Energy Required* per ha (KWh)
	l/s/ha	**m**	**w**	**KWh**
January	1.56	56	857.0	26.9
February	1.30	56	714.2	22.4
March	0.34	56	186.8	5.9
April	0.60	56	329.6	10.3
May	0.08	56	43.9	1.4
June	0.82	56	450.5	14.1
July	1.32	56	725.2	22.7
August	1.42	56	780.1	24.5
September	1.02	56	560.3	17.6

October	0.08	56	43.9	1.4
November	0.20	56	109.9	3.4
December	1.12	56	615.3	19.3
Total				169.9

*Considering 20 hour pump operation and 164 lt fuel consumption per hour, the total fuel consumption per day per unit land for 10 pumps is 3,280 lt.

6.10 Thrust Restraint for Delivery Pipe

Changes in pipe direction (normally 3 degrees), cross sectional area, or isolation points develop additional forces which act on the pipe and cause the need for thrust restraints. Thrust restraint for transmission pipelines would typically be by use of welded or mechanically restrained joints or poured in place concrete thrust blocks depending upon pipe type. Thrust restraints developed by friction between the pipes and surrounding soil depend upon the pipe coating and the pipe zone backfill material used. Thrust restraint should be designed to accommodate the combination of static and transient pressures, plus a realistic safety factor (typically 1.5). Restraint also would be used on steep slopes or where future excavation may allow the pipeline to separate. For the large diameter steel pipe welded thrust restraint is recommended (HE and MWH, 2005).

6.11 Delivery Pipe Cover for Roadway Crossing

For other non-paved and low traffic roads, open cut crossings are expected to be generally allowed. In some cases, installation of a cased pipe in the open cut to avoid future roadway excavations for repair of the pipeline could be considered. As a rule, the minimum burial for either the pipeline or the casing would be 2.3m below finished grade (HE and MWH, 2005). Deeper cover may be desirable to avoid pipeline damage from future utility excavations.

6.12 Flood protection Embankment

The major purpose of the design of embankment is to protect the main pump station from flooding during the design flood occurrence. At the station the river bank level is much higher than the corresponding high flood level in the river. However the pump house floor level is kept at 370.0 meter to meet the requirement for NPSH of the pumps. This level is much below the high flood level that requires dyke to exclude the house at the bank from the river course.

The design of dike will follow the same criteria that are used for homogenous embankment dam. However, the specifications for the dike embankment are not that rigged.

6.12.1 Type of Embankment

The type of material selected for this design is the excavated materials (silty clay materials). If availability and accessibility condition allows other embankment materials can be explored. The type of embankment selected is homogenous embankment. The permeability of the bank material is in the range of semi permeable to permeable. The downstream side of the embankment is provided with 500mm thick reinforced concrete (C-25) which will be formed monolithically with the pump sump bay and one side of the wall of the pump house.

6.12.2 Height of Embankment

The total embankment height from deepest bottom up to top bank level varies depending on the location. However, the top level is set at 386.5 meter level. This level is decided based on the observed flood level (during site visit) and anticipated free board.

6.12.3 Top Width of Embankment

The top width is generally kept 5 meters considering it will also serves as access road for inspection and maintenance. The embankment shall extend in the upstream and downstream of the pump site to safeguard the pump station during high flood.

6.12.4 Embankment Side Slope

The upstream and downstream slopes of the embankment are mainly depends on the type of material used to construct the embankment. Considering the general practice for short height homogenous embankment 1V:2H to 1V:2.5H slope can be used safely. Checked for stability of the embankment can be made by analysis on vulnerable section under worst scenarios. Safety of the embankment during HFL of Omo river (384m above mean sea level) is evaluated in Geo-Slope using range of soil property. In this regards, provision of 1V:2H side slope on the downstream side seems unsafe when analyzed for HFL if the embankment height exceeds 8m. Therefore, sides slope of 1V:2.5H shall be adopted under such site specific condition. The downstream side slope is valid for embankment which is away from the vertical concrete shear wall.

6.12.5 Embankment Erosion Protection

500 mm thick stone pitching over the embankment material can be used as erosion protection on the upstream slope side of the embankment. The downstream side slope of the embankment consists of vertical concrete shear wall at the sump bay and slope in the range of 1V:12H to 1V:2.5H depending on the site condition and soil material used for the embankment.

6.13 Settling Basin Design

6.13.1 Planning

Designing the facilities for sediment removal from irrigation water has long been based on the principles of Stokes' law. The sedimentation basins were designed on the basis of retention time for discrete particles or sediment to settle which a concept is still persists with several modifications. The facilities can be called tanks, basins, ponds or lagoons, basically, the facility is commonly used in irrigation canals in which the in flowing water contains non-cohesive sediment particles. The facility design method utilizes the relation between the particle settling velocity and the forward velocity. Furthermore, the basin should be designed in such a way that suspended particles settle within as short distance as possible and allow adequate space for sediment accumulation. In some design this purpose is achieved by providing an adverse slope in the direction of flow and so that the settling depth is so limited with forward velocities low enough to avoid suspending settled particles in suspension. However, in order to utilize the purpose of the facility up to their expectation the structures must be designed for maximized sedimentation. The following design is based on Class I type sedimentation tank which considers unlimited settling of discrete particles in a continuous flow.

Settling basin is one of an important structure where the plan source of irrigation water is direct river intake where the river flow is characterized by high concentration of sediment transport. In this regard, several water resource study report for the water source of this project (i.e. Omo river) and field visit observation carried out has revealed the presence of high concentration of sediment. The sediment is composed of predominantly silt, clay and fine sand particles during normal flow.

For a continuous flow sedimentation basin, which is the case here, the rectangular shaped basin can be used. According to Camp (1953) the basin is divided in to four zones which affect settling process, namely the inlet zone, theoretical effective settling zone, sludge zone (beneath the settling zone), and outlet zone.

6.13.2 Design considerations

The design principle of settling basin considers the following points;

I. The settling basin must have length and width dimensions which are large enough to allow settling of the sediments but not so large that the basin is over expensive and bulky.
II. It must allow for easy flushing out of deposited sediments to undertaken an acceptable or reasonable frequency/intervals.
III. Flushing water must be led carefully away from the structure in order to avoid erosion of soil in the surrounding area of the basin structure.
IV. Sufficient capacity must be allowed for collection of sediment.

6.13.3 Basin layout and arrangement

- In order to satisfy the requirement for good hydraulic performance the settling basin structure is arranged to have four main components: inlet zone, settling zone, outlet zone and sediment removal and or basin draining sluice.
- To ensure the continuous operation of the irrigation system two settling basins parallel to each other are provide so that one of the basin remain in operation while the other is under cleaning operation of the deposited sediment .

6.13.4 Sediment Design Particle Size

The hydraulic design of a settling basin is normally begins with analyzing the quantity and quality of sediment carried by the river and determining the necessary degree of removal on the basis of theory and practical experience but in the absence of the enough sediment data the minimum limiting size of the suspended matter allowed to deposit in the settling basin is determined to be 0.2mm (sand), which is the minimum diameter of particles acceptable in irrigation water.

6.13.5 Flow through Velocity

The highest permissible flow-through velocity *V* should also be specified, considering that particles once settled should not picked up again. According to *Camp as cited by Mosonyi (1991)*, the critical flow-through velocity is estimated from:

$$V = a\sqrt{d}$$

Where *d* is the equivalent diameter of the smallest sediment particle to be settled in (d = *0.2mm)* and *a* is constant given as a = *0.44 for d = 0.2mm* (Mosonyi, 1991).

However, Monsonyi (1991) depicted that velocities other than the computed flow through velocity will ensue along the length of the settling basin during operation.

6.13.6 Settling Velocity

The terminal settling velocity for discrete particle sedimentation is given by (Depeweg and Mendez, 2007)

$$W_s = \sqrt{\frac{4}{3}\frac{gd(\Delta - 1)}{C_D}}$$

And

$$\Delta = \frac{\rho_s}{\rho_w}$$

Where W_s is terminal settling velocity, ρ_s is mass density of particle in kg/m^3, ρ_w mass density of water in kg/m^3, *g* is acceleration due to gravitation (9.81 m/s^2), *d* is diameter of particle in m, C_D dimensionless drag coefficient.

The drag coefficient is not constant. It varies with the particle Reynolds number and the shape of the particle. The particle Reynolds number can be given by;

$$R_e = \frac{W_s d\mu}{\rho_w} = \frac{W_s d}{\nu}$$

Where μ is the dynamic viscosity of water in N.s/m^2 and ν is the kinematics viscosity of water in kg.m/s.

Therefore, the following relationship for drag coefficient and Reynolds number can be used.

$$C_D = \frac{24}{R_e} \qquad for \qquad R_e < 1$$

$$C_D = \frac{24}{R_e} + \frac{3}{\sqrt{R_e}} + 0.34 \qquad for \qquad 1 < \ R_e < 1000$$

The fall velocity of a spherical particle of diameter 0.2mm and relative density of 2.65 is found to be around 3.0cm/s (0.030m/s) at around 25^{o}C temperature.

6.13.7 Settling Basin Size

The size of a settling basin must have length and width dimensions which are large enough to cause settling of the sediments but not so large that the basin is over expensive and bulky. The length, width and of the settling basin is computed as follows.

Generally the depth of horizontal settling basin should not excessively high in order to avoid heavy excavation work. The depth of horizontal flow settling basin in waterpower project ranges between 1.5 and 4 meter with corresponding velocity not higher than from 0.4 to 0.6 m/s (Mosonyi, 1991).

The length of the settling basin depends on the required distance *L* for settling of the sediment. So a sediment particle with a fall velocity W_s and flowing with the water velocity *V* require a settling length *L* when it enters the settling basin at a height *y* above the bed (fully settled). Therefore, the following relationship can be established (Ankum, 2004)

$$\frac{V}{W_s} = \frac{L}{y} \qquad and \qquad V = \frac{Q}{yb}$$

Therefore, the required minimum length of the settling basin can be obtained by rearranging the above equation;

$$L_{min} = \frac{Q}{W_s b}$$

Accordingly, for the design discharge, basin width of 13m and fall velocity of 0.03m/s; 26 m minimum length of settling basin is required.

An important fact to be considered in design is velocities other than the computed flow through velocity will ensue along the length of the basin. Uniform velocity distribution over the cross section build up only at a certain distance away from the inlet and out let section which result in introduction of transition. Therefore, the flow will expand away from the inlet zone and contract at the exit before discharging over the weir or sill. Owing to this effect the length of the settling basin should be increased to some extent to what has been obtained by computation (Mosonyi, 1991). Therefore, a safety factor of *K* (usually, 1.5 is assumed) is used for adjustment of length of the basin.

On the other hand, the width of basin, *b* can be computed from the continuity equation as:

$$b = \frac{Q}{yV}$$

Analysis for minimum settling basin length has been carried out in Table 6.13. The proposed design size of the settling basin will provide an acceptable range of velocity of flow in the basin.

Table 6.13 Analysis of minimum basin length for range of expected sediment particle size

Width of basin, b	Terminal fall velocity Ws for range of particle sizes in m/s			Head over sill, *H* in m	Sediment entrance height, *y* in m	Basin flow velocity, *V* in m/s
	0.02	0.03	0.035			
	Minimum length of settling basin L in m					
8	62.5	41.7	35.7	0.81	1.7	0.75
9	55.6	37.0	31.7	0.75	1.6	0.69
10	50.0	33.3	28.6	0.70	1.5	0.65
11	45.5	30.3	26.0	0.66	1.5	0.60
12	41.7	27.8	23.8	0.62	1.5	0.57
13	38.5	25.6	22.0	0.59	1.4	0.54
14	35.7	23.8	20.4	0.56	1.4	0.51
15	33.3	22.2	19.0	0.53	1.4	0.48
16	31.3	20.8	17.9	0.51	1.4	0.46
17	29.4	19.6	16.8	0.49	1.3	0.44
18	27.8	18.5	15.9	0.47	1.3	0.42
19	26.3	17.5	15.0	0.46	1.3	0.40
20	25.0	16.7	14.3	0.44	1.3	0.39

In general, the hydraulic behaviour of long narrow tanks is superior to that of wide low velocity tanks and in practice a minimum length to width ratio, L/W, of 2 - 3 is adopted from hydraulic consideration. The summary of result for the settling basin is presented in Table 6.13.

Table 6.14 Summary of settling basin size

Parameter	Symbol	Unit	Value
Sediment Size	*d*	mm	0.2
Fall velocity	*Ws*	m/s	0.03
Length of settling basin	*L*	m	42
Width of settling basin	*b*	m	13

6.13.8 Inlet Zone

The main function of the inlet is to gradually decrease the turbulence and avoid all secondary currents in the basin. This is achieved by decreasing the flow velocity through gradually increasing the flow cross-section, i.e., by providing gradual expansion of the width and depth.

In order to achieve a uniform approach of water over the whole chamber width, the transition is designed using the formula.

$$I = \frac{B - b}{2tan\theta} \leq \frac{L}{3}$$

Where *I* is length of the transition zone in m, *b* is settling basin width in m, *B* is width of the inlet to transition in m, θ is expansion angle (θ=21°) and *L* is the length of basin in m.

$I = \frac{17-13}{2tan21}$ = 5.2m ≤ 8.4m, hence the design is safe.

The head loss in the inlet transition is determined according to USBR (1978);

$$\Delta h_e = \alpha_{in} \frac{(V_1^2 - V_2^2)}{2g}$$

Where Δh_e is the head loss in the inlet transition in m, α_{in} is the head loss coefficient (usually 0.2 for open channel), V_1 is the flow velocity in the head canal towards the transition in m/s, V_2 is the flow velocity in the settling basin in m/s.

The bed slope of the inlet section should have a flatter slope (usually not steeper than 1 in 100) to prevent any settling out of sediment occurring before the basin (MacDonald, 1990).

However, such design arrangement for settling basin can be possible in river intake (either direct or through head work arrangement). In this regards, the water discharge in to the settling basin from a delivery pipe connected to a centrifugal pump. Therefore, some modification is required to be made. Therefore, the inlet zone of the settling basin is designed to serve as a plunge pool for the flow discharging from the delivery pipe. The water will enter to the two compartment of the settling basin though the inlet sluice gate.

6.13.9 Outlet Zone

This is a kind of transition provided following the settling zone to facilitate getting back the flow into the conveyance system with the design velocity by gradually narrowing the width and depth. The outlet transition wall is designed to flare at 1.3H: 1V. The head loss is computed by (USBR, 1978);

$$\Delta h_{ex} = \alpha_{out} \frac{(V_3^2 - V_2^2)}{2g}$$

Where Δh_{ex} is the head loss in the exit transition in m, α_{out} is the head loss coefficient (usually 0.5 for open channel), V_3 is the flow velocity in the tail end canal downstream of transition in m/s, V_2 is the flow velocity in the settling basin in m/s.

6.13.10 Flushing design

In order to scour the deposited sediment from the settling basin, it is essential to provide flushing facility. The flushing facility shall have a sloping bed in order to produce scouring velocities so that sediment deposited will be cleared away from the settling basin. The flushing is controlled by provision of sluice gates.

For the assumed D_{50} size of 0.2mm scouring velocity V_{SC}= 1m/s from Figure 6.3, The bed slope required in the settling basin to provide this scouring velocity can be calculated as (MacDonald, 1990):

$$q_s = \frac{Q_d}{B}$$

Where q_s is the specific discharge in m^3/s/m, Q_d is the design discharge in m^3/s, and B is the width of the settling basin in m.

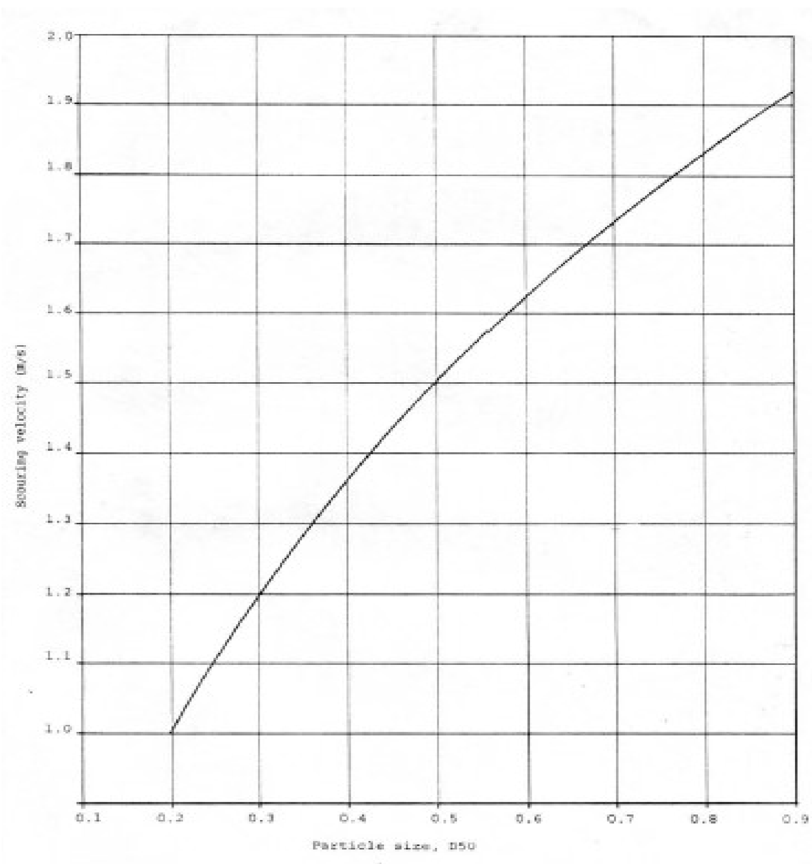

Figure 6.3 Scouring velocity in sand trap (MacDonald, 1990)

The scouring depth of flow in the basin can be

$$y_s = \frac{q_s}{V_{sc}}$$

Where y_s is the scouring depth of flow in basin in m and V_{sc} is the scouring velocity in m/s.

Therefore, the minimum bed slope of the basin is

$$S_o = \left(\frac{nV_{sc}}{y_s^{2/3}}\right)^2$$

Where S_o is the bed slope in m/m and n is the Manning roughness coefficient (usually 0.015 for concrete and 0.02 for masonry work).

The size of the sluice gate and its invert level are prime importance for successful flushing of deposited sediments. If insufficient capacity is provided, the scouring velocity will not develop in the downstream end of the settling basin. The size of the sluice gate and its invert level should be based on the following criteria;

$$y_g = \frac{Q_d}{B_g}$$

Where y_g is the depth of water upstream of the sluice gate in m and B_g is width of sluice gate.

The depth of the sluice gate invert below the bed of the settling basin at the downstream end is given by

$$y_g + 0.05 - y_s$$

The slope of the sluiceway outfall downstream of the sluice gate should be sufficient enough to produce supercritical flow in the channel. The hydraulic calculation and summary of the parameters of the settling basin is presented in table 6.15.

Table 6.15 Summary of settling basin parameters

Parameters	**Symbol**	**Unit**	**Value**
Bed width of settling basin	B_w	m	13
Scouring velocity in basin	V_{sc}	m/s	1
Scouring depth of flow in basin	y_s	m	0.77
Required minimum bed slope of basin	S_c	m/m	0.0003
Width of flushing sluice gate	B_g	m	2.5
Depth of water upstream of flushing sluice gate	y_g	m	4.00
Invert level of flushing sluice gate (provided)		m	2.9
Basin slope provided	S_o	m	0.012

Inlet Invert level of basin at plunge pool		m	419.1
Outlet Invert level of basin at flushing sluice gate		m	418.60
Maximum water level in settling basin (design)			423.20
Free board of settling basin	*FB*	m	0.50
Top wall level of settling basin	*TBL*	m	423.70
Overflow weir Sill level at exit to outlet zone		m	422.40
Head loss in transition at Inlet zone of basin	Δh_e	m	0.03
Head loss in transition at outlet zone of basin	Δh_{ex}	m	0.02
Head loss in sluice gates		m	0.15
Calculated water level at main canal inlet		m	423.00
Designed water level in main canal		m	423.00

6.14 Design of Boosting pump station

The main canal can only irrigate command below 420 m level. The FSL at the end of main canal is 421 m. A Boosting pump station will lift the water in to a new canal that starts 3500m away from the end of the main canal with destination elevation of 490m above mean sea level. A gravity flow will commence again in the new canal. The Boosting pump station left the water at an FSL of 489 m at the beginning of the canal. The total length of the delivery pipe is 3500m.

The lined part of the main canal can be used as a sump bay for the suction steel pipe having 600mm in diameter. The delivery pipe is 500mm diameter steel pipe. The design discharge of the boosting pump station is 4.0 m^3/s and the total static lift head is 68m. The boosting pump arrangement is shown in figure 6.4.

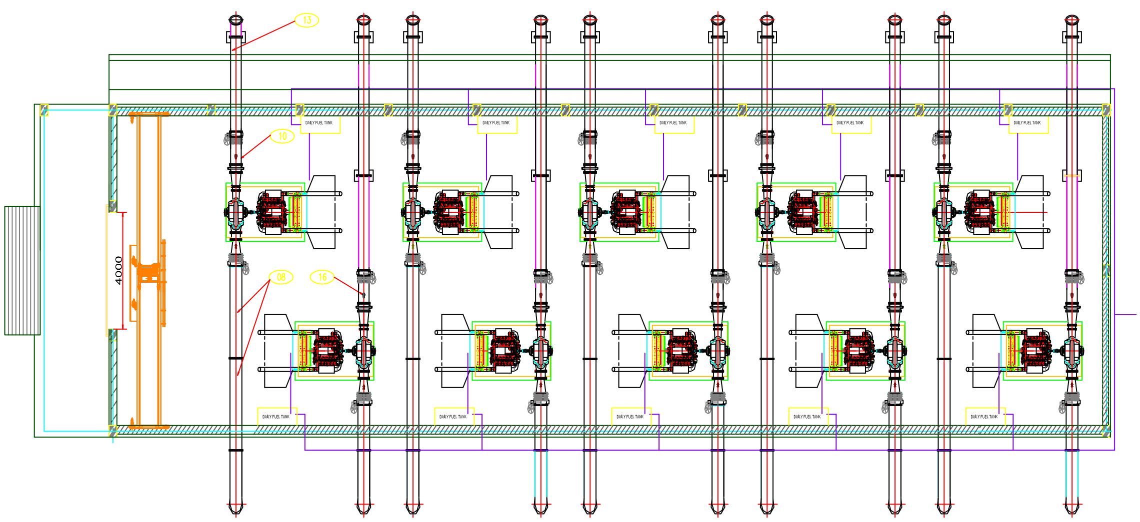

Figure 6.4 Pump house arrangement for boosting pump station

6.14.1 Net Positive Suction Head

The Net Positive Suction Head Available for the boosting pump station is also calculated and presented in Table 6.15.

Table 6.16 NPSHA of boosting pump

H_{bar}	h_s	H_{vap}	h_{fs}	$\sum h_m$	h_{voi}	**Total**
9.88	-1.5	0.76	0.04	0.12	0	7.46

6.14.2 Total Dynamic Head

The static lift of the boosting pump station is almost 68m. The length of the delivery pipe is 3,500m and the associated head loss due to friction is assumed to be 14m. .

6.14.3 Power and Energy Requirement

The energy requirement for the boosting pump station is also presented in Table 6.16.

Table 6.17 Monthly energy requirement per unit land for boosting pump station

Months	**Duty**	**Rating head***	**Power**	**Energy requirement**
	(l/s/ha)	**(m)**	**W**	**KWh**
January	1.56	81	1239.6	38.9
February	1.3	81	1033.0	32.4
March	0.34	81	270.2	8.5
April	0.6	81	476.8	15.0
May	0.08	81	63.6	2.0
June	0.82	81	651.6	20.4
July	1.32	81	1048.9	32.9
August	1.42	81	1128.3	35.4
September	1.02	81	810.5	25.4
October	0.08	81	63.6	2.0
November	0.2	81	158.9	5.0
December	1.12	81	890.0	27.9
Total				**245.80**

*Almost 14m head loss due to friction is assumed in 500mm diameter and 3,500m long delivery pipe which depends on pump discharge around 300l/s.

7. Structural Design

7.1 General

Omo Farm Irrigation Project consists of reinforced concrete structures that require special attention such as intake, Irrigation conduit, pump house, and sump.

The structure is subjected to different types of loading with varying magnitude within their design period. Within this period, the structures shall withstand the loads, with tolerable damage and deliver services for which it is designed. So that it delivers its intended services, the structures has been designed to safely carry extreme load (ultimate load) with %5 probability of being exceeded .Moreover, it is designed to carry service load exhibiting negligible cracking and deformations on to the structures.

Generally speaking, ultimate limit state of design approach has been followed to design the structure. Under limit state design approach, structures are designed for service as well extreme loads.

For the service loads, especial attention was paid to checking whether stresses induced is stress which leads to considerable cracking of the structures. The checking is so important because the structure is exposed to water which may result in corrosion of the re-bars, if the crack has excessive width. However, by satisfying the precondition set by codes to control requirement for the crack width, it enables to minimize the possibility of concrete being corroded. In addition, is possible to further lower possibility of re-bar corrosion by limit permeability of concrete. Limiting permeability of concrete is possible by choosing right components and their mixing ratio during mix design. Therefore, by limiting crack width and the permeability of concrete, durability of concrete will be ensured.

The responses of the structures to extreme loads and the service loads were determined with help of SAP2000, software which has capability of static as well as dynamic finite element analysis. The latest version, version 15, of the software was employed for the same purpose. The responses were obtained at critical sections of the structures, as result of the analysis. The responses of special interest were bending moments and shear forces acting along the base of structures.

The induced bending moments shear forces were compared with capacity of the concrete reinforced critical sections. The determinations of the capacity were based on the recommendations of ACI code. In addition to it, the provisions of BS were invoked to help determinations of the minimum amount re-bars required for purpose of controlling temperature induced as well as shrinkage induced stresses when the external load triggered stresses were found to be well below the design tensile strength of the concrete section of the concrete.

The stability analysis and structural designing for the sumps and intakes has been conducted as per the following codes:-

1. BS8007: Designing of Concrete Structures for Retaining Aqueous Liquids.

2. EUROCODE-2: Designing of Concrete Structures.
3. EUROCODE-8:Designing of Structures for Earthquake Resistance
4. ACI-318-08:-BUILDING REQUIRMENTS FOR STRUCTURAL CONCRETE

In addition to checking its capacity with regard to stresses caused by service as well as ultimate loads, the resistance of the dam to sliding and overturning has been investigated as per requirements of EM-1110-2-2200.

7.2 Design Inputs

Results of the previous hydrological, geological and geotechnical studies study provided inputs for structural analysis and design of the sump and the intake. In addition to these reports, drawings issued after hydraulic design, provided design in puts for structural modeling of the structures and subsequent analysis and design of the same.

The analysis and designing started by selecting appropriate grade of the concrete and specifying design foundation material property. The properties of the concrete and soil foundation and their respective partial safety factors are as specified here below.

7.2.1 Material Properties

Material inputs for designing of the of concrete structures:-

- Unit weight of concrete=24KN/m^3,
- Unit weight of embankment soil=18 KN/m^3,
- Characteristic cubic compressive strength of concrete =30Mpa.
- Characteristic cylindrical tensile strength of concrete =3.8Mpa.
- Design cylindrical tensile strength of concrete =2.53Mpa.
- Characteristics minimum yield strength of re-bars=413N/mm^2
- Design shear strength of plain concrete=0.44Mpa.
- Modulus elasticity of reinforced concrete=23Gpa.
- Angle of internal friction of embankment material=28.14^0,
- The minimum allowable pressure of foundation material =420Kpa.
- The Poisons ratio of reinforced concrete=0.2
- The martial partial safety factor utilized for concrete and steel are 1.5 and 1.15 respectively.
- Coefficient of friction for foundation material=0.25

7.2.2 Inputs from Hydrological Study

Maximum and minimum pool levels (384m and 373.03m above sea levels) which had been fixed based on the hydrological study and crop requirement demand were used as inputs for purpose of modeling static load due to static water walls of sump and intake. For purpose of structural design the phreatic line has been assumed to be horizontal in the embankment, giving the same levels as mentioned here above near the walls outside faces.

7.2.3 Inputs from Hydraulic Design

Civil drawings, the result of hydraulic design, were used as source preliminary dimension for modeling and the conducting structural analysis and designing of the structures. Finally, the preliminary dimensions of the structures have been revised in order to meet stability requirements and flexural requirement.

7.2.4 Inputs from Seismic Hazard Analysis

Seismic activities in Ethiopia are generally said to be confined to Afar and the main Ethiopian rift valley. The Main Ethiopian Rift (MER), which is part of the East African Rift System, and Afar Depression are considered to be locus of volcanic and seismic activities as they represent extensional tectonics in action. The MER meets the two oceanic rifts, namely Red Sea and Gulf Aden in Afar Depression/Triangle forming three-rift (RRR) triple junction.
The Omo Valley Farm project area is located in the Sothern part of the main Ethiopian rift valley, which is seismically active area. According to the seismic Zoning map of Ethiopia (Figure: 7-1), the project site falls under Zone 4 – corresponding to a zone of major damage where the seismic ground shaking would produce intensity VIII and above.

The horizontal and vertical loads under pseudo-static analysis are represented by appropriate seismic coefficient to give the design acceleration as a fraction of the acceleration due to gravity.

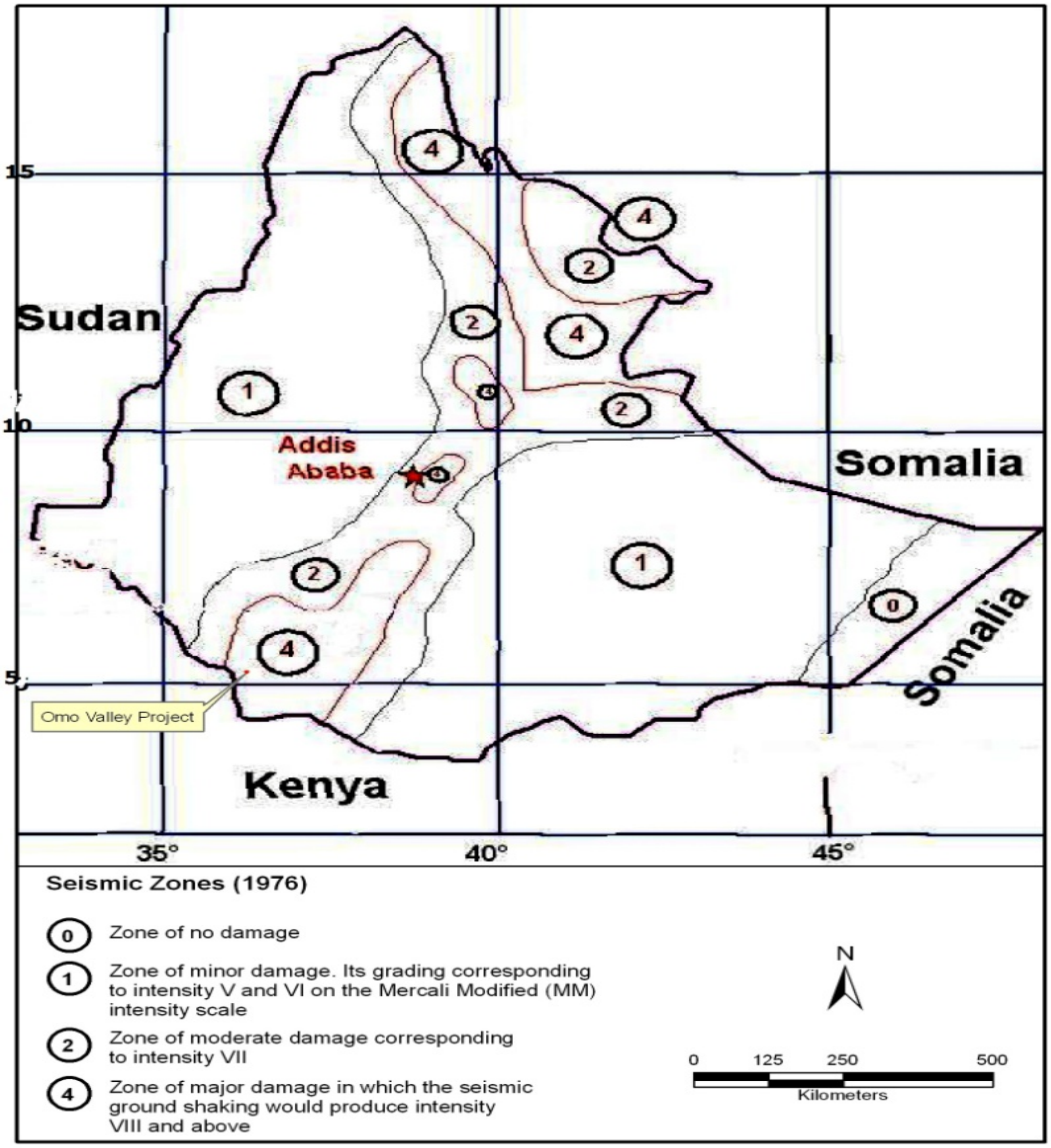

Figure 7.1 Seismic Zoning Map of Ethiopia

Remarks: The '<u>Seismic Risk Map</u>' produced by <u>Laike Mariam Asfaw(1986)</u> for a hundred period and 0.99 probability shows that the study area <u>falls within 8MM scale.</u>

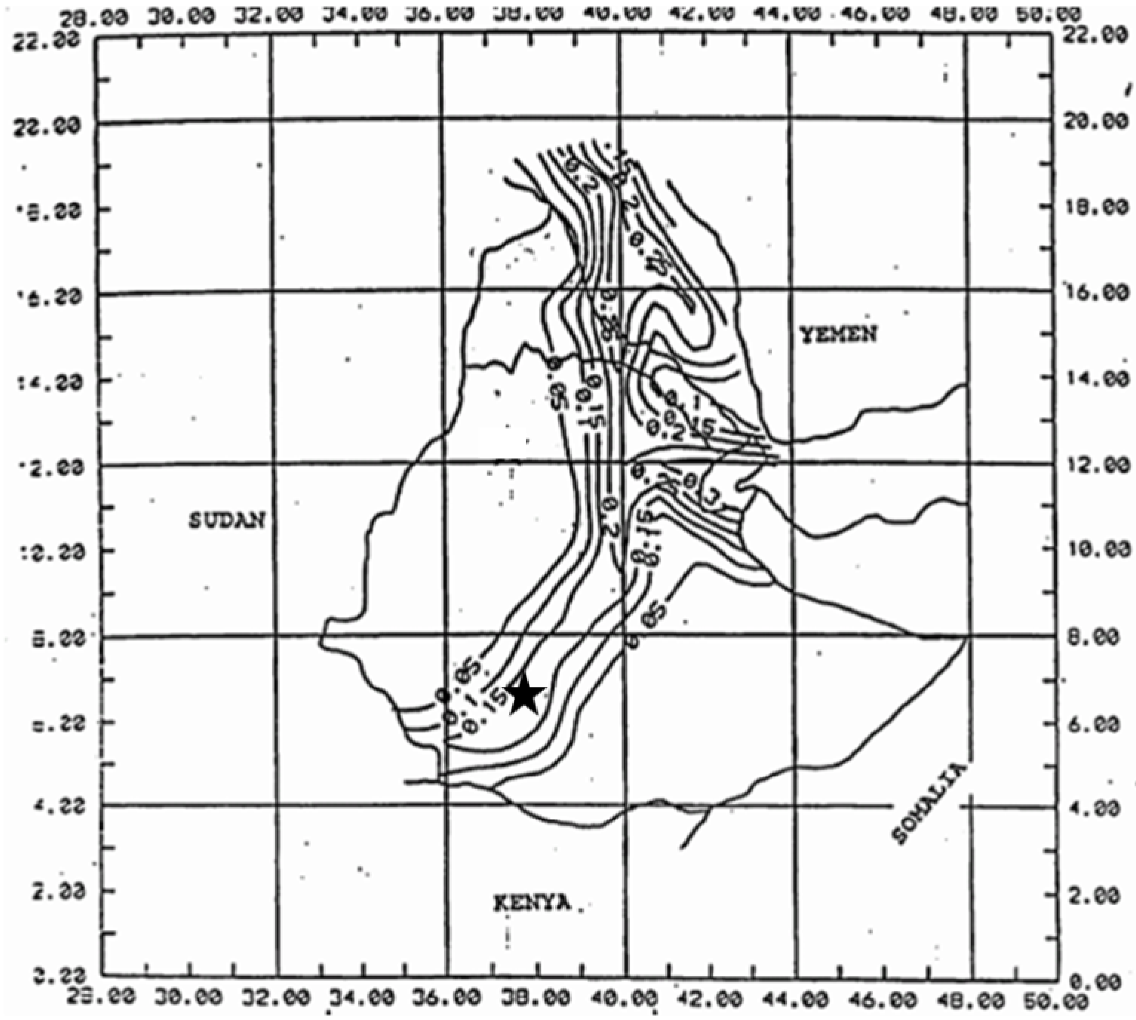

Figure 7.2 Seismic Hazard Map of Ethiopia and its Northern & Eastern Neighboring Countries, Contours indicate peak ground accelerations as a fraction of g. The Black Star Indicates the Approximate Location of Omo Valley Farm site.

Figure7.2 shows seismic hazard map of Ethiopia prepared by the Institute of Geophysical Observatory at Addis Ababa University for a Design Base Earthquake (DBE) with a return period of 300 years (return period for DBE of dams/civil structure is generally taken to be 300 to 400 years). This map shows peak ground acceleration contours as a fraction of acceleration of gravity *g* and has been used as the basis for seismic design of several other dams in Ethiopia. The black star on this map shows the approximate location of the farm site in Omo-Gibe River Basin with geographic coordinates of 05^{O} 10' to 05^{O} 16' Northing and 36^{O} 12' to 36^{O} 17' Easting.. Based on this map, the nearest contour to the farm site is with a ground acceleration of 0.15g. Therefore, according to this hazard map, the design horizontal peak ground of acceleration would be 0.15.

Regarding vertical peak ground accelerations, EUROCODE-8 recommends the vertical peak ground acceleration could be taken as the 90% 0f horizontal ground acceleration for structural member analysis and design purpose.

From the peak ground acceleration, horizontal seismic coefficient was determined, as explained here below .Moreover; the peak ground acceleration has been utilized to estimate the response spectrum that will be input earthquake load for seismic analysis.

$\alpha_h = \left(\frac{2}{3}\right) * \left({}^{a_g}\!/_{g}\right)$, where α_h is seismic coefficient, a_g is peak ground acceleration, and g is ground acceleration.

α_h =(2/3)*(0.15g/g)=0.10

Therefore, for purpose of analyzing the hydrodynamic load with seismic coefficient method, 2/3 of the peak ground acceleration has been utilized as design ground motion. As result, 0.10 has been adopted as horizontal seismic coefficient.

7.2.5 Inputs from geological investigation

Geological investigations revealed that silty clay formations exist below foundation area of pump house, irrigation conduit and sump. The silt clay is believed to exist up to 2.5m from original ground level. It has been assumed that uniform soil distribution exist up to foundation level of the structures. Laboratory test result revealed that the silt soil has average cohesion of 48.52KN/m^2, angle of internal friction of 28.14^0 and unit weight of 18KN/m3.

Using the engineering property, bearing capacity has been calculated based different approach as described in the geotechnical report. AS result, different value of allowable bearing capacities has been obtained. However, 420KN/m3 has adopted as representative value of the bearing capacity of foundation material of sump, pump house, intake and conduit.

7.3 Specification of construction materials

Construction material properties of the structures were specified in view of ensuring their durability and strength so that they can withstand loads and adverse environmental conditions, with tolerable damage, and carry out their intended functions properly during their design life.

Next to specification of the construction materials, it was assumed that substances harmful to durability of concrete do not exist in foundation material and in reservoir water. Therefore, during construction it should be ascertained that the harmful substances do not exist. The validity of this assumption shall be assured based on results of pertinent investigations.

If harmful substances found to exist in the foundation as well as in water in contact with structures, then necessary measures should be taken to neutralize their effect onto durability of the concrete.

From durability point of view, EUCODE-2 specifies minimum concrete based on class of exposure of the structure. As per the code, the exposure condition of the intake and sump structures may be categorized as class XC2, concrete subjected to permanent wetting and occasional drying.

For expose class of XC2, the code recommends a minimum concrete characteristic strength of C30/37.Therefore, the strength of concrete has been selected to be C30/37.

Concerning re-bars, the code calls for usage of re-bars whose minimum yield strength that falls in between 400Mpa. and 600Mpa. So that its recommendation will be applicable, depending on the above statement the minimum yield strength of re-bars has been chosen to be 420Mpa.

7.4 Structural object modeling

The 3-D structural models of sumps and intakes have been prepared by simplifying the configuration of the structure so as to enable easy in computation during analysis. The modeling has been carried out with help of finite shell elements. Figure-1 and Figure-2 here below shows the 3-D models of the intake and sump.

Since there is expansion joint between them, the sump and the intake have been modeled as separate structures in regard to their response to external load. However, hydraulically they are expected to as unit and for that matter water stop is inserted in between order to make the joint water tight. The meshes of the models have been continuously refined continuously until the difference between consecutive mesh reaches level where it could be considered negligible.

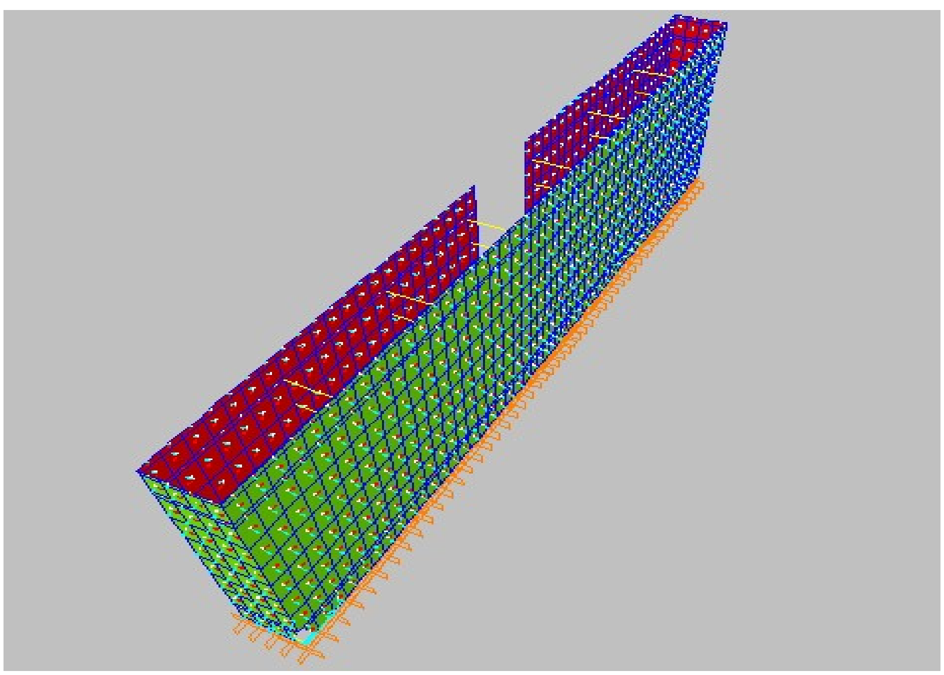

Figure 7.3 3-D Shell Model of the Sump

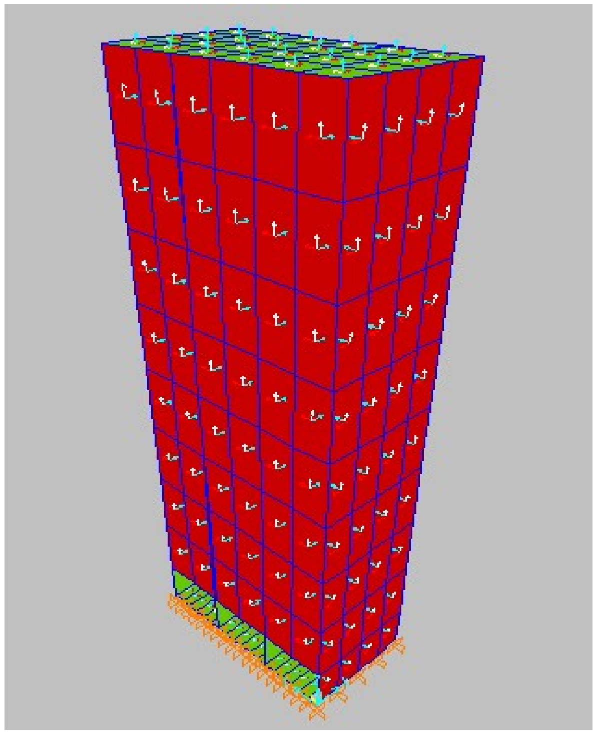

Figure-7.4:3-D Shell Model of the Intake

Being height of 14.65m, the sump requires huge slab thickness to support the internal water load and external earth pressure loads. In order to reduce the load effect, the wall thickness beams at horizontal distance of 7m and vertical distance of 2.0m has been introduced.

The following dimensions have been used for modeling of the structures for purpose of analysis and design.

Intake structure has the following preliminary dimension:-

1. overall internal plan dimension=2.5mX4.85m
2. height=14.80m
3. Has three compartments of overall internal plan dimension=1.22mX2.5m.
4. Two partition walls of 0.6m.

Irrigation conduit has been the following preliminary dimensions:-

5. Internal diameter =1.2m
6. Total length of 40m

7. Wall thickness of 0.15m
8. Precast unit of 1m length has been proposed

The sump has the following preliminary dimension:-

9. Overall internal dimension of 48.75mX3m
10. Overall internal height=13.8m

Final thickness of walls of the structures has been fixed after conducting stability analysis ,shear and flexural design.

7.5 Estimation of Service and Ultimate Loads

Estimation of the service and ultimate loads was concerned with calculation of the loads that are exerted directly or indirectly onto the structures. The loads that act onto the sumps and intake, pump were estimated for purpose of stress analysis and subsequent designing of its sections.

The service loads were applied for crack width analysis at base of the structures and at corners where the walls meet. First the service loads were estimated followed by calculation of the ultimate loads by multiplying the service load with their load factors.

The following loads have been considered:-

1. Hydrostatic load (F),
2. Vertical water pressure (F_v),
3. Self-weight of the structures (D),
4. Static earth pressure (H),
5. Seismic inertial load (D_i),
6. Hydrodynamic load (F_E). and
7. Dynamic earth pressure (H_E) load.

The loads combine in following way to give **U**, based on recommendation given by the code:-

Service loads combinations

8. 1.0L+D1.0+1.0F,
9. 1.0L+1.0D+1.0H,
10. 1.0D+1.0L,and

Ultimate loads combinations

11. 1.4(D+F),
12. 0.9D+1.2F,
13. 1.2(D)+1.6(L+H),
14. 1.2D+ 1.6H +D_i
15. 1.2D+1.2F+ D_i
16. 1.2(D+F)+ D_i
17. 1.2D+1.6H
18. 1.4F+0.9 (F_v+D)

C) Loads imposed during construction.

19. 1.2D+1.6(L_s+H)

Compliance with serviceability and ultimate limit state requirements have been checked after determining, the envelops of bending moments, and shear forces.

For instance, crack width limit state has been checked after determining envelops of the service load combinations stated. For crack width calculation, bending moment envelop of the service load combinations has been determined.

7.5.1 Hydrostatic Load

The hydrostatic loads were calculated as linearly distributed loads and the unit weight of the waste water has been taken to be 10KN/m^3.When the water level is at its maximum, 14m in the sump and 13.2m in the intake have been taken as water heights at which water pressures are exerted on the all internal walls of the respective structures. Moreover, it is assumed that the same magnitudes of water pressure are expected to act on water front wall of intake and the sump.

It has been assumed that ground water level is well below foundation of intake and sump and it will not exert uplift load on to the structures. However, further investigation shall be conducted during construction phase in order to check validity of this assumption.

If level of ground water is in region where it affects the response of the structure or itself exerts significant magnitude of uplift pressure, then the stability analysis and structural design shall be revised based on the information directly obtained from the investigation or/and information inferred from the investigation.

7.5.2 Earth Pressure Load

The front walls of sump and intake faces backfill earth pressure, the static and dynamic distribution of earth pressure has been determined based on their respective earth pressure coefficient .Mononbe-okabe's approach has been followed to determine the earth pressure coefficient and estimate the earth pressure.

The following relation has been utilized to calculate the dynamic earth pressure coefficients.

$$K_{AE}=\frac{\sin^2(90+\theta-\phi)}{\cos\theta\,\sin^2(90+\theta+\phi/2)\left[1+\sqrt{\dfrac{\sin 1.5\,\phi\,\sin(\phi-\theta-\beta)}{\sin(90+\phi/2+\theta)\,\sin(90+\beta)}}\right]^2}$$

Where $\theta=\tan^{-1}k_h$, α = wall slope to horiz. (90° for a vertical face), ϕ = angle of internal friction, β = backfill slope, and δ = wall friction angle.

At rest, active, passive and active dynamic earth pressure coefficients are 0.73, 0.54, 2.0, and 0.07 respectively, (K_h=0.075) has been taken to be one half of the normalized peak ground acceleration (0.15).

Surcharge loads due to parking of heavy vehicle near the walls and surcharge of load during compaction of the back fills has been given due considerations. The surcharge load due compacting machine and parking heavy vehicle near the retaining wall are of magnitude 48KN/m2 and 12KN/m2 respectively.

7.5.3 Loads due to Temperature Change and Shrinkage

Deformation loads are expected to be imposed onto the sumps and intakes due to seasonal or/and daily temperature change which may range from 10^0 to 25^0. For all slabs and walls, no need of considering to the temperature load so long as the movement joints are provided.

7.5.4 Seismic Inertia load

The inertia seismic load and the weight of the concrete walls have been calculated using the software SAP2000.For estimation of the seismic inertial loads; Modal response spectrum method has been used. The peak ground acceleration in three orthogonal directions has been used to define, design response spectrum for the each structure. The response spectrums in the corresponding directions were determined based on the recommendations of EUROCDE-8 for rock foundation and 5% damping ratio.

As per the code, for purpose of seismic inertia analysis, the magnitude of the vertical component will be 0.9 times of the vertical component. So, the magnitude of vertical ground acceleration was specified to have peak ground acceleration magnitude of 0.135g for peak ground acceleration of value 0.15g.

Using the peak ground accelerations, design response spectra or target response of the earth quake were determined .the design response spectra has been derived by multiplying the peak ground acceleration with standard shape of response spectra which are available in Eurocode-8. As per the code,

$$0 \le T \le T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5 - 1)\right] \quad \text{Eqn (1)}$$

$$T_B \le T \le T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \quad \text{Eqn (2)}$$

$$T_C \le T \le T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5\left[\frac{T_C}{T}\right] \quad \text{Eqn (3)}$$

$$T_D \le T \le 4s : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5\left[\frac{T_C T_D}{T^2}\right] \quad \text{Eqn (4)}$$

Where :

$S_e(T)$ is the elastic response spectrum

T is the vibration period of a linear single degree of freedom system,

a_g is the design ground acceleration on type A ground,

η is the damping correction factor with a reference value of η= 1 for 5% viscous damping,

T_B is the lower limit of the period of the constant spectral acceleration branch,

T_C is the upper limit of the period of the constant spectral acceleration branch,

T_D is the value definin g the beginning of the constant displacement response range of the spectrum,

S is the soil factor,

For ground rock foundation (ground type A) similar to that of sump and intake, the value of TB, TC, TD and S will be as given in the table here below.

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0

Figure 7-5 and 7-6 depict one of the horizontal response spectrum and the vertical response spectrum develped based on the reccommendatoin of Eurocode-8.

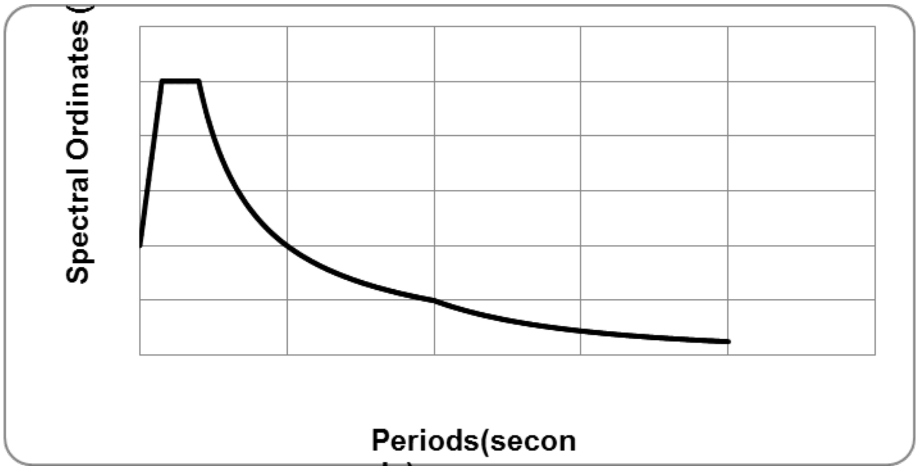

Figure 7-5: Graph of One the Horizontal Response Spectrum

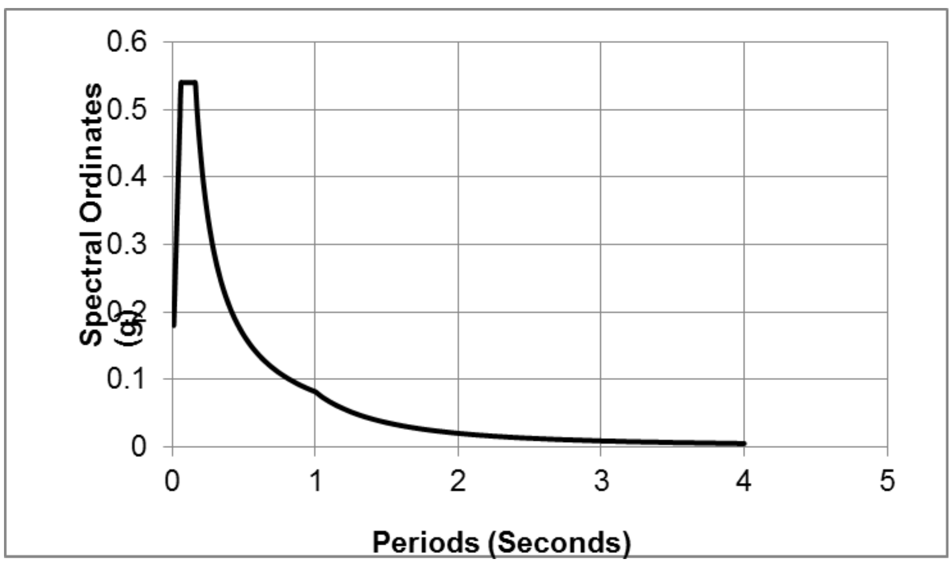

Figure 7-6: Graph of the Vertical Response Spectrum

In turn, the response spectra were used to carry out dynamic analysis using modal response spectrum. Modal analysis has been carried out in order to determine sufficient number of modes required for the modal response spectrum analysis. The number of modes has been fixed in such way that the participating modal mass ratio reaches in the three orthogonal directions at least 90%.

Sufficient number of natural modes has been taken as sufficient number of mode to capture significant modes in the three directions. SRSS is technique used for modal and directional combination.

7.6 Response of the Structures to External Loads

The loads determined in previous sections were applied to the 3-D finite element model of the structure. The responses of the structure were estimated with help SAP2000 analysis. Moments about horizontal and vertical axis respectively, for vertical walls were determined at the critical sections. For foundation slab they are moments about an axis parallel to short directions and about an axis parallel to long directions. In addition to the moments, shear forces at the critical section were determined.

7.7 Designing of Walls and Foundation Slabs

Vertical walls as well as foundation has been designed for flexure using 1000mm wide strip(b) and their respective thickness (h) and applying design relations set out for beam. In designing the walls for a factored negative or positive moment (M_u), the depth of the compression block is given by **a.**

$$a = d - \sqrt{d^2 - \frac{2|M_u|}{0.85 f'_c \phi b}}$$

Where d is effective depth, M_u is ultimate moment and Φ is strength reduction factor.

The following value has been adopted for designing:-

Φ=0.9 for flexure (tensioned controlled),

Φ=0.75 for shear.

The reinforcement area (A_s) required to resist tensile force caused by M_u has been determine as follows:-

$$A_s = \frac{M_u}{\phi f_y \left(d - \frac{a}{2} \right)}$$

Whether A_s is above the Maximum allowable limit has been checked in such way that its geometric reinforcement ratio (ρ) is less than the maximum geometric ratio (ρ_{max} =0.012).The maximum geometric ratio is taken to 75% of the balanced geometric ratio (ρ_b). In turn the balanced geometric ratio has been obtained using the following relation.

$$\rho_b = 0.85\beta_1 \frac{f'_c}{f_y} \frac{\epsilon_u}{\epsilon_u + \epsilon_y}$$

Where,β1 is factor=0.85,

f'_c is cylindrical compressive strength of age 28 days,
f_y is minimum yield strength of re-bar,
ε_u is maximum compressive strain of concrete=0.0035,and
ε_y is is strain of re-bars corresponding to at stress equal to the minimum yield strength.

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APPENDIX-A: MECHANICAL DETAILS OF THE MAIN PUMP STATION

General

This report presents the Design of transfer pumping station for Omo Valley Farm Irrigation Development Project located in Lower Omo River Basin, Hamer woreda. The area to be covered by this project is 5,600 ha gross irrigable land. Two pump stations are designed for irrigating the whole area: Main and Booster stations.

The main pump station will be supplied via an intake structure on the river which supplies a short canal directly connected to the respective intake well of the pump stations. At the pump station each pump shall discharge to the main canal through a separate delivery pipe lines the size of which shall be calculated later. The booster station covers the area above 422 m contour line up to 490m contour line having net area of 2,000 ha. It takes water from the main canal through intake canal to the suction sump where the pumps draw the required water.

With regard to the design discharge and static head of the pump stations, layout drawings of the project area and the following data were provided.

Pump Station	Area hectare	Discharge Req'd. m3/s	Total Lift (m)	Length of pipe (m)
Main PS	1726	9.3	56	500
Booster PS	54.3	3.9	68	3,500

Eligibility of Pump Set Suppliers

It is important that the pump set installation is simple, reliable, sustainable and easy to operate. The sustainability of an installation of this nature is dependent on good maintenance which will sometimes require support from the pump supplier or their agent. The pump set supplier will therefore be limited to one who can demonstrate the provision of an effective support service and extensive experience of working in Ethiopia. The Client’s choice of supplier must meet these requirements.

Good Suction Performance

In choosing the pumping station location and depth as well as selecting the pumps consideration was given to ensure that NPSH available always exceeds the NPSH required by the selected pump at all operational points, including operational points where the pumps are running out beyond their duty point but within the acceptable operational range of the pump. The pumping station depth was determined using the following NPSH calculation.

The NPSH (available) ≥ NPSH (required by the pump)

NPSH (available) = atmospheric pressure + the static head at the pump suction – the water vapour pressure – friction losses in the suction pipe work to the station - the safety factor.

In this relation, the followings were assumed:

Local atmospheric pressure = 9.743 metres.

The maximum water temperature of 45 degrees C giving a vapour pressureof 0.904metres.

A safety factor of 2 metre.

Selection of pump types

For the design volume and head of Omo valley basin pump stations, pump units with a radial impeller construction are the best choice. The pump units will be driven by suitably powered diesel engines as there is no EEPCO power in the area and on the request of the client.

With reference to pump and drive unit connection and configuration, two main alternatives were brought into attention of the Client.

a) Wet pit, Vertical shaft centrifugal pump units.
b) Horizontal split surface centrifugal pump units.

These alternatives require different designs of the pumping layout and inlet sump or wet-well, for best operation. A short channel connected to the respective pump stations will provide the supply for both alternatives and therefore, construction of an intake channel and gate chamber is needed.

The main difference among the alternatives relates to the dimension of the intake canal and chamber required and the layout of the pump house. Decision was given to the horizontal shaft centrifugal pump because the client has already purchased centrifugal pumps of 1500 m/hr discharge and 65m head.

Pumping Station Pipe Sizing

For the design of suction and discharge piping and the transmission pipeline from the pump station to their respective delivery, a flow velocity limit of 1.5, 3 and 2 meter/sec is adopted for, suction pipes, discharge pipes (common & independent) and transmission pipes. The following formula is used for calculating pipe sizes.

$$V = 1.274*Q / D^2$$

Where V = velocity of water in pipe in m/sec.

Q = discharge under the selected pumping arrangement in m^3/sec.

D = diameter of selected pipe in meters.

The calculations were made for the pumping arrangements of 9 +1 (nine duties and 1 stand-by). The results and the selected diameter of the pipes are shown in the table below. The pipes are sized based on the selection of commercially available pipes with the minimum diameter but satisfying the suction and discharge velocity limits.

Table 1. Pipe Design Parameters

Station discharge, (l/s)	10,000
No of duty pumps	9
No of stand-by pumps	1
Total No of pumps	10
Single pump discharge, l/s	1000
Suction manifold pipe diameter, mm	900
Suction manifold water velocity,	1.57
Discharge manifold pipe diameter, mm	800
Discharge manifold water velocity,	1.99

NB: - As it can be seen from the result all the requirements for the design of suction and delivery pipe sizes is met. Therefore, the following pipe diameters are selected to be the final sizes for all the booster stations.

Suction pipe diameter, mm = 900
Discharge pipe diameter, mm= 800

Pumping System Head Calculation

The head losses through the water piping between the intake wet well to the delivery inlet level is computed in the following table. The friction head loss in pipes is calculated from equations:

$$H_{Fr} = f \cdot \frac{L}{D} \cdot \frac{V^2}{2 \cdot g} \qquad f = 0.25 / \log_{10}^2 \left(\frac{k_s}{3.71 \cdot D} + \frac{5.74}{\mathrm{Re}^{0.9}} \right) \qquad \mathrm{Re} = \frac{V \cdot D}{\nu}$$

Substituting and re-arranging:

$$H_{Fr} = \frac{L}{g} \cdot \left[\frac{\frac{\sqrt{8} \cdot Q}{\pi}}{2 \cdot D^{2.5} \cdot \log_{10} \left\{ \frac{k_s}{3.71 \cdot D} + 4.62 \cdot \left(\frac{\upsilon \cdot D}{Q} \right)^{0.9} \right\}} \right]^2$$

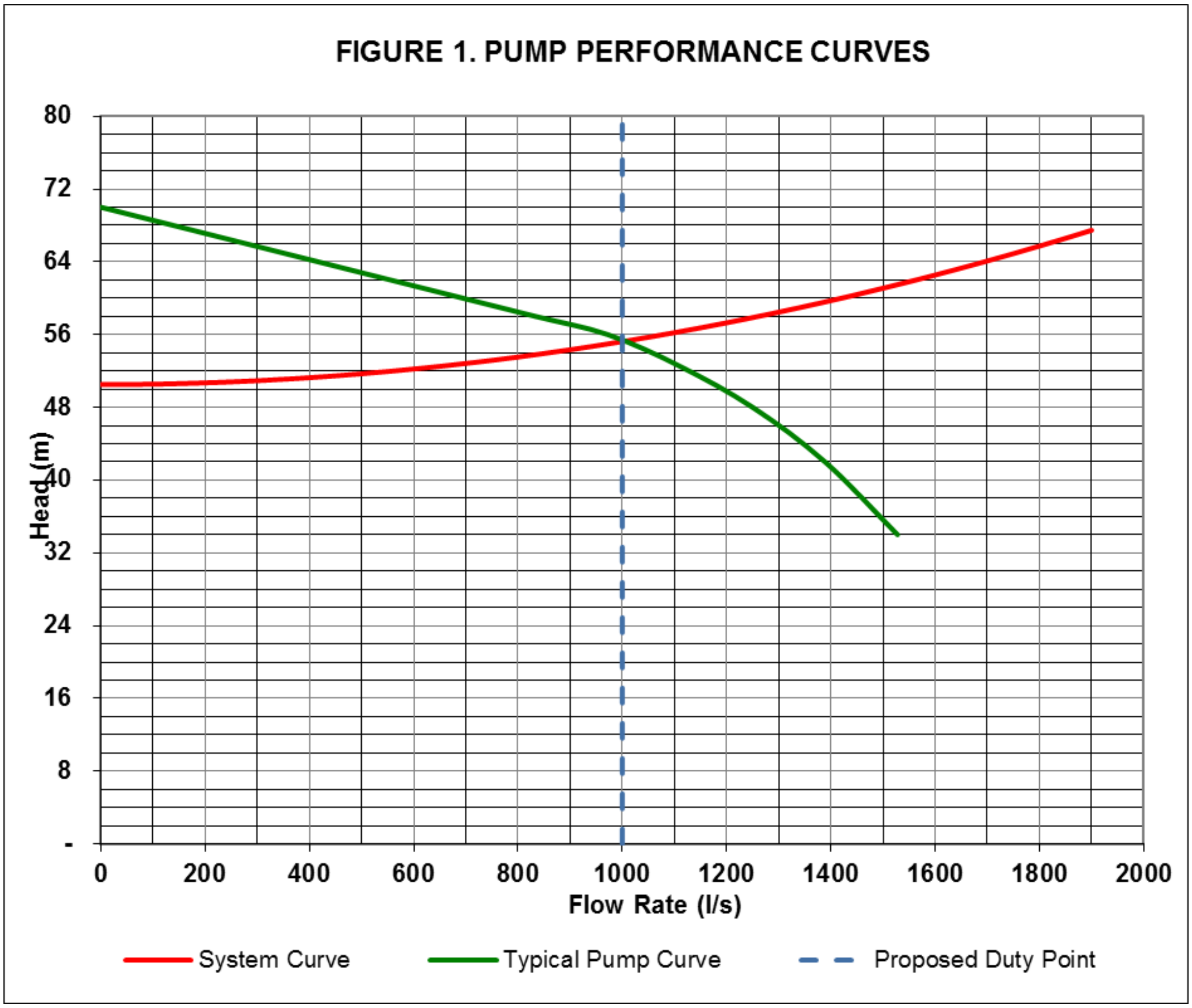
The calculations were made for the pumping arrangements of 9 +1 (nine duties and 1 stand-by). The results are shown in the table below.

Table 2. Pump System Head

Station discharge, m^3/hr	33,480
Station discharge, (l/s)	10,000
Intake min water level masl	301.56
Intake max water level masl	303.18
Delivery pipe inlet level masl	423.14
Average static head (m)	54.84
Maximum static head (m)	55.34
Minimum static head (m)	54.34
No of duty pumps	9
No of stand-by pumps	1
Discharge side, transmission pipe length (m)	500
Discharge side, transmission pipe dia. (mm)	800
Roughness coefficient (mm)	0.3

System heads are calculated under minimum and maximum head conditions for operating situations of various water level combinations at the suction and discharge side water levels. This will give us the minimum and maximum system head envelope which contains all possible system operation points.

The following graph shows the system curves for the maximum head conditions with a typical pump curve selected from Kirloskar Brothers Limited, a potential pump supplier of the client.



NB: - As it can be seen from the result horizontal split double suction surface centrifugal pump with capacity of Q=1000l/s and H=56m is recommended.

Net Positive Suction Head

The net positive suction head (NPSH) is the absolute pressure of the fluid at the pump center line or impeller eye as it enters the pump suction. Two values of NPSH are important in pump selection. These are NPSH required ($NPSH_{req}$) and NPSH available ($NPSH_{av}$).

The elevation difference required between the pump inlet level and the water level in the wet well depends on the NPSH requirement of the selected pump. Thus, NPSH requirement of the pumps is one of the main parameters affecting the pump selection.

The value of NPSHav mainly depends on the site elevation and layout of pump suction piping system, and is given by:

$$NPSH_{av} = H_{abs} + H_s - h_L + H_{vap}$$

Where:

$NPSH_{av}$: available net positive suction head, m
H_{abs}: absolute pressure on the surface of the liquid in the suction or reservoir (usually atmospheric pressure), m
H_s: suction head at the pump suction, m
h_L: total head loss due to friction, entrance, valves, and fittings in the suction piping, m

For any installation it is recommended that the NPSH available should exceed the NPSH required by one meter or more.

Table 3. NPSH Determination

Pump discharge at duty point, l/s	1000
NPSH required at selected pump duty point, m	3.54
Altitude of water pump station, masl	372
Absolute Pressure at pump station altitude, m	9.743
Vapor Pressure of Water at 45 ^{0}C, m	0.904
Friction loss in suction pipes and fittings H_L, m	1.56
Suction head at the pump suction H_s, m	0
$NPSH_{av}$	7.279
$NPSH_{av}$ - $NPSH_{rm}$	3.739 Ok!

The result shows that there is ample $NPSH_{av}$ at the site for suction pipe & fittings installation condition adopted.

Booster Pumping Station

The calculations were made for the pumping arrangements of 10+2 (ten duties and 2 standby) with separate delivery pipe line for each pump discharge. The results are shown in the table below.

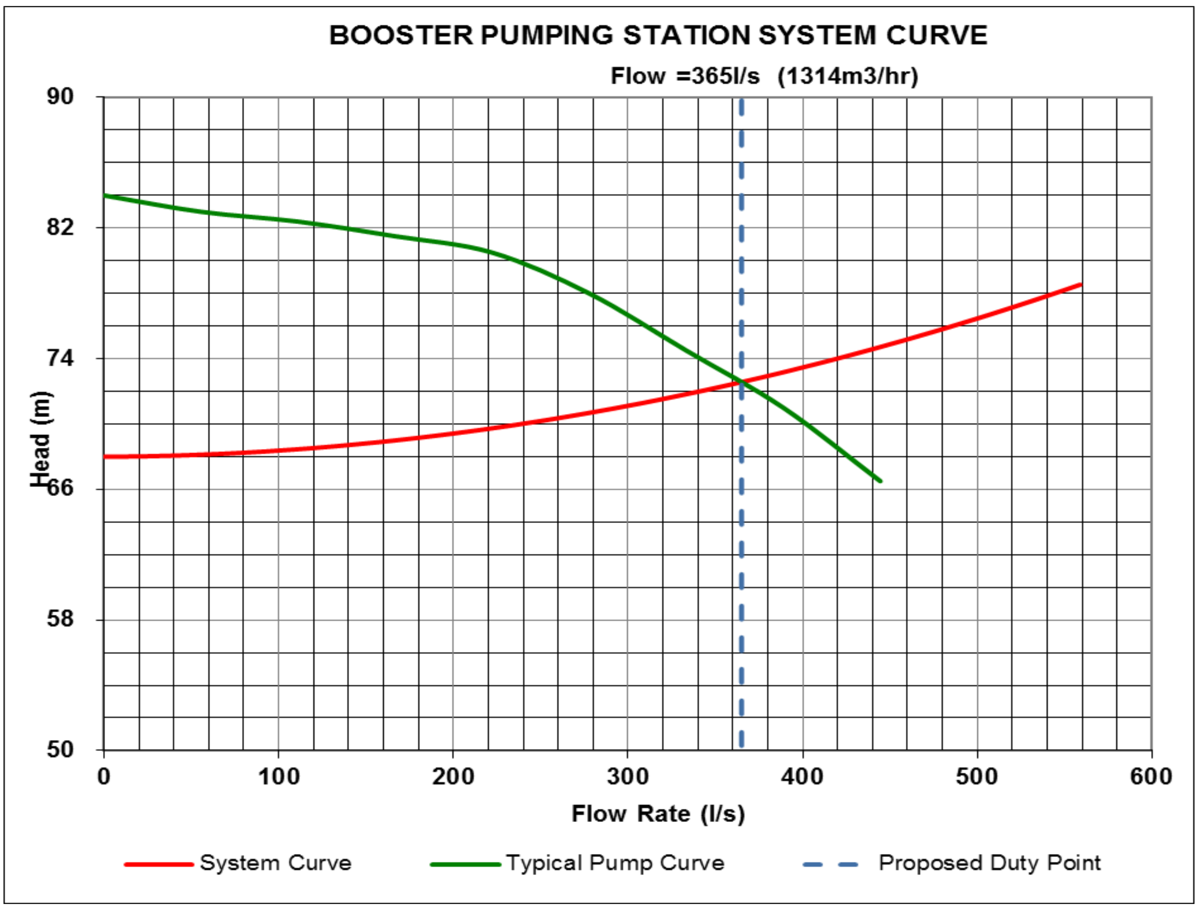
Table 4 Pipe Diameter Determination

Station discharge, m^3/hr	14,040
Station discharge, (l/s)	3900
Intake min water level masl	421.53
Intake max water level masl	421.53
Delivery pipe inlet level masl	489

Average static head (m)	73.93
No of duty pumps	10
No of standby pumps	2
Discharge side, transmission pipe length (m)	3500m
Discharge side, transmission pipe dia. (mm)	700
Roughness coefficient (mm)	0.3

System heads are calculated under minimum and maximum head conditions for operating situations of various water level combinations at the suction and discharge side water levels. This will give us the minimum and maximum system head envelope which contains all possible system operation points.

The following graph shows the system curves for the average head conditions with a typical pump curve provided by the client from a possible supplier.



NB: - As it can be seen from the result the existing pump SDS 300-500 which is already at hand with the client will be running approximately at Q=365l/s and H=72.5m at the existing site condition and shall have efficiency of more than 82%. But the additional pump sets should be rated with Q=417l/s and H=73m.

Net Positive Suction Head

The net positive suction head (NPSH) is the absolute pressure of the fluid at the pump center line or impeller eye as it enters the pump suction. Two values of NPSH are important in pump selection. These are NPSH required ($NPSH_{req}$) and NPSH available ($NPSH_{av}$).

The elevation difference required between the pump inlet level and the water level in the wet well depends on the NPSH requirement of the selected pump. Thus, NPSH requirement of the pumps is one of the main parameters affecting the pump selection.

The value of NPSHav mainly depends on the site elevation and layout of pump suction piping system, and is given by:

$$NPSH_{av} = H_{abs} + H_s - h_L + H_{vap}$$

Where:

$NPSH_{av}$: available net positive suction head, m
H_{abs}: absolute pressure on the surface of the liquid in the suction or reservoir (usually atmospheric pressure), m
H_s: suction head at the pump suction, m
h_L: total head loss due to friction, entrance, valves, and fittings in the suction piping, m

For any installation it is recommended that the NPSH available should exceed the NPSH required by one meter or more.

Table 5 NPSH Determination

Pump discharge at duty point, l/s	417
NPSH required at selected pump duty point, m	___
Altitude of water pump station, masl	420
Absolute Pressure at pump station altitude, m	9.686
Vapor Pressure of Water at 45 ^{0}C, m	0.904
Friction loss in suction pipes and fittings H_L, m	0.5
Suction head at the pump suction H_s, m	-1.5
$NPSH_{av}$	6.782
$NPSH_{av}$ - $NPSH_{rm}$	_____

The result shows that there is ample $NPSH_{av}$ at the site for suction pipe & fittings installation condition adopted. But the NPSH required by the pump is not provided on the technical information submitted with pumps, therefore it was not possible to confirm the suitability of the pumps for the given site condition mentioned above at this point.

APPENDIX-B: MECHANICAL DETAILS OF THE SLIDE GATES (GATE CHAMBER)

The provided gates are: 1.22x1.22m Service and emergency gates respectively.

Design data:

Gate opening clear height-h = 1.22m

Gate opening clear width-w = 1.22m

Maximum water level = 381masl

Sill level of gate = 370.52masl

Material selection:

Gate and stiffeners = structural steel

Seal = rubber

Wheel = hardened steel/wrought steel

Seal seat = stainless steel

Bearing = antifriction bearing

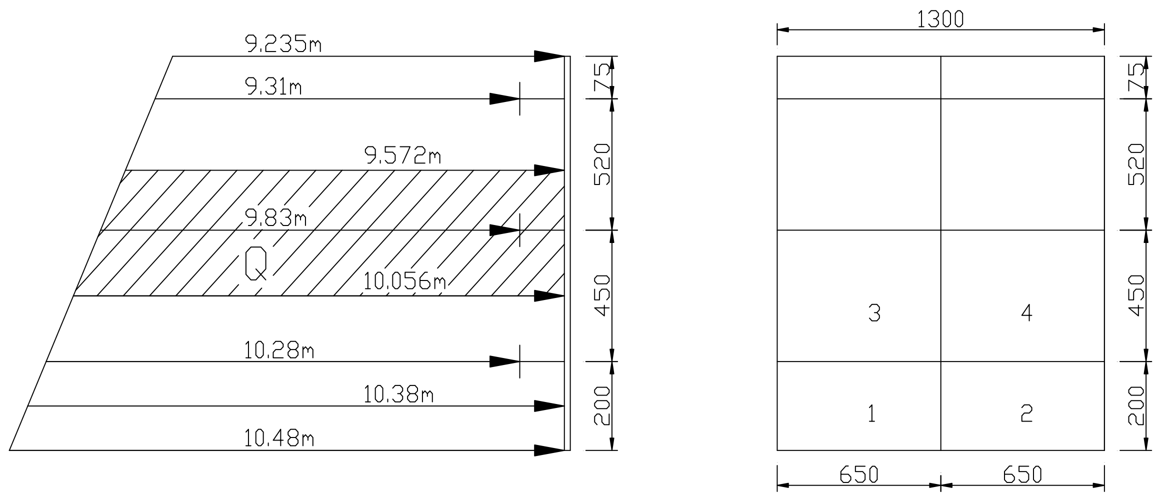
Selected and assumed design parameters:

Permissible shear stress for gate material	1120kgf/cm2
Permissible bending stress for gate material	1600kgf/cm2
Permissible combined stress inside skin plate	2040kgf/cm2
Permissible bearing stress for gate support frame material	2080kgf/cm2
Permissible bending stress for gate wheel track material	600N/mm2
Permissible contact stress for wheel material	3349.5N/mm2
Brinell hardness number (BHN) for wheel material	461
Permissible gate deflection (1/800)	1.625mm
Maximum shear stress for wheel material	1110N/mm2
Modulus of elasticity 'E' - steel	2.1 x 10^6kgf/cm2
Friction coefficient (antifriction bearings)	0.015
Friction coefficient rolling (steel on steel)	1
Friction coefficient sliding (steel on rubber)	0.65

Specific weight of water (γ) 1t/m^3

The total load on the gate is to be shared by three horizontal beams. Let position of the first beam be at a depth of h_1=10.28m, the second at h_2= 9.83m and the third at a depth of h_3= 9.31m.

One vertical stiffener is provided at the center of the gate. The pressure diagram is divided in to series of horizontal strips representing the selected number of horizontal beams.



The total load on the lower horizontal beams is:-

$$Q = \frac{1}{2}\gamma\left(10.056^2 - 9.572^2\right)$$

= 46.6KN/m

Skin plate thickness determination

Flat plate bending formula is used to determine the skin plate thickness

$$\sigma = \frac{k * p * a^2}{100 * t^2}$$

Where:-

σ = bending stress in flat plate

a, b = bay width (a = shorter side, b = longer side)

t = plate thickness

k = non dimensional factor depending on the values of a and b and boundary conditions

The following table shows computation of the skin plate thickness

Panel	Shorter side(a) (m)	Longer side(b) (m)	Aspect ratio(b/a)	Factor (k)	Pressure (N/m^2)	Allowable stress (N/m^2)	Plate thickness (m)
1	0.2	0.65	3.25	75	103800	1.57E+08	0.004454
2	0.2	0.65	3.25	75	103800	1.57E+08	0.004454
3	0.45	0.65	1.44444	47.3	100560	1.57E+08	0.007833
4	0.45	0.65	1.44444	47.3	100560	1.57E+08	0.007833

From the above table the skin plate thickness selected is 8mm.

Horizontal beams

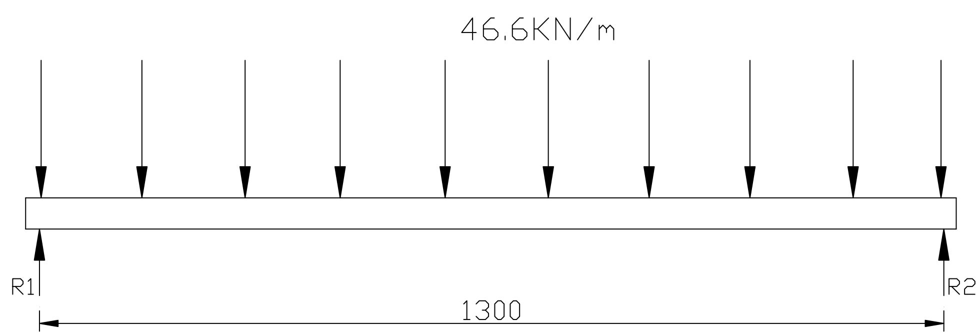
The horizontal beams are designed as simply supported beams carrying uniformly distributed load. The sizing calculation is carried out for the bottom most loaded beam.

Load supported by the beam is:-

$$w = \frac{1}{2}\gamma\left(10.056^2 - 9.572^2\right)$$

= 46.6KN/m

Load diagram



Taking the bearing length of the beam 1.3m for simply supported beams carrying uniformly distributed load, the maximum bending moment occurs at the center and it is given by:-

$$M_{\max} = \frac{wl^2}{8}$$

= 9.844KN.m

Reactions R1, R2:-

$$R1, R2 = \frac{wl}{2}$$

$= 30.29KN$

The allowable bending stress is given by:-

$$\sigma_{al} = \frac{M_{\max}}{z}$$

Where

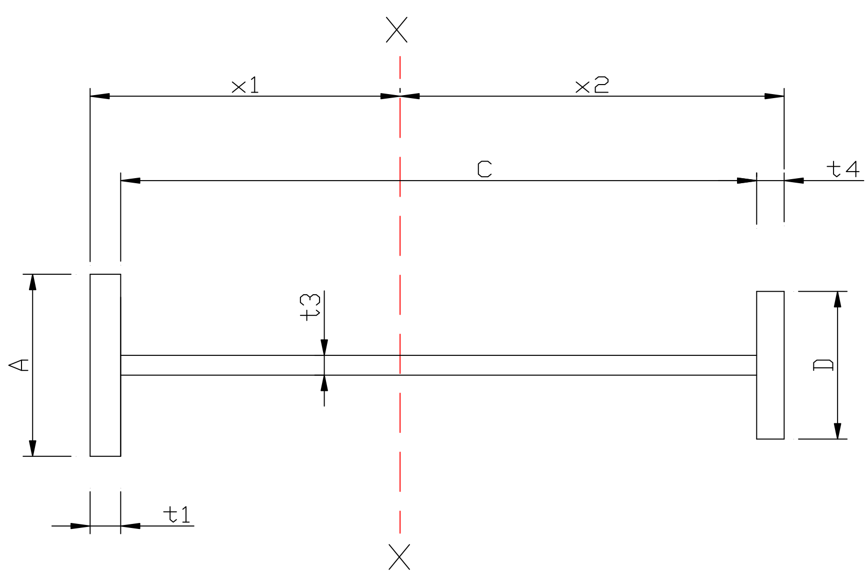
Z = the required section modulus of the beam

Allowable stress σ_{al} = 157Mpa

$$Z_{req} = \frac{M_{\max}}{\sigma_{al}}$$

$= 0.062702x10^6mm^3$

The selected beam section has the following dimensions:



Co-acting width of skin plate is given by, A=50*t_1

Where:-

t_1 = thickness of skin plate

= 50*8

= 400

Depth of web plate:-

C = 150mm

Section property of the beam:-

$$Z_{x1,x2} = \frac{I}{x_1, x_2}$$

Where,

I = moment of inertia of the beam.

x_1,x_2 = distance of neutral axis from both ends.

Deflection:-

For simply supported beams carrying uniformly distributed load the beam deflection is given by:-

$$\delta = \frac{5wl^4}{384EI}$$

Where

δ = deflection

w = distributed load on the beam

l = length of the beam

E = modulus of elasticity of the beam material

I = moment of inertia of the section

Stresses in the beam:-

$$\sigma_{x1,x2} = \frac{M_{\max}}{z_{x1,x2}}$$

Allowable deflection:-

$$\delta = \frac{L}{800}$$

Where,

L= length of the beam

Table below shows the result of analysis.

Co-acting Width of skin plate, A [mm]	400
Co-acting Thickness, t_1 [mm]	8
Web Plate Width , C [mm]	150
Web Plate Thickness , t_3 [mm]	8
Flange Plate Width, D [mm]	100
Flange Plate Thickness, t_4 [mm]	8
Beam depth	166
Summation of [AX]	242000.00
Summation of [A]	5200.00
Distance of Neutral Axis, X1 [mm]	46.54
, X2 [mm]	119.46
Moment of Inertia, I_{X1} [mm4]	5807532.94
Moment of Inertia, I_{X2} [mm4]	0.0
Moment of Inertia, I_{X3} [mm4]	3845332.54
Moment of Inertia, I_{X4} [mm4]	10669360.16
Moment of Inertia, I_X [mm4]	**20322225.64**
Section Modulus of the beam, Z_{X1} [mm3]	**436675.92**
, Zx_2 [mm3]	**170115.22**
Horizontal Stiffener Length, L [mm]	1300
Hydrostatic Load on Stiffener, [N/m]	46600

Modulus of elasticity, E [N/mm2]	210000
Beam Deflection, [mm]	**0.406**
Allowable Deflection, [mm]	**1.625**
Maximum bending moment [N.m]	9844.25
Allowable bending stress [N/m^2]	157000000
Section modulus required [mm^3]	**62702.2293**
Stress,σx_1 [N/m^2]	**22543606.1**
Stress,σx_2 [N/m^2]	**57868132.69**

Wheel design

Design data

Total load on the gate (P)	151KN
Number of wheels	4
Load on each wheel (p)	37.666KN
Wheel diameter taken (d_w)	150 mm
Radius of wheel crowning / wheel radius - R1/R2	5
BHN of wheel material	461
Elastic modulus (N/mm^2)	$210x10^3$
Factor of safety	3
Permissible contact stress for wheel material	3349.5 N/mm2
Max. Shear stress not to exceed (2.41 x BHN)	1111 N/mm2
Critical stress on the tread (0.169 x BHN –15.174)	62.74 N/mm2
Allowable stress (σ_{al})	20.9 N/mm2
Poisson’s ratio (μ)	0.25

Projected area required (A)

$$A = \frac{p}{\sigma_{\text{al}}}$$

$= 1802mm^2$

Thread width (b)

$$b = \frac{A}{d_w}$$

= 12mm

Take b = 50mm

Maximum shear stress

$$\frac{B}{A} = \frac{R_1}{R_2} = 5$$

$$B + A = \frac{1}{2}\left(\frac{1}{R_1} + \frac{1}{R_2}\right)$$

= 0.008

From Fig. (Curves for determination of stress in wheels) for $\frac{B}{A} = 5$

K = 0.34

$$\frac{\Delta p}{a^3} = 0.22$$

$$\frac{Z_1}{a} = 0.24$$

$$\frac{\Delta}{a}\left({}_1Z_z -{}_1Y_y\right) = 0.21$$

Evaluation of elastic property and shape property

$$\Delta = \frac{2\left(1 - \mu^2\right)}{E\left(A + B\right)}$$

$$\Delta = \frac{2\left(1 - 0.25^2\right)}{210 * 10^3\left(0.008\right)}$$

= 0.00111

$$\frac{\Delta p}{a^3} = 0.22$$

$$a = \sqrt[3]{\frac{\Delta * p}{0.42}}$$

$$= \sqrt[3]{\frac{0.00111 * 37666}{0.22}}$$

= 5.758

$$\frac{\Delta}{a}\left({}_1 Z_z - {}_1 Y_y\right) = 0.21$$

$$\left({}_1 Z_z - {}_1 Y_y\right) = \frac{0.21 * a}{\Delta}$$

$$\left({}_1 Z_z - {}_1 Y_y\right) = \frac{0.21 * 75.758}{0.0011}$$

= 1083.57N/mm^2 maximum differential of stress components

Maximum shear stress (τ_{max}) = $\frac{1}{2}\left({}_1 Z_z - {}_1 Y_y\right)$

= 0.5*1083.57N/mm^2

= 541.78N/mm^2 which is less than the allowable shear stress of 1111 N/mm^2

Contact stress

Z_1 = 0.24*a

= 0.24*5.758

= 1.38

Minimum depth of penetration = 2*Z_1

= 2.76mm

Semi major and semi minor axis of ellipse of contact are

a = 5.758mm

b = k*a

= 1.957mm

Contact stress = $\frac{3 * p}{2 * \pi * a * b}$

= 1595.78N/mm^2 < 3349.5 N/mm^2

Shaft diameter

Load on the shaft (p) = 37666N

Allowable shear stress of shaft material (τ) = 48x10^{6}N/m^2

$$\tau = \frac{p}{A}$$

$$d = \sqrt{\frac{4 * p}{\pi * \tau}}$$

= 0.0316m

Take d = 60mm

Dead weight of gate, G:-

G= 0.5ton

Opening and Closing force required (wheel Type)

Dead weight of gate (G) = 0.5ton

Total hydrostatic load on the gate (P) = 151KN

Radius of wheel (R) = 75mm

Radius of wheel shaft (r) = 30mm

Rubber width = 100mm

Thickness of gate at top (t_t) = 216mm

Thickness of gate at bottom (t_b) = 166mm

Unit water load on the side seal (N/mm) = 0.123KN/mm

Force of sealing (μ=0.65) = 2x0.65x100x0.123KN/mm

= 16.05KN

Unit water load on the top seal (N/mm) = 0.1195KN/mm

Force of sealing (μ=0.65) = 0.65x100x0.1195KN/mm

= 7.77KN

Total sealing force = 23.83KN

Rolling and sliding friction in wheel

(μ_s = 0.015, μ_r = 1)

$$f_w = \frac{P}{R}(\mu_s r + \mu_r) = 2.91KN$$

Total friction load (W) = 26.74KN

= 2.674ton

Weight of water above the gate (F_1) = 2.4ton

Buoyancy (up trust force) (F_2) = 2.1ton

Opening force required (F_o)

$F_o = 1.2W+1.1G+F_1-F_2$

= 4.055ton

= 40.55KN

Closing force (F_c)

$F_c = 1.2W+F_2-1.1G-F_1$

= 2.36ton

= 23.6KN

Opening and Closing force required (Sliding Type)

Dead weight of gate (G) = 0.5ton

Total hydrostatic load on the gate (P) = 151KN

Friction force (W) (μ=0.65) = 0.65x151KN

= 98.15KN

= 9.8ton

Buoyancy (up trust force) (F_2) = 2.1ton

Opening force required (F_o)

$F_o = 1.2W+1.1G-F_2$

= 10.12ton

= 101.2KN

Closing force (F_c)

$F_c = 1.2W+F_2-1.1G$

= 13.32ton

= 133.2KN

Actuator – power operated actuator with 45KN opening and 25KN closing capacity with operating speed of 0.5 – 1m/min for fixed wheel type gate and 105KN opening and 135KN closing capacity for sliding type gate.