ADDIS ABABA UNIVERSITY

Performance Assessment of Diversion Headwork Implemented for Irrigation

(Case Study on Fantale Irrigation Based Integrated Development Project)

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Declaration

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, a required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

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Abstract

Many diversion head-works for irrigation have been designed and constructed in the preceding years. Some of them are performing successfully, but it has been observed in various reports that some of the schemes have failed to serve the purpose for which they are designed. Therefore it is important to deal with the performance assessment of diversion head works implemented for irrigation.

A case study on Fantale diversion head-work carried out for this research work and attempt is made to understand the hydraulic and structural problems encountered. This serve for different purposes such as for construction of new diversion structure (either as replacement for existing structures, or as an entirely new structure), rehabilitation of existing structures (from minor repairs to complete re-engineering either to maintain existing function, or to meet new requirements) decommissioning of the structures.

Surveying data at the head-work cross section and other supporting data's were used to estimate the amount of sediment deposition. The analyses of the data's shows that there is a clear reduction of flow area or opening size at the canal head regulator.

The main factors that caused below performance of Fantale diversion head-work are sedimentation and formation of shoals upstream of both under-sluice and weir component, selection of diversion structure type, malfunctioning of gates, seepage under the weir and the under-sluice, reduced opening size at the canal head regulator, failure to flush sediment regularly, entry of silt in to the canal.

Key words: Analysis, Diversion, Downstream, Gates, Head work, Regulator, Scour, Seepage, Silt, Stability, Upstream, Under-sluice, Weir.

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List of Symbols

A = Area of opening

B = Total length of the floor

B =Width of the weir base.

C_d =Discharge coefficient, 0.64

D =depth of downstream cut off below downstream floor level

e = eccentricity

f = Silt factor for foundation material

g = Gravity, 9.8 m^3/s

G =Density of construction material for apron

GE = Safe exit gradient

H = Head of water i.e. difference of u/s and d/s water level

H = Maximum static head (pond level -downstream floor level)

∑H = summation of all horizontal load components

i_{cr} =Critical hydraulic gradient

$\sum M$ = Summation of all moments about the toe of the structure

Q = Discharge intensity per unit width (i.e. under sluice, weir

Q = Discharge

R' = Lacey's scour depth

t	= thickness of apron at a point
$\sum V$	= summation of all vertical loads
$\sum Vf$	= summation of vertical forces excluding the base reaction
X	= distance of the resultant force from toe
γ_{sat}	= Unit weight of saturated soil
γ_w	=Unit weight of water
α	= symbol alpha (ratio of total length of apron to depth of d/s pile
λ	= symbol lambda (a function of ratio of total length to depth of downstream pile)

List of Abbreviations

D/S	Downstream
FIBIDP	Fantale Irrigation Based Integrated Development Project
IIMI	International Irrigation Management Institute
NBCBN	Nile Basin Capacity Building Network
OWWDSE	Oromia Water Works Design and Supervision Enterprise
NTNU	Trondheim Norwegian University
U/S	Upstream

1. Introduction

1.1 Background

The majority of population of Ethiopia is dependent on rain fed agricultural production for its livelihood. However, estimated crop production is not close to fulfill the food requirements of the country (Lambisso,R, 2005).

Irrigation is one means by which agricultural production can be increased to meet the growing demands in Ethiopia (Awulachew et al. 2010). A study also indicated that one of the best alternatives to consider for reliable and sustainable food security development is expanding irrigation development on various scales, through river diversion, constructing micro dams, water harvesting structures, etc. (Lambisso,R, 2005)

The total potential irrigable land in Ethiopia is estimated to be around 3.7 million ha (Awulachew et al 2010). Current irrigation schemes cover about 640,000 ha across the country (IWMI, 2010). This means that a significant portion of irrigable land in Ethiopia is currently not irrigated. This means that there are potential opportunities to vastly increase the amount of irrigated land.

Different irrigation techniques such as diversion structures, storage, pumped etc. can be used. Diversion headwork structures are engineering facilities built across rivers or canals to store water and/or divert it from its original course. Among these, low-head diversion structures are extensively used in irrigation projects to divert water to a canal from either a canal or a natural river by raising the water level upstream (Fantale, Tibila).

Within these five years (2010-2015) Ethiopia plans to significantly increase its irrigated land from the current 640,000 ha to about 1.8 million ha, through small-scale irrigation, rainwater harvesting, and other medium- and large-scale irrigation projects (IWMI,2010). This 280 percent increase from current irrigation levels will require tremendous resources, including funding, human capacity, infrastructure, and other human and capital investments.

Thousand ha
Zone 1
Zone 2
Zone 3
Design capacity
241
315
84
640
Small scale
Non-/under-performing
78
116
10
204
Medium and large
Estimated underperformance2
13
Abandonment1
13
13
Actual capacity
34
310
66
410

1.2 Statement of the problem

Main causes for under performance would be on the head work or conveyance system. It would be on the head work in the sense that the designed hydraulic and structural parameters not reflect the real parameters and there would also be operational problems.

Functions of diversion headwork implemented for irrigation are to raise the water level on its upstream side, regulate the supply of water in to canals, control entry of silt in to canals, creates a small pond (not reservoir) on its upstream and provides some pondage, help in controlling the fluctuation of water level in river during different seasons (Garge S.K, 2005).

Fantale Irrigation Based Integrated Development Project is irrigation project located on Awash River at about 50km south west of Metehara town with a total command area of 27,000 hectares. It has problems in regulating the supply of water in to the canal, in controlling entry of silt in to the canal, and in controlling the fluctuation of water level in the river during different seasons. In addition problem of sedimentation in and around the intake works.

The above problems would cause change of irrigation schedule, reduction of main canal capacity and reduce the discharge capacity of the head work. It would lead to not only under performance, but also it would cause total failure of the head work structures.

This research work is aimed to identify the major causes of underperformance of the diversion headwork components of Fantale Irrigation through performance assessment. This will serve from minor repairs to complete replacement, either to maintain existing function, or to meet new requirements.

1.3 Research questions

The following questions are the main factors that are dealt in this research work.

- What are the reasons for the reduction water withdrawn from the head work?
- Why the head-work failed to control entry of silt and debris in to the main canal?
- What factors contribute for the formation of shoals and sediment deposition in front of the weir proper and under sluice respectively?

- ❖ Is there any structural problem such as failure due to uplift, overturning shear and sliding?
- ❖ Is there any relationship between the different components of the head work?

This study will go deeper into scientific analysis to answer the above mentioned questions by categorizing the problems as design, operational and construction problems through performance assessment.

1.4 Objective

1.4.1 General objective

- ❖ The primary objective of this research work is to identify the major causes of the underperformance of the diversion headwork components of Fantale Irrigation through performance assessment.

1.4.2 Specific Objectives

- ❖ To assess hydraulic performance of the Fantale diversion head work: the headwork must provide the desired hydraulic performance throughout the full range of flow conditions.
- ❖ To assess the structural integrity: the weir must be able to resist the onerous hydraulic and structural loading throughout its design life, without the need for excessive maintenance expenditure.

1.5 Significance of the research

In Ethiopia recently lots of Irrigation projects were under construction and lot of design works were done for the future expansion for food security of the increasing population. The country has experienced many cases of failure of diversion head works and below capacity for many decades but there is no any information for the causes of their failures (IWMI,2010).

The following are some of the uses of this research work

- ❖ To identify the major causes of the underperformance of the diversion headwork components of Fantale Irrigation through performance assessment.

- ❖ This research work aims to provide some information to parties engaged in maintenance and improvement of the existing Fantale Irrigation head work, so as to ensure that mistakes are avoided and opportunities are not missed.

- ❖ Performance assessment of diversion head works implemented for irrigation can also serve for different purposes such as for construction of new diversion structures, either as a replacement for an existing structure, or as an entirely new structure, rehabilitation of existing structures, from minor repairs to complete re-engineering, either to maintain existing function, or to meet new requirements, decommissioning of a structure.

1.6 Organization of the thesis

The thesis is organized in to five chapters. Chapter one deals with introduction that covers the general background, the problem statement, objectives of the research, the research question, Literature review, significance of the research, and organization of the study. Chapter two deals with the methods and materials used. Chapter three covers result and analysis. Chapter four discuss about each hydraulic and structural performance indicators. Chapter five deals with conclusion and recommendations.

2. Literature Review

2.1 Head works

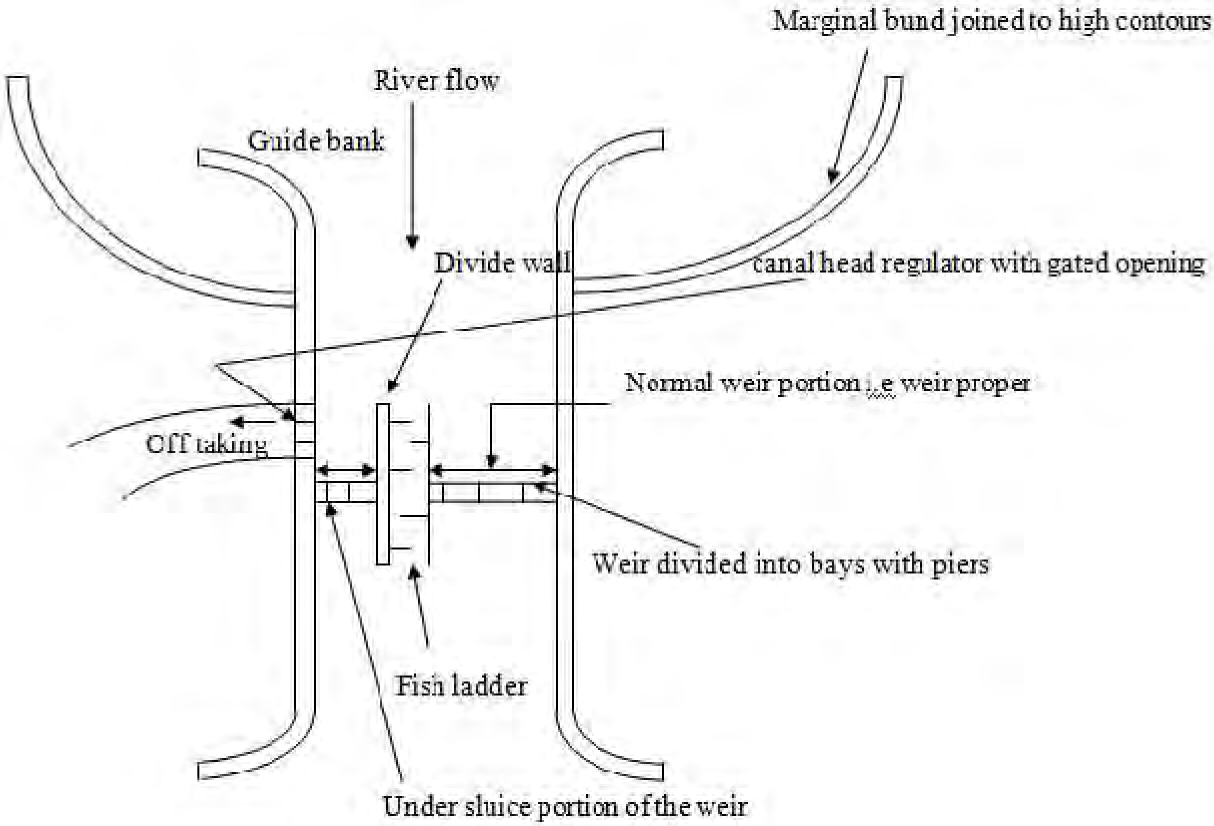
Head works are barriers across a river at the head of an off taking main canal. Head works can be either diversion head works or storage headwork (G.L Asawa, 2008). Diversion head works, are constructed at the head of the canal, in order to divert the river water towards the canal, so as to ensure a regulated continuous supply of silt free water with a certain minimum head in to the canal.

Storage head work is a barrier constructed across the river valley to form the storage reservoir. The water is supplied to the canal from the reservoir through the canal head regulator. This serve as multipurpose functions like hydroelectric power generation, fishery, flood control, etc (Garge S.K, 2005).

Diversion headwork provides an obstruction across a river, so that the level of the water is raised and water is diverted to the channel at required level. The flow of water in the canal is controlled by the canal head regulator. This increased water level helps the flow of water by gravity and the increasing the commanded area and reducing the water fluctuation in the river (Garge S.K, 2005).

As per Bibhabasu M, (2012) headwork serves the following purposes:

- To rise the water level at the head of the canal
- To control the intake of water into the canal.
- To control the entry of silt in to the canal and to control deposit of silt at the head of the canal.
- To store water for small period of time so that water is available throughout the year.
- To control the fluctuation of water level in the river during different season.


Marginal bund joined to high contours
River flow
Guide bank
Divide wall
canal head regulator with gated opening
Normal weir portion i.e weir proper
Off taking
Weir divided into bays with piers
Fish ladder
Under sluice portion of the weir

Under sluices /scouring sluices are openings provided at the base of the weir or barrage. These openings are provided with adjustable gates. Normally, the gates are kept closed. The suspended silt goes on depositing in front of the canal head regulator. When the silt deposition becomes appreciable the gates are opened and the deposited silt is loosened with an agitator mounting on a boat. The muddy water flows towards the downstream through the scouring sluices so the gates closed. But, at the period of flood, the gates are kept opened (Bibhabasu Mohanty, 2012).

The divide wall is a long wall constructed at right angles in the weir or barrage; it may be constructed with stone masonry or cement concrete. On the upstream side, the wall is extended just to cover the canal head regulator and on the downstream side it is extended up to the launching apron (Bibhabasu Mohanty, 2012).

To form a still water pocket in front of the canal head so that the suspended silt can be settled down which then later be cleaned through the scouring sluices from time to time. It controls the eddy current or cross current in front of the canal head. It provides a straight approach in front of the canal head. It resists the overturning effect on the weir or barrage caused by the pressure of the impounding water (Bibhabasu Mohanty, 2012).

Fish ladder is provided just by the side of the divide wall for the free movement of fishes. Rivers are important sources of fishes. The tendency of fish is to move from upstream to downstream in winters and from downstream to upstream in monsoons. This movement is essential for their survival. Due to construction of weir or barrage, this movement gets obstructed, and is determined to the fishes. In the fish ladder, the fable walls are constructed in a zigzag manner so that the velocity of flow within the ladder does not exceed 3 m/s. The width, length, and height of the fish ladder depend on the nature of the river and type of weir or barrage (Bibhabasu Mohanty, 2012).

A structure which is constructed at the head of the canal to regulate flow of water is known as canal head regulator. It consists of a number of piers which divide the total width of the canal in to a number of spans which are known as bays. The pier consists of tiers on which the adjustable gates are placed. The gates are operated from the top by suitable mechanical device. A platform is provided on the top of the pier for the facility of operating the gates. Again some piers are constructed on the downstream side of the canal head to support the roadway (Bibhabasu Mohanty, 2012).

Functions of canal head regulator are; it regulates the supply of water entering the canal, it controls entry of silt in the canal, it prevents the river flood from entering the canal (Bibhabasu Mohanty, 2012).

Entry of silt in to canal which takes off from head-works can be reduced by constructed certain special works called silt control works. These works may be classified in to the two types silt excluders and silt ejectors.

Silt excluders are those works which are constructed on the bed of the river upstream of the head regulator. The clearer water enters the head regulator and silted water enters the silt excluder. In this type of works the silt is therefore removed from the water before it enters the canal (Bibhabasu Mohanty, 2012).

Silt ejectors also called silt extractors are those devices which extract the silt from the canal water after the silted water has travelled a certain distance in the off taking canal. These works are therefore constructed on the bed of the canal and little distance downstream from the head regulator (Bibhabasu Mohanty, 2012).

River training works are required near the weir site in order to ensure a smooth and an axial flow of water and thus to prevent the river from outflanking the works due to a change in its course. The river training works required on a canal head-work are guide banks, marginal bunds, spurs or groynes (Bibhabasu Mohanty, 2012).

2.2 Concept of Performance

Performance is the degree to which a system achieves its objectives. But objectives differ for individual systems and may be reset from time to time by a management decision. Abernethy (1989) has given the following definition of performance: “The performance of a system is represented by its measured levels of achievement in terms of one, or several, parameters which are chosen as indicators of system’s goals.”

(Murray Rurst, D.Hammond and snellen, 1992) have commented that the above definition (by Abernethy 1989) is output oriented only. According to them, the definition totally disregards the resource utilized, and the environmental impacts in achieving the level of outputs.

Perhaps the definition given by Small and Svendsen (1992) does give due consideration to the points raised by Murray Rust and Snellen (1991). This improved the concept of performance is given as follows: "performance of a system as encompassing the totality of both its activities-inputs and the transformation of the inputs into intermediate and final outputs and the effect of these activities on system itself and on its external environment."

From a different angle, the definition of performance as given by Abernethy (1989) is simpler and more practical. The points raised and additions proposed could be considered as essential tools for the assessment of performance that is to determine whether the performance results are acceptable or not.

In this study, performance assessment is restricted to a component of an irrigation system generally called the diversion head-work. The focus is on identifying the major causes of failure of the Fantale Irrigation head-work.

2.3 Performance Standards

In order to address concerns related to headwork design in a systematic way performance standards developed by Lysne.D.K et al. (2003) have been discussed below.

A) Withdrawal of water

Headwork of a diversion structure needs to be capable of abstracting the amount of water required for power generation/irrigation and bypassing the surplus. The diversion structures have to be designed such that it is able to extract design discharge from the river even during dry season. Diversion weir along with the intake diverts and controls the abstraction of water into the conveyance system.

A submergence of the intake is required so that the water level in the river is high enough for necessary abstraction of flow even during dry seasons and for the prevention of air entrainment in the conveyance system.

B) Passage of floods, including hazard floods

The headwork structure needs to be designed to facilitate a safe passage of the design flood without causing serious damages to the headwork. Flash floods due to natural hazards such as overtopping should be handled with some structural damages.

C) Passage of ice, trash and floating debris

Accumulation of the debris in front of the intake causes significant changes in the flow pattern near the intake. Increase in turbulence level and head loss across the intake are some of the resulting consequences. Thus, the design needs to allow the passage of all ice, trash and floating debris with the use of debris gates and trash racks.

Trash racks in front of the intake prevent the passage of undesired materials through the intake. The velocity across the inlet should be maintained in order to be able to clean the trash rack manually. A hydraulic loss over the trash rack also needs to be considered, which is a function of the water velocity and the geometry of the trash rack.

D) Passage of sediments

The design of the headwork must prevent the bed-load from approaching the intake and causing clogging of the intake like in figure above. The design needs to facilitate the passage of bed-load through sluiceways without causing significant structural damages to the headwork components.

The run-of-river schemes in sediment-loaded rivers need to be designed such that most of the sediment is transported along the river flow that is remaining after abstraction of water into the waterways. The transportation of sediment with the river flow can be obtained by two ways separation of the sediments before the intake and flushing of sediments from the intake structure.

The inlet of the intake needs to be placed above the intake bed such that the bed load and sediments in the lower layer of the flow are separated from upstream the intake at all flow conditions.

E) Bed control at the intake

In order to avoid the river bed from building up at the intake and causing clogging and uneven flow distribution, the intake either needs to be located close to the spillway gates or should be equipped with under-sluices.

F) Exclusion of suspended sediments and air

Suspended sediments need to be removed from the diverted water with the use of settling basins to avoid sediment problems in the waterways and the hydraulic machineries. In order to avoid air entrainment problems in the conveyance system air vents need to be designed.

G) Flushing of settled sediments

Efficient flushing of the sediments from the under sluice pocket needs to be ensured such that its capacity remains unaltered. The removal of sediment from the basins is usually done by flushing with the u se of flushing gates. A dead storage is, however, provided where sediments are accumulated between two consecutive flushing, which further depends on the sediment load and the flushing method.

Some methods may require closure during flushing, whereas others allow a continuous operation during flushing. The flushing systems can be classified according to Table 1 (Lysne et al., 2003).

Table 1 Classification of flushing system.

Flushing Arrangement			
Close down during flushing		In operation during flushing	
Conventional gravity flow flushing	Excavators and manual unloading	continuous flushing	Intermittent flushing

2.4 Previous studies on performance of Irrigation Structures.

A number of irrigation schemes have been designed and constructed in Ethiopia in the previous years. However, while some schemes are performing successfully, it has been observed in various reports that some of the schemes have failed to serve the purpose for which they are intended. In line with this, recent study report for the Amhara region (Asfaw, 2004) has been used as a bench mark to conduct related study in the southern region.

To this end, a research by (Lmbisso,R, 2005) which aims to evaluate the design practices and performances of small scale irrigation structures in Southern Nations Nationalities and Peoples Regional Government (SNNPR). In addition to the hardware problems, institutional, planning, social and economic problems contributing to the failure are also highlighted.

Robel's primary objective was to investigate the cause of failure of existing small scale irrigation schemes of the region to learn a lesson and generate knowledge on practices and performances for future practical application and compile set of recommendations for planning, design, implementation and operation of irrigation schemes.

In the research, desk study was made on existing small scale irrigation systems and practical field visits were conducted to 15 sites found in the region. In the desk study, the available relevant data on 11existing irrigation structures and post implementation review reports on the status of existing schemes in the south region is collected from the regional irrigation authority.

From the study and design reports, the current design practice for irrigation structures is examined and from the practical visits undertaken on 15 sites, the extent and frequency of hardware and software problems on the schemes is assessed (Lambisso R, 2005).

In the analysis, the frequency of each problem among the sites, type of problem in each site and problem ranking has been done. Better insights on hardware and software aspects of the problems have been acquired via interviews with community and technical personnel using relevant structured questionnaire. Out of the 26 sites considered for the analysis, 13 are observed to have main canals highly charged with sediment. The silt load is observed to come either along with the river water (suspended and bed load) or as a runoff from upstream nearby catchment (Lambisso R, 2005).

Headwork sedimentation was observed at 11 of the 26 sites. Accordingly, some 42% of the sites are affected with this problem. Headwork sedimentation refers to the overall submergence of the weir proper, wing walls and appurtenant structures such as gates due to settlement of sediment and the bed level rise of the river channel. The problem of main canal seepage is observed on 9 of the 26 schemes considered for the purpose of the analysis. This means that the problem is prevalent on 35% of the schemes (Lambisso R, 2005).

The foundation seepage is also observed in the region as being one of notable problems. The analysis of water seepage under the foundation is very important and must be given due attention. Failure to apply the proper sub- surface flow theories and practices may result in complete failure of the structure (Garg, 1989). During the survey, 1 of the 26 sites (4%) is seen to fail due to this problem (Lambisso R, 2005).

Damage on downstream bed is observed on 5 of the 26 (19%) of the surveyed sites and it is attributed to improper hydraulic design that arises from poor knowledge of the energy dissipation and impact of sediment on the structure (Novak et. al, 2001). Drying out of river flow was observed at the 3 of the 26 sites considered for the analysis. In other words, about 12% of the schemes were seen having no water in their river channels (Lambisso R, 2005).

According to (Lambisso R, 2005).considerable number of schemes (18%) have already totally failed because of the various problems such as main canal siltation, sedimentation of the headwork, problem of seepage through foundation, main canal seepage, scouring of downstream bank, drying of rivers, damage on impervious and flexible apron, change of river course, damage of under sluices and damage on cross drainage works & a significant number are performing below their capacity.

Accordingly the researcher suggested, the government and various other agencies involved in the subject need to revise the approach towards irrigation development by: integrating local research with modern irrigation development by creating mutual relationship between research and irrigation development, promoting irrigation development activities based on local knowledge and community participation, Building the capacity of the technical personnel involved in the subject so that better skills can be gained in planning, design and implementation of projects, integrating software and hardware aspects of irrigation schemes rather than focusing largely on design and construction alone as hardware components. Failure to consider these elements may result in subsequent failure of other schemes (Lambisso R, 2005).

The other research used as a bench mark for this research is a research work by Nile Basin Capacity Building Network entitled design and operation of diversion. The objective of the research was to assess the existing river diversion systems in some countries of the Nile Basin and to identify categorically problems related to them.

The research work conducted by NBCBN also assesses the existing river diversion systems in some countries of Nile Basin and tries to identify categorically problems related to them. The objective of the research was to systematically compile and build a database of existing both traditional and modern diversion system, to review critically the existing design criteria and to identify the limitation in these criteria, to suggest remedial measures in a form of design and operation guidelines or procedure to improve the performance of existing conventional diversion systems and suggest to the use of alternative design.

The research work was aimed at assessing the existing river diversion systems in the Nile Basin countries and at identifying the, hydraulic, hydrologic, and structural problems related to them. The specific objective of their research was to systematically compile and build a data base of existing both traditional and modern diversion systems to review critically the existing design criteria and identify limitation in these criteria, to suggest remedial measures in a form of design and operation guideline or procedures to improve the performance of existing conventional diversion systems and suggest the use of alternative design.

The following table indicates the category of existing problems in diversion systems and frequency of the problem on 84 small scale irrigation projects inventoried in north, central and southern Ethiopia.

Table 2 Problem category and weight (NBCB Research Cluster, 2005)

No	Problem category	Weight
1	Site selection	31%
	Clogging of under sluice	1
	Sedimentation problem on headwork	1
	Change in river course	1
	Damage on main canal and farmlands	1
	Main canal siltation	1
2	Structure selection	25%
	Sedimentation problem on headwork	1
	Damage on weir proper	1
	Clogging of under sluice and outlet	1
	Prevalence of downstream scouring	1
3	Hydrology and sediment consideration	13%
	Clogging of under sluice and outlet	1
	Main canal siltation	1
4	Hydraulic design of weirs and components	69%
	Prevalence of d/s scouring	1
	Damage on main canal and farm land	1
	Damage on d/s apron	1
	U/s flooding	1
	Damage on retaining wall	1
	Seepage problem under the weir	1
	Change in river course	1
	Clogging of under sluice and outlet	1
	Damage on d/s cutoff	1
	Damage on sill (if any)	1
	Damage on divide wall	1
5	Structural design of weir and components	25%

	Damage on intake gate	1
	Damage on scouring sluice gate	1
	Damage on weir proper	1
	Damage on divide wall	1
6	Scheme operation	13%
	Damage on intake gate	1
	Damage on scouring sluice gate	1

Taunsa Barrage located on Indus River at about 39 km south of Taunsa Sharif town and 16 km north west of Kot Adu was constructed from 1954-1958 to provide weir controlled irrigation supplies to originally flood fed areas on both banks of the river alongwith some new lands in Thal desert area. This Barrage serves 2.351 million acres (951,400 hectares) besides diverting flows from Indus River to the Chenab River through Taunsa Panjnad (TP) Link Canal. The barrage also serves as an arterial road bridge, a railway bridge, and crossing for gas and oil pipelines, telephone line and EHV transmission lines (S.M.A. Zaidi et.al, 2010).

The barrage soon after its completion in 1958 ran into multiple problems like oblique right sided river approach to the barrage causing heavy siltation in DG Khan canal with reduction in its capacity, excessive retrogression of water levels on the downstream, damage to stilling basin floor, breakdown of subsurface flow monitoring system and defects developing in mechanical installations. Extensive repairs were carried out during 1959-62 and periodically thereafter and in 2003 the latest, but problems persisted. Punjab Government constituted committees of experts in 1966 and 1973 but no specific measures, were taken to address the problems that continued to aggravate (S.M.A. Zaidi et.al, 2010).

Thus a paper by S.M.A. Zaidi et.al presented the introduction to the history of Taunsa Barrage, the problems, their impacts, proposed remedial measures, design and construction of works and finally evaluation of performance of the project, after 4 years of completion which is a total success story of the project implementation.

In the paper high sediment intake into the canal was considered to be due to: Excessive sediment concentration in the right pocket and DG Khan canal intake being located on the inner side of the river approach curve, Quite a few potent hill torrents discharging into the river from the right side a short distance on the upstream of the barrage bring very heavy course sediment charge in the right half of the river channel, Due to use of semi still pond regulation, the right pocket used to get silted up frequently and DG Khan canal received the supply direct from the river channel with heavy concentration of sediment.

On account of heavy retrogression of tail water levels (TWLs), the safe discharging capacity of the barrage had been reduced to almost half the design capacity and had also indirectly affected the safe limit of head across the barrage (S.M.A. Zaidi et.al,2010).

As a remedy, a subsidiary weir with a crest RL 424.0 was constructed 925 ft downstream of the barrage gate line. The sub-weir was expected to raise the TWLs of the barrage to the elevation required (as a minimum) for formation of hydraulic jump (S.M.A. Zaidi et.al, 2010).

2.5 Theoretical framework

2.5.1 General

No scheme is assumed to operate as designed hence occurrence of problems in hydraulic structures is not uncommon (Robel Tilaye, 2009). However, problems which hinder the ease of operation and utilization of resources substantially with far reaching consequences are given due attention in this research mainly in this sub topic the performance indicators of river diversion headwork. The standards developed by Lysne et al. (2003) performance indicators are classified as hydraulic and structural performance indicators for this research work.

2.5.2 Hydraulic performance indicators

Hydraulic performance-the diversion structure must provide the desired hydraulic performance throughout the full range of flow conditions, from low summer flow to flood. The hydraulic performance indicators selected for this research work are passage of sediment, prevalence of scouring, seepage problem under the weir, clogging of under sluice and outlet, passage of flood and withdrawal of water. These indicators are selected from a previous study reports which are highlighted in the literature review section.

A) Passage of sediments

According to Lauterjung, (1984), sedimentation is the tendency for particles in suspension to settle out of the fluid in which they are entrained and come to rest against a barrier. This is due to their motion through the fluid in response to the forces acting on them: these forces can be due to gravity, centrifugal acceleration or electromagnetism.

Under sluice is the one of the technical measure to scour silt deposited in front of canal regulator and control silt entry in the canal. Under sluices are provided with adjustable gates. Normally, the gates are kept closed. The suspended silt goes on depositing in front of the canal head regulator and flushed out. The depth of sediment stored upstream of the headwork is measured using graduated staff rod and tape.

According to (Bibhabasu Mohanty, 2012) entry of silt in to canal which takes off from head-works can be reduced by constructed certain special works called silt control works. These works may be classified in to the two types silt excluders and silt ejectors.

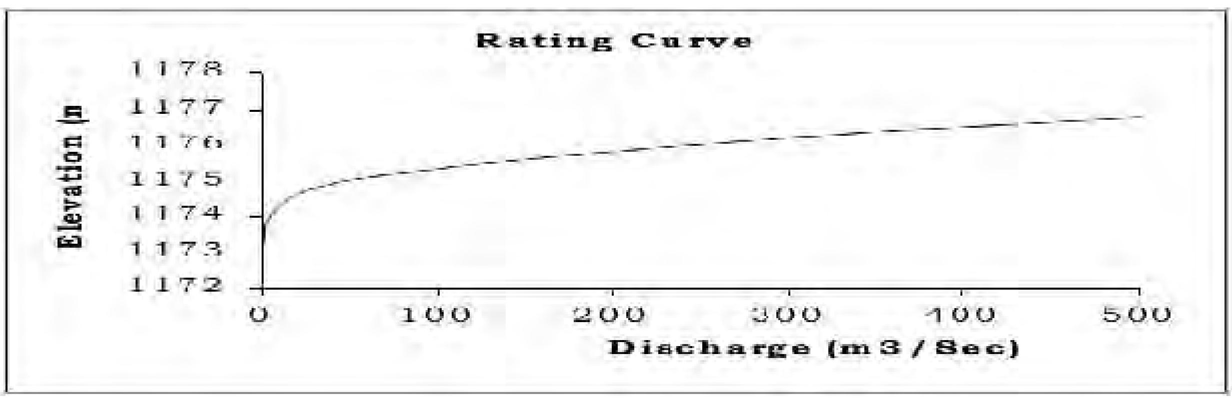
Silt excluders are those works which are constructed on the bed of the river upstream of the head regulator. The clearer water enters the head regulator and silted water enters the silt excluder. In this type of works the silt is therefore removed from the water before it enters the canal.

Silt ejectors also called silt extractors are those devices which extract the silt from the canal water after the silted water has travelled a certain distance in the off taking canal. These works are therefore constructed on the bed of the canal and little distance downstream from the head regulator (Bibhabasu Mohanty, 2012).

B) Passage of floods, including hazard floods and Withdrawal of water

A flood may be defined as an overflow coming from some river or from some other body of water. When the water overflows the banks of the river, the river is said to be flooded (Garg, 2005).

Whenever an important structure is to be constructed on a river or in the vicinity of a river it must be properly planned and designed keeping in view the damage to which it is susceptible and



$Q \quad C_d \quad A \qquad gh$

h- Head of water i.e. difference of

u/s and d/s water level

g- Gravity, $9.8m^3/s$

C) Seepage Analysis

Seepage of water is one of the major problems which have an effect upon hydraulic structures. Therefore, the seepage under the hydraulic structures can be considered one of the most important factors in the hydraulic structures safety (G.L.Asawa, 2008).

The seepage usually occurs in the impervious soils because of the differential pressures due to differences in water level between upstream and downstream. Seepage flowing below the foundation of hydraulic structures founded on permeable soils exerts pressure on the structures and tends to wash away soil under it. Excessive uplift pressure and piping are often the main cause of damage of the stability of the structure and may cause its failure (G.L.Asawa, 2008).

To identify weather there is a problem of seepage which causes uplift pressure and piping failure there was no any access as the whole structure is covered with water. But in this research to implemented khoslas method of independent variable to now the existing situation of Fantale diversion headwork and has also checked the exit gradient using the formula below:

$$GE = \left(\frac{H}{d}\right) \times \frac{1}{\pi\sqrt{\lambda}} \qquad (2.2)$$

Where, GE = Safe exit gradient

H = Maximum static head (pond level -downstream floor level)

d = depth of downstream cut off below downstream floor level

λ = another variable defined as = $(1+ \sqrt{1+\alpha^2})/2$

$\alpha = b/d$, b = Total length of the floor

In addition Geo-Slope, Seep/w software was also used to analyze the seepage under the diversion structure.

D) Prevalence of scouring

Scour is a natural phenomenon caused due to the erosive action of flowing stream on alluvial beds which removes the sediment around or near structures located in flowing water. These endanger stability of the structure by shearing.

Scouring occurs during floods and when the water flow with very high velocity over the structure. These have been checked by integrating both physical observation and flow velocity measured on Fantale diversion headwork.

E) Clogging of under sluice

The under sluices are openings provided at the base of the weir. These openings are provided with adjustable gates. Normally, the gates are closed. The crest of the under sluice portion of the weir is kept at a lower level (1-1.5) than the crest of the normal portion of the weir. The suspended silt goes on depositing in front of the canal head regulator (G.L.Asawa, 2008).

When the silt deposition becomes appreciable the gates are opened. The muddy water flows towards the downstream through the scouring sluices. The gates are then closed. But, at the period of flood, the gates are kept opened.

This Research has identified this by physical observation of the under sluice gates operation, checking debris and sediment upstream of the under sluice. The problems may also be design, operational and construction problems.

Table 3 Hydraulic performance indicators & their related head-work components.

Hydraulic performance indicators	Related components of a Head-work
Passage of sediment	Under sluice, barrage, gates ,trash rack
Withdrawal of water / discharge capacity	Canal head regulator, under sluice, weir
Seepage	Cutoff depth, length and thickness along the upstream and downstream impervious floor.
Scouring	Depth of upstream and downstream sheet pile, suitable length and thickness of launching apron at the upstream and downstream side.
clogging	Gates , trash rack, sill level

2.5.3 Structural performance indicators

After designing the components of the structure based on the surface and subsurface flow considerations, the structure should have to be checked for stability. The forces and moments acting on the corresponding structure are then calculated and the structure is checked for its stability against overturning and sliding. If the structure is unsafe for the conditions at the time of assessment the reasons shall be investigated (Baban, 1995).

The forces that act on a diversion structure, especially weir section has given due attention. The structural performance indicators selected for this research work are stability against uplift, overturning and sliding.

A) Stability against Uplift

Uplift force exists on the structure because of the subsurface flow of water underneath it. This uplifting pressure head decreases from upstream to downstream (Baban, 1995). To insure the stability against uplift the necessary apron thickness at different points along a longitudinal section are provided.

First the maximum unbalanced head between the uplifting pressure head and depth of surface water above the apron is calculated at points along the structure from high flood flow, pond level flow and static water cases. The hydraulic grade line and the depth of surface water for the corresponding flow cases are calculated to find the maximum unbalanced head at a point from the three flow cases. Then the necessary thicknesses are calculated from the density of apron material.

B) Stability against Overturning

Stability against overturning is the next important factor to take into consideration. In big structures such as most low-head diversion structures, where the forces that act on them are distributed. It is necessary to keep the stabilizing moment more than the destabilizing moments.

C) Stability against Shear and Sliding

The aprons of the structure and the weir body are considered for stability against shear and sliding. The structure may slide in the flow direction if there are not enough grips between the base and the foundation. To prevent this happening, the vertical forces are checked to be adequate, compared to the horizontal forces, to supply static friction that would keep the structure intact in its place (Baban, 1995).

Applied to well-constructed mass concrete, FSS on a horizontal plane should not be permitted to exceed 0.75 for the specified normal load combination. FSS may be permitted to rise to 0.9 under the extreme load combination (P. Novak, 4th edition).

3. Method and Material

3.1 Study Area

3.1.1 Location

The study area, Fantale Irrigation Based Development Project, is located 0n Awash River at about 4Km upstream of Bole Bridge. It is near Nura Hera state farm camp number 3 which is about 50 km south west of Metehara town and at about 160 km east of Addis Ababa. It is located at latitude 8 58′and longitude39 54′. The site has been visited for this research from 5-20 June 2013.

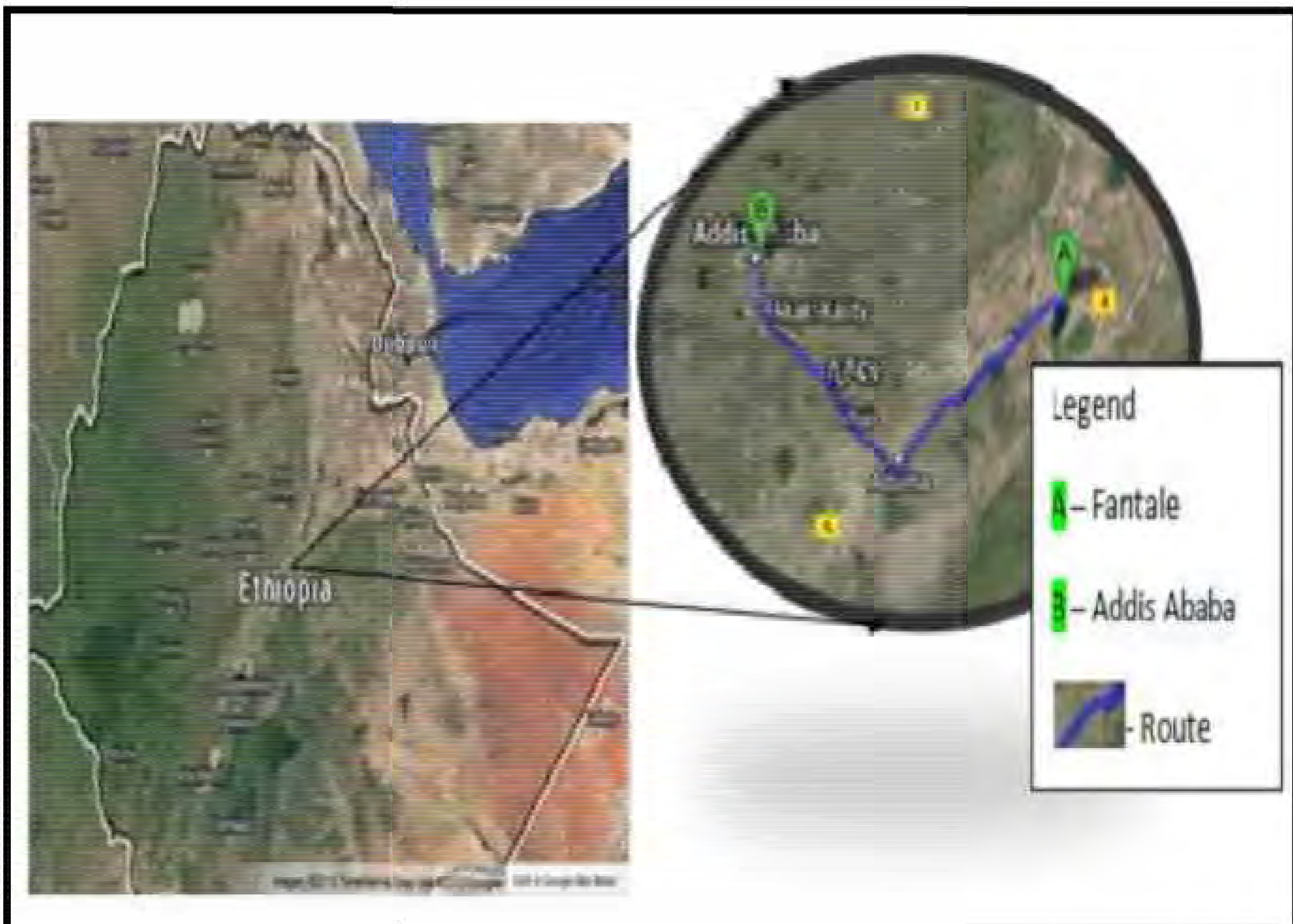


Figure 6 Geographical location of Fantale Irrigation Diversion Head-work

3.1.2 Description of Fantale Irrigation

Fantale Irrigation Based Integrated Development Project (FIBIDP) located in Fantale and Boset Woredas of East Shoa is large-scale irrigation based development project in Oromia. It is also the first large-scale irrigation project designed and constructed by the capacity of a construction government enterprise (Oromia Water Works Construction Enterprise).

The dominant command area is situated west of Metehara town and it is bound to the foot slopes of mount Fentale on the north, Lake Beseka on the east, Rift valley escarpment on the west and rocky land (young lava flow) on the south. More precisely it falls in between 8^0 50' and 9^0 04' N latitude and 39^0 41' and 39^0 52' E longitude. The command area has covered gross area of about 27,000 hectares with altitude ranging from 970 to 1020 m above sea level.

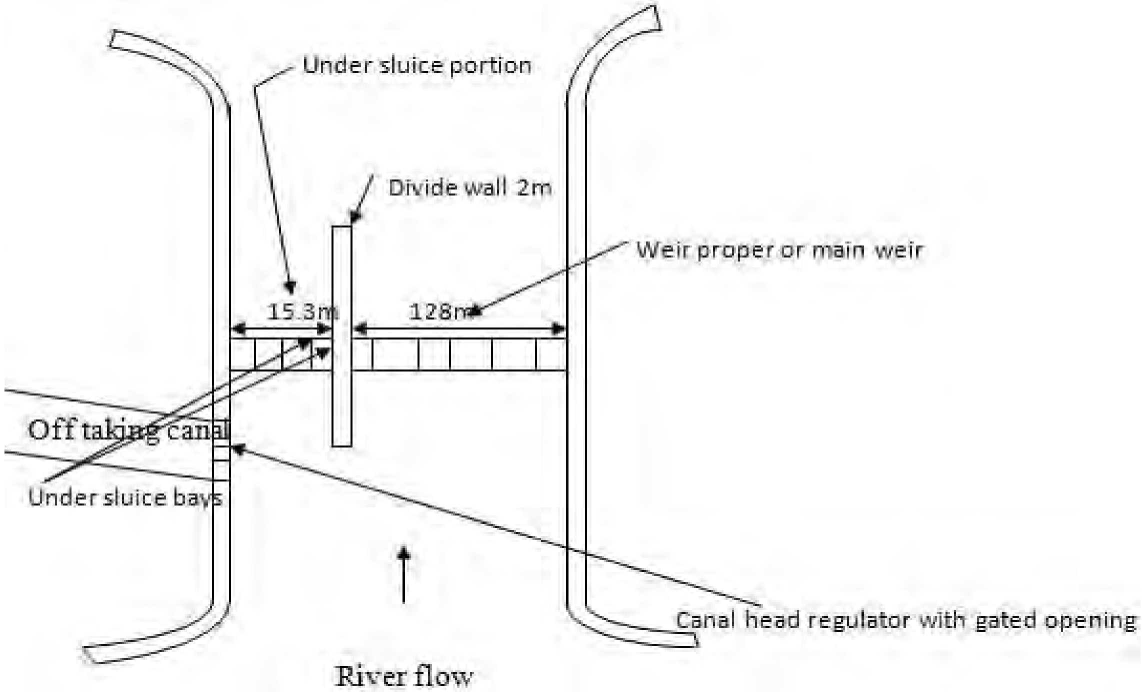
The total area covered was about 27,000 hectares out of which areas for the expansion of Metehara Sugar Factory (MSF) covered 11,000 hectares. The project area covers Kawa and Huluko from Boset district; and Gidara, Turo Bedenota and Tututi kebeles from Fentale district.

The water source for the whole command area of the project is the Awash River. The diversion site on Awash River and the Kawa command area can be accessed via Metehara Sugar Factory (MSF) 25 km on all-weather road and 5 km dry weather access road.

Irrigation water for abstraction is raised by weir at the headwork of the project and conveyed through Conveyance main canal of 49.3 km and two Branch Canal 38 & 31 km with pertinent structures and Pump including Electro-mechanical equipment for Kawa command area.

3.1.3 Description of Fantale Diversion Head-work

The main features of Fantale Irrigation diversion head work are weir, under sluices, divide wall, and canal head regulator. The total water way (four bays of the under sluice each 2.7m wide, three piers each 1.5m thick, one divide wall 2m thick, width of ungated weir 128m) is 145.3m.


Under sluice portion
Divide wall 2m
Weir proper or main weir
15.3m
128m
Off taking canal
Under sluice bays
Canal head regulator with gated opening
River flow

From all the above considerations the permanent structure for the diversion weir / head works for Fentale Irrigation based development project has been located at Awash River at about 4 km upstream of Bole Bridge.

3.2.2 Selection of the diversion weir axis

In the selection, due consideration was given to the flat topography, the shallowness and the meandering nature of the river, besides the area to be flooded during the floods and existence of the village close to the right bank.

3.2.3 Selection of type of diversion structure

If the difference between the pond level and crest level is within 1.5m the pond level can be maintained by falling shutters, then weirs without gates are provided. However, if the difference is more than 1.5m, a gate controlled weir is necessary which is called a Barrage.

The barrage has the following advantages over a weir.

- ❖ It has a better control on the river inflow and out flow as also on the discharge in the main canal

- ❖ Since complete control on the river discharge is possible with proper regulation area in front of the barrage can be maintained free of shoals and sediment deposition.

- ❖ With the help of scouring sluices comparatively sediment free water is fed in the off taking canal

- ❖ Relative merits and demerits of having a barrage or a weir for diverting the river water are:

- ❖ In the barrage, the sill can be kept at a lower level and the pond level can be maintained with the help of gates, whereas in the weir the sill of the weir will has to be kept at the pond level to maintain the pond level resulting in to higher afflux for the same water way.

- For maintaining the pond level and to have better control on the river inflow and out flow, large number of gates have to be provided in the barrage while in the weir by keeping the sill level at pond level no gate need to be provided.

- Large numbers of gates in barrage not only result into heavy initial cost but would also need regular heavy expenditure on their operation and maintenance while in weir no such cost is involved.

- In the barrage, the afflux at high flood level can be kept minimum where as in weir option the afflux cannot be reduced beyond a certain limit as the sill of the weir is kept at pond level.

- In the barrage as the piers are provided in the bays, a bridge if required can be provided at little extra cost where as in weir no bridge can be provided in the weir portion. In the proposed diversion work, however, no road bridge is required to be provided.

- The scouring sluices have to be provided in both the cases

In view of the heavy additional cost involved and relatively fewer advantages in the barrage option, it is proposed to provide a weir with no gates and crest at the pond level and scouring sluices with gates.

3.2.3 Design of weir and under-sluice

Like any other hydraulic structure the design of weir and under sluices consists of two main phases:-

- The hydraulic design
- The structural design

The hydraulic design basically consists of the following steps:

- Fixing of design flood discharge.
- Preparation of the rating curve.

- Fixation of pond level.
- Determination of optimum waterway and afflux for the design flood discharge.
- Evaluation of effect of retrogression.
- Determination of levels of crest and upstream floor of the weir and under sluice bays.
- Level and length of the downstream floor.
- Determination of depth of upstream and downstream cut offs.
- Determination of the total length of the floor from the exit gradient considerations.
- Design of protection works.

a) Design flood discharge:

For the purposes of design of items other than free board, a design flood of 50 years frequency has been adopted. For the design of free board, 500 year flood has been adopted. Thus design floods of 492 cu m/sec and 635 cum/sec (which have been computed in the Hydrological and Meteorological study report) have been adobted for the design of weir and for free board respectively.

b) Rating Curve

No past observed flow data for the selected site was available and thus the sufficient information and data required for developing a rating curve was not available. In the absence of detailed data a preliminary rating curve has been prepared by computing the discharges at different water levels from the available topographical map, the cross section and the longitudinal profile by using the Manning's equation.

c) Pond level

Pond level in the under sluice pocket upstream of the canal head regulator and upstream of weir portion is generally obtained by adding the working head to the designed full supply level of the canal. The working head shall include the head required for passing the design discharge into the canal the head losses in the regulator and for possible rise of FSL in the canal due to silting in the head reach of the canal. The pond level is normally fixed a minimum of 1.0 m above the full supply level in the off taking canal. The pond level has however been fixed at EL 1171.16 m, which is 0.92 m above the full supply level in the main canal.

d) Optimum Water Way and Afflux

In deep and confined rivers with stable banks the overall water way (between abutments) including thickness of piers should be approximately equal to the actual width of river at the design flood. A likely figure to adopt for water way is given by the following formula representing Lacey's wetted parameter:-

$$P = 4.83\sqrt{Q} \quad \text{......} (3.1)$$

Where, P is the overall water way in meters and

Q is the design flood discharge in cum/sec for 1 in 50 year frequency.

For a design flood 492 m^3 / sec, (1m 50 years flood) the Lace's waterway works out to 107 m. However, a total waterway of 145.3m has been provided for the weir (under sluice portion + weir portion) to limit the afflux up stream of weir.

Four bays of under sluices each 2.7 m wide	10.8 m
Three piers each 1.5 m thick	4.5 m
One divide wall 2.0 m thick	2.0 m
Width of un-gated weir	128.0 m
Total	145.3 m

With this water way the design floods of 492 cum/sec/and 635cum/sec shall pass with the affluxes water levels of 1178.60m and 1178.90m respectively, which will not create any appreciable heavy submergence on the upstream.

e) Crest and Upstream Floor Level of the Weir Portion & Scouring / Under Sluice Bays

The width of the under sluice portion is determined on the basis of the following considerations.

- ❖ It should be capable of passing at least double the canal discharge to ensure good capacity.
- ❖ It should be capable of passing at least 20% of the maximum flood discharge at high floods.
- ❖ It should be capable of passing fair weather freshets and low monsoon floods for obviation overtopping of weir.
- ❖ It should be wide enough to keep the approach velocities sufficiently lower than the critical velocities to ensure maximum settling of suspended silt load.

The crest level of the weir is kept at the pond level, as no shutters or gates are proposed to be provided on the weir. Having tentatively decided the crest levels as well as the waterway of the under-sluice and the weir proper, it is necessary to check that the maximum flood discharge passes down the works without excessive afflux.

The crest and floor level of the under-sluice bay has been kept at 1173.10 m and 1171.40 m. The crest and floor level of the weir portion has been kept at 1177.16 m and 1174.00 m respectively.

With these levels the designed flood of 492 cum/sec shall pass at a water level of 1178.60 m, which hereafter will be termed as design H.F.L. The flood of 1 in 500 years frequency (635 cum/sec) will pass at a water level of 1178.90 m.

f) Divide Wall:

The length of the divide wall has been fixed such that,

- ❖ It does not extend beyond the upstream end of head regulator
- ❖ Generally satisfactory results are obtained if it covers two thirds the width of the head regulator
- ❖ Downstream divide wall shall extend up to the end of the downstream apron

- The thickness of the divide wall has been kept as 2.0 m

g) Upstream and Down Stream Protection Works

The impervious floor of a weir or barrage is protected on the upstream as well as on the downstream by loose apron. In the immediate vicinity of the impervious floor a certain portion of loose apron is made non- launching. The non-launching apron prevents the scour hole to travel close to the floor whereas the launching apron is designed to launch along the slope of the scour hole to prevent further scooping out of the underlying bed material.

3.2.4 Structural design

Having fixed the hydraulic profile of the regulator and general layout of wings and abutments structural design of various components was done on the basis of following general criteria.

The following loads have been considered for designing

- Self weight wing wall and weight of earth retained
- Water pressure
- Earth pressure
- Uplift pressure - Although free draining backfill material is specified for backfill and pipe drains are specified to be installed at the bottom of the backfill, full uplift is considered acting at the base of the footing and only 15 % of the uplift at the junction of masonry retaining wall and concrete footing.
- Earthquake

In absence of laboratory test results the following data have been used for the design of the retaining walls/abutments:

- Weight of stone masonry in 1:4 cement 2000 kg/m^3
- Weight of dry backfill 1600 kg/m^3
- Weight of saturated backfill 1800kg/m^3
- Weight of foundation concrete 2400 kg/m^3
- Angle of repose (ø) = $30^0$and cohesion
- Coefficient of friction between concrete and masonry = 0.7

- Coefficient of friction between concrete and foundation = 0.6

3.3 Data Collection

The beneficiary or user community that is obtaining the services, field measurements and observations are the primary sources of data for the study. In order to achieve the objectives of the study secondary data are also used. These data are obtained from the Oromia water works construction enterprise, project offices, and agriculture & rural development offices that are found at grass root level. In addition to these literatures, different project documents or proposals, project evaluation and completion reports are also refereed.

3.3.1 Primary data collection

Field observation at Fantale project was made to identify where different parameters of the head-work must be taken. Main canal discharge, sediment depth upstream of the canal head regulator, water levels at carefully selected points of the project was taken in collaboration with the Oromia water works construction enterprise workers. The opening heights of the gates provided at the under sluice and canal head regulator were also measured on the site for this research work.

3.3.2 Secondary data collection

Secondary data used for this research were collected from responsible bodies and officials. These data include design discharge of the head-work, climatic data, water demand on the command area, and dimensions of the headwork components for the project. In general the design report was also revised to get some insight in to the head work components and these help in identifying the sensitive parameters for the problems on the head work.

3.4 Methods and materials

On first stage previous studies, documents and papers related to diversion headwork have been revised and desk study have been undertaken to identify the key issues. In the desk study, the available relevant data on existing irrigation structures (i.e. Fantale diversion headwork) and post implementation review reports on the status of existing schemes was also important to be collected from the regional irrigation authority.

Field visit surveys have been conducted for gathering out of data for the purpose of describing the nature of existing conditions and to compare existing conditions with the design target of the diversion headwork on Fantale. The survey conducted is cross sectional or one shot data gathering which is economic and efficient covering a wide target.

To assess the operation and functionality of constructed diversion head works as well as operations of each component parts of the head work, field visit survey coupled with primary data obtained by interviewing beneficiary and administrative officials at Woreda as well as project manager of the irrigation project have been conducted. The secondary dates' are also collected from Oromia Water Works Design and Supervision Enterprise design reports.

For the hydraulic and structural analysis of the Fantale irrigation based integrated development project (i.e. head work) the survey datas collected were of the river cross section, upstream and downstream of the headworks. These data's are taken out of the design of the head work prepared by Oromia Water Works Design and Supervision Enterprise in 2009 G.C. Therefore this set of data's were used for the hydraulic analysis and adequacy check of the existing headwork and previously proposed design by the consultant of the project.

The first parameter that is assessed for this study is the design flood i.e. passage of floods, including hazard floods. The flood magnitudes taken from the hydrological study for the project are flood of 1 in 50 years recurrence is equal to 492m^3/s, flood of 1 in 100 years recurrence is equal to 550m^3/s, flood of 1 in 500 years recurrence is equal to 632m^3/s.

Discharge through the sluice way and canal head regulator is given by submerged orifice flow equation:

$$Q = Cd \times A \times \sqrt{2gh} \quad (3.2)$$

Where, C_d - Discharge coefficient, 0.64

A - Opening area

g – Gravity, 9.81 m/

h – Upstream water level minus downstream water level

Discharge over the weir is calculated by: Q_2 = Cd (L- nKH (H) ^1.5……………….(3.3)

Where, C_d = is the coefficient of discharge

L = net width of weir (waterway)

Q_2 = discharge over the weir

n = number of end contractions

K = coefficient of end contraction ~ 0.1

Seepage analysis tackles water seepage below Fantale weir structure (on the weir foundation), the quantity of seepage, pressure head and exit gradient were calculated using both Geo-Seep/w and khoslas Method of independent variable (see the appendix 3).

Geo-Seep/w is a finite element package that can be used to model the fluid flow and pore-water pressure distribution within porous materials such as soil and rock. Its comprehensive formulation makes it possible to analyze both simple and highly complex seepage problems. The objective is to examine the pore water pressure conditions in the foundation beneath a water retention structure and to estimate the seepage losses through the foundation.

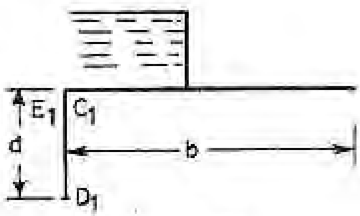
The steps of this model are: Defining problems (input), Solving problems, Contouring and Graphing Results (output).Fantale weir model have been carried out by using this model and the above steps to predict the seepage under the weir foundation.

For designing hydraulic structures such as weirs or barrages on pervious foundations khosla has evolved a simple, quick and an accurate approach, called Method of Independent Variables. In this method a complex profile like that of a weir is broken in to a number of simple profile each of which can be solved mathematically. Mathematical solutions of the flow nets for these simple standard profiles have been presented in the form of equations given in figure below which can be used for determining the percentage pressures at various key points. The simple profiles which are most useful are:

- ❖ A straight horizontal floor of negligible thickness with a sheet pile line on the u/s end and d/s end (Fig.9 a and b)
- ❖ A straight horizontal floor depressed below the bed but without any vertical cut-offs (Fig.9 c)
- ❖ A straight horizontal floor of negligible thickness with a sheet pile line at some intermediate point (Fig.9 d).

The key points are the junctions of the floor and the pile lines on either side and the bottom point of the pile line, and the bottom corners in the case of a depressed floor. The percentage pressures at these key points for the simple forms in to which the complex profile has been broken is valid for the complex profile itself, if corrected for;

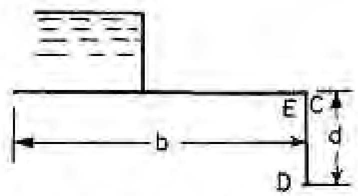
- ❖ Correction for the mutual interference of piles
- ❖ Correction for thickness of floor
- ❖ Correction for the slope of the floor



$$\phi_{C_1} = 100 - \phi_E$$

$$\phi_{D_1} = 100 - \phi_D$$

(*a*)



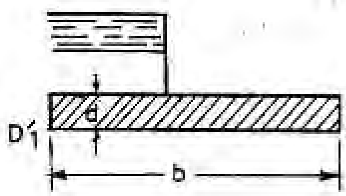
$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$$

where $\lambda = \dfrac{1 + \sqrt{1 + \alpha^2}}{2}$

$\alpha = \dfrac{b}{d}$ (respective)

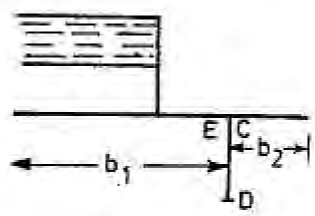
(*b*)



$$\phi_{D'} = \frac{2}{3}(\phi_E - \phi_D) + \frac{3}{\alpha^2}$$

$$\phi'_{D_1} = 100 - \phi_{D'}$$

(*c*)



$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$$

$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

$$\phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$$

where $\lambda = \dfrac{\sqrt{1 + \alpha_1^2} + \sqrt{1 + \alpha_2^2}}{2}$

$\lambda_1 = \dfrac{\sqrt{1 + \alpha_1^2} - \sqrt{1 + \alpha_2^2}}{2}$

$\alpha_1 = b_1/d$

$\alpha_2 = b_2/d$

(*d*)

In order to check the value of exit hydraulic gradient when the maximum total head (H) equals 3.08 m. The depth of the last sheet pile at downstream (d) equals 5.5 m, and the length of the hydraulic structure floor (b) equals 31.6 m for the weir part.

The critical hydraulic gradient (i_{cr}) was calculated from:

$$i_{cr} = \frac{\gamma_{sat} - \gamma_w}{\gamma_w} \quad \text{...(3.4)}$$

Also, by using value of unit weight of saturated soil underneath the weir structure (γ_{sat}) equals 18kN/m, and unit weight of water 9.807kN/m. The calculated value of the critical hydraulic gradient (i_{cr}) equals 0.835.

The existing exit gradient for Fantale weir and under sluice using the formula:

$$G = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}} \quad \text{..(3.5)}$$

The other option in analyzing seepage below the Fantale weir structure is to use Geo-studio, Seep/w analysis. It is a finite element package that can be used to model the fluid flow and pore-water pressure distribution within porous materials such as soil and rock. Its comprehensive formulation makes it possible to analyze both simple and highly complex seepage problems.

The discretization of this model into a finite element mesh is calculated as quadrilateral regions and drawn in the problem domain. Inside each region, any number of finite elements can automatically be generated.

The steps of this model are: Defining problems (input),Solving problems, Contouring and Graphing Results (output).Fantale weir model have been carried out by using this model and the above steps to predict the seepage under the weir foundation.

The study tackles water seepage below Fantale weir structure (on the weir foundation), the quantity of seepage and pressure head and was calculated using (Geo-seep/w) model.

For checking the adequacy of the upstream and downstream cut off depth to Lacey's equation is used:

$$R = 1.35\left(\frac{q^2}{f}\right)^{0.33} \qquad (3.6)$$

To insure the stability against uplift the necessary apron thickness at different points along a longitudinal section are provided. For checking the thickness of the weir and under sluice for downstream impervious floor, the equation below

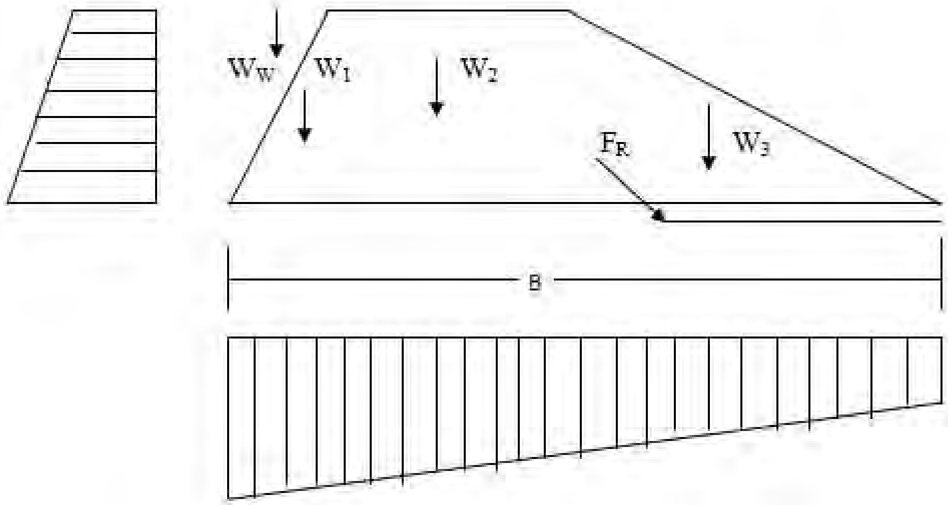
$$t = \frac{h}{G-1} \qquad (3.7)$$

Where, t= thickness of apron at a point, h= the unbalanced head between the uplifting pressure head and surface water depth, G=Density of construction material for apron. Equation above was used to calculate the required thickness of floor and compare it with the provided thickness which must be equal to or larger than values of the required thickness.

For checking the stability of the headwork against overturning both the expected worst conditions and existing situation are considered for analysis. The forces and moments acting on the corresponding structure are then calculated and the structure is checked for its stability against overturning.

The forces and moments taken in to account for the worst case are weight of the weir itself (w1, w2, and w3), weight of water supported on the upstream slope (Ww),uplift forces (U), horizontal hydrostatic pressure (p), upward vertical earthquake forces, and horizontal inertia forces due to earthquake (see figure 10).

In the worst case it should be reminded that the highest flood level is considered on the upstream side and empty on the downstream side and also the weight of water overtopping the weir is also neglected. The water on the top of weir is neglected because it acts downward and it supports the structure against overturning see figure below.



$$e \quad \frac{B}{6}$$

$$e \quad \frac{B}{2} \quad X \quad \frac{B}{6}$$

$$\frac{M}{V}$$

$$M$$

$$V$$

X= distance of the resultant of the forces from the toe

B= width of the weir base

For checking the stability of the diversion structure against shear and sliding both the expected worst conditions and existing situation are considered for analysis. The forces and moments acting on the corresponding structure are then calculated and the structure is checked for its stability against shear and sliding (see appendix 4).

In the worst case it should be reminded that the highest flood level is considered on the upstream side and empty on the downstream side and also the weight of water overtopping the weir is also neglected. In case of existing situation the water level and uplift considered for analysis are based on existing situation.

The structure may slide in the flow direction if there are not enough grips between the base and the foundation. To prevent this happening, the vertical forces are checked to be adequate, compared to the horizontal forces, to supply static friction that would keep the structure intact in its place.

Factor of safety against shear and sliding can then be defined as the ratio of the summation of all horizontal load components, $\sum H$, to the summation of all vertical loads, $\sum V$, on the plane considered, i.e. for a horizontal plane:

$$\frac{\sum H}{\sum V} \qquad (3.10)$$

Applied to well-constructed mass concrete, FSS on a horizontal plane should not be permitted to exceed 0.75 for the specified normal load combination. Factor of safety against shear and sliding may be permitted to rise to 0.9 under the extreme load combination (Novak p., Moffat, Nalluri C. Narayanan R. 2001).

From the study and design reports, the current performance of the diversion headwork is examined and from the practical visits undertaken on the site, the extent of the problems on the structure is assessed.

Materials used for this research work are stopwatch, tapes, and digital camera, staff rods have been used for measuring distance, time, depth, velocity and discharge of water in the different parts of the headwork. Staff rod has also been used to find the depth of sediment stored upstream of the under sluice part.

4. Result and Discussion

4.1. Hydraulic performance indicators

4.1.1 Passage of floods, including hazard floods and Withdrawal of water

There is no extreme flooding occurred after the structure begins operation. The flood discharge capacity of the head work is greater than the designed discharge capacity.

Table 4 Calculated discharge capacity of the under sluice and weir

Discharge Capacity of Under Sluice(Q_1)	Discharge Capacity of the weir (Q_2)	Discharge Capacity (Q_1+Q_2)
111.5 m^3/ s	402.8 m^3/ s	514.3 m^3/ s
116.02 m^3 / s	534.8 m^3 / s	650.82 m^3/ s

In table 4 Q_1 and Q_2 are calculated using equation 3.1 and 3.2 and it indicates the total calculated discharge capacity of the head-work (514.3m^3/s) is greater than the design flood (492m^3/s). This implies that the head-work is safe against flood even more than the design flood. The result also implies that there is no extreme flooding occurred after Fantale diversion structure begins operation.

The existing minimum water level of the Fantale head work is 1178.08m which is above the design water level which is 1177.16m. The headwork can feed the water required by the command area throughout the year.

Using equation 3.2 the existing flow of water in to the main canal is 11.5m^3 /s but the amount of water required to be diverted to the main canal is 18m^3 /s. This implies that there is a shortage of water by an amount 6.5 m^3/s to meet the demand of the command area (see appendix 2).

The above result is the consequence of malfunctioning of the gates. To achieve the required $18m^3/s$ discharge the two gates should have to be opened by an amount 0.78m or one gate should have to be opened by an amount 1.565m. But gates provided both on the under sluice and canal head regulator are not functioning because of incomplete construction procedure and failure.

4.1.2. Passage of sediments

The accumulation of high silt load at the head regulator, high sediment entry to the main off taking canal is prevalent. The sediment load deposited in front of the head regulator cannot be flushed out. Sediment deposition and formation of shoal in front of the weir proper is also prevalent.

Gate number one on the canal head regulator (the gates are numbered one to two, west to east) jammed in the fully closed position and gate number two remains stuck at height of 1m opening. The gates provided at the under sluice were also not operating.

Design does not take in to account actual field condition into consideration for selecting type of diversion structure. The design of Fantale Diversion Head work considered the technical measures to be taken merely for selecting the type of diversion structure.

Capacity of the design flow to transport different size of sediment is not determined for the case of Fantale diversion structure. In addition to the above reasons failure to provide trash rack on the head regulator is also one of drawback of Fantale diversion headwork which results debris entry in to the main canal.

The designed under sluice sill level that is the lowest river bed level at the weir axis was 1173.10m but the existing under sluice level is raised to a level of 1174 i.e. about 0.9m depth sediment deposited at the back of head regulator.

4.1.3 Seepage Analysis

The existing exit gradient for Fantale diversion weir and under sluice is obtained to be 0.0965 and 0.189 respectively. Adequate for the safe performance of the Fantale diversion weir and

Foundation condition	u/s water level (m)	d/s water level (m)	Head(m)	Height of sub soil H.G line above the datum					
				u/s pile line			d/s pile line		
Actually existing water level	1178.08	1176.1	3.08	$\square_{E1}$ 100%	$\square_{D1}$ 77.13%	$\square_{C1}$ 71.66%	$\square_{E2}$ 32%	$\square_{D2}$ 25%	$\square_{C2}$ 0%
				3.08	2.38	2.21	0.99	0.77	0

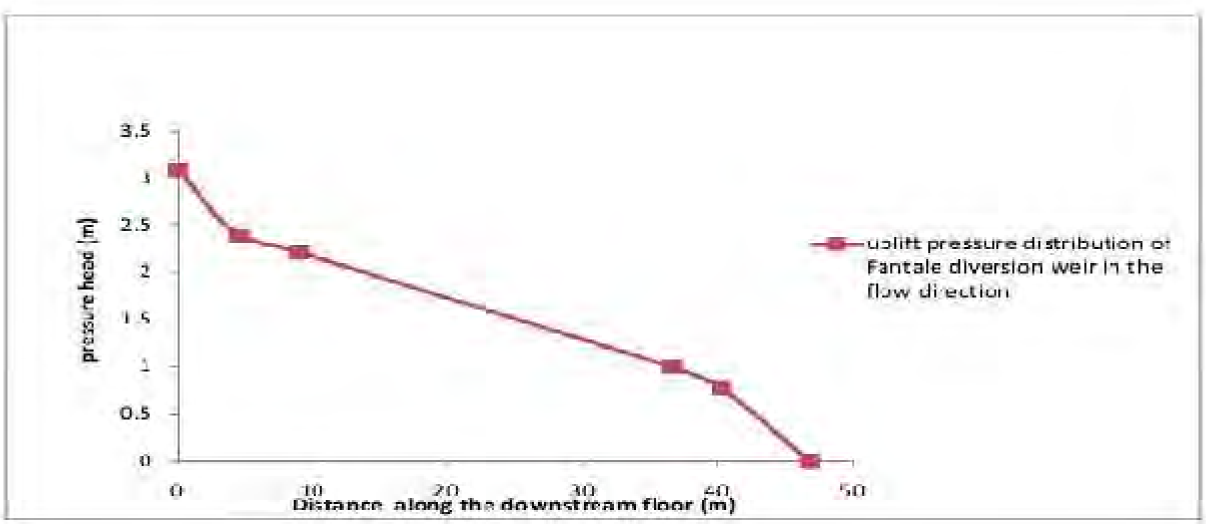


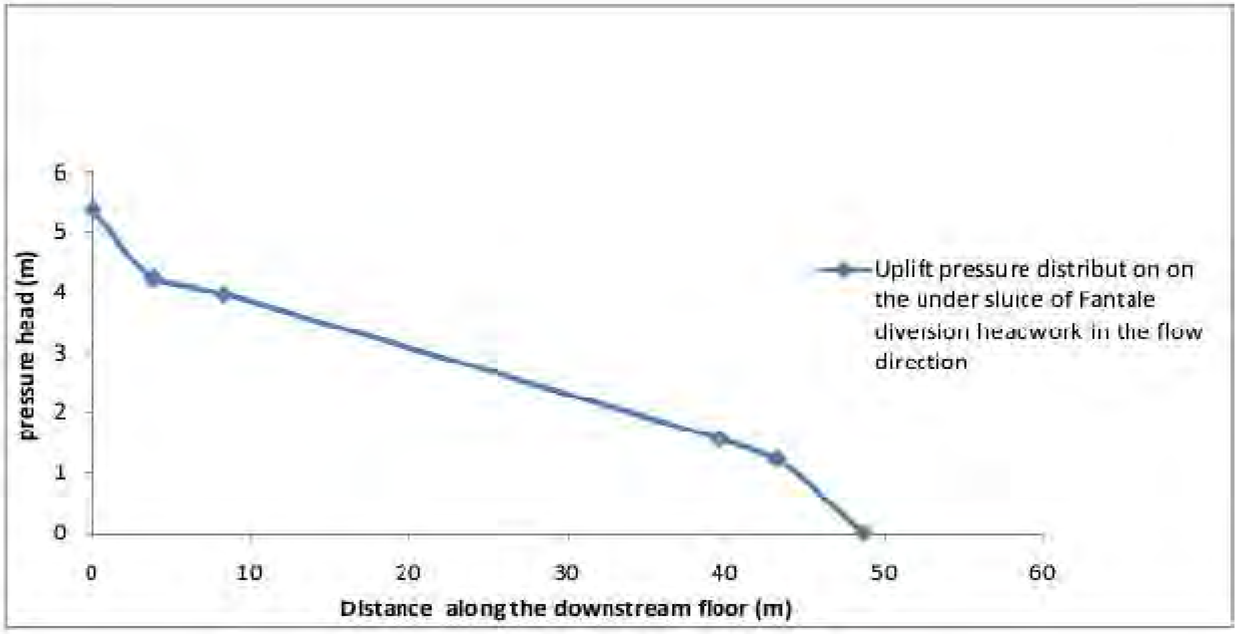
Figure 11 is the graph of distance along the downstream floor of the weir versus the calculated pressure heads. The graph shows that the upli ft pressure decreases as the creep length progresses to the downstream end, but this does not mean that the residual uplift pressure head does linearly vary from upstream to downstream. This means that the thickness required at the upstream is thicker than to the downstream.

This implies that that accurate representation of the subsurface flow phenomenon is quite important for proper design of diversion structures in particular. It also provides the uplift pressure at each point which in turn serves as an input to find the required floor thickness at each point.

Using the method of independent variables the pressures at different key points are calculated with khoslas simple profiles and the values are corrected for the piles mutual interference and thickness of the floor. And the corrected pressures at various key points of Fantale under sluice structure are tabulated below.

Table 6 Corrected pressures at various key points of Fantale under sluice structure

Foundation condition	u/s water level (m)	d/s water level (m)	Head(m)	Height of sub soil H.G line above the datum					
				u/s pile line			d/s pile line		
Actually existing water level				$\square_{E1}$ 100%	$\square_{D1}$ 78.8%	$\square_{C1}$ 73.76%	$\square_{E2}$ 29.1%	$\square_{D2}$ 23%	$\square_{C2}$ 0%
				5.38	4.24	3.968	1.568	1.23	0



6
5
4
3
2
1
0
pressure head (m)
0
10
20
30
40
50
60
Distance along the downstream floor (m)
Uplift pressure distribut on on the under sluice of Fantale diversion headwork in the flow direction

Table 7 Required thickness for downstream floor of Fantale weir

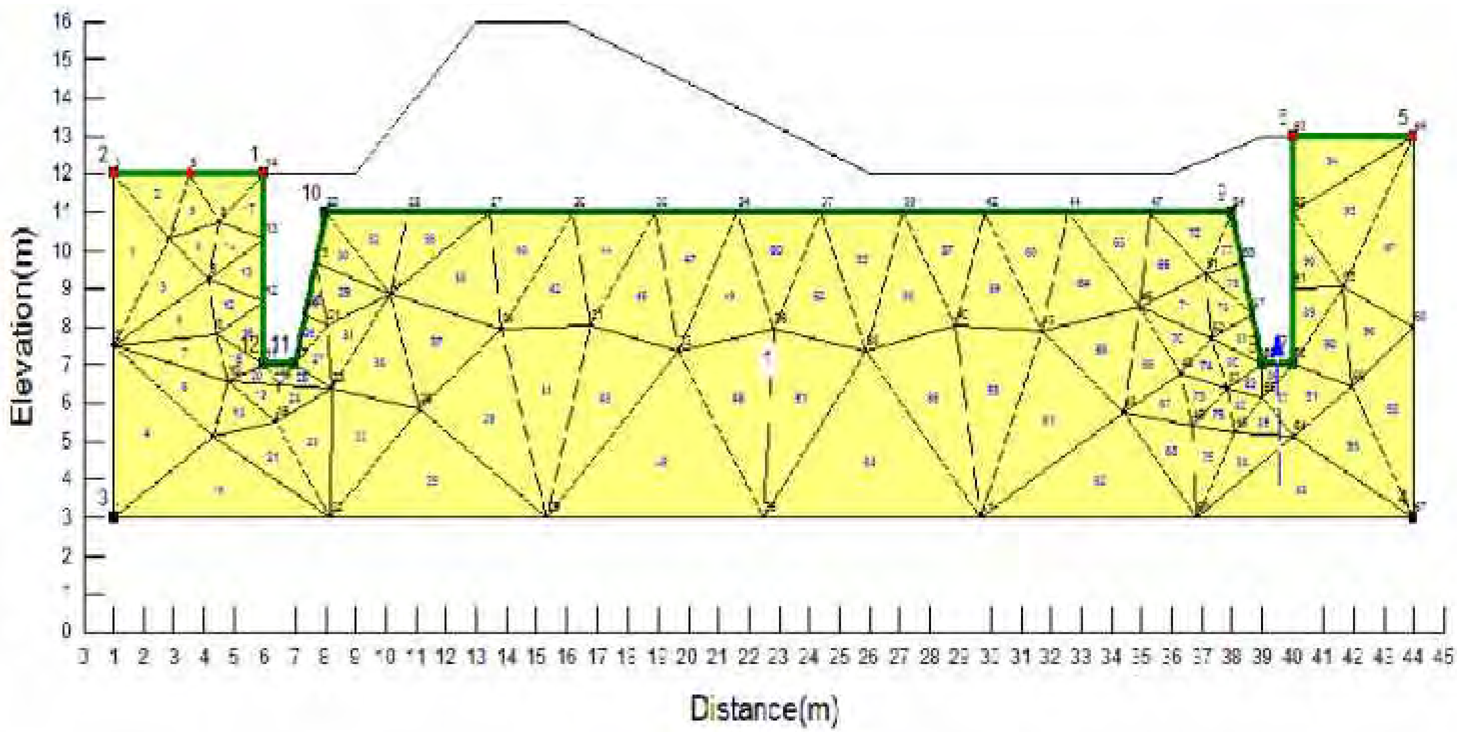
Distance to d/s floor	17.8	19	23	25	26	27.8	28	29	31	31.6
Calculated floor(t)(m)	1.13	1.09	0.954	0.885	0.852	0.79	0.785	0.745	0.651	0.623

Table 8 Required thicknesses for downstream floor of Fantale under sluice

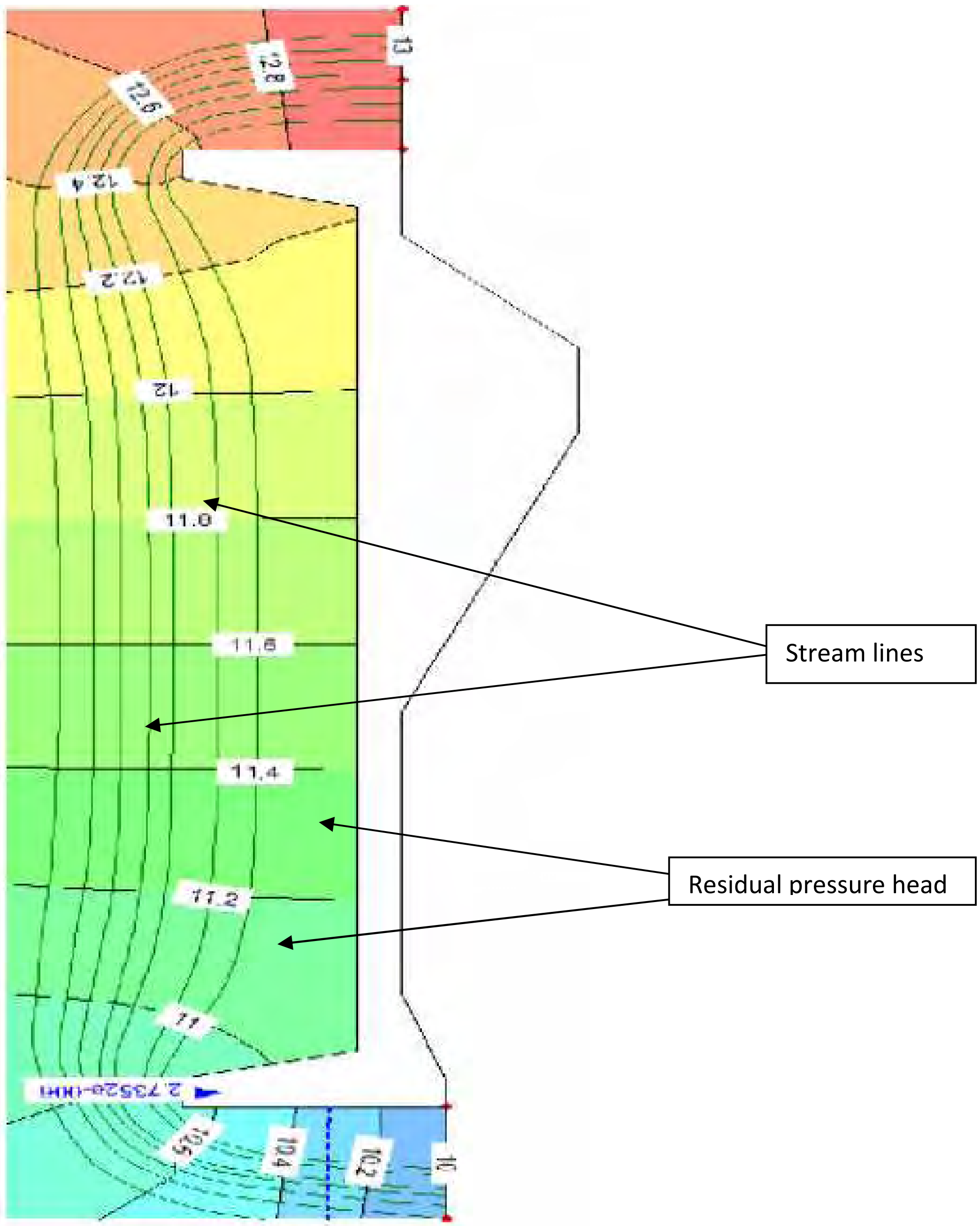
Distance to d/s floor	15.1	17	18	20	21	22.45	25	26.3	27	28	29	30	31
Required floor(t)(m	2.2	2.09	2.03	1.9	1.85	1.76	1.62	1.54	1.5	1.44	1.38	1.32	1.26

The other option in analyzing seepage below the Fantale weir structure is to use Geo-studio, seep/w analysis. It is a finite element package that can be used to model the fluid flow and pore-water pressure distribution within porous materials such as soil and rock. Its comprehensive formulation makes it possible to analyze both simple and highly complex seepage problems.

The steps of this model are: Defining problems (input), solving problems, Contouring and Graphing Results (output). Fantale weir model have been carried out by using this model and the above steps to predict the seepage under the weir foundation.



Elevation(m)
Distance(m)



Stream lines
Residual pressure head

The value of quantity of total seepage (q) is 2.735E-6 m3/sec/m, and Figure 14 illustrates the seepage flow path underneath the weir foundation and the quantity of seepage, (Numbers in the Figure referred to the head at each location with respect to the soil depth under weir is 10 m).

4.1.4 Prevalence of scouring

By using value of discharge intensity per unit width that is 4.75 for under sluice and 1.7 for the weir portion. The calculated values of the cutoff depth are 3.76m and 2.95m (using equation (3.6)) for the under sluice and weir portion respectively.

When the provided piles at the upstream and downstream is compared with the calculated values, the provided depth must be greater than the calculated lacey's scour depth which comes true in case of Fantale diversion headwork.

In case of Fantale as the bed of the river is fairly hard, there is not likely to be any appreciable scour on the downstream of the under sluice bays and weir portion, no extra protection is therefore, provided on the upstream and downstream side. Fantale diversion headwork is protected from both upstream and downstream scouring.

4.1.5 Clogging of under sluice

In case of Fantale diversion headwork efficient flushing of the sediments from the under sluice pocket is not ensured. Gates on the under-sluice (jammed in the fully closed position). The under sluice opening is clogged with the deposited sediment.

Capacity of the design flow to transport different size of sediment is not determined for the case of Fantale diversion structure. This intern leads to accumulation of sediment.

4.2 Structural performance indicator

4.2.1 Stability against Uplift

Tables 9 and 10 show the calculated required floor thickness for points below downstream floor of the under sluice and weir, and the provided floor thickness for these points. The distance values of these points were calculated from the under sluice and weir structure from upstream side.

The uplift pressure head below the floors of both the under-sluice and the weir are obtained from the khoslas method of independent variables analysis in section 3.1 of this research work. The calculated required floor thicknesses are obtained by hand calculation using equation (3.7) .

Table 9 Required thicknesses for downstream floor of Fantale under sluice

Distance along the downstream floor	Uplift pressure head below the floor(m)	Calculated required floor thickness(t) in (m)	Provided floor thickness(m)
15.1	2.86	2.2	2
17	2.72	2.09	2
18	2.64	2.03	2
20	2.486	1.9	2
21	2.41	1.85	2
22.45	2.3	1.76	2
25	2.1	1.62	1
26.3	2	1.54	1
27	1.95	1.5	1
28	1.87	1.44	1
29	1.795	1.38	1
30	1.72	1.32	1
31	1.64	1.26	1

Table 10 Required thicknesses for downstream floor of Fantale weir

Distance along the downstream floor	Uplift pressure head below the floor(m)	Calculated required floor thickness(t) in (m)	Provided floor thickness(m)
17.8	1.468	1.13	1
19	1.416	1.09	1
23	1.24	0.954	1
25	1.15	0.885	1
26	1.107	0.852	1
27.8	1.03	0.79	1
28	1.02	0.785	1
29	0.969	0.745	1
31	0.846	0.651	1
31.6	0.809	0.623	1

4.2.2 Stability against Overturning

The stabilizing moment is greater than destabilizing moment both for highest flood level and existing situation (see appendix 4.a). Factor of safety against overturning for weir section is 1.635 which is in between the recommended values (i.e. 1.5-2.0). There is no lifting up of the structures heel and the structure is not susceptible to any tension on the base.

4.2.3 Stability against shear and Sliding

Fantale diversion structure is checked for stability against shear and sliding and the result manifested that the diversion structure is safe against shear and sliding both for worst condition and existing situation on the site at the time of investigation (see table 11).

Table 11 Stability against shear and sliding

Factor of safety against shear and sliding			
	Calculated	Recommended	Remark
For worst case	0.67657	0.9	Safe against shear and sliding
For existing case	0.2511	0.75	

4.3 Discussion

A diversion structure has to be designed in such a way that it may be able to pass a high flood of sufficient magnitude (called the design flood) safely. Therefore while designing structures it has to be considered a flood value against which these structures can be designed to be safe. It can neither use a very high value nor a very low value because very high value needs much more investment and that of very low value may cause damage on the structure.

So considering the above facts the design of Fantale diversion structure uses a design flood of 50 years frequency for the purposes of design of items other than free board. For the design of free board, a design flood of 500 years flood has been adopted. Thus design floods of 492m^3/s and 635m^3/s (which have been computed in the hydrological and meteorological study report) have been adopted for the design of weir and for free board respectively.

From table 4 in the previous chapter the total discharge capacity of the head-work is greater than the design flood. This implies that the head-work is safe against flood even more than the design flood. The result also implies that there is no extreme flooding occurred after Fantale diversion structure begins operation.

Diversion weir is situated in the river and its function is to dam up the water level in order to ensure a constant minimum depth of water upstream of the weir and to allow the quantity of water for operational purposes (amount of service water) to be diverted from the river irrespective of the regime.

The head regulator was also expected for a discharge capacity of at least 18m^3/s which is the demand at the command area. The discharge has been checked for both full operations i.e. when the two gates fully open and when the regulator operates only 0.5m head in case of severe silt accumulation.

The existing minimum water level of the Fantale head work is 1178.08m which is above the designed minimum water level that is 1177.16m.This implies that the headwork can feed the water required by the command area throughout the year. But this cant proof whether the amount required for the command area is meeting or not.

The existing flow of water in to the main canal is $11.5m^3/s$ (measured value) but the amount of water required to be diverted to the main canal is $18m^3/s$. This implies that there is a shortage of water by an amount $6.5m^3/s$ to meet the demand of the command area.

As it can seen from the result the minimum water level can feed the required amount of water to the canal but the amount of water withdrawn in to the canal at the time of investigation implies that the demand is not meet.

The above result is the consequence of malfunctioning of the gates. Using orifice flow equation to achieve the required $18m^3/s$ discharge the two gates should have to be opened by an amount 0.78m or one gate should have to be opened by an amount 1.565m . But gates provided both on the under sluice and canal head regulator are not functioning because of incomplete construction procedure and failure.

As stated above the other function of diversion headwork is to control entry of silt in to the canal. But Fantale diversion head-work faces accumulation of high silt load at the head regulator, high sediment entry to the main off taking canal.

The sediment load deposited in front of the head regulator cannot be flushed out. Sediment deposition and formation of shoal in front of the weir proper is also prevalent.

With the discharge each channel entrains solids in the form of suspended matter or bed load. The suspended matter consists of small solid particles of various size held in suspension by buoyant forces in the water or by turbulence. In the water they are scarcely visible to the naked eye. Peak discharges of an intensive brown, for example, suggest a high solid matter concentration.

The bed load consists of solids such as fine sand, gravel with a small diameter of up to about 3 mm, or coarse material (gravel, stones of various sizes). The bed load is always transported on the river bottom.

The origin of the solids in the discharge of a channel can be attributed to a great number of causes; for example,

- ❖ Surface erosion as a result of precipitation, chiefly in catchment areas with sparse vegetation cover,
- ❖ Erosion in the river bed, in old branches, in reservoirs, and on foreshores, particularly in the case of peak discharges,
- ❖ Pieces of plants and their decomposition products.

A particle can be transported in the discharge both as bed load and as suspended matter. An exact delimitation is not possible, as the influences in particular the flow velocity - can be very different according to the discharge character. (Graf, w.h. 1984)..

The capacity of the design flow to transport different size of sediment is not determined for the case of Fantale diversion structure. Due to its transporting capacity mentioned above the upstream face of the weir and different gates are exposed to boulders approaching at higher floods. The impact from the boulders on to the weir and gates at the under sluice and canal head regulator will create severe damage. This will lead to malfunctioning of the gates and the whole diversion structure.

Before an intake structure is planned, it is therefore necessary to obtain information on the solid matter transport upstream of it and in the area of the intake structure so as to be able to estimate the influence of the structure upon the deposit of solid matter in front of it and the erosion behind it, and to determine the type of structure to be used. (Graf, w.h.1984). But in case of Fantale it didn't consider this and this is one of design problems.

The presence of sediment loads behind the diversion structure and subsequent emergence of the sediment island upstream of it, not only causes serious challenges for the operational efficiency of these structures but also change the hydraulic gradient and divert the flow towards the downstream side while elevating the river bed and subsequent difficulties in water abstraction .This problem is prevalent in Amibara diversion head in Afar regional state work mainly.

Because the gates cannot be manipulated as required silt deposited upstream of the Fantale diversion structure is not flushed away to the downstream of the river. This caused accumulation of sediment to a depth of 0.9m in the under-sluice pocket which more than the recommended 0.5m depth. Due to this the silt entry in to the canal is not controlled by the diversion structure.

Those gates jammed in the fully closed position on the under-sluice caused the minimum water level increase on the upstream which is not good at the time of high flood. The failure of gates at the canal head regulator has also decreased the amount of water withdrawn to the canal.

The passage of floating debris and trash above the weir and the bed load sluice was not found satisfactory because no trash rack is provided. The only component that can pass the floating debris and trash is the under sluice. This is also the other factor for the bed level increment upstream of the canal head regulator.

The design does not take in to account actual field condition into consideration for selecting type of diversion structure. To hold off the bed load the natural hydraulic behavior of the river can be profited from or technical measures taken should be considered. But the design of Fantale Diversion Head work considered the technical measures to be taken merely for selecting the type of diversion structure.

To assist diversion of silt free water to the main canal, complete control on the river discharge is required with proper regulation area in front of the headwork. This can be achieved by providing a barrage than the weir which is actually provided on the site because a barrage has lower crest height with higher gate height as compared to weir and this will serve to flush sediment deposited upstream of the weir structure easly. Therefore this can be considered as one of the reasons for the actually physically observed problem in sediment deposition and formation of shoal in front of the weir proper.

The water percolating through the foundation exerts an upward pressure on the impervious floor, called the uplift pressure. If the weight of the floor is not adequate to counterbalance the uplift pressure, it may fail by rupture.

Figure 10 shows the total pressure head distribution underneath Fantale weir floor in downstream section .The graph shows that the upli ft pressure decreases as the creep length progresses to the downstream end , but this does not mean that the residual uplift pressure head does linearly vary from upstream to downstream. This means that the thickness required at the upstream is thicker than to the downstream. This implies that that accurate representation of the subsurface flow phenomenon is quite important for proper design of diversion structures in particular. It also provides the uplift pressure at each point which in turn serves as an input to find the required floor thickness at each point.

Figure 11 in section 4.1.3 also shows the total pressure head distribution underneath Fantale under-sluice floor in downstream section. The graph shows that the uplift pressure decreases as the creep length progresses to the downstream end, but this does not mean that the residual uplift pressure head does linearly vary from upstream to downstream. This means that the thickness required at the upstream is thicker than to the downstream. This implies that that accurate representation of the subsurface flow phenomenon is quite important for proper design of diversion structures in particular. It also provides the uplift pressure at each point which in turn serves as an input to find the required floor thickness at each point.

Figure 13 in section 4.1.3 shows seepage of water underneath Fantale weir foundation using Geo studio 2004. It has also provided the same result as the khosla's method of independent that the uplift pressure decreases to the downstream. It also provides the uplift pressure at each point. In addition it has also an advantage over khosla's method of independent variable because this software shows all the stream lines and simplifies tedious hand calculations of Khosla's method which might cause mistakes.

Table 5 in section 4.1.3 shows the required thickness for downstream floor of Fantale under sluice section. From the table we can understand that the provided floor thickness is not adequate because the provided floor thickness is less than required floor thickness along more than half of the distance. This means that the weight of the floor is not adequate to counterbalance the uplift pressure, and it may fail by rupture.

Table 6 in section 4.1.3 shows required floor thickness for downstream floor of Fantale weir section. From this table also we can see that the provided floor thickness is not adequate because the provided floor thickness is less than required floor thickness along more than half of the distance.

The thickness provided is only 1m throughout the cross sectional length which is not adequate especially along the upstream floor length. This means that the weight of the floor is not adequate to counterbalance the uplift pressure, and it may fail by rupture. Site is not accessible to observe any rupture on the floor because it has already been covered with water flowing downstream.

Piping/undermining occurs below the weir if the water percolating through the foundation has a large seepage force when it emerges at the d/s end of the impervious floor (Garge, 2005).

When the seepage force exceeds a certain value, the soil particles are lifted up at the exit point of the seepage. With the removal of the surface soil particles, there is further concentration of flow in the remaining portion and more soil particles are removed. This process of backward erosion progressively extends towards the upstream side, and a pipe-like hollow formation occurs beneath the floor. The floor ultimately subsides in the hollows so formed and fails. This type of failure is known as piping failure (Garge, 2005)

The existing exit gradient for Fantale diversion weir and under sluice is obtained to be 0.0965 and 0.189 respectively. Therefore the results obtained are considered adequate for the safe performance of the Fantale diversion weir and under sluice against piping because the exit gradients calculated are less than the critical exit gradient (0.835).

The analysis showed that Fantale diversion headwork is protected from both upstream and downstream scouring. This so because the provided piles at the upstream and downstream is compared with the calculated values, the provided depth must be greater than the calculated lacey's scour depth which comes true in case of Fantale diversion headwork.

The bed of the river is fairly hard on the downstream end, there is not likely to be any appreciable scour on the downstream of the under sluice bays and weir portion, no extra protection is therefore, provided on the upstream and downstream side. However, an inverted filter of 0.9m thick in three layers overlain by 1.0m × 1.0m × 1.0m cement concrete blocks has been provided in a length of 4.3m. The space between the cement concrete blocks had been 50mm and filled with coarse gravel.

The gates were not operated during the inspection. However, operation of gates before inspection revealed, gates on the under-sluice (jammed in the fully closed position). Gate number one on the canal head regulator (the gates are numbered one to two, west to east) jammed in the fully closed position and gate number two remains stuck at 1m height. Which intern causes cloggage of the under sluice. This has also obviously caused the increase of the water levels on the upstream side of the head work structures.

It is necessary to keep the stabilizing moment more than the destabilizing moments. In case of Fantale diversion structure this criterion is analyzed and the result implied that the stabilizing moment is greater than destabilizing moment both for highest flood level and existing situation (see appendix 4.a).

Factor of safety against overturning for weir section is 1.635 which is in between the recommended values (i.e. 1.5-2.0). This implies that the structure is safe against overturning.

There is no lifting up of the structures heel and the structure is not susceptible to any tension on the base because the resultant force is within the middle third (see appendix 4).

Fantale diversion structure is checked for stability against shear and sliding and the result manifested that the diversion structure is safe against shear and sliding both for worst condition and existing situation on the site at the time of investigation.

Table 9 shows that the factor of safety against shear and sliding for both the worst case and existing condition is safe because the calculated factor of safety is in recommended range.

5. Conclusion and Recommendation

5.1. Conclusion

Clear opening height reduction or flow area reduction to only 1m at the canal head regulator due to river bed level rise and malfunctioning of gates is the cause for reduction of water supplied to the main canal to 11.5m^3/s.

The floor thickness actually provided both below the weir and under-sluice of the head work is not fully adequate to counter balance the uplift pressure due to seepage below the structures. There is no piping under the foundation of both the under sluice and weir proper.

Entry of silt and debris in to the main canal is prevalent because the capacity of the design flow to transport sediment not determined, gates are not functional both at the head regulator and under sluice to flush out sediment and trash rack not provided at the canal head regulator.

The sediment deposited upstream of the weir proper because there is no complete control on the river discharge with proper regulation area and the deposited sediment not flushed out because the gates are not under operation.

The foundation of the Fantale diversion structures (weir & under-sluice) is safe against piping at the time of investigation, but the uplift pressure below the diversion structures can't be counter balanced by the weight of downstream floor. This means that the provided floor thickness is not adequate& this will lead to failure by rupture of the floor provided.

Malfunctioning of the gates caused sediment deposition in front of the canal head regulator, the sediment deposited clogged the under -sluice this intern allowed entry of silt in to the main canal.

5.2. Recommendation

Complete control on the river discharge is required with proper regulation area in front of the headwork. This can be achieved by providing a barrage than the weir which is actually provided on the site. Therefore this can be considered as one of the reasons for the actually physically observed problem in sediment deposition and formation of shoal in front of the weir proper.

The gates should have to be repaired or changed to mechanical operating system and trash racks should have to be provided for a good performance of the head work.

Operational problems cause inefficient functionality of the project. Proper operational trainings have to be provided for someone from the Water Use association or beneficiaries. The trainer has to show the trainees how & when operation of especially the sluice gates.

The capacity of the design flow to transport different size of sediment is not determined for the case of Fantale diversion structure. Due to its transporting capacity the upstream face of the weir and different gates will be exposed to boulders approaching at higher floods. The impact from the boulders on to the weir and gates at the under sluice and canal head regulator will create severe damage. This will lead to malfunctioning of the gates and the whole diversion structure. So the design should consider transporting capacity of the design flow to prevent the weir and especially sluice gates from damage.

Supervision and evaluation of performance of the head-work on the site should have to be carried out on a certain time interval to preserve the diversion structure from failure due to hydraulic and structural problems. This will serve for proper implementation and functionality of the project.

Performance assessment of diversion head works implemented for irrigation should have to be undertaken on different projects because it can serve for different purposes such as for Construction of new diversion structures, either as a replacement for an existing structure, or as an entirely new structure, rehabilitation of existing structures, from minor repairs to complete re-engineering, either to maintain existing function, or to meet new requirements, decommissioning of a structure. Therefore it is important to assess performance of diversion head works before the diversion structures are endangered by different factors enumerated in this research work as it is said “prevention is better …..”.

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Appendixes

Appendix 1: Onsite Inspection and Field Survey

A detailed checklist has been developed and followed for the project structures to document the observations of each significant structural and hydraulic feature. Particular attention has been given to detect evidence of leakage, erosion, seepage, undue settlement, displacement, tilting, cracking, deterioration, and improper functioning; to verifying the adequacy and quality of maintenance and operating procedures; and to observing significant post construction changes.

The use of photographs for comparison of previous and present conditions, documentation of new or progressive problems, and inspection records has been included as a part of the inspection program.

The inspection includes appropriate features and items, including but not limited to the following:

1. water level rise in the river to an extent required for feeding the off-taking canal;
2. Regulation and control of the river flows.
3.

Maximum flood discharge	
Normal Pond level upstream	
Highest flood level upstream & emergency pond level	
Highest flood level downstream	
Under sluice (Bay)	
Weir	
Width of pier and divide walls	
Total width between flanks	
Crest RL in the under sluice Bay	
Crest RL in the weir	

upstream floor RL in the under sluice	
upstream floor RL in the weir	
downstream floor RL in the under sluice section	
downstream floor RL in the weir section	
Total length of impervious floor	
I) Under sluice Bay	
II) Weir Bays	
Downstream scour depth	

4. Retrogression of levels on the D/S of head work and consequential issues including reduction in discharge capacity
5. Sedimentation problems
6. Problems associated with gates structure, their operation, and hoists
7. Checking whether the canal suffers with excessively high coarse sediment entry at the head regulator and consequent siltation in the head reach reducing the canal capacity.
8. Structural Cracking- Concrete structures should be examined for structural cracking resulting from overstress due to applied loads.
9. Movement-Horizontal and Vertical Alignment-Concrete structures should be examined for evidence of any abnormal settlements, heaving, deflections, or lateral movements.
10. Junctions-The conditions at the junctions of the structure with abutments or embankments should be determined.
11. Water Passages-All water passages and other concrete surfaces subject to running water should be examined for erosion, cavitations, obstructions, leakage, or significant structural cracks.
12. Seepage or Leakage-The faces, abutments, and toes of the concrete structures should be examined for evidence of seepage or abnormal leakage, and records of flow of downstream springs should be reviewed for unusual variation with reservoir pool level. The sources of seepage should be determined, if possible.

13. All monolithic construction joints should be examined to determine the conditions of the joint and filler material, any movement of joints, or any indication of distress or leakage.
14. Foundation-The foundation should be visually examined to the extent possible for damage or possible undermining of the downstream toe.
15. Abutments- The abutments should be examined for signs of instability or excessive weathering.
16. Control Gates and Operating Machinery- the structural members, connections, hoists, cables, and operating machinery and the adequacy of normal and emergency equipment should be examined and tested to determine the structural integrity and verify the operational adequacy of the equipment.
17. Headwater and Tail water Gages-The existing records of the headwater and tail water gage measurements should be examined to determine the relationship between these and other instrumentation measurements such as stream flow, uplift pressures, alignment, and drainage system discharge with the upper- and lower-water surface elevations.
18. The structures and all features should be examined for any conditions that may impose operational constraints on the cooling facilities such as silt or debris accumulation at the water intake or discharge.

Appendix 2: Calculations on withdrawal of water

The existing flow of water in to the main canal is taking place within only one gate opening which is opened about 1m height . Using submerged orifice equation below the discharge in to the main canal can be estimated:

$$Q = C_d \times A \times \sqrt{(2gh)}$$

Where, C_d -Discharge coefficient, 0.64

A - Opening area

g – Gravity, 9.81 m/s^2

h – Upstream water level minus downstream water level

1178.08-1175.45= 2.63

Q= 0.64*2.5*1*$\sqrt{2 \times 9.81 \times 2.63}$

Q=11.5 m^3/s

To maintain the 18 m^3/s the discharge required by the command area the two gates should have to be opened by an amount:

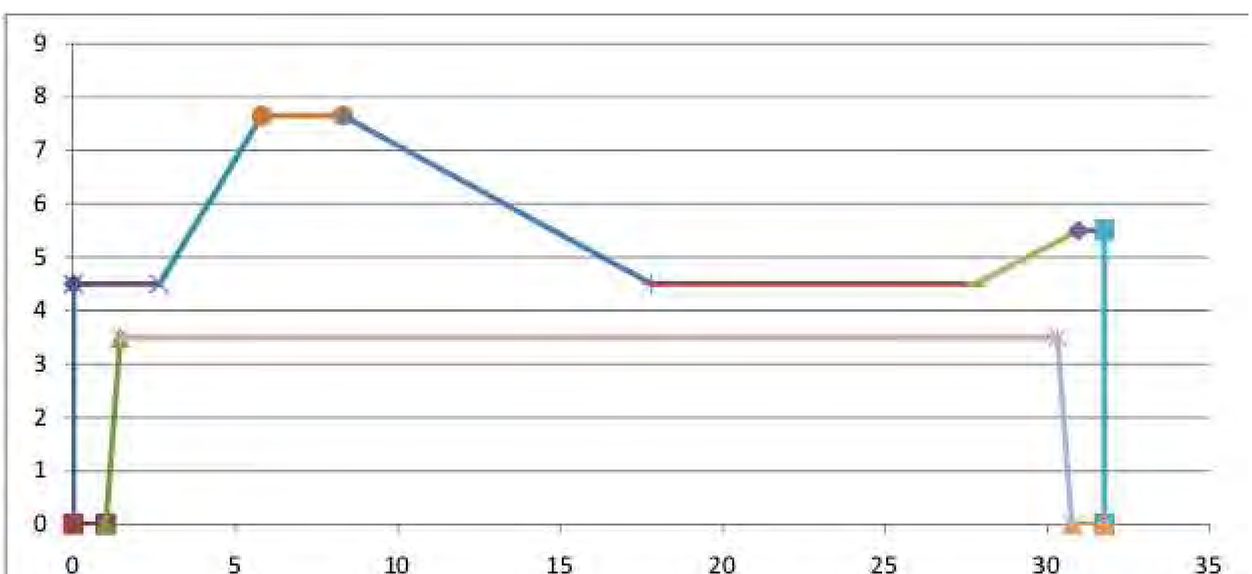
$$18 = 0.64 \times (X \times 5) \times \sqrt{2 \times 9.81 \times 2.63}$$

X=0.78m

One gate should have to be opened by an amount:

$$18=0.64*(X*2.5)*\sqrt{2 \times 9.81 \times 2.63}$$

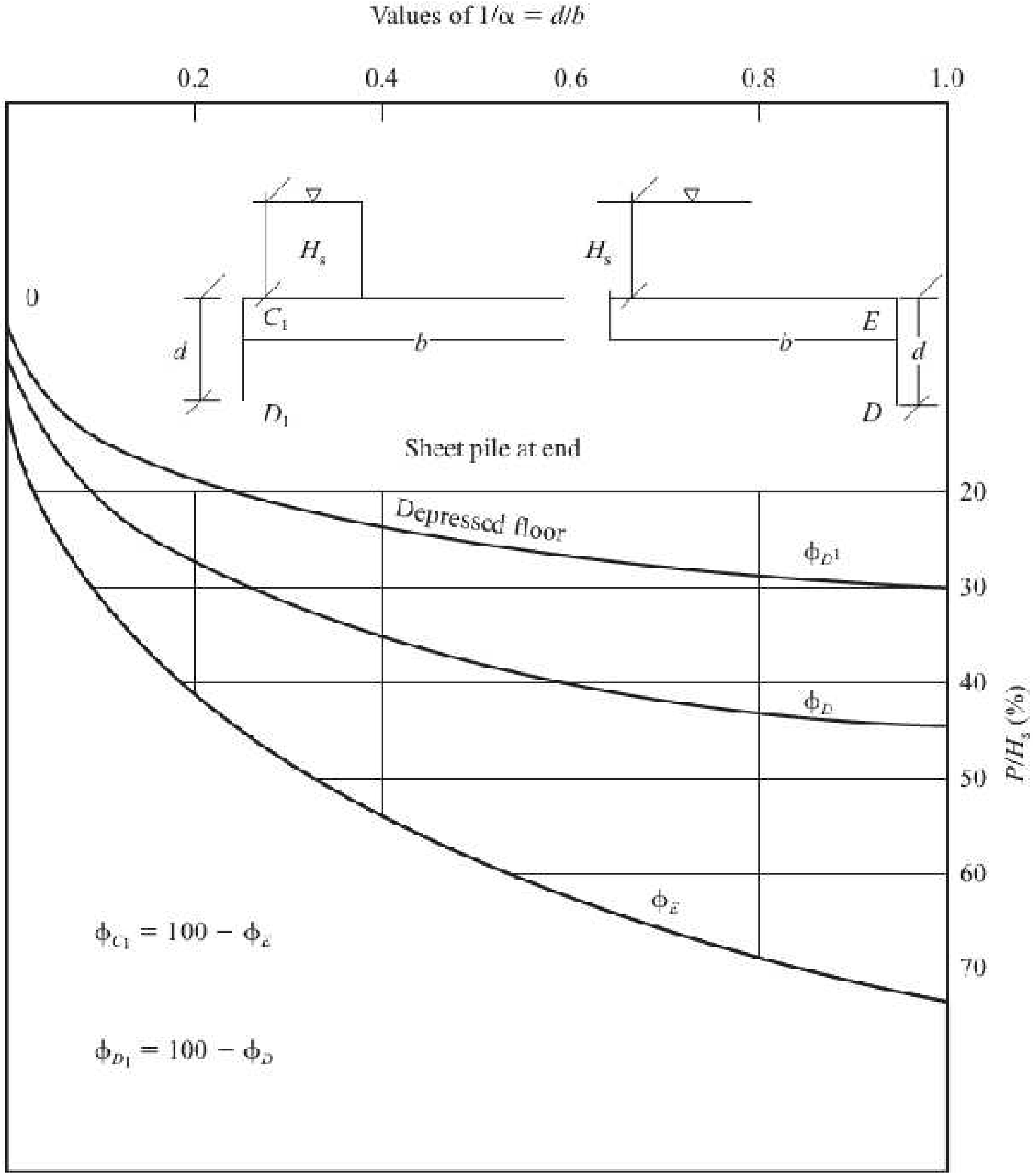
18=0.64*(X*2.5)*7.2 $\quad X = \frac{18}{11.5}$, X=1.565m



9
8
7
6
5
4
3
2
1
0
0
5
10
15
20
25
30
35

simple standard profiles have been presented in form of equations given in figure below and curves given below, which can be used for determining the percentage pressures at various key points(Garge,2005).

The simple profiles which are most useful for Fantale are: A straight horizontal floor of negligible thickness with a sheet pile on the up the upstream end and downstream end.



Values of $1/\alpha = d/b$
0.2
0.4
0.6
0.8
1.0
0
H_s
H_s
C_1
E
b
b
d
d
D_1
D
Sheet pile at end
Depressed floor
ϕ_{D^1}
ϕ_D
ϕ_E
20
30
40
50
60
70
P/H_s (%)
$\phi_{C_1} = 100 - \phi_E$
$\phi_{D_1} = 100 - \phi_D$

The values obtained from the simple profiles need to be corrected for mutual interference of piles and for thickness of the floor.

Using this technique the corrected pressures at various key points of Fantale weir structure are tabulated below.

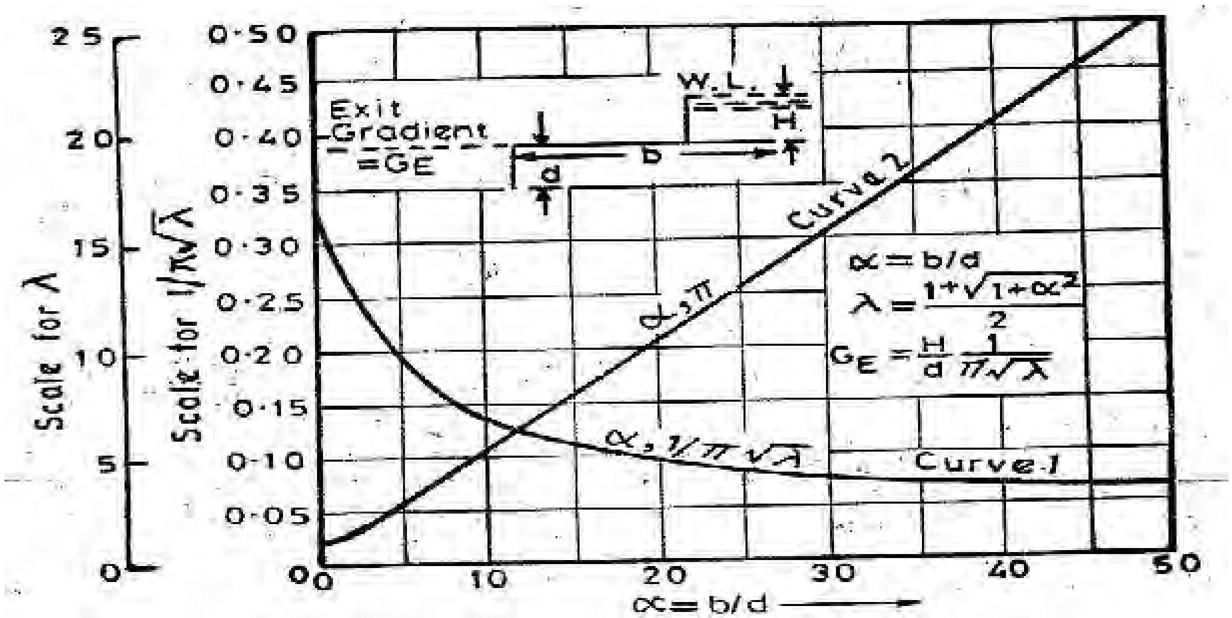
Upstream pile line	Downstream pile line
$\square_{E1}$= *100%*	$\square_{E2}$=32.45%
$\square_{D1}$=77.13%	$\square_{D2}$=25.02%
$\square_{C1}$=71.66%	$\square_{C2}$=0%

The exit gradient is also computed using

$$G = \frac{H}{d} \square \frac{1}{\pi\sqrt{\lambda}}$$

For a value of $\alpha = \frac{31.6}{5.5}$ = 5.74, $\frac{1}{\pi\sqrt{\lambda}}$ from curves of plate below is equal to 0.172

Exit gradient (GE) is 0.0965 this means that 1 in 10.4 which is less than the recommended value 1 in 7. Therefore no piping occurred under the foundation of the weir.



Upstream pile line

ϕ_{E1} = *100%*

ϕ_{D1}=78.8%

ϕ_{C1}=73.76%

Downstream pile line

ϕ_{E2}=29.148%

ϕ_{D2}=22.9%

ϕ_{C2}=0%

$$G \quad \frac{H}{d} \quad \pi \ \lambda$$

$$\alpha \quad \frac{31.6}{4.5} \qquad \frac{1}{\pi \ \lambda}$$

Exit gradient (GE) is 0.189 this means that 1 in 5.3 which is less than the recommended value 1 in 5. Therefore no piping occurred under the foundation of the under sluice.

Appendix 4: Summary of the calculations implemented for structural stability analysis

a) For the worst loading condition

		Magnitude of force in KN			
Name of force	Designation if any	Vertical forces Downward=+ve Upward= -ve	Horintal forces Towards upstream=+ve d/s= -ve	Lever arm in m	Moments about toe in KN Anti clock wise(+ve) Clockwise(-ve)
Weight of weir itself	W_1	(+) $\frac{1}{2}$ 3.15 □ 3.14 □ 24 = 118.7		13.05	+1549.035
	W_2	(+)2.5*3.14*1*24= 188.4		10.75	+2025.3
	W_3	(+)$\frac{1}{2}$ □ 9.5 □ 3.14 □ 1 □ 24 = 357.96		6.33	+2265.8868
		$\sum V_1$= 665.06			

					$\sum M_1$= +5840
Weight of water supported on U/s slope	w_w	(+)$\frac{1}{2}$ 3.14 3.15 10 = 49.5 $\sum V_2$= 49.5		14.1	697.95 $\sum M_2$= 697.95
Uplift forces	U_1 U_2	(-)15.15*1.54*10=233 (-)$\frac{1}{2}$ 2.21 15.15 10 = 167.4075 $\sum V_3$= -400.7175		7.575 10.1	(-)767.32325 (-)116.34328 $\sum M_3$= -3458.1
Horizontal hydrostatic pressure	P_1 P_2		-$\frac{1}{2}$*3.14*3.14*10 =-49.298 -2.36*10*3.14*1 =-74.104	1.05 1.57	-51.7629 -116.34328

			$\sum H_1$=-123.402		$\sum M_4$=-168.10
Upward vertical earth quake forces 0.05w		-0.05*665.06 $\sum V_4$= -33.253			-0.05*5840.22 $\sum M_5$= -292.01
Horizontal inertia forces due to earthquak			-0.1*W_1=-11.87 -0.1*W_2=-18.84 -0.1*W_3= -35.796 $\sum H_4$=-66.506	1.05 1.57 1.05	-12.4635 -29.5788 -37.5858 $\sum M_6$=-79.628

b) For the existing condition at the time of investigation

		Magnitude of force in KN			
Name of force	Designation if any	Vertical forces Downward=+ve Upward= -ve	Horizontal forces Towards upstream=+ve d/s= -ve	Lever arm in m	Moments about toe in KN Anti clock wise(+ve) Clockwise(-ve)

Weight of weir itself	W_1 W_2 W_3	(+) $\frac{1}{2}$ 3.15 3.14 24 = 118.7 (+)2.5*3.14*1*24=188.4 (+)$\frac{1}{2}$ 9.5 3.14 1 24 = 357.96 $\sum V_1$= 665.06		13.05 10.75 6.33	+1549.035 +2025.3 +2265.8868 $\sum M_1$= +5840
Weight of water supported on U/s slope	w_w	(+)$\frac{1}{2}$ 3.14 3.15 10 = 49.5 $\sum V_2$= 49.5		14.1	697.95 $\sum M_2$= 697.95
Uplift forces	U_1 U_2	-15.15*1.54*10=233 (-)$\frac{1}{2}$ 2.21 15.15 10 =		7.575 10.1	(-)767.32325 (-)116.34328

		167.4075 $\sum V_3$= -400.7175			$\sum M_3$= -3458.1
Horizontal hydrostatic pressure	P_1 P_2		-49.298 -29.516 $\sum H_1$=-78.814	1.05 1.57	$\sum M_4$=-98.1030