VISVESVARAYA TECHNOLOGICAL UNIVERSITY
BELAGAVI - 590018

A

REPORT ON

EXTENSIVE SURVEY PROJECT (21CVMP67)

For the partial fulfilment of requirement of degree of

BACHELOR OF ENGINEERING IN
CIVIL ENGINEERING

SUBMITTED BY

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DEPARTMENT OF CIVIL ENGINEERING

SRI VENKATESHWARA COLLEGE OF ENGINEERING
Vidyanagar, Bengaluru -562157

2023 – 2024

SRI VENKATESHWARA COLLEGE OF ENGINEERING
Vidyanagar, Bengaluru -562157

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DEPARTMENT OF CIVIL ENGINEERING

CERTIFICATE

This is to certify that, the project report entitled **'EXTENSIVE SURVEY PROJECT'** is being submitted by:

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Of 6th Semester B.E (Civil) in partial fulfilment of the requirements for the award of DEGREE in BACHELOR OF ENGINEERING under the discipline of the **Visvesvaraya Technological University, Belgaum** during the academic year 2023 - 2024. It is certified that all corrections/suggestions indicated for internal assessment have been incorporated in the report deposited in the department library. The project has been approved as it satisfies the academic requirement in the report of the project work in 6th semester prescribed for Bachelor Degree.

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05 Aug 2024

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Sir Mokshagundam Visvesvaraya (1861 – 1962)

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We would also like to express our special thanks to **Mrs. LAVANYA S, Assistant professors**, camp officers for having enlighten the extensive survey camp in successful direction.

We would also like to thank all other faculty members and non - teaching staffs for their support and encouragement.

ABOUT GADENAHALLI VILLAGE, BANGLORE NORTH

Gadenahalli village is located in Bangalore North Tehsil of Bangalore district in Karnataka, India.

According to Census 2011 information the location code or village code of Gadenahalli village is 612886. Gadenahalli village is located in Bangalore North Tehsil of Bangalore district in Karnataka, India. It is situated 12km away from sub-district headquarter Bangalore North and 12km away from district headquarter Bangalore. As per 2009 stats, Bettahalasooru is the gram panchayat of Gadenahalli village.

The total geographical area of village is 141.43 hectares. Gadenahalli has a total population of 604 peoples. There are about 122 houses in Gadenahalli village. As per 2019 stats, Gadenahalli villages comes under Byatarayanapura assembly & Bengaluru North parliamentary constituency. Bbmp is nearest town to Gadenahalli which is approximately 12km away.

Gadenahalli - Village Overview	
Gram Panchayat :	Bettahalasooru
Block / Tehsil :	Bangalore North
District :	Bangalore
State :	Karnataka
Pincode :	562157
Area :	141.43 hectares
Population :	604
Households :	122
Assembly Constituency :	Byatarayanapura
Parliament Constituency :	Bengaluru North
Nearest Town :	Bbmp (12 km)

MAP OF GADENAHALLI VILLAGE

ABOUT BETTAHALASUR (13.1538583, 77.56977666) BEHIND MVIT COLLEGE

13.1538583, 77.56977666 is located in Bangalore North Tehsil of Bangalore district in Karnataka, India. It is situated 22km away from sub-district headquarter.

According to Census 2011 information the location code of bettahalsur village (**13.1538583, 77.56977666)** is **612895**. Bettahalsur village is located in Bangalore North Tehsil of Bangalore district in Karnataka, India. It is situated 10km away from sub-district headquarter Bangalore North and 10km away from district headquarter Bangalore. As per 2009 stats, BETTAHALSOOR is the gram panchayat of 13.1538583, 77.56977666.

The total geographical area of village is 598.25 hectares. Bettahalasur has a total population of 3,573 peoples, out of which male population is 1,817 while female population is 1,756. Literacy rate of bettahalasur village is 73.05% out of which 78.43% males and 67.48% females are literate. There are about 900 houses in bettahalasur village. Pincode of bettahalasur village locality is 562157.

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CHAPTER-1

NEW TANK PROJECT

1.1 Introduction

Water is the greatest resource of humanity It not only helps in survival but also helps in making life comfortable and luxurious. Besides various other uses of water, the largest use of water in the world is made for irrigating lands. Irrigation may be defined as the science of artificial application of water to the land, in accordance with the 'crop requirements' throughout the 'crop period' for full-fledged nourishment of the crops.

1.1.1 Necessity of Irrigation in India

India is a tropical country with a vast diversity of climate, topography and vegetation. Rainfall in India varies considerably in its place of occurrence, as well as in its amount. Even at a particular place, the rainfall is highly erratic and irregular, as it occurs only during a few particular months of the year. Crops cannot, therefore, be raised successfully, over the entire land, without providing artificial irrigation of yields.

1.1.2 Earthen Dam

Earthen dams are used for the storage of water for irrigation, daily uses such as drinking and other domestic purpose. These dams have been built since earliest times. These dams are however for limited heights. For the design of earthen dams, the foundation conditions & materials of construction are thoroughly investigated. A purely homogeneous type earth dam is composed of single kind of material. In these dams, internal drainages system is provided in the form of horizontal filter drain. Existing bund is in inadequate to store water for irrigation & water supply nearby villages. Hence it is planned to purpose new bund to fulfil the present & future requirement of water. The site chosen for the new bund has greater capacity of storing water compared to the existing bund.

1.1.3 Types of Earthen dams

Earthen dams can be of the following three types,

- Homogeneous Embankment types
- Zoned Embankment types
- Diaphragm types

Homogenous Embankment Type

The simplest type of an earthen embankment consists of a single material and is homogenous throughout. Sometimes a blanket of relatively impervious material may be placed on the upstream face. A purely homogenous section is used, when only one type of material is economically or locally available. Such a section is used for low to moderately high dams and for levees. Large dams are seldom designed as homogenous embankments. A purely homogenous section possesses the problems of seepage, and huge sections are required to make it safe against piping, stability, etc. due to this, a homogenous section is generally added with an internal drainage system.

Zoned Embankment Type

Zoned Embankments are usually provided with a central impervious core, covered by a comparatively pervious transition zone, which is finally surrounded by a much more pervious outer zone. The central core checks the seepage. The transition zone prevents piping through cracks which may develop in the core. The outer zone gives stability to the central impervious fill and also distributes the load over a large area of foundation.

Diaphragm Type Embankments

Diaphragm type embankments have a thin impervious core which is surrounded by earth or rock fill. The impervious core, called diaphragm is made of impervious soil, concrete, steel, timber or any other material. It acts as a water barrier to prevent seepage through the dam. The diaphragm is placed either at the center as a central vertical core or at the upstream face as a blanket. The diaphragm must also be tied to the bed rock or to a very impervious foundation material. The thickness equals or exceeds these limits, it is considered to be of Zoned Embankment type.

1.2 Selecting a Suitable Preliminary Requirements for an Earthen Dam

The preliminary design of an earth dam is done on the basis of existing dams of similar characteristics and the design is finalized by checking the adequacy of the selected section for the worst loading condition.

- Freeboard
- Top width
- Upstream and Downstream slopes
- Line of seepage or Phreatic line in earth dams

1.2.1 Freeboard

Freeboard or minimum freeboard is the vertical distance between the maximum reservoir level and the top of the dam (i.e. the crown or crest of the dam). The vertical distance between normal pool level or spillway crest in the top of the dam is termed as normal freeboard. The freeboard must be sufficient enough, as to avoid any such possibility of overtopping. Values of freeboard, for various heights recommended by U.S.R.B. are given in the table 1.1.

Table 1.1: U.S.R.B Recommendation for freeboard for Earth dams

Spillway type	Height of dam	Minimum freeboard over MWL
Uncontrolled (i.e. free) spillway	Any height	Between 2m to 3m
Controlled spillway	Height less than 60m	2.5m above top of gates
Controlled spillway	Height more than 60m	3m above top of gates

1.2.2 Upstream and Downstream slopes

The side slopes depend upon various factors such as the type and nature of the dam and foundation materials, height of the dam, etc. The recommended values of side slopes are tabulated in table 1.2.

Table 1.2: Preliminary Dimensions for low Earth Dams

Height of dam in meters	Maximum freeboard in meters	Top width(a) in meters	U/S slope (H:V)	D/S slope (H:V)
Up to 4.5	1.2 to 1.5	1.85	2:1	1.5:1
4.5 to7.5	1.5 to 1.8	1.85	2.5:1	1.75:1
7.5 to 15	1.85	2.50	3:1	2:1
15 to 22.5	2:1	3.00	3:1	2:1

1.3 Salient Features of Bund

The purposed project includes the selection of reservoir site, bund, weir, tank and canal alignment work. The area surroundings the best suited for the irrigation, also being main occupation of the locality. Reservoir provided is more than sufficient to provided supply water to the fields during times of requirement.

It is the type of storage reservoir, which is primarily used for water supply and irrigation. It is constructed to store the excess water during the period of large supply and release it gradually as a when needed. The engineering geological and hydrological investigation show that the catchment area has less or minimum percolation losses.

Catchment area has adequate capacity without submerging excessive land & other properties. The water stored in suitable for the project undertaken. The soil & rock mass at the reservoir site do not contain any objectionable minerals and salts.

1.4 Types of storages in reservoir

1. Useful storage

The quality of water available between FRL and DSL is known as use full storage. It is the actual quality of water which can be drawn from reservoir for the purpose for which water is stored.

2. Dead storage

The quality of water available below DSL is known as dead storage. It is provided in reservoirs to accommodate sediments.

3. Surcharge storage

Excess of water available above FRL up to the gate is known as surcharge storage.

1.4.1 Types of Levels

1. Maximum water level

Maximum water level is the one at which water level is measured during highest floods.

2. Full reservoir level

Full reservoir level refers to the max water level in reservoir up to crest of the spillway in normal operating condition of the reservoir.

3. Dead storage level

Dead storage level refers to the minimum water level to be maintained in the reservoir in normal operating condition of the reservoir.

1.4.2 Gross Command Area (GCA)

GCA is the total area bounded within the irrigation boundary of the project, which can be economically irrigated without considering the limitation of the quantity of available water. It includes the cultivable as well as uncultivable area.

Ex: - ponds residential areas, reserved forest etc.

1.4.3 Cultivable Command Area (CCA)

CCA is the cultivable part of the gross command areas, and includes all the land of gross command area (GCA) on which cultivation is possible. It includes fallow land, which can be made cultivable. It doesn’t include non-cultivable land like ponds, populated areas, roads, reserved forests, etc.

1.5 Surplus weir

Weir is an overflow section from which water is discharged. Protective works in the form of aprons, etc. are required to keep the bed from erosion. Where the foundation is hard rock no protective works are necessary. Diversion weirs are usually 3 to 10metres high and their primary function is to raise the river level for diverting he water into the canal. A weir is generally placed at right angles to the direction of flow of the river. The required height of weir must be determined from the consideration of the stream flow during low flow period.

1.5.1 Conditions for Stability of weir

- There must be tension in the masonry or in the contact plane between the weir and the foundation and there must be no overturning.
- There must be no tendency to slide on the joint with the foundation or any horizontal plane above the base.

- The maximum toe and heel pressures on foundation should not exceed the prescribed safe values.

1.6 Stability Analysis

Resisting moment (M_r), overturning moment (M_o), horizontal forces ($\sum H$), vertical force ($\sum V$) are calculated using pressure diagram.

Factor of safety against overturning is calculated by

$FOS = M_r/M_o$

Factor of safety against sliding is calculated by

$FOS = (\mu+\sum V)/\sum H$

1.7 Stability of slopes

1.7.1 Types of slopes: -

- **Infinite slope: -** They have dimensions that extend over grate distances and the soil mass is inclined to the horizontal.
- **Finite slope: -** A finite slope is one with a base and top surface, the height being limited. The inclined faces of earth dams, embankments and excavation and the like are all finite slopes.

1.7.2 Importance of Slope Stability Analysis

When the ground surface is not horizontal a component of gravity will try to move the sloping soil mass downwards. Failure of natural slopes (landslides) and man-made slopes has resulted in much death and destruction. Some failures are sudden and catastrophic; others are widespread; some are localized. Civil engineers are expected to check the safety of natural and slopes of excavation. Slope stability analysis consists of determining and comparing the shear stress developed along the potential rupture surface with the shear strength of the soil.

1.7.3 Causes of slope failure

1. **Erosion: -** The wind and flowing water causes erosion of top surface of slope and makes the slope steep and thereby increase the tangential component of driving force.
2. **Steady seepage: -** Seepage forces in the sloping direction add to gravity forces and make the slope susceptible to instability. The pore water pressure decreases the shear strength. This condition is critical for the downstream slope.
3. **Sudden drawdown: -** In this case there is reversal in the direction flow and results in instability of side slope. Due to sudden drawdown the shear stresses are more due to saturated unit weight while the shearing resistance decreases due to pore water pressure that does not dissipate quickly.
4. **Rainfall: -** Long period of rainfall saturate, soften, and erode soils. Water enters in to existing cracks and may weaken underlying soil layers, leading to failure, for example, mud slides.
5. **Earthquakes: -** They induce dynamic shear forces. In addition, there is sudden build-up of pore water pressure that reduces available shear strength.
6. **External loading: -** Addition loads placed on top of the slope increases the gravitational forces that may cause the slope to fail.
7. **Construction activities at the toe of the slope: -** Excavation at the bottom of the sloping surface will make the slopes steep and thereby increase the gravitational forces which may result in slope failure.

1.7.4 Types of failure

Broadly slope failure are classified into 3 types as

1. **Face (slope) failure:** This type of failure occurs when the slope angle (ß) is large and when the soil at the toe portion is strong. as shown in fig1.1 (a)

2. **Toe failure:** In this case the failure surface passes through the toe. This occurs when the slope is steep and homogeneous. as shown in fig1.1(b).
3. **Base failure:** In this case the failure surface passes below the toe. This generally occurs when the soil below the toe is relatively weak and soft. as shown in fig1.1(c).

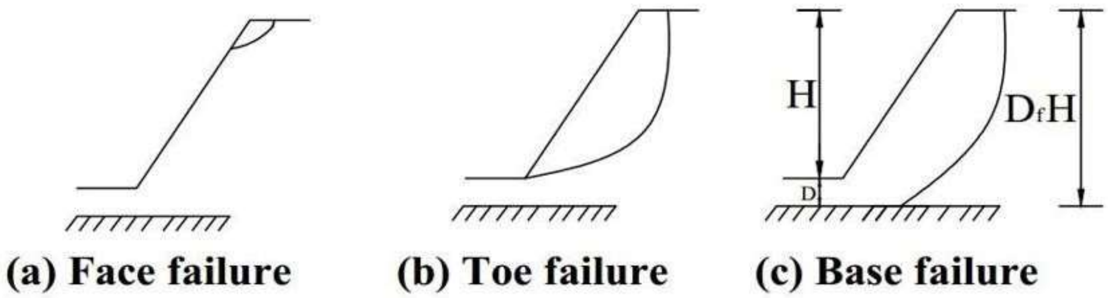

Fig 1.1: Types of failures

1.7.5 Methods of Stability Analysis

1. Total stress analysis for purely cohesive soil.
2. Total stress analysis for cohesive- friction (C-Φ) soil – (Swedish method of slices or method of slices)
3. Effective stress analysis for conditions of steady seepage, rapid drawdown and immediately after construction.
4. Friction circle method.
5. Taylor's method.

1.8 Steps Involved in New Tank Project

As the existing bund is in inadequate to store water for irrigation & water supply to the nearby villages. Hence it is planned to purpose new bund to fulfill the present & future requirement of water.

- Site selection for an irrigation tank.
- Reconnaissance surveying.
- Preliminary investigation.
- Centre line alignment of bund.
- Capacity contouring.
- Block levelling for waste weir and sluice.
- Irrigation canal alignment

1.9 Factors considered for site selection

For any site the topographical and the geographical features play an important role, therefore the following features are to be considered before selection of site.

- The water storage capacity must be large for minimum height of bund.
- The site must be located in a narrow valley.
- Good impervious strata must be available for moderate depth.
- The materials for construction should be available locally.
- The length of the bund must be minimum as far as possible.

Based on the above factors site is selected and further project are done

- Reconnaissance survey.
- preliminary investigation. Detailed survey.
- Geological investigation.
- Hydrological investigations.

1.9.1 Reconnaissance Survey

Geological Survey of India (GSI) is an official agency to survey the entire country. GSI has prepared the topo-sheets by dividing the entire land into number of square segments. As the proposed new tank is of smaller capacity, topo-sheets are not being used, instead a reconnaissance survey has been conducted to locate the site for the tank and then to fix up the bund-line.

1.9.2 Preliminary Investigation

- Rough levelling work to determine the topography of the site.
- To study the nature of ground profile, height of the bund, length, storage capacity, area covered under u/s side of the bund, nature of soil (to find out stability for earthquakes etc.)
- Feasibility for constructing hydraulic structures like canals, waste weir, sluices etc.
- To determine the length of bund in order to align the centreline of the bund.
- To find out the availability of materials, labours required for constructing the bund.
- To roughly determine the runoff of the catchment area covered under u/s, and also to design the structures, which help in effective usage of stored water.

1.9.3 Geological Investigations

The geological investigations are carried out to have detailed information of the following.

- Water tightness of reservoir and ground water conditions in the region.
- Suitability of foundations for the dam.
- Geological & structural features such as folds, faults, joints of the basin.
- Type and depth of superficial deposits.
- Location of permeable and soluble rock materials.
- Location of quarries for construction of the dam.

The geology of the catchment must be studied from point of secure foundation and also since it affects the runoff percolation. The special requirement for the geology of the reservoir is that there should be no danger of serious leakage when the ground is under pressure from full head of water of the reservoir.

1.9.4 Hydrological Investigations

It is an important aspect of the reservoir planning. The capacity of irrigation canal will depend on the available supplies from the reservoir.

The following studies are conducted

- Collection and process of rainfall data of catchment.
- Study of runoff pattern at the tanksite.
- Determination of storage capacity to a given demand.
- Study of flood hydrograph to obtain the worst flood condition

1.10 Project Site Details

Name of the project: - construction of storage reservoir and canal work at THEERTHARAMESHWARA

Village: - THEERTHARAMESHWARA

Catchment area: -1.0km2

Average rainfall in the area: - 1683mm

1.11 Topography

Topography of the project area is such that the reservoir site is at highest level & area that has to be irrigated has situated at lower level. About 10000 & 12000 people inhabiting in the village are coming under the command area.

1.12 Climate

The project area receives rainfall during the period from June to October. Rainfall data of fairly long term for the catchment area is available. The average rainfall for 20 years is 1683mm. the maximum temperature of the area is about 370.

1.13 Survey Work

The Survey Work was conducted for knowing the reservoir capacity and to design the Earth Bund. Rainfall data for the past 30 years is collected for the survey conducted area. Runoff is calculated from the strange table taking 60% rainfall data dependently. Detailed Survey Includes

- **Alignment of centreline of the bund**
  It is established by use of theodolite
- **Taking L/S and C/S of the bund along centreline**
  It is done by using dumpy level and levelling staff at 10m intervals with 30m width on either side of the bund
- **Taking block levels at sill level and waste weir in site**
  Block levelling is done for 20m*20m @ sill level. Block levelling is done for 60m*40m @ waste weir
- **Tracing water spread contours**
  In the case of capacity contours, the bottom level is determined and a suitable contour interval is decided, the contours are than drawn using AUTOCADD. The area of each contour is determined by digitalizing the contour in AUTOCADD further from which total area of the reservoir is determined.
- **Canal alignment at sill level**

1.14 Hydraulic structures

The hydraulic structures which are supposed to be constructed in the site in order to store the water and to fulfil demands of the people for various purposes. The hydraulic structures are:

- Earthen Dam.
- Surplus weir.
- Canal drop.
- Irrigation canal.

1.14.1 Earthen Dam: The earthen dam structure is shown in fig 1.2.

Fig 1.2: Earthen Dam

1.14.2 Design Criteria for Earth Dams

- A fill of sufficiently low permeability should be developed out of the available materials, so as to best serve the intended purpose with minimum cost.
- Sufficient spillway and outlets capacities should be provided so as to avoid the possibility of overtopping during design flood.
- Sufficient freeboard must be provided for wind setup, wave action, frost action and earthquake motions.
- The seepage line (i.e. phreatic line) should remain well within the downstream face of the dam, so that no sloughing of the face occurs.
- There is a little harm in seepage through a flood control dam. If the stability of foundations and embankments is not impaired, by piping, sloughing, etc. but conservation dam must be as watertight as possible.
- There should be no possibility of free flow of water from the upstream to the downstream face.
- The upstream face should be properly protected against wave action, and the downstream face against rains and waves up to tail water. Provisions of horizontal berms at suitable intervals in the downstream face maybe thought of, so as to reduce the erosion due to flow of rain water.
- The portion of the dam, downstream of the impervious core, should be properly drained by providing suitable horizontal filter drain or toe drain, or chimney drain.
- The upstream and downstream slopes should be so designed as to stable under worst condition of loading. These critical conditions occur for the upstream slope during sudden drawdown of the reservoir and for the downstream slope during steady seepage under full reservoir.
- The upstream and downstream slope should be flat enough, as to provide sufficient base width at the foundation level, such that the maximum shear stress developed remain well below the corresponding maximum shear strength of soil so as to provide a suitable factor of safety.
- Since the stability of the embankment and foundation is very critical during construction or even after construction, due to the development of excessive pore pressure and consequent reduction in shear strength of soil, the embankment slope must remain safe under this critical condition also.

1.15 Design calculation

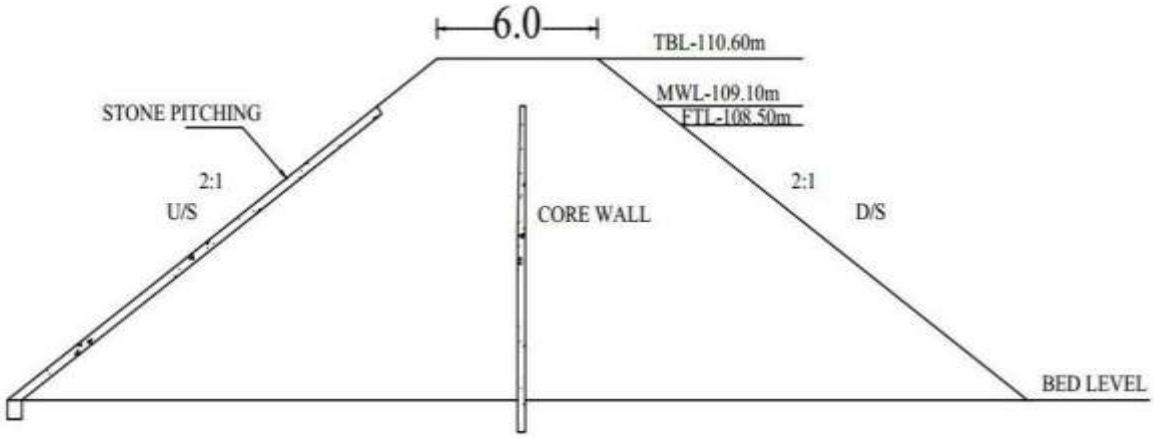

Fig 1.3: Typical cross section of bund

1. **Rain fall data**

   Annual average rainfall obtained from IMD is 1683mm or 168.3cm.

2. Run off

from stranger's run off percentage table, run off percentage for catchment designed as good for monsoon rainfall of 168.3cm is 60 %. Run off = 168.3x (60/100) = 100.98cm.

3. Computation of catchment area and volume

The catchment area of proposed new tank is determined from the top sheets and is found to be 15Km^3. From the rainfall records, the average rainfall is 168.3cm. Average rainfall of that year is taken as (2/3 or ¾) of mean annual rainfall

Catchment area = 1.0 km^2. (Obtained from topo sheet)
Annual run off = 0.23m.
Inflow volume = 1x10^{6}x0.23
Inflow= 230000m^3
Assuming 80 % of inflow volume to be stored.
Volume stored = (80/100) x230000=184000m^3

4. Contour details

Table 1.3: Contour details

SL NO	CONTOUR	CUMMULATIVE AREA(m^2)	CUMMULATIVE VOLUME(m^3)
1	100.500	480.00	180.00
2	104.000	8910.00	27578.25
3	105.000	3510.00	30447.5
4	106.000	11930.00	42374.75
5	107.000	10190.00	52564.75
6	108.500	39160.00	91725.00

From contour details
Reduced level corresponding to inflow volume

Volume (m^3)	Reduced level(m)
273109.71	+97.000m
166342.0	+95.500m

1. 1 RL of Full tank level (FTL) = +108.5m.
2. 2. RL of Maximum water level (MWL) = 108.5+0.60 = +109.1m
3. RL of Top bund level (TBL) = 109.1+1.500 = +110.6m.

Assuming 10% 0f the total volume stored as dead storage.
Dead storage = 9172.5m3
RL for Dead storage (DSL) can be get by interpolation

Volume (m^3)	Reduced level (m)
180.00	+100.50
27398.25	+100.40

By interpolation, for 9172.5m3 RL = +101.65m.

1. Height of dead storage level = 1.68 m.
2. Height of full storage level = 10.630m

1.16 Earth work calculation

Data obtained from Field survey

Top level of dam =108.500m
Width of the top of dam = 6.0m
Upstream slope = 2:1
Downstream slope = 2:1

Table 1.4: Estimation of Earth Work of Earthen Bund

SL.NO	RL OF GL (m)	TBL (m)	HT. OF THE BUND	AREA (m^2)	INTERVAL (m)	VOLUME (m^3)
1	110.6	110.6	0	0	10	0
2	108.995	110.6	1.605	10.918	10	17.523
3	106.925	110.6	3.675	28.800	10	105.84
4	105.365	110.6	5.235	45.112	10	236.16
5	104.355	110.6	6.245	56.970	10	355.77
6	103.455	110.6	7.145	68.395	10	488.68
7	102.580	110.6	8.02	80.280	10	643.845
8	102.660	110.6	7.94	79.160	10	628.530
9	102.495	110.6	8.105	81.475	10	660.350
10	102.050	110.6	8.550	87.850	10	751.117
11	101.585	110.6	9.015	94.725	10	853.945
12	100.845	110.6	9.755	106.110	10	1035.103
13	100.690	110.6	9.910	108.564	10	1075.869
14	100.070	110.6	10.53	118.620	10	1249.06
15	99.970	110.6	10.63	120.278	10	1278.55
16	99.990	110.6	10.61	119.946	10	1272.627
17	99.995	110.6	10.605	119.863	10	1271.147
18	103.855	110.6	6.745	63.217	10	426.398
19	106.655	110.6	3.945	31.45	10	124.070
20	108.255	110.6	2.345	16.819	10	39.440
21	110.600	110.6	0.00	0	10	0
				∑A=1438.61	**TOTAL VOLUME**	12514.024

EARTH WORK REQUIREMENT=Total +16% compaction. = 12514.184m^3

1.17 Stability analysis

By using Swedish slip circle method for a cohesive – friction (c-Φ) soil,

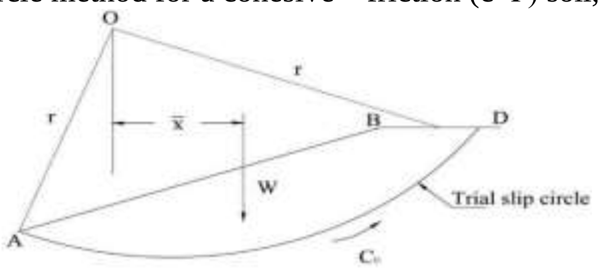

Fig 1.4: Φ_u= 0 Analysis

Fig 1.4 shows a slope AB, the stability of which is to be determined. The method consists in assuming a number of trial slip circles, and finding the factor of safety of each. The circle corresponding to the minimum factor of safety is the critical slip circle. let AD be a trial slip circle, with r as the radius and O as the centre of rotation. Let W be the weight of the soil of the wedge ABDA of unit thickness, acting through its centroid. The driving moment MD will be equal to WX where X is the distance of line of action of W from the vertical line passing through the center of rotation.

The distance r of the centroid of the wedge, from centre of rotation O, can be determined by dividing wedge into a number of vertical slice and dividing the algebraic sum of moment of weight of each slice by the weight of wedge.

C-Φ Analysis:

In order to test the stability of the slope of a c-Φsoil, trial slip circle is drawn, and the material above the assumed slip surface is divided into a convenient no of vertical strips of slice.

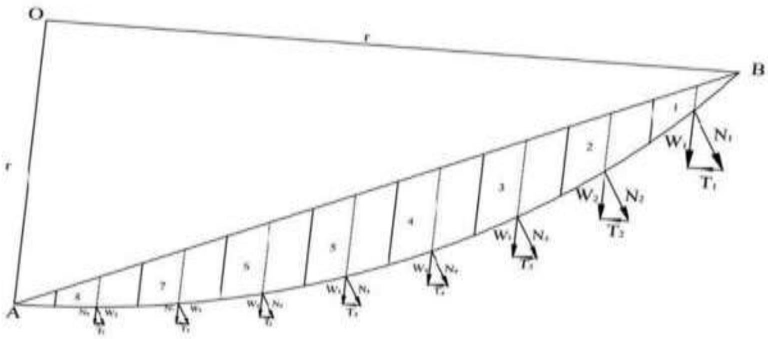

Fig1.5: Stability analysis

The forces between slices are neglected, and each slice is assumed to act independently as a column of soil of unit thickness and of width b. the weight W of each slice is assumed to act at its centre. If this weight of each slice is resolved into normal (N) and tangential (T) components, the normal components will pass through the centre of rotation (O), and hence do not cause any driving moment on the slice.

The procedure is follows

1. Draw the slope to scale
2. A trial slip circle AB with radius ‘r’ is drawn from the centre the rotation ‘o’.
3. Divide the soil mass above the slip surface into convenient number of slice (more 5 is preferred).
4. Determine the area of each slice A_1, A_2, A_3……………………A_n
   Where, A = width of the slice *mid height
   A= b x z
5. Determine the total weight W including external load if any as,
   W = γ b z =γ A
   Where, γ = unit weight
   b = width of slice
   z = height of slice
   the reactions R1 and R2 on the sides of the slices are assumed equal and therefore do not have any effect on stability.
6. Normal component, N = W COS β
   Tangential component, T = W SIN β
7. The values of N and T are tabulated and assumed up as shown in the table 1.5
8. The factor of safety is calculated as follows.
   Sliding moment = r ∑T
   Restoring moment = r (c r θ+∑N tan Φ)

Factor of safety, F S = (c r θ+∑N tan Φ)/∑T

To conducting the laboratory experiment for direct shear test, to find out the C-ϕ values, C =7.8kg/cm2 and Φ = 28^{o}

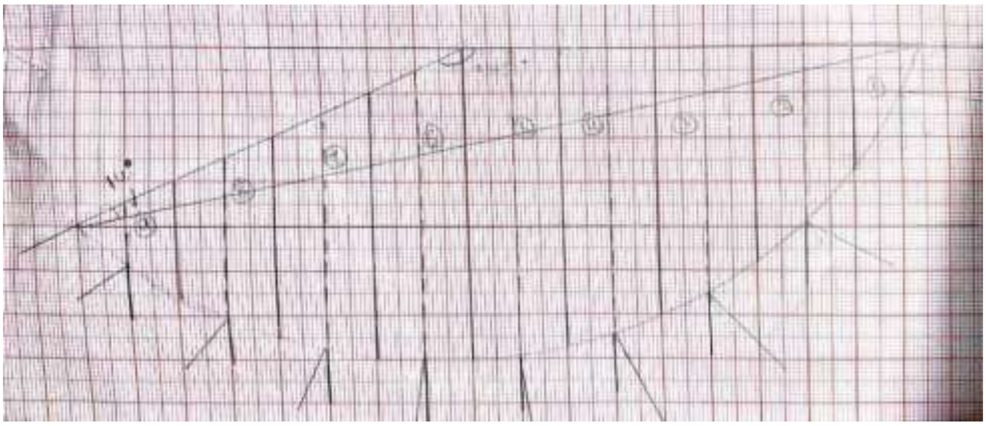

Table 1.5: Normal and tangential component of various slices in the slope

SL NO	Width (b) in m	Mid height (z)	Area (A) (m^2)	Unit weight (γ)	Weight	Angle (θ)	N=Wcos θ	T=Wsin θ
1	2	1.7	3.4	18	61.2	80	10.627	60.270
2	2	5.8	11.6	18	208.8	50	134.204	159.95
3	2	7.6	15.2	18	273.6	33	229.46	149.013
4	2	8.6	17.2	18	309.6	20	290.93	109.89
5	2	9.1	18.2	18	327.6	90	323.57	51.250
6	2	8.5	17	18	306	80	303.022	42.59
7	2	7.1	14.2	18	255.6	19	241.675	83.21
8	2	5.1	16.2	18	183.6	34	152.211	102.66
9	2	2.4	4.8	18	86.4	49	56.683	65.21
							∑N=1742.39	**∑T=768.8**

Factor of Safety = (c r θ+∑N tan Φ)/∑T Where, L=rθ

Assuming the values r, r =10m

= (7.8x2.2) + (1742.39xtan 38˚)/768.8

FOS = 1.23

1.18 Details of earthen bund

1. Full reservoir level (FSL) = +108.50 m.
2. Maximum water level (MWL) = +109.10m.
3. Top bund level (TBL) = +110.60 m.
4. Dead storage level (DSL) = +101.65m.
5. Free board = 1.5m.
6. Height of bund = 8.10m.
7. Top width of bund (carriage way) = 6.00m

1.19 Weir

In general, the above purpose can be accomplished by constructing a barrier across the river so as to rise the water level on the upstream side of the obstruction and thus to feed the main canal taking off from its upstream side at one or both of its flanks as shown in fig 1.6.

Fig 1.6: Weir

Diversion head works include the connection of waste weir to dispose of the surplus water. The waste weir is constructed to dispose of excess water during flood seasons. Length of the weir should be such that the quantity of water estimated as maximum flood discharge likely to enter from the catchment into the tank can be disposed should be properly designed and should have adequate capacity to dispose of the entire surplus water at the time of worst design period of with a depth of water over the weir equally to the difference between them.

1.20 Design of Surplus Weir Steps

Data available:
Top bund level :(T.B.L) = 110.6m.
Full tank level: (F.T.L) = 108.5m.
Maximum water level (M.W.L) =109.1m

a) Estimation of flood discharge entering the tank

Isolated catchment area of the group of tanks = M= 1.0km^2
Flood discharge entering the tank is determined by the formula,
Q = CM2/3 Assume Ryve's co-efficient =C=9
Q = 9x (1.0)2/3
Q=11.79 cumecs

b) Length of Surplus Weir

Water is to be stored up to a level of +108.50m. i.e., F.T.L of tank is +108.50m and so, the crest level of the surplus weir has to be kept at +108.50m.
Submersion of foreshores lands is limited to +109.1m. i.e., M.W.L of tank is to be kept at +109.1m
Therefore, head of discharge over the weir is 109.1-108.5(M.W.L-F.T. L) =0.60M
To provide grooved dam stones 20cm x20cms. Will be fixed in the centre of the crest at one-meter interval with top at M.W.L
The weir may be assumed as a broad-crested weir .so the discharge per meter length Of the weir is given by

Q=2/3Cd h √2gh =1.66×h2/3 Q=1.66× (.60)2/3
Q=1.18cumecs
Clear length of surplus weir required =17.00/1.18
L=14.00m

Since, the dam stone is to be fixed on top at one meter clear intervals, the number of openings will be 14.
So, the numbers of dam stones required are 13.

Size of dam stone 20cms x20cms and the projecting length above crest will be 60cms. Therefore, the overall length of surplus weir between abutments is14+(13x0.2) =17.00m Provide an over length of 18.0m.

c) Weir

Crest level =108.50m (F.T.L)

Top of dam stone =109.10m (M.W.L)

Level where hard soil at foundations is mat with is 103.86m

Taking foundation about 0.50m deeper into hard soil the foundation level can be fixed at 103.36m.the foundation concrete may be usually 0.50m thick.

Top of foundation concrete =+103.86m

Height of weir above foundation =108.5-103.86

H=5.14m

d) Crest width

Generally, the crest width is assumed as equal to 0.55(√H+√h), where H is the height of weir and h is the head over the weir (both H and h expressed in meters)

a =0.55(√H+√h) =.055× (√5.14+√.60)

a =1.670≈1.70m

This gives a crest width of about 1.40m. This width may be adopted.

e) Base width

Check the stability of the weir such that the resultant thrust due to overturning water pressure when water on the upstream side is up to the top of shutters and the weight of masonry of the weir passes through the middle third.

In cases, the maximum overturning moment due to water thrust is equal to

b = (H+S)/(√ρ-1)

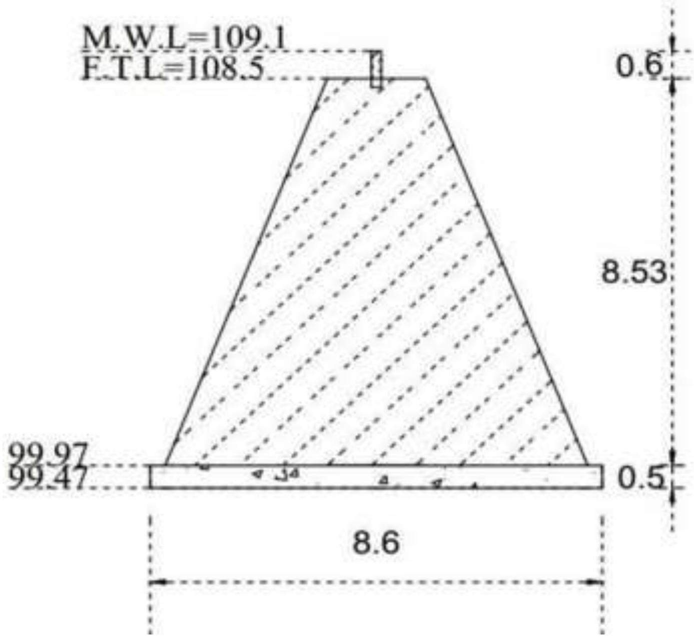

Fig 1.7: Cross Section of Weir

b = (5.14+0.6)/√ (2.3-1)

b= 5.00m

Where H is the height of weir above the foundation and S is the height of shutters. The slopes of weir on either side being same, the restoring moment M of the weir due to the weight of masonry is

Where, p= specific gravity of masonry.

H=height of weir.
a =crest width of weir=2.0m.
b = base width of weir.
S =height of shutter above weir crest =0.60m

f) Abutments, wings and return

The width of abutments, wings and returns will all be uniformly 0.50m with a front batter of 1in8 lengths wings walls must be enough to completely encase the tank.

- **Abutments**

Portion AB is called the abutment .it has its top level same as that bund at +110.6m and has its length at top same as that of the top width of bund.

The height of abutment above foundation = 110.6-103.86=6.74m.

The bottom width required AB is 6.74×0.4=2.704 ≈ 3m.

The section as indicated in figure 4.3 can be adopted. The wall BD is called the upstream wing wall. The section of the wing wall at B is the same as the section of the abutment.

This wing walls starts sloping down from B till it reaches about 30cms above MWL that is a level of 109.1+0.30=9.30 at C. So, the portion of wing wall BC will be having its top sloping down from =110.60 to 109.40m.

- **Section of Wing Wall at c**

Height of wall above top of foundation 109.4-103.86=5.54m.
Base width required,
=5.54×0.4 = 2.29 or 2.5m. Adopt the section.

The top width from B to C is the same as 0.5m but the bottom width gets slowly reduced to 2m. At section B to 2.50m at section C.

- **Level Wing and Returns**

Since the level wing and return, that is portion CD and DE have to be throughout 30cms. Above MWL the same section of the wall at C can be adopted.

- **Upstream side transition**

In order to give an easy approach, the upstream side wing wall may be splayed as shown, that is generally at 1 in 3.

- **Downstream side Wings and Returns**

As the water after passing over the weir goes down rapidly to normal MWL in the water course, the wings and returns need not to be so high as those on the upstream side. The wing walls from A to F will slope down till the top reaches the ground level at F. the section of wing walls at A will be the same as that of the abutment.
The top of wing wall at F may be fixed at +105.30 same as the ground level.
So, the height of wall above foundation concrete is 105.3-104.00=1.30m.
The base width required is 1.30×0.4=0.52m or adopt a minimum base width of 1.0m.

- **Downstream transition**

The downstream side wings are given a splay of 1in 5 as shown in fig.4.2.

- **Aprons of the Weir**

The ground level at the site of weir is =+105.3m

- **Upstream aprons**

Generally, no aprons are required on the upstream side of the weir. However, it is desirable to provide a puddle apron as shown in fig.4.6. It is also sometimes provided with a nominal rough stone apron 30cms. Thick
packed well on puddle clay apron

In case where the head of percolation is great, in order to reduce the length of aprons on the downstream side of the weir, it is necessary to provide upstream side solid apron. This apron is not subjected to any uplifts and hence can be of nominal thickness. However, this acts in considerably reducing the creep length and consequently reduces lengths and thicknesses of the aprons, downstream of the weir.

- **Downstream Aprons**

Since the ground level is falling down to +104.7m in a distance of about 6.0m, it is desirable to provide a single solid apron.

The aprons may be designed for the hydraulic gradient of 1 in 5 so that the residual gradient at the exit of the aprons can be limited to 1 in 5 which is safe enough and will not start undermining the structure.

Maximum uplift pressure is experienced on the downstream aprons when the water level in the tank is up to top of dam stone level, i.e., to +109.10m with no water on the downstream side.

The total uplift head acting =109.1-108.1=1.0m

If residual uplift gradient is to be limited to 1/5, we required aprons to be accommodate a total creep length of 1×5=5m.The upstream water as to be percolated under the foundation of the weir, if it has to establish any uplifts under the aprons. Assuming the puddle apron formed on the upstream of the weir to be not imperviousness, the water will start percolating from A at a level f +108.50m and reach B and C then it will follow CD under the foundation concrete. From here, it will follow the least path from D to E under the end cut-off and then appear at F, i.e., the lower solid apron. So the total length of percolations:

Then we require accommodating a total creep of 10m

AB+BC+CD+DE=2.94+(2*0.3*5) +CE+1.94

CE=11.0m

This length must not be less than 11.00m, if the structure is to be safe

Therefore, CE+5.00=11.0

CE=6.0m

The total length of the solid apron from the body wall as provided in the drawing is 6m and this will be enough.

These can be reduced if the upstream side puddle clay apron is really impervious.

To ensure safety, the whole upstream side apron can be packed with stone and well grouted with cement concrete.

At the end of the apron, retaining wall of the downstream side apron, a nominal 3 to 5m length of Talus with a thickness of 50cms.May be provided as a safety device.

- **Thickness of Solid Aprons**

The maximum uplift on the apron floor is felt immediately above point D in the sketch. Assuming thickness of 80cms of apron the bottom level of apron is +102.36m creep length from D to the bottom of apron is 1.94m.

Total creep length from point A on the upstream side up to the above D under the solid apron is

2.94+3.0+1.94=7.88m

Therefore, Head lost is percolation along the path up to the point

= 8.0÷5= 1.60meters

Residual head exerting uplift under the apron

=1.60-1.0 =0.6 meters.

Since the bottom of the apron is above the assumed tail water elevation, the weight of concrete fully takes care of the uplift, as there is no loss of weight in concrete due to buoyancy.

Each meter depth of concrete can withstand a head of 2.25m by the self-weight of apron alone. Allowing an extra 20% thickness to withstand any variations, the thickness of apron required is (0.60/2.25) × (6/5) =0.32m or say 32cm

So, provide the first solid apron as 32cm thick.

Table 1.6: Details of surplus weir

Sl. No	Specification	Details
1	Top bund level	110.600m
2	Maximum water level	109.100m
3	Full tank level	108.500m
4	Dead storage level	101.650m
5	Upstream slope	2:1
6	Downstream slope	2:1

1.21 Canal

A canal is an artificial channel is shown in fig 1.8, generally trapezoidal in slope constructed on the ground to carry water to the fields from the river or from the reservoir.

Fig 1.8: Canal

Classification based on the function of canal,

- Irrigation canal.
- Carrier canal.
- Feeder canal.
- Navigation canal

Based on the capacity of the water in the reservoir the canal is designed as an irrigation canal. An irrigation canal carries water to the agricultural fields.

1.21.1 Canal alignment

- A Canal has to be aligned in such a way that it covers the entire are proposed to be irrigated with shortest
- Possible length and at the same time its cost including drainage work is minimum.
- A shorter length of canal.
- Insures less losses of head due to friction, smaller loss of discharge due to seepage and evaporation so that addition.
- Area can be brought under cultivation.

According to alignment the channels may be divided into three types,

1. Ridge canal
2. Counter canal
3. Side slope canal

Based on capacity of the water in the reservoir in the canal is designed as an irrigation canal. Based on canal alignment the canal is designed as a side slope canal.

1.22 Design Procedure for Irrigation Canals

Cross section of an irrigation canal:

A typical most desired section of a canal is shown in drawings. This section is partly in cutting and partly in filling, and aims at balancing the quantity of earthwork in excavation with that in filling. Some times when natural surface level is above the top of the bank, the entire canal section will have to be in cutting, and it shall be called ‘canal in cutting’. Similarly, NSL lower than the bed level of the canal, the entire canal section will have to be built in filling, and it is called ‘canal in filling or canal in banking’.

- **Side slopes:** The side slope should be such that they are stable, depending upon the type of the soil. A comparatively steeper slope can be provided in cutting rather than in filling, as the soil in the former case shall be more stable.
  1H: 1V (1:1) to 1.5H: 1V (1.5:1) slope in cutting, and 1.5H: 1Vto 2H: 1V in filling, are generally adopted.
- **Berms:** Berm is the horizontal distance left at ground level between the toe of the bank and top edge of cutting.
- **Freeboard:** The margin between FSL and bank level is known as free board. The amount of free board depends upon the size of the channel.

1.22.1 Design procedures [By Lacey’sTheory]

- First of all, the longitudinal section of the existing ground along the proposed canal alignment is plotted on the suitable scale (say 1cm=100m for horizontal scale 2cm=1m for vertical scale)
- A suitable canal slope is assumed from Lacey’s diagrams for an approximate discharge and silt factor. It is made consistent with existing slope.
- A trail slope line is marked for drawing FSL line keeping in view guidelines already given. The depth and position of falls, etc. is decided tentatively.
- The channel is then designed from its toe reach towards its head reach, kilometres to kilometres.
- The discharge required in the channel in the given reach, for required irrigation potential, is worked out and losses are added so as to calculate to required discharge. The cumulative losses of a particular km and the next below it.
- The channel is now designed for the discharge and assumed bed slope, by Kennedy’s theory, using Garret’s diagram. The bed width depth ratio is kept within specified limits.
- The bed slope, FSL, falls etc. Are all adjusted using intelligence, judgment and knowledge.
- • The bed levels, water depths etc. Are drawn on L section. The cross section at every km is than drawn, using canal standards.

Canal design

Crop	Delta (cm)
Rice	120
Maize	25
Cotton	50

Total capacity of reservoir = 9172.5m3 - DSL
=91725m3 - 180 m3
=91545 m3

Assuming evaporation and conveyance losses = 20%

Volume = 18309m3
Area to be irrigated = volume /depth
For, rice seeds, 18309/.12
= 15257.5 m2 (Assume 50 hectares)

(Taking oil seeds which consumes more water compared with maize and peas)Area to be irrigated will be 50 hectare an each side canal to be irrigated will be 25 hectare.

1.22.2 Design of Regime Canal by Lacey's theory

Data available:
Discharge = $0.06m^3/s$
Area to be irrigated = 50 hectare
Duty for cotton =775 ha/cumecs
Rice,
Discharge = Q = area/duty = 50/775 = $0.064m^3/s \approx 0.06\ m^3/s$
Silt factor, (f) =1.76x√dm
Where,
d_m=average particle size in mm
From table no = 4.6 of S.K Garg book of silt fine soil, dm=0.73
f = 1.76x√0.73
f = 1.50

Step-1 Calculation of velocity (V):
V = $[QF^2/140]1/6$
V = 0.314m/s

Step-2 Hydraulic mean depth (R)
R = $5/2[V^2/F]$
R =$5/2[(.314)^2/0.704]$
R = 0.164 m

Step-3 Area of channel section(A)
A= Q/V
A = 0.06/.314
A= $0.19\ m^2$

Step-4 Wetted parameter (P)
P = 4.75×√Q
P = 4.75×√0.06
P = 1.16 m
Depth of channel, D = P-√P2-(6.944×A)/3.472
D = 1.16-√ $(1.16)^2$-(6.944×0.19)/3.472 = 0.3 m
Breadth of channel = (B) = P-2.236(D) = 1.16-2.236(0.3) = 0.50m

Step-5 Bed slope(S)
S = [(f5/3/ (3340× (Q)1/6
S = [(1.5)5/3/ (3340 × (0.06)1/6]]
S = 1/1065

Step-6 Details of canal

i. Side slope =1:1
ii. Bed slope =1/1065
iii. Depth =0.3 m
iv. Breadth =0.5 m

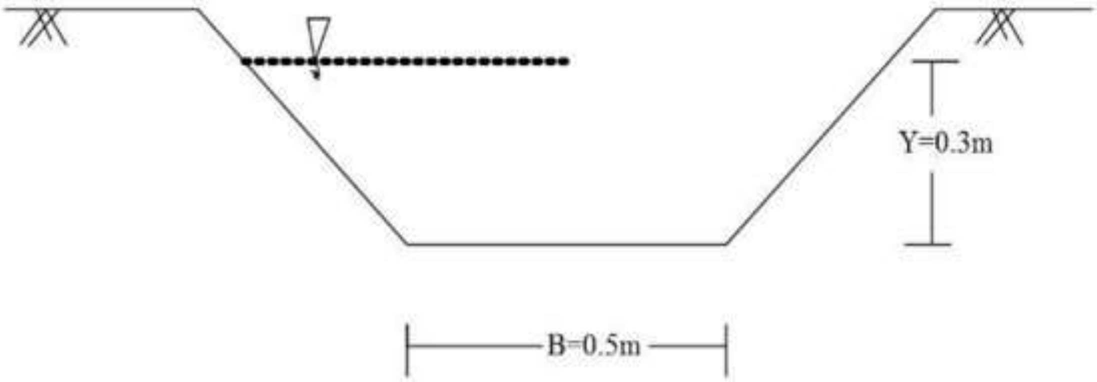

Fig 1.9: Cross section of canal

CHAPTER-2

WATER SUPPLY AND SANITARY PROJECT

2.1 Introduction

One of the major essential requirement for human existence is Water; with other elements such as Air, Food, Heat and Light. The use of water by man, plants and animals is Universal. Without it there can be no life. The availability and quality of water plays an important role in determining the quality of Life of the people.

The planning, designing, financing and operations of water systems are complex understandings and they require high degree of skills and judgment. The work of construction and maintenance of water supply is generally undertaken by Government agencies mostly consisting of Civil Engineers and Environmental Engineers.

About 70% of earth's surface is covered with water. 97% of the water on the earth is filled with salt and other minerals and humans cannot drink this water although the salt can be removed, it is a difficult and expensive process.

Around 2% of the water on the earth is glacier ice at the north and south poles. This ice is fresh water and could be melted; however, it is too far away from where people live to be usable. Less than 1% of all the water on the earth is fresh water that we can actually use. We use this small amount of water for drinking transportation heating and cooling, industry, and many other purposes. A water supply scheme for Gadenahalli village is planned and designed for the project.

2.2 Extensive survey project

Gadenahalli village is located in Bangalore North Tehsil of Bangalore district in Karnataka, India.

2.2.1 Main objective

The Main objective of this project is to provide pure and safe drinking water supply with proper sewerage system to the proposed village.

2.2.2 Selection of Survey Project Site

Before commencement of the survey work we have made reconnaissance survey to locate the site for intake works, water treatment plant, overhead tank, rising main, gravity main and distribution system for the proposed village.

Detailed survey work is made for the entire project work and is given below

- Block levels for the proposed Intake works.
- Block levels for the proposed Water Treatment Plant.
- Block levelling for proposed Overhead Tank.

2.3 Water supply project

Intake works, Water Treatment Plant, Rising Main, Over Head Tank, Distribution system. All the drawings connected to the above two stages are given in detail, in a separate bounded report after the selection of site for the proposed village, detailed survey work has been carried out. Using survey data, we have made the following designs. Based on the prepared Layout for the proposed village and the no. of occupants' in

2.4 Necessity

The use of water is increasing rapidly with our growing population. Already there are acute shortage of both surface and underground water in many parts of the country. In order to ensure the availability of sufficient quantity of good quality water, it becomes almost imperative in a modern society, to plan and build suitable water supply scheme, which may provide potable water to the various sections of the community in accordance with their demands and requirements.

Gadenahalli village is not provided with proper and sustained water supply system to convey water to the houses. Water is obtained in these villages mainly through bore-wells and a small reservoir tank constructed mainly for the purpose of irrigation of nearby farm lands. The ground water resource is fast depleting due to the continuous usage of water and it may cause shortage of available water in the case of draught and if monsoon fails and the existing water supply is inefficient to meet the basic needs of people and for the development of these villages. This inconvenience can be avoided by constructing an efficient water supply scheme for the above village.

2.5 Water demand

It is essential to determine the quantity of water that is required daily before designing a proper water works project. The demand can be meet, only if there is sufficient source for supply otherwise a compromise has to be made in either the water demand or the source of supply of water.

2.5.1 Types of Water Demand

The various type of water demand, which a city may have, maybe broken down into following categories;

- Residential or Domestic use: This includes the water required for various activities such as drinking, cooking, bathing, washing of clothes, utensils, and flushing of water closets. According to the manual on water supply and treatment prepared by ‘Ministry of Urban Development, New Delhi’; recommends 135 lpcd (liters per capita per day) for communities with population up to 20,000.
- Institutional use: The requirement of water for various institutions such as hospitals, hotels, schools and colleges, offices, railway stations, etc. should be assessed and provided in addition to other demands.
- Industrial use: The quantity of water required for industrial use depends upon the type and number of industries present and are likely to be built in near future is considered in water supply project planning. The 3 villages considered for this project has no industries that can be accounted for and there is less likelihood for scope of future industries in these areas.
- Public use: This includes the water required for public utility purposes such as road washing, sanitation and public parks. A nominal amount of 5% of the total consumption may be added to meet this demand on an arbitrary basis.
- Fire demand: In a densely populated and industrial area, fire generally breaks out and may lead to serious problem. The villages considered are not so densely populated and the chance of breaking out large fire is rare.
- Water system loss: This includes the water lost in leakage due to bad plumbing or damaged meters, stolen water due to unauthorized water connection, other losses and wastes.

2.5.2 Sources of water:

The following are common sources of water.

1) Surface water
2) Sub surface water
3) Water obtained from reclamation

1) Surface water

Surface water is the one which is available as run-off from a catchment area, during rainfall or precipitation this runoff flows either into streams or into undrained lakes. The runoff water flowing into streams can either be stored in a reservoir by constructing a dam across it, or be diverted into a water supply channel.

2) Sub surface water

The largest available source of fresh water lies underground. The term ‘ground water’ refers to this water, which is stored by nature, under-ground in the water bearing formation of earth’s crust. The total ground potential is estimated to be one third the capacity of oceans. The main source of ground water is precipitation.

3) Water obtained from reclamation

- Desalination saline or brackish water may be rendered useful for drinking purposes by installing desalination plants.
- Re-use of treated waste water effluent or waste water can be treated suitably so that it may be re-used.

2.5.3 Factors affecting per capita supply:

The design per capita supply shall be minimum of 135 lpcd for rural areas. Water demand shall be calculated for the projected population for the design period of the project which is30 years.

1) Size of the city: The per capita demand for big cities is generally large as compared to that for smaller towns. This is because of the fact that in big cities, huge quantities of water are required for maintaining clean and healthy environments. For example, big cities are generally severed, and as such required large quantities of water.

2) Climatic conditions: At hotter and dry places, the consumption of water is generally more, because more of bathing, cleaning, air-cooling, sprinkling in lawns, gardens, roofs etc., are involved. Similarly, in extremely cold countries, more water may be consumed, because the people may keep their taps open to avoid freezing of pipes, and there may be more leakage from pipe joins, since metals with cold.

3) Types of gentry and habits of people: Rich and upper class communities generally consume more water due to their affluent living standards. Middle class communities consume average amounts, while the poor slum dwellers consume very low amounts. The amount of water consumption is thus directly dependent upon the economic status of the consumers.

4) Industrial and commercial activities: The pressure of industrial and commercial activities at a particular place increases the water consumption by large amounts. Many industries require really huge amounts of water (much more than the domestic demand) and as such, increase the water demand considerably.

5) Quality of water supplies: The quality and taste of the supplied water is good, it will be consumed more, because in that case, people will not use other sources such as private wells, hand pumps, etc. Similarly, certain industries such as boiler feeds, etc which require standard quality waters will not develop their own supplies and will use public supplies, provides the supplied water is up to their required standards.

6) Pressure in the distribution system: If the pressure in the distribution pipes is high and sufficient to make the water reach at 3rd or eve 4th storey, water consumption shall definitely be more. This water consumption increases of two reasons:

- People living in upper storeys will use water freely as compared to the case when water is available scarcely to them.
- The losses and water due to leakage are considerably increased if this pressure is high. For example, if the pressure increase from 20 m head of water, to 30 m head of water, the losses may go up by 20 to 30 percent.

7) Development of sewerage facilities: As pointed out earlier, the water consumption will be more, if the city is provided with 'flush system' and shall be less if the old 'conservation system' of latrines is adopted.

8) System of supply: The water may be supplied continuously for all the 24 hours of the day, or may be supplied only for peak periods during the morning and evening. The second system i.e. the intermittent supplies, may lead to some saving in water consumption due to the losses occurring for lesser time and a more vigilant use of water by the consumers. But at many places, the intermittent supplies may not give much saving over the continuous supplies, because of the following reasons:

- In intermittent supply system, water is generally stored by consumers in tanks, drums, utensils, etc. for non-supply periods. This water is thrown away by the even if unutilized as soon as the fresh supply is restored. This increases the wastage and losses considerably.
- People have a general tendency to keep the taps open during non-supply hours, so that they may come to know of it as soon as the fresh supply is restored. Many a times, water goes on flowing unattended even after the supply is restored, thus resulting in wastage of water.

9) Cost of water: If the water rates are high, lesser quantity may be consumed by the people. This may not lead to large savings as the affluent and rich people are little affected by such policies.

10) Policy of metering and method of charging: Water tax is generally charged in two different ways:

- On the basis of meter reading (meters fitted at the head of the individual house connections and recording the volume of water consumed).
- On the basis of certain fixed monthly flat rates.

2.6 Design period

The water supply project includes construction of dams, reservoirs, intake structures, water treatment works and distribution networks. These all works cannot be replaced easily or capacities increased conveniently for future expansions. While designing and constructing these works, they should have sufficient capacity to meet the future demand of the town for number of years. The number of years for which the designs of water works have been done is known as design period. The water works are designed for design period of 20-30years as per CPHEEO manual. For water supply project the design period is taken as 30years.

Water supply project under normal circumstances may be designed for a design period of 30 years. This 30 years' period is to be counted after the completion of the project. Hence, a completion time of about 2 years may also be added to this design period.

2.6.1 Factors affecting design period

1. Useful life of component structures and the chances of their becoming old and obsolete. Design period should not exceed those respective values.
2. Ease and difficulty that is likely to be faced in expansions, if undertaken at future dates. For example, more difficult expansions mean choosing a higher value of design period.
3. Amount and availability of additional investment likely to be incurred for additional provisions. For example, if the funds are not available, one has to keep a smaller design period.

4. The rate of interest on the borrowings and the additional money invested. For example, if the interest rate is small, a higher value of the design period may be economically justified and, therefore, adopted.
5. Anticipated rate of population growth, including possible shifts in communities, industries and commercial establishments. For example, if the rate of increase of population is less, a higher figure for the design period may be chosen.

2.7 Population forecasting

Population forecasting is the method of predicting the future population at the end of design period considering the previous year's population, of the area to be considered. Population at the end of the design period is used for calculating annual average daily draft.

Table no: 2.1 Details of population of previous year

Year	Gadenahalli population
2001	5492
2011	5741
2016	10000

Note: Only previous 3 decades' population data could be obtained; as earlier decades' data doesn't exist.

a. Estimating the future population depending upon the known population of previous decades is known as population forecasting.
b. There are many methods which are used in population forecasting namely, Arithmetic Increase Method (AIM), Geometric Increase Method (GIM), Incremental Increase Method (IIM), Master Plan Method, Simple Graphical Method, Decreasing Rate of Growth Method, Comparative Graphical Method, and Logistic Curve Method.

2.7.1 Arithmetical increase method:

It is the simplest method of population forecast, though it gives lower results. In this method, the increase in population from decade to decade is assumed to be constant. $P_n = P_0 + n$

Where,

P_o= last known population of the village.

P_n = future population at the end of n decades.

x = average increment per decade.

Year	Population	Increase in population
1991	5492	249
2001	5741	
2016	10000	

∑= 4259

$$X = \frac{\sum X}{n} = \frac{4259}{2} = X = 2130$$

Population in 2017

$P_n = P_o + n$

P_{2017}=10000+0.1 X 2130

P_{2017} = 10213

Population in 2017

$P_n = P_o + n$

P_{2027}= 10000+1 X 2130

P_{2027}= 12,343

Population in 2037 = 14,473

Population in 2047 =16,603

Hence design Population is 16,603 for the year 2047

2.7.2 Geometrical increase method

In this method, it is assumed that the percentage increase in population from decade to decade is constant. This method gives higher results since the percentage increase never remains constant. The manual water supply and treatment recommends using this method.

$P_n = P_0 (1 + (r / 100))$ n

Where, P_n = future population at the end of n decades, P_0 = present population,

r = Average percentage increase in population per decade.

n = number of decades

r = $^{1/n}\sqrt{r_1 . r_2 . r_n}$

Year	Population(No)	Increase in population	% of growth rate
2001	5492	249	(249/5741)×100=4.33%
2011	5741		
2016	10000	4,508	(4,508/5492)×100=82%

$\sum$ **=4,259**

Population in 2017

r = √r₁×r₂ t

r=√4.33×82 2

r=18,85%

P_{2017}=10000(1+ (18.85/100))$^{0.1}$

Population in 2017 = 10,174

Population in 2027

P_{2027}=10174(1+ (18.85/100))^{1}

Population in 2027 = 12,092

Population in 2037 = 14,371

Population in 2047 = 17,079

Therefore, total design population at end of 3 decades: 17,079

2.7.3 Incremental increase method

This method combines both arithmetical average method and geometrical average method. In this method growth rate is not assumed to be constant.

$$\mathbf{P_n = P_o + n\,x' + \frac{n(n+1)}{2} \times y'}$$

Where, P_n = future population at the end of n decades,

P_0 = present population,

x'= average increment per decade.

n = number of decades

y'= Average of incremental increase of known decade

Population	Increase in population	=Σx/n	Y' = Incremental increase in population
5492	249	2130	4259
5741	4508		
10000			
	Σ = 4259		**Σy'=4259**

$'P_{2017}$ = 10000 + 0.1(2130) + 0.1(0.1+1) 2 ∗ 4259

$'P_{2017}$ = 10,447

$'P_{2027}$ = 16,836

$'P_{2037}$ = 23,225

$'P_{2047}$ = 33,873

2.7.4 Decreasing rate growth rate method

Year	Population	Increase in population	%Increase in population	Decrease in % increase
2001	5492			
2011	5741	249	4.33% 82.00%	77.67%
2016	10000	4508		

Decrease in percentage increase =77.67%

a. The expected population at the end of the year 2017

$$P_n = P_0 + \frac{\text{\% incremental of last decade} - \text{\% decrease}}{100} \times \text{population of decade}$$

$$P_n = P_0 + \frac{\text{\% incremental of last decade} - \text{\% decrease}}{100} \times P_0$$

$$P_{2017} = 10,000 + \frac{82 - 4.33}{100} \times 10,000$$

$\mathbf{P_{2017} = 17,767}$

b. The expected population at the end of the year 2027

$$P_{2027} = 17,767 + \frac{77.67 - 4.33}{100} \times 17,767$$

$\mathbf{P_{2027} = 30,797}$

c. The expected population at the end of the year 2037

$$P_{2037} = 30,797 + \frac{77.67 - 4.33}{100} \times \mathbf{30,797}$$

$\mathbf{P_{2037} = 52,038}$

d. The expected population at the end of the year 2047

$$P_{2047} = 52,038 + \frac{77.67 - 4.33}{100} \times 52,038$$

$\mathbf{P_{2047} = 85,675}$

2.7.5 Simple graphical method

- A graph is plotted between time and population.
- The curve is then smoothly extended up to the desired year.
- Population of a particular year can be known referring to the graph.
- But simple graphical method gives very approximate results as the extension of the curve is done by the intelligence of the designer.

2.7.6 Master plan method

- Only those expansions are allowed which are permitted or proposed in master plan.
- In master plan method a city is divided into various zones like residence, commerce and industry and the population density is also fixed.
- For example, there may be 5 persons living in a residential plot and there may be 10000 plots in a zone. Then total population of this zone, when fully developed, can be easily worked out as 5*10000=50000

2.7.7 Comparative graphical method

- In this method the cities having conditions and characteristics similar to the city whose future population is to be estimated are, first of all selected. It is then assumed that the city under consideration will develop, as the selected similar cities have developed the past.

- This method has a logical background, and if statistics of development of similar cities are available, quite precise and reliable results can be obtained.

2.7.8 The ratio method or the apportionment method

- In this method the forecasting future population of a city or a town, the city's census population record is expressed as the percentage of the population of the whole country.
- A graph is then plotted between time and these ratios, and extended up to the design period, so as to extrapolate the ratio corresponding to the future design year.
- In order to do so, the local population and the country's population for the last four to five decades is obtained from the census records. The ratios of the population to national population are worked out for these decades.

2.7.9 The logistic curve method

- It was explained earlier that under normal conditions, the population of a city shall grow as per the logistic curve.
- P.F. Verhulst has put forward a mathematical solution for this logistic curve. According to him, the entire curve AD can be represented by an autocatalytic first order equation. Since, the design period is fixed the next step is to determine the population of town, the population of town depends upon the factor like birth, death, migration, and annexation.

2.8 Common demands of a community

It is very difficult to precisely assess the quantity of water demanded by the public, since there are many variable factors affecting the consumption. Certain thumb rules and empirical formulas are, therefore, generally used to assess this quantity, which may give fairly acute results. The use of a particular method or a formula for a particular case has, therefore, to be decided by the intelligence and foresightedness.

The various types of water demands, which a city may have, may be broken down into the following classes:

i. Domestic water demand
ii. Industrial water demand
iii. Institution & commercial
iv. Demand for public use
v. Fire demand

i. Domestic water demand

Minimum domestic water consumption for communities with population up to 20,000 and without flushing system at water supply through stand post is 70 to 80lpcd (as per IS-1172-1993) Let us consider the demand as 135 lpcd. It includes all types of uses like drinking, cooking, bathing etc.

ii. Fire demand

In thickly populated and industrial areas fires generally breakout and may lead to serious damages, if not controlled effectively. Big cities, therefore, generally maintain full fire-fighting squads. Fire-fighting personnel require sufficient quantity of water, so as to through it public water supply schemes for fighting the fire. The quantity of water required for extinguishing of fires should be easily available & kept always scored in storage reservoir Fire demand usually taken as 1 liter/head/day.

Total water demand = 135 LPCD

GOI prefers the geometrical increase method Therefore by this method we have total design population at end of 3 decades: 17,079

1. Average daily demand = Water demand ×Population
   = 135 × 17,079

= 23,05,665 lit/ day

2. Maximum daily draft may be assumed as 180% of annual average of daily draft
   M D D = $\frac{180}{100} \times 2305665$
   D = 4150197 lit/ day
   D = 4.15 MLD
3. Maximum hourly draft of the maximum day, It may be assumed as 270% of annual average hourly draft Therefore, Maximum hourly demand of maximum day
   = $\frac{270}{100} \times 2305665$
   = 6225295 lit/ day
   =6.22 MLD

2.9 Water treatment

Water is not pure the treatment process is essential for this project. Ideal water for drinking means it should be free from all the impurities, undesirable taste and odors with reasonable temperature, color, etc. The treatment process involved for purification of water is as follows:

- Screening
- Aeration
- Plane sedimentation
- Sedimentation aided with coagulation
- Filtration
- Softening
- Disinfection
- Miscellaneous treatments

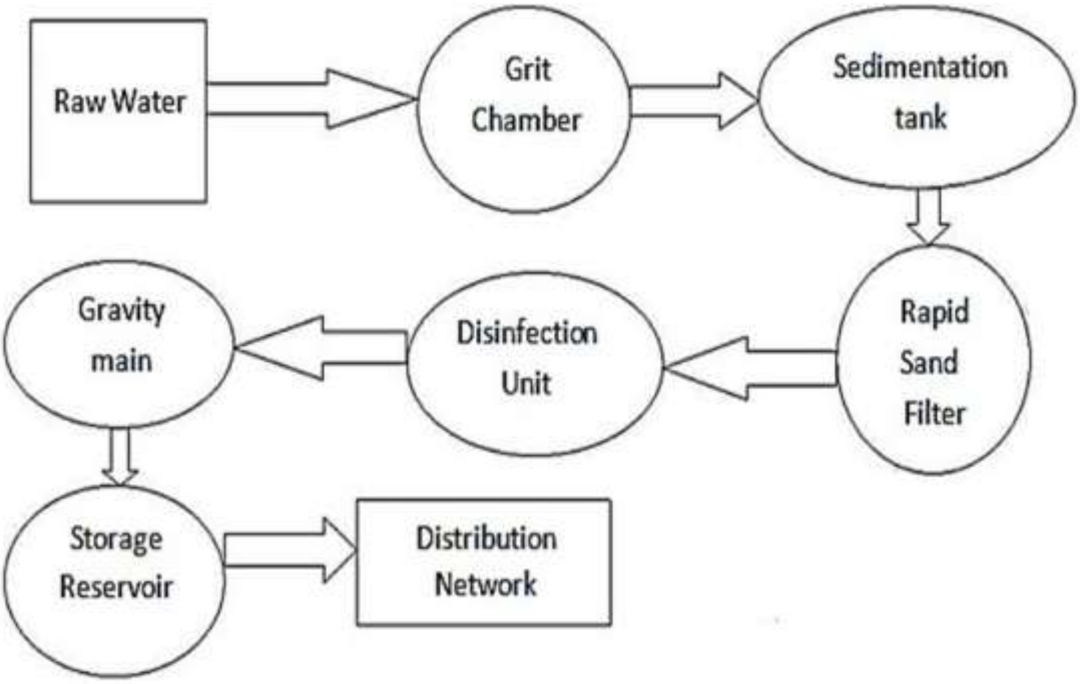

Fig. No. 2.1 Flow Diagram of Water Treatment Plant

2.9.1 Design of intake structure

The basic function of intake structure is to help in safely withdrawing of water from source over a predetermined range of pools levels and then to discharge this water into the withdrawal conduit. From conduit, it flows to the Jack well to pump the water to the treatment plant, in case the treatment plant is at higher elevation then the intake structure. The factors for selecting site for intake structure are:

- The site should be near treatment plant.
- The structure must be located in the purer zone.
- The site should permit greater withdrawal of water.
- Intake must be located at a place from where it can draw water even during the driest periods of the year.

RL of High flood level	= 101.00m
RL of Normal water level	= 100.00m
RL of Maximum water level	= 99.50m
RL of Bed level	= 97.00m

The intake structure for flecting the water from the stream may be designed for maximum daily draft for 6.22Mld

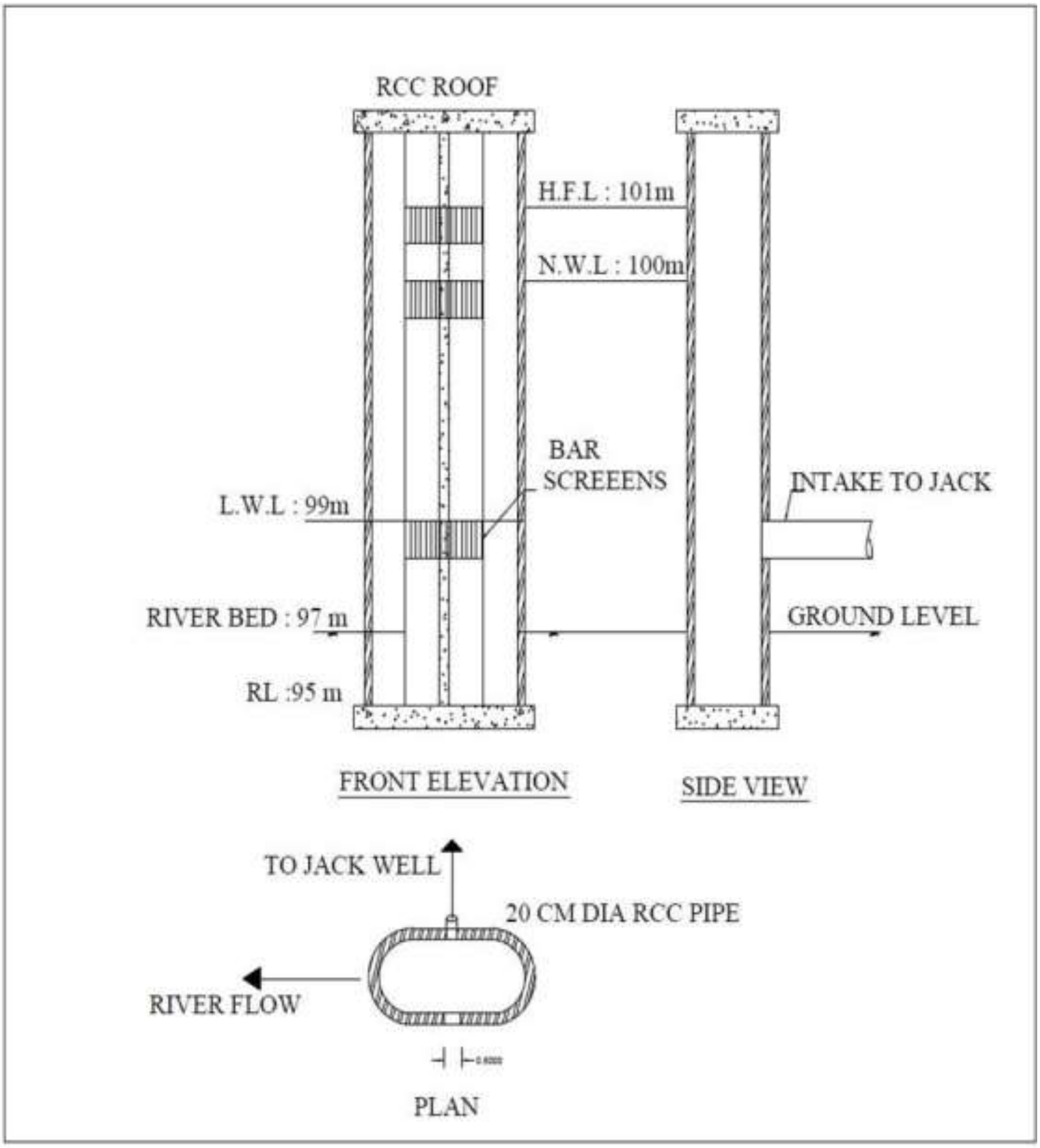

Fig. No: 2.2 Intake structure

Assuming the pumping hour is taken as 8 hours per day

The discharge to be pumped = 0.216 m^3/s

Opening should be fitted with the bar screen made of 12mm diameter steel bars say 50mm opening.

Let the velocity to the bar screen be limited to 0.16 m/sec

Area of opening required in each level = Q/V = 0.216m^3/sec

Let us provide 1m height of screen opening, then the clear length of opening required 1.4m.

No of opening required = 1.4 0.05 = 27No's =28No's

No of bar's provided 28No's

Length occupied by the bars (Assume 12mm dia. bars) = 28×0.012 = 0.336m

Total length of the screen = 0.336+1.4 =1.68~2m

Let us provide 2 ports at each level the size of each port will then be 0.2m height×1m length. In all there will be 6 screened ports.

2 @ each of the 3 levels.

2 screened ports lowest water level.

2 screened ports normal water level.

2 screened ports high flood level.

There ports can be fitted in an oblong well consisting of rectangular length of 0.8m (sufficient to fix 2 bar screens each of length 0.1m) and provided with circular ends,The well can have a width of 0.4m This inlet well can be sunk into the river bed by 2m below the river bed to provide space for accumulation of silt and sand so let's keep the bottom of inlet well @95.00m.

Also let us provide a free board of 1.5m over the river HFL to fix the bottom level of the roof of the well.

Hence provide inlet level of roof @105.800m

The height of inlet well will be then = 105.8-95 = 10.8m

Design of gravity pipe connecting intake well and jack well.

The intake pipe shall be designed to flow by gravity @ max velocity of say 1.2m/s.

Q=0.216m^3/s

V=1.2m/s

Area of pipe required = $\pi d^2/4 = \text{Q/V}$

$$\pi d^2/4 = 0.216/1.2$$

d = 0.5m

Diameter 0.5m hence use 50cm diameter

RCC intake pipe giving velocity of, $V = \boldsymbol{Q}/\boldsymbol{v}$

$$V = \mathbf{0.216/3.142 * 4 * 0.5^2}$$

V = 1.1m/s

Using manning's formula, we have

$$V = \frac{1}{N} \times R^{2/3} \times \sqrt{S}$$

$$V = \frac{1}{0.017} \times R22.63^{2/3} \times \sqrt{S}$$

$$\text{S} = 0.387 = \frac{1}{2.58}$$

Say 1 in 3

Hence lay 0.5 m dia. intake pipe @ a gradient of 1in 3.

2.9.2 Design of rising main

For a given discharge, if smaller diameter of pipes is selected the velocity of flow increases. However, the increased velocity results in higher frictional loss and hence increases total pumping head, which requires increased HP of the pump. This leads to higher pumping cost and may offset the reduction in initial cost due to the smaller diameter pipe. Normally, the combined effect is a net increase in cost. On the other hand, if too large a diameter of the pipe is used the cost of pumping will be less, but the initial cost of the pipe will be more than resulting saving in pumping cost. This initial investment in cost of pipeline and pumps has an annuity, which depends on the rate of interest and period of repayment of loan taken for capital

investment. The annual operating cost of the pumps will vary depending on HP/KW of pumps. For the most economical condition we must choose such a pipe size whose annuity due to initial cost together with the annual pumping cost will make the total annual expenditure minimum. The size of such a pipe is called ‘economical size of the pipe’.
The optimum velocity for most economical sizes of pipes is likely to be about 1m/sec.
a) Design of raising main

Maximum Daily Demand =4.15Mld= 4150 m^3/day
Length of Raising Main =880m
Period of pumping =4hrs
Avg. Daily Demand =2305665lit/day

Discharge through the pipe =Avg. Daily Demand × 24/period of pumping
= 2305665×24/4
=13.83×106lit/dsay
=13.83×106 /103×24×60×60=0.160cumecs

Length of Raising main =880m
H_s =10.0m

b) Delivery head

Elevation of rising main
H_d =Lowest point level of reservoir – RL of rising main @ pump house
Total head lift = (H_s+H_d) =10+10.8 = 20.8m
Let’s use the most economical diameter for pipe= 0.97 to 1.22$(Q)^{1/2}$

Q = 0.160cumecs
D= 1.22× $(0.160)^{1/2}$ = 0.50m
D = 500mm
Head Loss HL= $(fLv^2)/2gd$

Where, HL= Head loss in meters
L= Length of raising main in meters
d = Diameter of pipe in metres
V = Mean velocity of flow through pipe in m/sec
g = Acceleration due to gravity in m/s^2
H_L = (0.04×880×1.22)/ (2×9.81×0.50)
H_L = 5.16m

Assume other loses in fittings @ 10% =0.313 HL =5.16m
Total head =H_S + H_d + H_L =20.8+5.16=25.96m
H =26.00 m

Power required to the pumps giving 0.16 m^3/sec at a head of 26.00m
B.H.P =ϒW×Q×H/η

Where, ϒW = Unit weight of water in kN/m^3
Q = Discharge to be pumped in m^3/sec
η = Efficiency of the pump
H = Total head of water in metres = 9.81×0.16×26.00/ (70/100)
H = 42.85Kw =57.46 HP =58HP
(Hence provide 5 pumps of 10HP and 1pump 8HP, with one additional pump of equal capacity as stand by for repairs etc.)

2.10 Pumps

In very few water supply schemes, the originate from mountainous areas such that the consumers get water supply entirely by gravity in majority of cases, pumping is required to lift the water from a river, lake or reservoir to the treatment plant. After the treatment, another lift mains or to the overhead storage tanks from where water may flow under gravity

2.10.1 Design of pump

- Delivery head = 4.80 m
- F = 0.04m
- Pumping hours. = 8 hrs.
- Distance b/w the source & intake& treatment plant L=490 m
- Efficiency of pump = 70%

Assuming velocity through the pipe = V=1.5 m/s A=Q/V

V=0.160/1.5 A=0.11 m^2

$\pi d^2 /4 = 0.11$

$d = (4/\pi \times 0.110)^{1/2}$

d = 0.40m = 40cm

Head loss, $h = f'l\ v^2/2gd$

Where, HL= Head loss in meters

L= Length of raising main in meters

d = Diameter of pipe in metres

V = Mean velocity of flow through pipe in m/sec

g = Acceleration due to gravity in m/s^2

$= 0.04\times490\times1.5^2/2\times9.81\times0.40 = 5.62$ m

$H = H_s+H_d+h$

Where, H_s = Suction lift in meters

H_d = Delivery head in meters

H_L = Head lost due to friction in meters

H = 10m+4.80m+5.62 =21 m

Water horse power of pump

WHP = ΥW QH/0.735

$= 9.81\times0.160\times21/0.735$

WHP = 45 HP

BHP=WHP/η =45/ (70/100)

BHP= 64 HP

Provide 6 pumps each of 10HP and 1pump 4HP

Five Running, one backup in case of emergency

2.11 Design of treatment plant units

2.11.1 Screens

Screens are provided before the intake works, water when derived from the surface sources, may contain suspended matter which may range from floating debris such as sticks, branches, leaves etc. to fine particles such as sand, silt etc., Causing turbidity. Screens serve as a protective device for the remainder of the plant rather than as a treatment process.

2.11.2 Design of screens: -

Peak flow = 6.22 MLD

= [6.22∗10^6/1000] x (24*60*60)

=0.072m^3/sec

Assuming the velocity (0.8 to 1m/sec) through screens = 0.8m/sec The net area of screen openings required = Q/V

A = 0.072/0.8 = 0.090

A = 0.1m

2.12 Prechlorination

2.12.1 Chlorination unit

Disinfection of treatment plant effluent is done to further reduce the pathogens. Chemical oxidants such as chlorine, ozone and hydrogen peroxide are used for disinfection. In this design chlorine is used as disinfectant.

a. Chlorine is very effective in destroying pathogenic bacteria on contact of low concentration and it remains effective for a long period.
b. It is easily available and has low cost.

Importance of chlorination:

a. Chlorination assists in the formation of floc in coagulation process together with other Chemicals.
b. Chlorination assists the process of treatment of some type of industrial waste.
c. Chlorination controls foaming in sludge digestion tank and controls the odour by the prevention of formation of hydrogen sulphide.
d. Chlorination controls possible fly nuisance due to sewage.
e. Chlorination increases efficiency of sewage treatment units.
f. Chlorination reduces BOD.
g. Chlorination removes oil, grease, etc. from the effluent.

2.13 Disinfection (Chlorine demand)

Data: Population =10174
Demand=135 lpcd
Maximum Daily Demand =4.15Mld

Assuming that the disinfection used in the Bleaching powder having 30% of available chlorine and 0.3 mg/lit of chlorine is for disinfection

Amount of chlorine required/day = 0.3×10-6×4.15×106kg /lit =1.25 kg =1250gm

Since amount of chlorine contained in 100kg of bleaching powder is 30% Amount of bleaching powder required daily =1.25×100/30 =4.160kg =4160gm

Annual consumption of BP=4.160 ×365
=1518.4
=1520kg

60% removal of suspended Solids depending on concentration and characteristics of solids in suspension.

2.14 Design of sedimentation tank

Maximum daily demand = 4.15 MLD
Detention period = 4 hrs.
Velocity of flow = 0.15 m/minute
Quantity of water to be treated during = 4.15*10^6*4/24
D.P of 4 hours = 691666.6
= 0.69 mld
= 0.69*10^6 litres
The capacity of tank required = 690 cum
Velocity = 0.15 m/min

The length of tank required = velocity × Detention period of flow = 0.15×4×60 = 36m
C/s area of the tank required = capacity of tank/ length of the tank = 690/36 =19.50m^2
Assuming the water depth in the tank as 2m
Width of the tank required = Area/depth = 4/2 =2m
Using a free board of 0.5m the overall depth=0.5+2=2.5m Hence, a rectangular sedimentation tank with an overall size of 36m×2m×2.5m can be used.

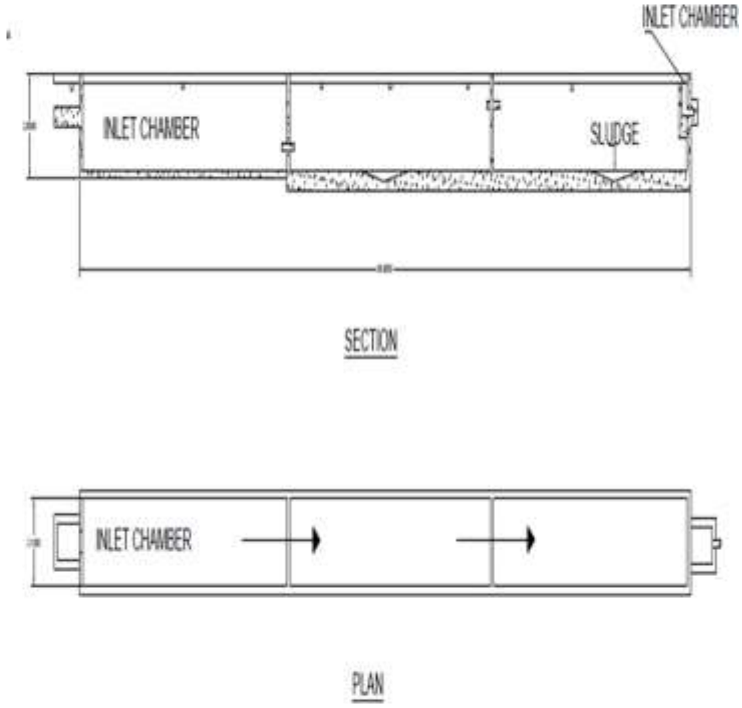

Fig No: 2.3 Sedimentation tank

2.15 Filters

Filtration is one of the most important operations in the water purification process. Though screening and sedimentation remove a large proportion of suspended matter, but they do not effectively remove fine floc particles, colour, dissolved minerals and micro-organisms. In filtration, water is passed through a filter medium in order to remove the particulate matter not previously removed by sedimentation. during the turbidity and colloidal matter of non-settle able type removed.

2.15.1 Design of filters

- Rapid sand gravity filters

Maximum daily demand = 4.15 MLD

Water demand / hrs = 4.15×106/24

=172916 lit/hr.

=173×103lit/hr.

Assume rate of filtration=8000lit/hr./m^2 (3000 to 6000 lit/hr./m^{2}of filter area) Area of the filter bed=water demand/rate of filtration =173×103/4000 = 43.23=44m^2

Since 2 units are to be required Area of each unit =44/2=22m^2

Assuming L=1.5B 1.5B^2=6

B (22/1.5)$^{1/2}$ = 4m L=1.5×4.0=6m

Hence 2 units of size 4.0m ×6.0m are required

One additional unit as standby may also be provided for break down repairs or cleaning operations (The depth of the tank may vary from 2.5 to 3m in order to achieve uniform distribution of water, the area of the filter units should not be kept larger, and is generally limited to about 10 to 80m2 for each unit)

Total = depth +free board =2.0 +0.5=2.5m

Sand layer =0.75m Gravel layer =0.6m (Water 0.6 to 15m, sand 0.75, gravel 0.6)

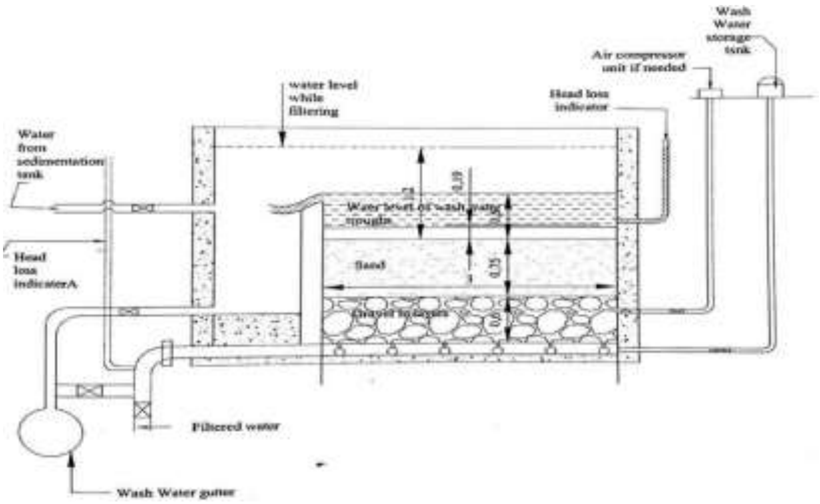

Fig No: 2.4 Typical section of a rapid gravity filter

2.16 Design of clear water reservoir

Distribution reservoir or Storage tanks are used to store the water so that the water is supplied whenever it is required. The main function is to meet the fluctuating demand with a constant rate of supply from the treatment plant.

Treated water stored in clear water reservoir = 415Mld = 4.15×106/103 = 4150 m^3/day

Assume 1/3^{rd} of discharge as a thumb rule for volume of tank /day =1384m^3

Assume depth of tank =5m

c/s area of tank =1384/5 =277 m^2

Providing circular tank, diameter of tank $\pi d^2/4 = 277$

d =18.77m

d= 19m

Assume free board of =0.5 m Total depth of tank = 5.5 m

2.16.1 Design of inlet pipe:

Assuming the average flow velocity of 0.9m/s and daily flow of 8hours

Discharge=4150m^3/ (8×60×60)

Area of inlet pipe required=discharge/velocity

= (4150/ (8×60×60) m^3× (1/.9) m^3/sec =0.16m^2 =16cm^2

Diameter of the inlet pipe = (4×16÷3.141)1/2

D=4.51=5cm

Diameter of the outlet pipe may be taken as 1.5 time that of inlet = 5+3= 8cm

2.17 Distribution system

Distribution system is meant to deliver the adequate amount of treated water to the consumers. In this system, proper networks of pipes are laid, to distribute the water with sufficient quantity and pressure, maintained in the system.

Depending upon the local conditions and orientation of roads, generally there are four different types of pipe networks are laid. They are,

a. Dead end system
b. Grid iron system
c. Ring system
d. Radial system

Each one of these has its own merits and demerits. Depending on the present conditions, for this project, pipe networks of dead end system can be laid, considering the following points,

a. Most of the area in the villages is covered under single main line.
b. There are very few number of sub lanes.

c. There is less number of planned networks of roads.
d. The network can be easily expanded, if any future development occurs.
e. Lesser number of cut-off valves can be provided.
f. Construction is simple, easy and economical.

The flow of water through the networks of pipe can be made possible by the following methods of distribution,

a. Gravitational system
b. Pumping system
c. Combined gravitational and pumping system.

The area under consideration is on the lower grounds, when compared to the source of supply and the treatment plant. The water treated from the treatment plant can be stored at a higher altitude to make the flow of watering the distribution networks, merely under the action of gravity. Also due to the large difference in altitude between the distribution reservoir and the supply points, sufficient head of water can be attained at various points.

2.17.1 Design of distribution system

Using **Manning's formula,** we have

$V=1/N\times R^{2/3}\times S^{1/2}$

Where,

N = Manning's constant

R= Hydraulic mean depth in metres S= Hydraulic gradient

Mains ϕ =150mm Branch ϕ=100mm Laterals ϕ=100mm

V=1m/s

N=0.012=1/0.012× (π/4×d2)/ (π×d)2/3×s1/2 S1/2 =0.012×4/D

S=0.017

S=1:59 Main line=685m

Branch=1.14m=1.2m

$V=1/0.012(150/4)^{2/3}\times (2/100)^{1/2}$

V=83.33×0.14 V=1.3m/sec

Main pipe lines, diameter=150mm

N=0.012

S=1:50 $V = (1/N) \times R^{2/3}\times S^{1/2}= (1/0.012) \times (0.15/4)^{2/3}\times (1/50)^{1/2}$

V=1.3m/sec

Main pipe lines of diameter 150mm having flow velocity of 1.3m/sec is provided with a slope f 1 in 50

2.17.2 Branches and laterals

Using Manning's formula, we have $V=1/N\times R^{2/3}\times S^{1/2}$

Where,

N = Manning's constant

R= Hydraulic mean depth in meters

S= Hydraulic gradient

$V= (1/0.012) \times (0.12/4)^{2/3}\times (1/50)^{1/2}$

V =1.1m/sec

Branch and lateral pipe lines of diameter 100mm having flow velocity 1.1m/sec is provided with a slope of 1 in 50

2.17.3 Water supply lines

The water will be supplied under pressure of 155mm with main pipe diameter 105mm (4 inch), branch pipelines diameter of 76.2mm (3inch), lateral pipe dia-19mm (0.75 inch).

2.18 Waste water treatment

2.18.1 Definitions:

- **Sewage:** sewage indicates the liquid waste from the community. It includes sullage, discharge from latrines, and toilets. Sewage is the mixture of waste products and water that's carried away through the underground drainage systems of pipes knows as sewers. These waste products can consist of such organic and inorganic materials derived from domestic, commercial and industrial waste streams together with storm-water run-off, such as human waste, garbage, mineral salts, floating debris, inert solids, plastics, rags and other debris.
- **Sullage:** It is a term used to indicate waste water from kitchen, bathroom and wash basin.
- **Sanitary sewage:** It is wastewater generated from the domestic and industrial establishments.
- **Sewer:** Sewer is an artificial underground closed conduit in which sewage is carried from the point of generation to discharge or disposal.

2.18.2 Sanitation:

Sanitation literally means measures necessary for improving and protecting health and well-being of the people. Sanitation is any system that promotes proper disposal of human and animal wastes, proper use of toilet, etc. Sanitation basically is a hygienic disposal or recycling of waste. It also considered as a practice that allows protecting health only with the help of hygienic measures. It is also commonly understood as term that is used for treatment of waste water.

Sewage treatment is the process of removing contaminants from wastewater and household sewage, both runoff (effluents), domestic, commercial and institutional. It includes physical, chemical, and biological processes to remove physical, chemical and biological contaminants. Its objective is to produce an environmentally safe fluid waste stream (or eated effluent) and a solid waste (or treated sludge) suitable for disposal or reuse (usually as farm fertilizer) Since treatment of sewage from even small towns along with solid waste management is necessary, the following systems are proposed:

a) Oxidation pond or Aerobic lagoon for sewage treatment
b) Sanitary landfill

2.18.3 Domestic sewage

Domestic sewage carries used water from houses and apartments; it is also called sanitary sewage. Domestic sewage is in the range of 180-200L/head. This is based on normal per capita water consumption of 150L/head/day, plus some leakage and sewer infiltration. Frequently, the domestic flow is taken to include normal commercial discharges from premises such as public houses, restaurants and similar establishments and a total figure of order of 225L/head per day may be appropriate based on major flow surveys.

2.18.4 Basic design considerations

In designing wastewater collection, treatment and disposal systems, planning generally begins from the final disposal point going backwards to give an integrated and optimum design to suit the topography and the available hydraulic head, supplemented by pumping if essential. Once the disposal points are tentatively selected, further design is guided by the following basic design considerations:

1. Engineering
2. Environment
3. Process
4. Cost

2.18.5 Design period

Sewage projects may be designed normally to meet the requirements over a thirty-year period after their completion. The period between design and completion should also be taken into account which should be between three to six years depending on the type and size of project. It is suggested that the construction of sewage treatment plant may be carried out in phases with an initial design period ranging from 5 to 10 years excluding the construction period.

- Design period is estimated based on the following:

Useful life of the component, considering obsolescence, wear, tears, etc.

1. Expandability aspect.
2. Anticipated rate of growth of population, including industrial, commercial developments & migration-immigration.
3. Available resources.
4. Performance of the system during initial period.

Table No 2.2 Design period for components of sewerage system and sewage treatment

Sl. No	Components	Recommended design period	Clarification
1	Collection System i.e. Sewer Network	30	The system should be designed for the prospective of 30 years as replacement is not possible during its use
2	Pumping Stations(Civil works)	30	Duplicating machinery within the pumping station would be easier/cost of civil works will be economical for full design period.
3	Pumping machinery	15	Life of pumping machinery is generally 15 years
4	Sewage Treatment Plant	30	The construction may be in a faced manner as initially the flows may not reach the design levels, and it will be uneconomical to build the full capacity plant initially.
5	Effluent disposal and utilization	30	Provisions of design capacities in the initial stages itself is economical.

2.19 Existing condition of sanitation in India

India has population of almost 1.2 billion people. 55% of this population (nearly 600 million people) has no access to toilets. Most of these numbers are made up by people who live in urban slums and rural areas. A large populace in rural areas still defecates in the open. Slum dwellers in major metropolitan cities, reside along railway tracks and have no access to toilets or a running supply of water. The situation in urban areas in terms of scale is not as serious as rural areas. However, what escalate problems in urban areas is poor sewerage systems and highly congested living.

Sewerage system if present at all, suffer from poor maintenance which offer leads to overflow of raw sewage. Today, cities are highly populated. Over 20 cities have over a million residents, including metropolises of Mumbai, New Delhi and Kolkata. In these places the existing sewerage systems, built to serve population of around 3 million people, can’t handle the wastewater produced by an average of 12-14 million residents.

Wastewater treatment facilities are Inadequate-India does not have enough water to flush out cities effluents, nor have enough sewage treatment plants. A report suggests that only 30% of water used is treated in India. The rest of water makes its own way into streams and rivers inducing major problem-water pollution. According to the country’s tenth 5-year plan, 75% of India’s surface water resources are polluted and 80% of this is due to sewage alone.

Needless to say this has a severe impact on human health. The water pollution aids the transmission of disease like diarrhea and other intestinal infections such as round worm and hook worm. Diarrhea alone accounts for over 5, 35,000 deaths in children under 5 years of age. Polluted water is breeding ground for mosquitoes. Mosquitoes, carriers of disease like Malaria and Dengue fever are responsible for another 3, 00,000 deaths in our country annually.

In addition to health issues, poor sanitary measures set India back by billions of dollars every year. Illnesses are costly to families and to the economy as a whole in terms of productivity losses and expenditures on medicines and health care. The economic repercussions are also evident in other areas like fisheries and tourism which are also hit by water related problems. As per World Bank statistics India's nominal GDP stands at 1.3 trillion dollars and we are currently ranked 11^{th} in the world on basis of nominal GDP. If we could cut down expenses incurred due to illnesses and lack of productivity due to illness, our economy would get the impetus it needs to flourish even more. This in turn would enable governmental agencies to improve sanitation standards and medical infrastructure which would in turn help improve living standards of people. Overcoming the demons of poor sanitation and addressing health issues arising out of the same will surely help us become a global superpower in a holistic sense.

Field procedure

- Levelling along the proposed main center line of the sewer line

Centre line for the sewer is marked from the treatment plant to the pumping station. Using ranging rod, levelling staff and dumpy level point along the centre line at every 10m are marked and staff reading are taken by holding the staff at the points and the levels of each points are reduced by applying Height of instrument method (collimation method) or Rise & Fall method. The survey is carried out by dumpy level. There are two methods of booking and reducing levels of the points from the observed staff reading.

1. Height of instrument method (collimation method)
2. Rise & Fall method

The first method that is Height of instrument is been used for booking and reducing the elevation of points. In this method Height of instrument is calculated for each setting of the instrument station by adding the back sight reading to the elevation of the bench mark (BM) or the change point. The reduced level of the turning point is then calculated by subtracting from the Height of instrument. The beginning point is back sighted at the distribution tank and the intermediate sights are sighted at a chainage of 10m.

2.20 Waste water treatment plant

Per capita sewage produced 80 % of per capita water supplied to the public Peak flow = 80 % Avg. Daily demand

$= 80/100 \times 230566$

$= 1840000$

$= 1840000$lit/day =1.84MLD

$= 1840000/ (24 \times 60 \times 60 \times 10^3)$

$= 21 \times 10^{-3} m^3/sec$

2.20.1 Grit chamber (Aerated)

Grit removal basins, such as grit chambers or grit channels or detritus tanks are the sedimentation basin placed in front of the waste water treatment plant to remove the in organic particles such as sand, gravel, grit, egg shells, bones and other nonputresible material that may clog channels or damage pumps due to abrasions and to prevent their accumulation in sludge digesters.

Many a times, the grit chambers are aerated by providing outside air through compressors. The diffused air creates spiral current within the grit, as to help in its settlement.

2.20.2 Field works carried out

a) Reconnaissance survey

Following information was collected in reconnaissance survey

- Topographical survey: from topo sheets, Google maps.
- Soil conditions: Red soil, hard rock.
- Present utility services like house connections for water supply and sewerage, electric and telephone poles, Cisterns.
- Grit chamber design

Maximum hourly demand = 6225295lit/day 80 % will be waste water

Maximum hourly demand = 4980236 lit/day

Velocity of flow = 0.9

Peak flow = 4980236 lit/day = 4980236/ (24×60×60×10^3) = 0.054 m^3/sec

Assume the peak flow rate as 3 times the average =0.054×3 =0.174 m^3/sec

Assume average Detention period = 3 min

Aerator volume = 0.174× (3×60) =31.32 = 32 m^3

In order to drain the channel periodically for cleaning and maintenance let us use 2 chamber.

Volume of 1 aerated channel =32/2 = 16m3

To determine the dimensions of aerated channel. Assume depth of 1m and width depth ratio as 2:1.

Width of channel = depth×2 =1×2=2m

Length of channel=volume of aerated channel/ (depth × width) =16/ (1×2) = 8m

Length =8m

Increase the length by 20% to account for inlet and outlet condition.

Length = 8× (20/100) =1.60m

Hence use two chambers each of **size 2.0m×1.6m×1m**

2.20.3 Oxidation pond

Stabilization ponds are open flow through earthen basin, specially designed and constructed to treat sewage and bio degradable industrial waste waters. Such ponds provide comparatively long detention periods, extending from a few days to several days during which time the waste get stabilized by action of natural forces. The term oxidation point was originally referred to that stabilization pond which received partially treated sewage. The results of the oxidation point treatment are the oxidation of the original organic matter and production of algae which are discharged effluent this results in a net reduction in BOD.

- **Design of oxidation pond**

Population=17079

Sewage produced=80% demand =80%×135 =108 lpcd

Assume 5day Bio Chemical oxygen demand (BOD) sewage=300mg/l

Quantity of sewage to be treated per day =17079×108 =1844532m^3 =1844ml.

BOD content per day=1844×300 = 553kg

Now assuming organic loading rate in the pond say 300kg/hectare/day Then, surface area=90kg/day/300kg/hectare

=553/300×hectare

= 553/300×$10^4$$m^2$

= 18443m^2

Surface area=18443m^2

Assuming the length of tank as, L=twice of its width B 2B^2=18443

B = $(18443/2)^{1/2}$

B=96m

L=2×B =2×96 = 192m (Depth -1 to 1.5m)

Using a tank with effective depth of 1.2m We have capacity of tank = L×B×h =192×96×1.2

Capacity =22118m^3
Now, DT=capacity/sewage flow =1844532/22118
DT =83.39=84 days
Hence use oxidation pond with length of 78m, width 39m, overall depth=1.2+1m and DP of 84 days.

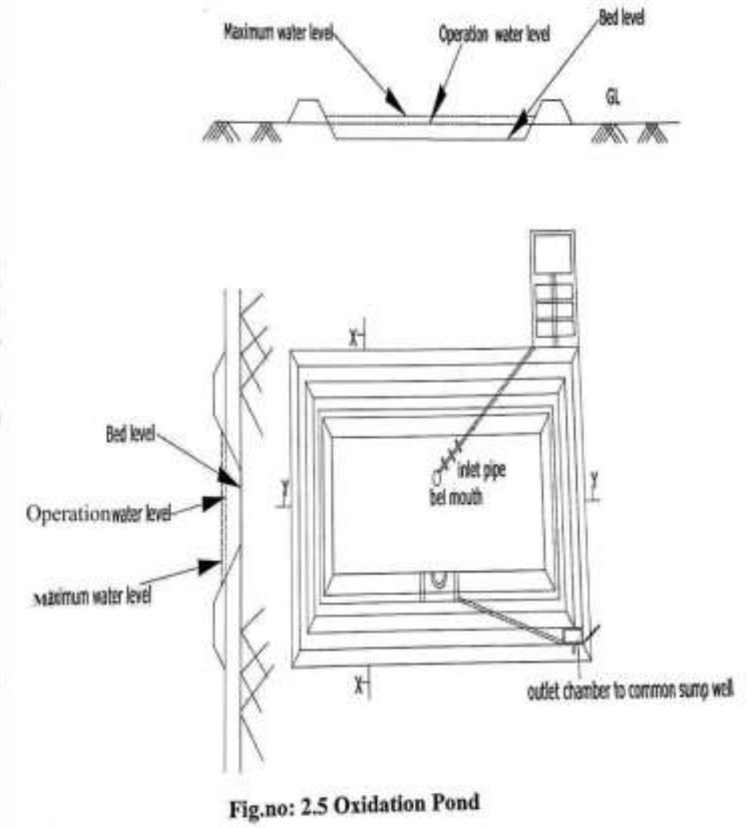

Fig. No. 2.5 Oxidation pond

Disposal:
After water is collected and treated it is disposed to nearby water source.

2.21 Solid waste management

- **Solid waste**

The term solid waste includes all those solid and semi-solid materials that are discarded by a community. The solid waste generated through domestic and commercial activities is classified as municipal solid waste and is also called 'REFUSE'.
In general, the solid wastes are usually divided into following two categories;

- Municipal solid waste
- Industrial solid waste

The safe disposal of solid wastes of a society was not a serious problem, as long as the population was small and the land available for assimilation of waste was large

- **Municipal solid waste**

Composition and quantity of the generated municipal solid waste or Refuse:

The municipal solid waste is a heterogeneous mixture of various kinds of solid waste which are not transported with water as sewage, and may include biodegradable food waste called garbage, and the non-putrescible solid waste like paper, glass, rags, metal items etc. called rubbish.

The quantity of municipal solid waste produced by a society depends upon the living standards and its residents.

2.21.1 Solid waste management

Solid waste management is the discipline associated with the control of waste generation, its storage, collection, transfer, and transport, processing and disposal in a manner that in accordance with best principles of public health, economics, engineering, conservation aesthetics, public attitude and environmental considerations.

2.21.2 Functional elements of solid waste management

The activities associated with management of solid waste from point of generation to final disposal have been grouped into 6 functional elements.

1) Waste generation:
2) On-site handling, storage, and processing:
3) Collection:
4) Transfer and transport:
5) Processing and recovery: and
6) Disposal

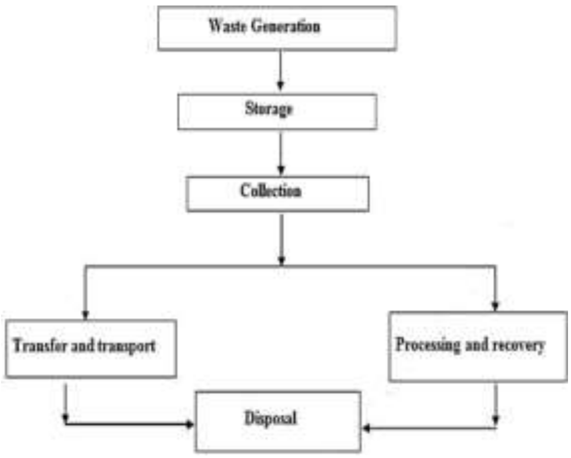

Fig. No. 2.6 Interrelationship of Functional Elements Comprising a Solid Waste Management System

Table.No.2.3: Description of the functional elements of solid waste management.

Functional element	Description
Waste generation	Those activities in which materials are identified as no longer being of value and are either thrown away or gathered together for disposal
On- site handling, Storage, and processing	Those activities associated with the handling, storage, and processing of solid wastes at or near the point of generation
Collection	Those activities associated with the gathering of solid wastes and the hauling of wastes after collection to the Location where the collection vehicle is emptied.
Transfer and transport	Those activates associated with (1) the transfer of wastes from the smaller collection vehicle to the larger transport equipment and (2) the subsequent transport of the wastes, Usually over long distance to the disposal site.

Processing and recovery	Those techniques equipment and facilities used both to improve the efficiency of the other functional elements and to recover usable materials, conversion products, or energy From solid wastes.
Disposal	Those activities associated with ultimate disposal of solid wastes including those wastes collected and transported directly to a landfill site, semisolid wastes (sludge) from wastewater treatment plants incinerator residue compost.

For the onsite storage of solid waste generated in selected area, stationary receptacles such as open masonry enclosures, concrete pipe sections are used. For collection of solid waste, curb side collection method is used. The system of collection used is stationary container system is used. For transportation motor vehicle transportation is used and finally the collected waste is 1disposed of by ordinary open dumping in low lying areas situated near by the village.

CHAPTER-3

HIGHWAY PROJECT

3.1 Introduction

Transportation contributes to economic, industrial, social & cultural development of any country. Transportation is a vital infrastructure for the speedy economic growth of a developing country, since every commodity produced needs transport at all stages from production to distribution. Highways have been rightly compared to the arteries of a human being and their importance in the social and economic up lift of a nation cannot be over emphasized. In the present era planning is considered as a pre-requisite and basic need for any new project or an expansion program. Thus highway planning is also a basic need for highway development.

3.2 Necessity of Highway Planning

In the present era, planning is considered as a pre-requisite before attempting any development programme. This is particularly true for any engineering work, as planning is the basic requirement for any new project. Thus highway planning is a basic need for highway development. Planning plays an important role when the funds available are limited whereas the total requirement is much higher. By proper planning we can execute the work for available funds or we can decide whether to take up project or not. This is the problem in developing country like India as the best utilization of available funds has to be made in a systematic and planned way.

The objectives of highway planning are given below,

1. To plan a road network for efficient and safe traffic operation, but at minimum cost.
2. Attainment of maximum utility.
3. Construction with locally available resources to minimize the cost of project.
4. Future requirements and improvements in view of anticipated developments.
5. Incorporation of technical recommendations framed by Indian Roads Congress (IRC) for various aspect of Highway construction.

3.3 Proposal

Highway project deals with the design of road and can be executed in following manner,

1) An alignment is selected and detailed surveys including cross section and longitudinal sections are carried out.
2) Plan of road alignment –longitudinal section is drawn to scale and final alignment is fixed, as per IRC specifications, confirming to class of roads.
3) Block levelling for the culvert.

3.4 Highway Alignment

The position or the layout of the central line of the highway on the ground is called the alignment. Horizontal alignment includes straight and curved paths. Vertical alignment includes level and gradients.

A new road should have aligned very carefully as improper alignment would result in following disadvantages,

- Increase in construction cost
- Increase in maintenance cost
- Increase in vehicle operating cost
- Increase in accident rate.

Once an alignment is fixed and constructed, it is not easy to change it due to increase in cost of adjoining land and construction of costly structures by the roadside.

3.4.1 Requirements of ideal alignment

The basic requirements of an ideal alignment between two terminal stations are that it should be:

1) The alignment should be short as possible.
2) The road should be easy for construction & maintenance.
3) The road should be safe.
4) The road should be economical

3.4.2 Factors Controlling Alignment: The various factors which control the highway alignment in general may be listed as follows,

1. Obligatory Points

These are the control points governing the alignment of the highways. These control points are divided into two categories, namely

i. Obligatory points through which the road alignment should pass. These will cause the alignment to often deviate from the shortest or easiest path.
ii. Points through which the alignment should not pass. These make it necessary to deviate from the proposed shortest alignment.

2. Traffic

The proposed alignment should suit the traffic requirements. Origin and destination study should be carried out in the area and desire lines be drawn showing the trend of traffic flow. The new road to be aligned should keep in view the traffic flow patterns and future trends.

3. Geometric design

Geometric design factors such as gradients, radius of curve and sight distance also would govern final alignment of highway. If straight alignment is aimed at, often it may be necessary to provide very steep gradients. As far as possible while aligning a new road, the gradient should be flat and less than the ruling or design gradient. Thus it may be necessary to change the alignment in view of design speed, maximum allowable super elevation and coefficient of lateral friction. It may be necessary to make the adjustment in the horizontal alignment of roads keeping view the minimum radius of the curve and the transition curves.

4. Economy

The alignment should be economical. The initial cost, maintenance cost & vehicle operation cost should be minimum, high embankment or deep cuttings are avoided & choose the balance cutting & filling. These factors also control the alignments of road.

3.5 Engineering Surveys for Highway Location

Before a highway alignment is finalized in highway project, the engineering surveys are to be carried out. The stages of engineering surveys are.

3.5.1 Map Study

Map study gives a rough guidance of routes to be surveyed in the field. The main features like river, hills and valleys, etc. are known by map study. By careful study of maps, the idea of aligning a new highway can be obtained.

3.5.2 Reconnaissance Survey

In this survey, the land along the various proposed highway routes are inspected. All the relevant details not available in the map study are collected and noted down.

3.5.3 Preliminary Survey

This is carried out to collect all the physical information's, which is necessary in connection with the proposed highway alignment. The quantity of earth work and cost of construction are worked out. The best proposal is selected after preliminary survey.

3.5.4 Detailed Survey

After preliminary survey, a detailed survey is carried out. Here Temporary Bench Marks are fixed and levelling works are performed. Here an elaborate and complete data are collected for preparing detailed plan and estimates of the project.

3.6 Surveying Details

In road survey, the first step is levelling. This is carried out using, instruments such as dumpy level, cross staff and levelling staff. Other instruments used are prismatic compass to note the bearings, arrows and ranging rods which are used to mark the points on the field. The levelling operation starts from the benchmark.

A benchmark is a point of known elevation. The road is aligned by two operations, namely,

1. Longitudinal Sectioning.
2. Cross Sectioning

3.6.1 Longitudinal Sectioning or Profile Levelling

Profile levelling is a process of determining the elevation of points at fixed intervals along the chain line. Here the line along which the section to be taken is marked by ranging rods, and the fore bearing of the line is taken with the help of a prismatic compass. The level is then setup at a point. The telescope is then directed to a staff, held on the temporary benchmark of RL 100.00m and the reading is taken. This reading is called as the back sight. Height of collimation is determined. All these readings are noted down in a level book. Then the intermediate sight is taken on the starting point of the line by holding the staff. For each setup, intermediate sights should be taken after the fore sight on the next turning station has been taken. To find the R.L. the intermediate sights are subtracted from the height of collimation. When the instrument is removed, a change point is selected and a staff is held on the same point and read it, which is fore sight.

It is then subtracted from the height of instrument to find the R.L. of the change point. The instrument is then transferred to the second position. Having adjusted the instrument, a back sight is taken on the change point just established. This reading when added to the change point gives the R.L. of the new line of collimation. Then successive intermediate sights are taken. The horizontal distances are plotted along the horizontal axis to some convenient scale and the distances are also marked. The elevations are plotted along the vertical axis. The various points obtained are joined by straight lines.

3.6.2 Checking the levels

For checking the levels, we use the technique called Fly Levelling. In this, we start from the last point and go to the first point, taking back sights and fore sights only, At the end, the last point should be in the vicinity of the bench mark. 3.6.3 Cross Sectioning Cross sections arc run at right angles to the horizontal profile and at either side of it for the purpose of lateral outline of the, round surface. It provides data for estimating quantities of earth work and other purposes. The cross sections are plotted in the same manner as the longitudinal sections.

3.7 Contouring

On a plan, the relative altitudes of the points can be represented by contour lines as they indicate the elevators directly. The area to be surveyed is divided into a number of squares. The levels on the comer of these squares are determined by direct levelling. The contour interpolation is done by graphical methods or by arithmetic calculation method.

3.8 Geometric Design

The geometric design of a highway deals with the dimensions & layout of visible features of highway such as alignment, sight distance, curves, super elevation & intersections etc. The geometrics of highway should be designed to provide optimum efficiency in traffic operation with maximum safety at reasonable cost. The designer may be exposed to planning of new highway network to meet the requirements of the anticipated traffic. The design also includes the sizes of drainage, aggregates, cross slopes, super elevation etc.

3.8.1 Terrain Classification

Topography and physical features play an important role in the location and design of a highway. The various design elements should be related to topographical features if an economical and sound design is to emerge. The classification of the terrain is normally done by means of the cross slope of the country. Terrain Classification is done according to Road User Cost Study.

Table 3.1: Terrain Classification

Sl. No.	Terrain Classification	Rise and Fall(m / km)
1	Plain	0-15
2	Rolling	16 – 30
3	Hilly	Over 31

3.8.2 Design Speed

“Design speed” is a speed determined for design and correlation of the physical features of a highway that influence vehicle operation. It is the speed to which a road is designed. It is the maximum safe speed that can be maintained over a specific section of a highway when conditions are favourable that the design features of the highway govern.

Table 3.2: Design speeds on rural highways

Road Classification	Design speed in Kmph for various terrains							
	Plain		Rolling		Mountainous		Steep	
	Rolling	Min	Rolling	Min	Rolling	Min	Rolling	Min
NH & SH	100	80	80	65	50	40	40	30
MDR	80	65	65	50	40	30	30	20
ODR	65	50	50	40	30	25	25	20
VR	50	40	40	35	25	20	25	20

3.8.3 Camber

Cross slope or camber is the slope provided to the road surface in the transverse direction to drain off the rainwater from the road pavement surface. It usually maximum at center of the road & at edge camber value is zero in a stretch.

Table 3.3: IRC recommended values of camber

SL. No.	Types of road Surfaces	Range of cambers in areas of rainfall range	
		Heavy	Light
1.	Cement concrete and high type bituminous surface	1 in 50 (2%)	1 in 60 (1.7%)
2.	Thin bituminous surfaces	1 in 40 (2.5%)	1 in 50 (2%)
3.	WBM and gravel pavement	1 in 33(3%)	1 in 40 (2.5%)

4.	Earth	1 in 25(4%)	1 in 33(3%)

3.8.4 Width of Roadway or Formation

It is the sum of widths of pavements or carriageway including separators if any and shoulders. Formation width is the top width of the highway embankment or the bottom width of the highway cutting excluding the side drains.

Table 3.4: Width of roadway recommended by IRC

Sl. No.	Road Classification	Width of roadway	
		Plain and Rolling terrain	Mountainous and Steep terrain
1.	NH & SH (a) Single lane (b) Two lane	12.0 12.0	6.25 8.80
2.	M D R (a) Single lane (b) Two lane	9.0 9.0	4.75 --
3.	O D R (a) Single lane (b) Two lane	7.5 9.0	4.75 --
4.	V R – single lane	7.5	4.0

3.8.5 Width of Pavement

The pavement or carriageway width depends on the width of traffic lane and number of lanes. The carriageway intended for one line of traffic movement is called a traffic lane. The lane width is determined on the basis of the width of the vehicle and the minimum side clearance which may be provided for the safety.

- When the side clearance is increased there is an increase in operating speed of vehicles and hence increases in capacity of the traffic lane.
- By considering these in view, a width of 3.75 m is considered durable for a road having single lane for vehicles of maximum width 2.44 m.
- For pavements having two or more lanes, width of 3.5 m per lane is considered sufficient.

3.8.6 Right of Way

Right of way is the area of land acquired for the road, along its alignment. The width of this acquired land is known as land width and it depends on the importance of the road and possible future development. A minimum land width has been prescribed for each category of road. A desirable range of land width has also been suggested for each category. While acquiring land for a highway it is desirable to acquire more width of land as the cost of adjoining land invariably increases very much, soon after the new highway is constructed.

Table 3.5: Recommended land width for different classes of roads (m)

Sl. No.	Road Classification	Plain & Rolling terrain		Mountainous and Steep terrain	
		Open areas	Built – up areas	Open areas	Built – up areas
1.	NH & SH	45	30	24	20
2.	M D R	25	20	18	15
3.	O D R	15	15	15	12
4.	V R	12	10	9	9

3.8.7 Shoulders:

It is the thin strip of land provided along the road edge for the emergency lane for parking or to repair the damaged vehicles. The minimum shoulder width recommended by IRC is 2.5m.

3.8.8 Sight Distance

The safe and efficient operation of vehicles on the road depends very much on the visibility of the road ahead of the driver. Thus the geometric design of the road should be done such that any obstruction on the road length could be visible to the driver from some distance ahead. This distance is said to be the sight distance.

Sight distance available from a point is the actual distance along the road surface, over which a driver from a specified height above the carriage way has visibility of stationary or moving objects.

Three sight distance situations are considered for design:

1. Stopping sight distance (SSD) or absolute minimum sight distance.
2. Safe overtaking or passing sight distance, and
3. Safe sight distance for entering into uncontrolled intersections.

1. Stopping Sight Distance (SSD)

Stopping sight distance (SSD) is the minimum sight distance available on a highway at any spot having sufficient length to stop a vehicle traveling at design speed, safely without collision with any other obstruction for the purpose of measuring the SSD, IRC has suggested the height of eye level of driver as 1.2 m and the height of the object as 0.15 m above the road surface.

2. Overtaking Sight Distance

The overtaking sight distance is the minimum distance open to the vision of the driver of a vehicle intending to overtake the slow vehicle ahead safely against the traffic in the opposite direction.

The overtaking sight distance or passing sight distance is measured along the center line of the road over which a driver with his eye level 1.2m above the road surface can see the top of an object 1.2 m above the road surface.

3.8.9 Design of horizontal alignment

1. **Horizontal curves**

When the centreline of the road changes the direction along the horizontal plane, horizontal curves are provided & the same are designed as follows.

2. **Super Elevation**

To counteract the effect of centrifugal force and to reduce the tendency of vehicle to overturn or skid the, outer edge of pavement is raised with respect to inner edge. Such provision of transverse slope is provided on horizontal curves.

3. **Extra Widening**

Extra widening is provided to provide extra space required for mechanical and psychological reasons along the horizontal curve, which is provided as per the table below.

Table 3.6: Extra Widening

SL. No.	Radius of curve (m)	Extra width for two lane road (m)
1	Up to 20m	1.5
2	20-40m	1.5
3	41-60m	1.2
4	61-100m	0.9
5	101-300m	0.6

6	Above 300m	Nil

4. **Horizontal Transition Curve**

A transition curve is introduced between the straight and a circular curve. It has a radius, which decreases from infinity at the tangent point to a designed radius of circular curve. The rate of change of radius of transition curve will depend on the equation of the curve or its shape.

5. **Set Back Distance**

Requisite sight distance should be available across the inner side of the horizontal curve. Lack of the horizontal curve, Lack of visibility in the lateral direction may arise due to obstruction, like wall hill cut, wooded area, high slope etc. the sight distance is measured along the middle of the inner lane. However, on single lane road, sight distance is measured along the centre of the carriageway. The setback distance depends on following factors;

i. Required sight distance.
ii. Radius of horizontal curve R.
iii. Length of curve, L_c which is greater or lesser than S.

I) When $L_c > S$

For narrow roads, such as single lane, m= R – R cos $\frac{\alpha}{2}$

Where, $\frac{\alpha}{2} = \frac{1 \times S}{2 \times \pi \times R}$ degrees

For wide roads, such as two or more lanes, m= R – (R- d) cos

Where, $\frac{\alpha}{2} = \frac{1 \times S}{2 \times \pi \times (R-d)}$ degrees

II) When $L_c < S$

m= R – (R- d) cos $\frac{\alpha}{2}$+( ) + $\frac{S - Lc}{2 \times \pi \times (R-d)} \frac{\alpha}{2}$

Where, m= set back distance.

α = central angle.

L_c =length of the curve.

S=sight distance.

R=radius in m,

d= centre line of road and the inside lane, m.

3.8.10 Design of Vertical Alignment

i. Gradient:

It is the rate of rise or fall along the length of the road with respect to the horizontal. I while aligning a highway, the gradient is decided for designing the vertical curve. Very steep gradients are avoided as it is not only difficult to climb the grade, but also the vehicle operation cost is increased. These are divided into four types.

Table3.7: Gradient according to IRC

Terrain	Ruling Gradient	Limiting Gradient	Exceptional gradient
Plain and rolling	3.3% (1 in 30)	5% (1 in 20)	6.7%(1 in 15)
Mountainous having elevation >3000m above MSL	5%(1in 20)	6%(1 in 16.7)	7%(1 in 14.3)
Steep terrain up to 3000m above MSL	6%(1 in 16.7)	7%(1 in 14.3)	8%(1 in 12.5)

a. **Ruling gradient:** It is the maximum gradient within which the designer attempts to design the vertical profile of the road.
b. **Limiting gradient:** It is the gradient steeper than the ruling gradient.

c. **Exceptional gradient:** It is the unavoidable steeper gradient provided for a smaller road stretch of the road.
d. **Minimum gradient:** It is the minimum gradient provided in the roads for the drainage of rain water as quick as possible. It will depend on the soil property, rainfall, runoff, etc.

3.8.11 Vertical Curve

Due to changes in grade in the vertical alignment of highway, it is necessary to introduce vertical curve at the intersections of different grades to smoothen out the vertical profile and thus ease off the changes in gradients for the fast moving vehicles.

The vertical curves used in highway may be classified in to two categories.

i. Summit curve or crest curves.
ii. Valley curve or sag curves.

1. **Summit curve:**

A curve with convexity up wards is called a summit curve. This occurs when an ascending intersects a descending gradient or when an ascending meets on other ascending gradient or an ascending gradient meeting a horizontal a summit curve is provided here as there is change in gradient matching the requirements of a summit curve.

2. **Valley Curve:**

A vertical curve, concave upwards is called as valley curve. This is formed when a descending gradient intersect an ascending gradient or when a descending gradient meets another descending gradient or when a descending gradient gains a horizontal path, they should be designed for:

i. Comfort condition
ii. Head light sight distance condition.

3.8.12 Pavement Design (IRC: SP 20-2002)

For the safety and comfort ability of fast vehicles road surface should be even along the longitudinal profile. The surface should also so be stable and unyielding in different conditions and it should allow the heavy load of traffic to move with least possible resistance. In order to provide a stable and even surface for traffic the road way is provided with a suitably designed and constructed pavement structure. Pavement is designed and to distribute wheel load in larger area and elastic deformation caused by the load to fall within the permissible limit.

The thickness of pavement is designed on the basis of projected number of commercial vehicles for the design life using the current commercial vehicles per day and its growth rate. Further, it requires the subgrade strength value in terms of CBR. It is expected that rural road will not have more than 450 CVPD in any case. The design chart may be referred to obtain the total pavement crust thickness required over the subgrade for the design life of pavement. Based on strength of granular materials that are used, the total design thickness is divided into base and subbase thickness.

3.9 Different design aspects

3.9.1 Terrain classification according to road user study

Average rise and fall (RF) of is given by;

$TC = h_1+h_2+h_{3\ldots\ldots\ldots}+h_m + h_n/\text{distance AB (km)}$

TC= 29 m

The terrain obtained is a **Rolling terrain**

3.9.2 Design speed

Design speed is selected according to class of road and type of terrain from table 3.2 (IRC: SP 20-2002)

There is a new proposed village road with construction of both earthen pavement and flexible pavement throughout the alignment. The design speed is 40kmph.

3.9.3 Stopping Sight Distance

SSD = (0.278Vt) + (V^2 /254f)

Where, V = design speed in kmph =40kmph.

S.S. D= Stopping Sight Distance.

t = reaction time of the driver in seconds = 2.5sec

f= longitudinal frictional coefficient = 0.4for 40kmph.

Then,

SSD = (0.278Vt) + (V^2 /254f) = (0.278*40*2.5) +(40^2/254*0.4)

= 101.25 ≈ 102 m = 43.54m

3.9.4 Overtaking Sight Distance (OSD)

Speed of overtaking vehicle V_a=40kmph

Speed of overtaken vehicle V_b= 24kmph

Therefore, V_a= 40/3.6 = 11.11 m/sec

V_b=24/3.6 = 6.670 m/sec

Avg. acceleration during overtaking vehicle, a = 1.11 m/sec2

Over taking sight distance (OSD) for two-way traffic

OSD = d1+d2+d3 = vbt+vbT+2S+vaT

d1 = Vbt = 6.67*2.5 =16.675 m

d2 =Vb*T+ 2S

S= 0.7Vb+6 = 0.7*6.67+6= 10.67m

T= √ (4S/a) = √ (4*10.67/1.11) = 6.20

d2= (6.67*6.20) + (2*10.67) = 62.7 m

d3= Va*T = 11.11*6.20 = 68.90m

Therefore, OSD = d1+d2+d3

OSD =16.675+62.70+68.90= 148.275m ≈ 150 m

Minimum length of overtaking zone

Min (OSD) = 3*OSD = 3*150 = 450 m.

Desirable length of overtaking zone

OSD (Max) = 5*OSD = 5*150= 750 m.

3.9.5 Radius Horizontal Curve Design

Road design speed =40kmph

Ruling minimum radius of the curve of ruling speed of 80kmph is given by

R_{ruling}= V^2/ 127(e+f)

Where,

e= Rate of super elevation

f= Design value for lateral friction coefficient = 0.15

v= Speed of vehicle m/sec

R= Radius of horizontal curve

g= acceleration due to gravity= 9.81 m/sec^2

e=According to IRC recommendations, maximum limit of super elevation 'e' in plain and rolling terrain is 7%

f= Maximum value of transverse skid resistance T force, for design purpose is 0.15

R_{ruling}= V^2/ (127(e+f)) = 40^2/ (127(0.07+0.15)) = 57.26 m ≈57 m

3.9.6 Design of Super Elevation:

The super elevation for 75% of design speed is neglecting the friction

e = V^2 /225R

$e = 40^2/225*57$

e= 0.124> 0.07

As the value of 'e' is 0. 124, which is greater than maximum limit of 0.07 it is not safe for speed of 80km/hr. Check for transverse skidding developed

$f = (V^2/ 127R)-e = (80^2/127*90)-0.07= 0.48$

The value of f obtained is slightly less than the allowable limit; the provided super elevation is safe against sliding of vehicles moving with design speed.

3.9.7 Extra Widening:

The extra widening required at the curve is given by,

We = Wm + Wps

$We= (nl^2/2R) + (V/9.5R^{0.5})$

Where,

n= number of traffic lanes.

l = Length of wheel base of longest vehicle, 6m.

V= design speed, kmph.

R=radius of horizontal curve.

$We= ((1*6^2)/(2*57))+(40/(9.5*57^{0.5}) =0.86m$

3.9.8 Design of Transition Curve:

1) Length of transition curve by considering the rate of change of acceleration.
   Length of transition curve is given by, $Ls = V^3 / (46.5CR)$
   Rate of change of centrifugal acceleration(C) is given by,
   C = 80 / (75+V) = 80/ (75+40) = 0.69m/sec³
   Therefore,
   $Ls = V^3 / (46.5CR) = 40^3/ (46.5*0.69*57) = 34.99$ m
2) Length of transition curve by considering the rate of introduction of super elevation.
   Considering that the road rotates with respect to inner edge.
   Ls = eN (W+We)

Where,

e =0.07

N= rate of change of super elevation is 150(Which is b/n 150 and 60, as per IRC 73-980-page no. 25)

W= width of carriage way =3.75 m

I = length of wheel base = 6m.

The extra widening required at the curve is given by,

$We = Wm + Wps= (nl^2/2R) + (V/9.5R^{0.5}) = ((1*6^2)/ (2*57)) +(40/ (9.5*5^{0.5})$

We=0.87 m.

Then,

L_s = eN (W+We) =0.07x 150(3.75+0.87) = 48.51m

3) Length of transition curve according to IRC. Length of horizontal transition curve is given by,
   $L_s = (2.7V^2 /R) = (2.7*40^2)/57=75.78m$
   The length of transition curve is highest of the above three values, therefore
   L_s=75.78m

3.9.9 Set Back Distance:

Given

S=m (SSD)

L_c= 42 m

R=57m

∴ L c> S

When L c> S
When length of curve Lc is greater than the sight distance

S = sight distance
R = radius of horizontal curve.
Lc= length of the curve.
D=distance b/w the center line of the road and the center line of the inside lane in.

The sight distance is measured along the middle of the inner side lane and the setback distance in m is given by

$\alpha/2 = 180S/(2\pi(R-d))$ degree
$\alpha/2 = 180*43.54/(2\pi\ (57-1.875)) = 22.62°$(degree)
$m=R-(R-d)\cos\alpha/2 = 57-(57-0.9375)\cos(22.62) = 5.25m \approx 6m$

Therefore, a setback distance of 6m on the inner side of the horizontal curve.

3.9.10 Design of Vertical Curve

1) **Length of Summit Curve**

Two cases are to be considered in deciding the length.

- When length of the curve is greater the sight distance [L>SSD].
- When length of the curve is less than sight distance [L<SSD].

Assuming L<SSD
L= $NS^2/4.4$
Where,

L= length of summit curve, m
S=stopping sight distance
N=deviation angle
H= height of eye level of driver above road way surface m =1.2m
h=height of the object above pavement surface m=0.125m
L= $0.08X\ 43.54^2 / (4.4) = 34.46$ m.

2) **Length of Valley Curve:**

Total Length of valley curve is given by;
L= $0.38\ (NV^3)\ ½ = 0.38\ (0.08 * 40^3)^{1/2} = 27.19$ m.

3.9.11 Curve Setting

Chainage (PI) = 630 m
Deflection angle Δ =340
Radius of curve = 300m
Peg interval = 10m
Tangent length (T) = R tan (Δ/2) = 300 tan (34/2)
T = 91.71m.
L= RΔ/180.
L= (3.1415 X300X34)/180= 178.02 m say 178m.

Chainage of the PC (T1) = chainage of PI –T= 630-91.71= 538.29 m.
Chainage of PT (T2) =chainage of T1+L = 538.29+178 = 716.29 m
Length of 1stsub chord, C= 540-538.29 = 1.71 m.
Length of last chord = C1 = 716.29-710 = 6.29 m
Number of normal chord = (710-540)/10= 17numbers.
Total number of chords = 1+17+1 =19

$\propto_1 = 1718.9C/R = (1718.9*1.71/300)/60 = 0°\ 9'47.86''$

$\propto_2$ to $\propto_{18} = 1718.9*10/300 = 0°57'17.8''$

$\propto_{19} = 1718.9 *6.29/300 = 0°36'\ 2.38''$

Δ_n= 16°59' 52.84"

3.9.12 Curve Setting Details

Table3.8: Curve Setting Details

Point	Chainage (M)	Chord length (M)	Tangential angle(δ)	Deflection angle (Δ)	Theodolite reading
T_1	538.29	-	-	-	-
A	540	1.71	0° 9' 47.86	0° 9' 47.86	0°9'
B	550	10	0° 57′ 17.8"	1° 7'5.66"	1°7'
C	560	10	0° 57′ 17.8"	2° 4'22.46"	2°4'
D	570	10	0° 57′ 17.8"	3° 1' 41.26"	3° 1'
E	580	10	0° 57′ 17.8"	3°58' 59.06"	4° 1'
F	590	10	0° 57′ 17.8"	4°56' 16.56"	5° 0'
G	600	10	0° 57′ 17.8"	5°53' 34.66"	6° 0'
H	610	10	0° 57′ 17.8"	6°50' 52.46"	7° 0'
I	620	10	0° 57′ 17.8"	7°48' 10.26"	7° 40'
J	630	10	0° 57′ 17.8"	8°45' 28.06"	8° 40'
K	640	10	0° 57′ 17.8"	9°42' 45.86"	9° 40'
L	650	10	0° 57′ 17.8"	10°40' 3.66"	10° 40'
M	660	10	0° 57′ 17.8"	11°37' 21.46"	11° 40'
N	670	10	0° 57′ 17.8"	12°34' 39.26"	12° 40'
O	680	10	0° 57′ 17.8"	13°31' 57.06"	13° 40'
P	690	10	0° 57′ 17.8"	14°29' 14.56"	14° 20'
Q	700	10	0° 57′ 17.8"	15°26' 32.66"	15° 20'
R	710	10	0° 57′ 17.8"	16°23' 50.46"	16° 20'
T_2	716.29	6.29	0° 36' 2.38"	16°59' 52.84"	17° 0'

Check: $\frac{\Delta}{2} = \frac{34}{2} = 17^{\circ}$

3.9.13 Design of Culvert

The pipes, which are to be provided, are non-pressure (IS-458) of MP3 grades.
The mean amount of rain fall = 969.6 mm =97 cm
The rain fall of a bad year is always taken 2/3 of mean amount of rain fall

Therefore, bad year rain fall is 2/3 *97= 64.67 cm
Run off co efficient usually assumed as 15% to 30%

Annual yield = (30/100) *64.67 = 19.39 ~ 20cm
Yield from catchment =10*106*20/100 =2.0*106cumecs/year

Therefore, yield @ site = 2.0*106cumecs/year =Q= 0.0634cumecs.

Assumed Velocity of flow, V= 2.0m/s

Width of the road= 12 m.

We know that Q=AV, where A is area of the pipe

A = Q/v =0.0634/2 = 0.031m² We know that, A = (3.14d²)/4

0.031= (3.14d ²)/4

Therefore, d = 0.20m

From IS 458-1971, adopt two Hume pipes of an internal diameter (d) 0.20m each with external diameter (D) 0.3m.

3.9.14 Soil Investigation Report

Sl. No.	Type of test	Result	Code of reference
1	Liquid limit	37.28%	1920 (part 5)
2	Plastic limit	23.49%	1920 (part 5)
3	Specific gravity of soil	2.36	1920 (part 3)
4	Grain size analysis	Cu = 5.25 Cc =2.09	1920 (part 4)
5	Light compaction	MDD=1.89 OMC =11%	1920 (part 7)
6	Heavy compaction	MDD=1.95 OMC =13%	1920 (part 8)
7	CBR Test	10 %	1920 (part 16)

3.9.15 Pavement thickness Design (IRC: SP 20-2002)

For the thickness design, CBR and traffic data are needed. These are obtained and thickness is designed according to IRC: SP 20-2002.

CBR = 10 %.

Traffic Volume= 15 CVPD (assumed).

Computation of Design Traffic:

A= P(1+r) n+x

A=Number of commercial vehicles per day for design

P= Number of commercial vehicles per day at last count

r= annual growth rate of commercial traffic (7.5%)

n= number of years between last count and year of completion of construction

x= design life in years

A= 15(1+0.075)1+15= 48 CVPD.

The below design chart may be referred to obtain the total pavement crust thickness

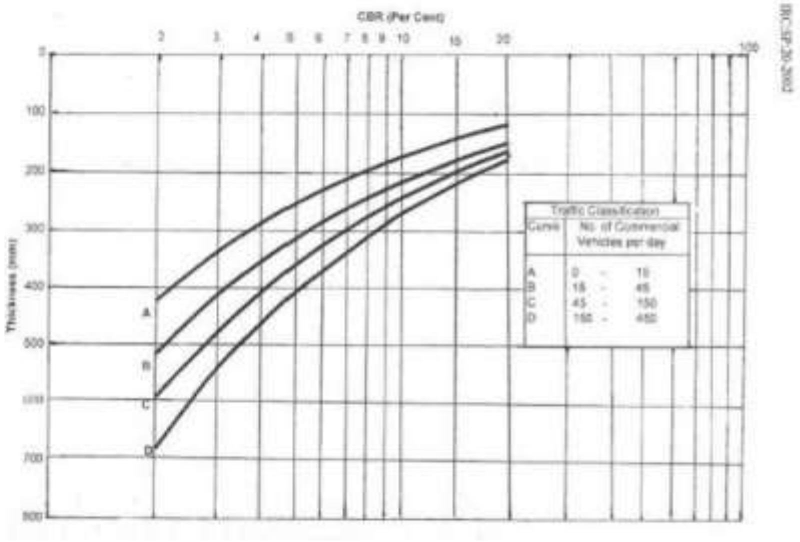

Fig.3.1: Pavement thickness chart

Total Pavement Thickness= 270 mm (from graph)
The total design thickness is divided into base and sub base thicknesses.

Sub Base = 150mm.

Base = 120 mm.

Surfacing= PMC or MSS can be provided if necessary for smooth riding surface.

3.9.16 The summary of our project is as follows

1. Road length :1,500.00m
2. Design speed :40kmph
3. Camber :1 in 50
4. Width of pavement :3.75 m
5. Width of roadway :7.0m
6. Width of the shoulder :1.8 m
7. Right of way :12 m
8. Culvert :01

3.9.17 Details of curves

Table 3.9: Details of curves

Sl. No.	Chainage	Type of curve
1	150.00	Vertical curve
2	600.00	Horizontal curve
3	350.00	Valley curve
4	550.00	Vertical curve
5	10.00	Valley curve

3.10 Estimation of earthwork for highway construction

Table3.10: Estimation of earthwork for highway construction

Station (or) Chainage (m)	Depth (or) Height (m)	Mean Depth in (m)	Area of Central portion in (m)	Area of Sides	Total sectional Area in (m)	Length Between Stations (m)	Quantity Bd+(Sd*d) *L	
		D	Bd	Sd*d	Bd+(Sd*d)	L	filling	Cutting
0	0					-		
10	0.42	0.21	2.52	0.08	2.6	10	26	-
20	0.54	0.48	5.76	0.46	6.22	10	62.2	-
30	1.72	1.13	13.56	2.55	16.11	10	161.1	-
40	1.03	1.375	16.5	3.78	20.28	10	202.8	-
50	1.13	1.08	12.96	2.33	15.29	10	152.9	-
60	1.23	1.18	14.16	2.78	16.94	10	169.4	-
70	1.27	1.25	15	3.125	18.125	10	181.25	-
80	1.35	1.31	15.72	3.43	19.15	10	191.5	-
90	1.34	1.345	16.14	3.618	19.785	10	197.58	-
100	1.38	1.36	16.32	3.69	20.01	10	200.1	-
110	1.39	1.385	16.62	3.83	20.45	10	204.5	-
120	1.23	1.31	15.72	3.43	19.15	10	190.5	-
130	1.12	1.175	14.1	2.76	16.86	10	168.6	-
140	0.92	1.02	12.24	2.08	14.32	10	143.2	-
150	0.47	0.695	8.34	0.966	9.306	10	93.06	-

160	0	0.235	2.82	0.110	2.93	10	29.30	-
170	0.24	0.12	1.44	0.021	1.461	10	-	14.61
180	0.52	0.38	4.56	0.216	4.776	10	-	47.76
190	0.81	0.665	7.98	0.663	8.643	10	-	86.43
200	0.78	0.795	9.54	0.948	10.488	10	-	104.88
210	0.83	0.805	9.66	0.972	10.632	10	-	106.32
220	0.74	0.785	9.42	0.924	10.344	10	-	103.44
230	0.77	0.755	9.06	0.855	9.915	10	-	99.15
240	0.75	0.76	9.12	0.866	9.986	10	-	99.86
250	0.92	0.835	10.02	1.045	11.065	10	-	110.65
260	0.97	0.945	11.34	1.339	12.679	10	-	126.79
270	1.03	1.0	12	1.5	13.5	10	-	135.
280	0.86	0.945	11.34	1.339	12.679	10	-	126.79
290	0.76	0.81	9.72	0.984	10.704	10	-	107.04
300	0.40	0.58	6.96	0.504	7.464	10	-	74.64
310	0	0.20	2.4	0.06	2.46	10	-	24.6
320	0.21	0.105	1.26	0.022	1.282	10	12.82	-
330	0.33	0.27	3.24	0.145	3.385	10	33.85	-
340	0.79	0.56	6.72	0.627	3.347	10	73.47	-
350	1.21	1.0	12	2	14	10	140.	-
360	1.51	1.36	16.32	3.699	20.019	10	200.19	-
370	1.73	1.62	19.44	5.248	24.688	10	246.88	-
380	1.91	1.82	21.84	6.624	28.464	10	284.64	-
390	1.94	1.925	23.1	7.411	30.511	10	305.11	-
400	2.0	1.97	23.64	7.761	31.401	10	314.01	-
410	2.05	2.025	24.3	8.201	32.501	10	325.01	-
420	2.07	2.06	24.72	8.487	33.207	10	332.07	-
430	1.95	2.01	24.12	8.08	32.2	10	322	-
440	2.02	1.985	23.82	7.88	31.7	10	317	-
450	1.88	1.95	23.40	7.605	31.005	10	310.05	-
460	1.83	1.855	22.26	6.882	29.142	10	291.42	-
470	1.45	1.64	19.68	5.379	25.059	10	250.59	-
480	1.27	1.36	16.32	3.699	20.019	10	200.19	-
490	1.06	1.165	13.98	2.714	16.694	10	166.94	-
500	1.11	1.085	13.02	2.354	15.374	10	154.74	-
510	0.93	1.02	12.24	2.08	14.32	10	143.2	-
520	0.62	0.775	9.314	1.201	10.515	10	105.15	-
530	0.27	0.445	5.34	0.396	5.736	10	57.36	-
540	0.19	0.23	2.76	0.079	2.839	10	-	28.39
550	0.76	0.475	5.7	0.338	6.038	10	-	60.38
560	1.05	0.905	10.86	1.228	12.088	10	-	120.88
570	1.23	1.14	13.68	1.949	15.628	10	-	156.28
580	1.39	1.31	15.72	2.574	18.294	10	-	282.94
590	1.58	1.485	17.82	3.307	21.127	10	-	211.27
600	1.88	1.73	20.76	4.489	25.249	10	-	252.49
610	1.96	1.92	23.04	5.529	28.569	10	-	285.69
620	2.17	2.065	24.78	6.396	31.176	10	-	311.76

630	2.23	2.2	26.4	7.26	33.66	10	-	336.6
640	2.11	2.17	26.04	7.064	33.104	10	-	331.04
650	1.37	1.74	20.88	4.54	25.42	10	-	254.2
660	1.55	1.46	17.52	3.197	20.717	10	-	207.17
670	1.93	1.74	20.88	4.541	25.421	10	-	254.21
680	0.97	1.45	17.4	3.154	20.554	10	-	205.54
690	0.86	1.915	22.98	5.500	28.48	10	-	284.8
700	0.79	0.825	9.9	1.020	10.92	10	-	109.2
710	0.72	0.755	9.06	0.855	9.915	10	-	99.15
720	0.64	0.68	8.16	0.694	8.854	10	-	88.54
730	0.59	0.615	7.38	0.567	7.947	10	-	79.47
740	0.27	0.43	5.16	0.277	5.437	10	-	54.37
750	0	0.135	1.62	0.027	1.647	10	-	16.47
760	0.11	0.055	0.66	0.006	0.666	10	6.66	-
770	0.19	0.15	1.8	0.045	1.845	10	18.45	-
780	0.22	0.205	2.46	0.084	2.544	10	25.44	-
790	0.16	0.38	4.56	0.288	4.848	10	48.48	-
800	0.10	0.13	1.56	0.033	1.593	10	15.93	-
810	0.02	0.06	0.72	0.007	0.727	10	7.27	-
820	0.66	0.34	4.08	0.173	4.253	10	-	42.53
830	0.03	0.345	4.14	0.178	4.318	10	-	53.18
840	0.02	0.025	0.3	0.0009	0.300	10	-	3.009
850	0.16	0.09	1.08	0.012	1.092	10	10.96	-
860	0.34	0.25	3	0.093	3.093	10	30.25	-
870	0.35	0.345	4.14	0.178	4.318	10	43.78	-
880	0.09	0.22	2.64	0.07	2.716	10	27.36	-
890	0.30	0.195	2.34	0.057	2.397	10	24.97	-
900	0.23	0.265	3.18	0.105	3.85	10	32.85	-
910	0.07	0.15	1.8	0.033	1.833	10	18.45	-
920	0.05	0.06	0.72	0.0054	0.725	10	7.27	-
930	0	0.025	0.3	0.0009	0.300	10	3.01	-
940	0.01	0.005	0.06	0.00003	0.060	10	0.6	-
950	0.08	0.045	0.54	0.003	0.543	10	5.44	-
960	0.14	0.11	1.32	0.081	1.338	10	13.68	-
970	0.20	0.17	2.04	0.043	2.083	10	21.34	-
980	0.09	0.145	1.74	0.031	1.771	10	18.24	-
990	0.09	0.09	1.08	0.012	1.092	10	1.108	-
1000	0.02	0.055	0.66	0.0045	0.664	10	-	6.645
1010	0.24	0.13	1.56	0.025	1.585	10	-	15.85
1020	0.04	0.14	1.68	0.029	1.709	10	-	17.09
1030	0.33	0.185	2.22	0.051	2.271	10	-	22.71
1040	0.32	0.325	3.9	0.158	4.058	10	-	40.58
1050	0.43	0.375	4.5	0.210	4.71	10	-	47.1
1060	0.33	0.38	4.56	0.216	4.776	10	-	47.76
1070	0.58	0.455	5.46	0.414	5.874	10	58.74	-
1080	0.55	0.565	6.78	0.638	7.418	10	74.18	-
1090	0.41	0.48	5.76	0.96	6.72	10	67.2	-

1100	0.33	0.37	4.44	0.273	4.713	10	47.13	-
1110	0.23	0.28	3.36	0.156	3.516	10	35.16	-
1120	0.26	0.245	2.94	0.120	3.06	10	30.6	-
1130	0.14	0.20	2.4	0.008	2.48	10	24.8	-
1140	0.13	0.135	1.62	0.027	1.647	10	-	16.47
1150	0.04	0.085	1.02	0.014	1.034	10	10.34	-
1160	0.26	0.15	1.8	0.045	1.845	10	11.84	-
1170	0.38	0.32	3.84	0.204	4.044	10	40.44	-
1180	0.23	0.305	3.66	0.186	3.816	10	38.16	-
1190	0.89	0.56	6.72	0.627	7.347	10	73.47	-
1200	0.76	0.825	9.9	1.361	11.26	10	112.061	-
1210	0.64	0.70	8.4	0.98	9.38	10	93.8	-
1220	0.55	0.595	7.14	0.708	7.848	10	78.48	-
1230	0.45	0.5	6	0.5	6.5	10	65	-
1240	0.40	0.425	5.1	0.361	5.461	10	54.61	-
1250	0.54	0.47	5.64	0.4418	6.081	10	60.81	-
1260	0.54	0.54	6.48	0.583	7.063	10	70.63	-
1270	0.51	0.525	6.3	0.551	6.851	10	68.51	-
1280	0.68	0.595	7.14	0.708	7.848	10	78.48	-
1290	1.72	1.2	14.4	2.88	17.28	10	172.8	-
1300	0.66	1.19	14.28	2.832	17.11	10	171.12	-
1310	0.60	0.63	7.56	0.793	8.353	10	83.53	-
1320	0.49	0.545	6.54	0.594	7.134	10	71.34	-
1330	0.51	0.5	6	0.5	6.5	10	65	-
1340	0.42	0.465	5.58	0.432	6.012	10	60.12	-
1350	0.14	0.28	3.36	0.156	3.516	10	35.16	-
1360	0.22	0.18	2.16	0.064	2.224	10	22.24	-
1370	0.28	0.25	3	0.125	3.125	10	31.25	-
1380	0.75	0.515	6.18	0.530	6.71	10	67.1	-
1390	1.05	0.9	10.8	1.62	12.42	10	124.2	-
1400	1.21	1.13	13.56	2.55	16.11	10	161.1	-
1410	0.92	1.065	12.78	2.668	15.04	10	150.48	-
1420	1.37	1.145	13.74	2.622	16.36	10	163.62	-
1430	1.14	1.255	15.06	3.150	18.21	10	182.1	-
1440	0.85	0.995	11.94	1.980	13.92	10	139.2	-
1450	0.30	0.575	6.9	0.495	7.395	10	-	73.95
1460	0.68	0.49	5.88	0.360	6.24	10	-	62.4
1470	0.62	0.65	7.8	0.633	8.433	10	-	84.33
1480	0.32	0.47	5.64	0.331	5.971	10	-	59.71
1490	0.27	0.295	3.54	0.130	3.67	10	-	36.7
1500	0.29	0.28	3.36	0.117	3.477	10	-	34.77
						TOTAL	10,248 m^3	8,475 m^3

Quantity of filling = 10,248 m^3
Quantity of cutting = 8,475 m^3
Difference = 1,774 m^3

CONCLUSION

The highway is designed to serve the people and helps in connecting the nearby existing roads. We proposed road which connects the wind mills over the hill in outskirts of Gadenahalli village to Gadenahalli village. From this work, the surrounding agricultural lands and plantations will be accessed to the proposed road which helps in transportation of raw materials. Transportation will be much easier than before. The people travelling through this road will have a healthy atmosphere and it gives good aesthetic appearance.

Hence there are chances of development of villages in the future.

CHAPTER-4

OLD TANK PROJECT

4.1 Introduction

Tanks are important for conserve precious water resources in semi-arid areas. It is well known that tanks traditionally performed a useful role in providing irrigation, water for domestic use, including livestock and for supporting livelihoods of the poor, protecting local environment and sustainable water resources. Tanks are small storage reservoir (ponds) created on the upstream of a small earthen dam constructed across a stream. The depth of water in a tank is usually less than 4.5 m. However, in exceptional cases it may be more than 4.5 m but not greater than 12 m. If the depth of water is exceeding 12 m the tank is termed as reservoir.

The tanks may be having independent catchment drawing their supplies from the run-off from the catchment areas. These tanks fully depend upon the rainfall in the catchment area. An old tank generally consists of the following.

- An earthen bund across the valley creating storage.
- A surplus weir to dispose of flood discharge.
- Sluice to feed the channel.
- Channel from the sluices to feed the command area.

Every rainy season surface run-off brings large amount of the silt in to reservoir. The deposited silt reduces storage capacity of the reservoir every year. Hence, old tank or irrigation tank generally faces the fallowing problems are,

1. Reduction in the gross capacity of the tank, due to silting.
2. Reduction in the safety of the bund, due to bad maintenance and wearing out of the standard dimensions of the bund.

The above two problems can be overcome by restoring the tank. Restoration of the tank is done by raising the height of the existing bund, thereby allowing for increased storage and improved safely. This operation is called restoration of an old tank.

4.2 Reasons for increasing the height of existing bund

The primary reasons for increasing the height of an existing bund are:

4.2.1 Increased storage capacity

The storage available at the upper bunds of a reservoir for a given increase in bund height can be significant depending upon the topography of the reservoir. In many cases, significant increased storage can be obtained with only a small increase in the height of the dam. This is because the surface area of an existing reservoir at the spillway level is large and adding a few meters to the reservoir depth can effectively increase the reservoir storage capacity by several thousands of hectare- meters.

4.2.2 Spill-way Adequacy

Inadequacy spillway capacity and the potential for the catastrophic overtopping of the crest of the bund under design flood conditions is the most prevalent dam safety issue faced today. In some cases, the original bund and spillway design criteria was less strengthening than criteria imposed by current regulation. In other cases, the hazard classifications of the dam increases after the original bund construction because of changes in downstream flood plain. By increasing the height of the bund, additional free board is provided resulting in increased reservoir surcharge storage capacity and a greater discharge capacity for the spillway.

4.2.3 Strengthening

Quite frequently the most feasible means of strengthening an existing bund to with stand the various loading combinations acting on the bund and possible an increase in the height of the dam. An obvious by- product of rising a bund for strengthening purposes is increased storage capacity.

4.3 Factors to be considered in designing raises for existing bund

The issues associated with the design of raise for an existing bund is that a special attention must be paid for developing a full understanding of the configuration and physical properties of existing bund so that the raised portion is compatible with the existing portion. However, there are a number of general factors that need to be considered and these are outlined briefly below.

4.3.1 Environmental permitting

The raising of an existing bund or dam result in higher reservoir water levels and corresponding greater areas of land in undated by which may have serious environmental implications other signified environmental permitting issues that could impact the design related to borrow sources, in stream flow releases, wet lands, archaeological sites and stream modification.

4.3.2 Spillway

Depending on the size of the raise and the resulting increase in reservoir level, a major modification of the existing spillway or perhaps even a completely new spillway is required. This can have a significant influence on the cost of the raise and careful planning of this aspect will be required to minimize costs.

4.3.3 Outlet works

Modifications to the outlet works of dams or bunds are often required as a part of the raise. Typically, this involves extending the outlet conduit to the new embankment toe location and possibly relocating the intake structure or sitting basin control towers located at the center line of bunds or dams also need to be raised.

4.3.4 Reservoir operation

Whether or not the reservoir can be lowered or emptied during construction will have significant influence on both feasibility and the design of the raise. If the reservoir can be lowered, the raised portion can be constructed partially upstream of the existing dam and thus could significantly facilitate the toe into the existing core of the dam. Moreover, depending on the materials availability and or environmental constraints on the location of borrow areas, borrow areas may be confined to the inundated portions of the reservoir. Has the added advantage of the increasing reservoir storage by an amount equal to the borrow volume? The benefits of an upstream or a partial upstream raise are offset by a need to improve extend the outlet works upstream. If the reservoir cannot be lowered during construction, materials for construction on the raise may have to be imported from the outside.

4.3.5 Central core

For a very small increase in height compared to the original height of the dam or bund it is typically most economical to the core of the addition to the core of the existing dam or bund. For significantly greater raises, a completely new core may have to be constructed such that existed dam or bund may not be feasible for use.

4.3.6 Drainage system

Embankment drains are frequently included in the design of raises to be existing dams or bunds. Even if the original dam/bund did not include a drain system the designer carefully evaluates whether drainage needs to be incorporated into the raised embankment section. The design considerations outlined above apply in the general sense to any proposed dam or bund raise. Yet, each dam or bund is unique and has its own site of design issues that need to be addressed.

4.4 Study area

The study area Gadenahalli village is located in Bangalore North Tehsil of Bangalore district in Karnataka, India. The tank is located at an elevation of 100.000 m with respect to MSL. The purpose of the project is to provide water stored for irrigation and for public water supply for the surrounding villages. However, because of silting up of the reservoir, the usable storage has been severely curtailed. Also it is evident from inspection that the original dimensions of the bund are worn out and the dam safety may be deficient. Corrective measures are now being designed to restore the usable storage and increase safety.

4.5 Survey to be conducted

4.5.1 Plan of the existed bund

The tracing of plan of the bund is done by using compass. The compass is set up at the beginning of bund and north direction is noted here and mark starting point (A) of bund. The point A is located by using basic principal of survey.

Procedure

- Plan of the existing bund is traced on a sheet with help of bearings that are with help of prismatic compass. Set the compass at zero chainage (from point A) and take the bearings at every 10 m interval. While taking the fore bearing note down back bearing also.
- Repeat the same procedure up to the point B.

4.5.2 Longitudinal and cross-section along the center line of the existing bund

Survey is conducted from starting point of the bund the longitudinal and cross section is taken. Longitudinal section is taken at every 10 m interval and cross section is taken at every 20 m chainages at an interval of 2.5 m on the both upstream and downstream side of bund.

Equipment's used

- Tape.
- Chain.
- Arrows.
- Ranging Rod.
- Dumpy level and Theodolite with tripod stands.
- Level Staff.
- Plane table with accessories

Procedure

1. The R.L of existing Top Bund Level (TBL) is determine by carrying fly levelling from permanent bench mark on to the top of the masonry abutment of surplus weir.

The R.L of the top of the abutment = R.L of the existing Top Bund Level (TBL)

Note: - The horizontal surface of the notch stone indicates the TBL. The notch stones are usually providing all along the length of the bund at regular intervals. Since the notch stones are missing on the bund, the top of abutment is taken as R.L of existing TBL.

2. A point along the center line of the existing bund is located whose R.L is one meter greater than the R.L of Top Bund Level (TBL). i.e., the height of the existing bund is raised by 1m.

R.L of proposed TBL = R.L of existing TBL + 1 m

3. On the extended center line of the existing bund, search for the R.L of proposed TBL at the ends of the bund, these points are the beginning and end points/ stations of the proposed bund. The location of these two points should be fixed with respect to three permanent objects.

R.L of beginning and end points = R.L of proposed TBL

4. R. Ls of longitudinal section is taken at every 10 m interval on the centreline of the bund from the beginning point to the end point of the proposed bund. Cross sections are taken at every 20 m intervals and where ever there is change in alignment of bund.
5. Salient features such as sluice; Beginning and end of surplus weir and R.L of crest of surplus weir (FTL) should be noted at respective chainages.
6. The cross section is taken at points L1, L2, L3----- and R1, R2, R3 as shown in below figure. The horizontal distances of 6m are with reference to the centreline on either sides of bund.

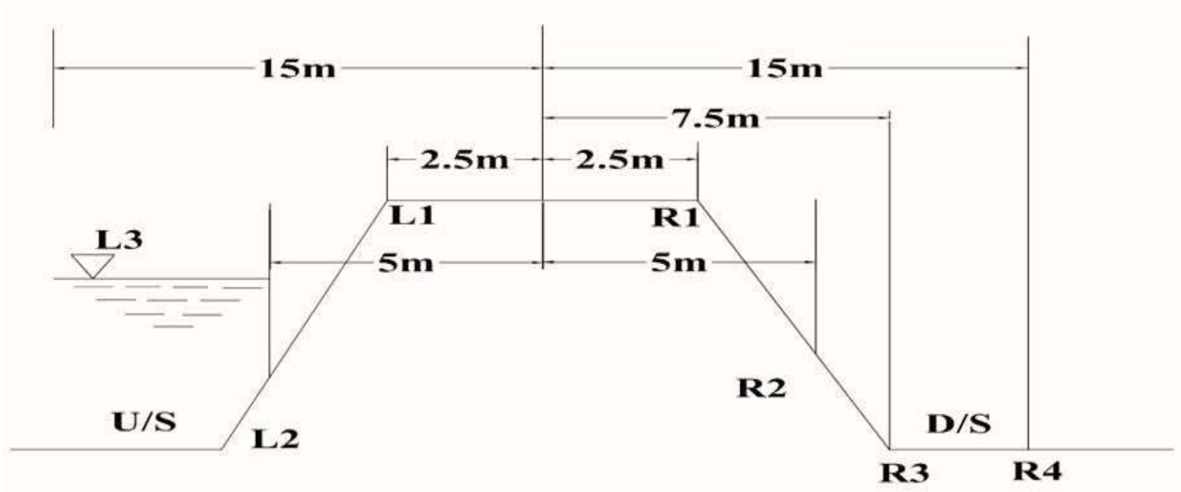

Fig 4.1: Cross sectional view of proposed bund

4.5.3 Capacity contours at the existing and proposed full tank levels

Center line of the bund along with the three well defined points is transformed from the plan on a new drawing sheet. Capacity contours are plotted by the method of direct contouring using tachometer for distance measurement. Scale 1cm: 10 m.

Procedure

1. Taking the sill of waste weir as bench mark point or any nearer permanent bench mark on the contour are located and determine the height of the instrument.
2. The plane table is set up over a good commanding station and oriented.
3. The plane table and levelling instrument are kept as close as possible each other, so that the distance of contour points are measured by tachometry and plotted on the plane table by method of radiation.
4. Whenever the plane table and level are shifted choose a commanding position by conducting reconnaissance.
5. The plane table and the level are shifted to the new commanding position and its position on the plane table sheet is marked.
6. Three-point problem shall be solved for every third plane table position in order to orient the table accurately.
7. The contour work should be ended at the starting point on the waste weir

4.5.4 Block Levelling

Block levelling is done for sluice gate and waste weir of 20m X 20m and 60m X 40m respectively. Using rope, chain, tape, Dump level, level staff, etc. the block levelling gives the clear contour profile of the measured area.

Procedure

1. Using tape, chain, rope lay the desired block required with certain interval.

2. Set the dumpy level near the block where all block points are clearly visible and take the temporary benchmark.
3. Note down the readings at block point of known interval using dumpy level and level staff.
4. Calculate the reduced level of the block points and plot the block on to the sheet and locate the required contour.
5. The reduced level of required contour is calculated using equation
$$\frac{\text{Highest RL of the block} - \text{lowest RL of the block}}{\text{No s of contours required}}$$
6. The distance of the contour is calculated by using equation
$$\frac{\text{Required RL} - \text{Lowest RL}}{\text{Highest RL} - \text{Lowest RL}}$$

4.6 Design Work

Table: 4.1 Salient Features of Old Tank Project

Sl. no	Specification	Details
1	Top width of bund	5 m
2	Total length of bund	495.6 m
3	Top bund level	+87.820 m
4	Maximum water level	+86.820 m
5	Full tank level	+85.820 m
6	Dead storage level	+84.220 m
7	Total height of bund	3.075 m
8	Upstream slope	1:1
9	Downstream slope	1.5:1

4.7 Rainfall Analysis

The yearly rainfall data of a tank as collected from the panchayath of Gadenahalli village is shown in table 4.2

Table: 4.2 Rainfall Data

Year	Rainfall in mm	Cumulative rainfall in mm
2018	994.00	994.00
2019	912.00	1906.00
2020	1023.40	2929.50
2021	909.00	3838.50
2022	947.00	4785.50
2023	845.00	5630.50

The total average rainfall data for the catchment of an old tank for 6 years is 939 mm. The rainfall of a bad year is always taken as 2/3rd of mean amount of rainfall. Bad year rainfall

$$\mathbf{l\ {}^{2}/_{3}\ \text{of}\ 939 = 626\ \text{mm}}$$

4.8 Calculation of yield at site

Runoff coefficient usually assumed as 15% to 20%.

Assuming 20%

Mean rainfall = 62.60 cm

Annual yield = 20/100 x 62.60 = 12.52cm

Yield from catchment = 1.2 x 10^6 x ($\frac{1.5}{1}$) = 0.150 x 106 cum/year

Yield at site = 0.150 x 106 cum/year

4.9 Calculation of storage capacity of tank

1. Areas of successive contours are measured using Planimeter or by constructing squares.
2. If A1, A2, A3… An, are the areas of successive contours, H being the contour interval, then by Prismoidal rule. The storage capacity can be calculated.

Using Prismoidal rule

V = [(A1 +An) +4(A2+A4 +A6+...) +2(A3+A5+ A7+...)] m^3

V= 2628.193m^3

The storage capacity of old tank is 2628.193m3 hence the yield of the storage tank in 2628.193m3

4.10 Design of the Surplus Weir

Estimation of flood discharge entering the tank. If the tank is an independent one, then the flood discharge can be easily estimated by using Reeve's formula, Q= CM2/3

Where,

Q = Flood discharge,

M = catchment area in sq.km

C = Reeve's coefficient = 6.5 to 15,

Catchment area = M = 1.2 sq. Km

Discharge, Q = $CM^{2/3}$

Q = 9 x $2^{2/3}$

Q = 10.16 m^3/sec

1. Length of Waste Weir

For a rectangular weir, the discharge is given by: -

H = MWL- FTL= 87.320– 86.820 = 0.5 m

Taking co- efficient of discharge, Cd = 0.60

$Q = \frac{2}{3} \times Cd \times L \times \sqrt{2g} \times H^{3/2}$

$10.16 = \frac{2}{3} \times 0.6 \times L \times \sqrt{(2\times9.81)} \times (0.5)^{3/2}$

L= 15.72 m≈ 16 m

Length of the weir including the dam stones of 20cm × 20cm to store the water up to MWL when necessary. Dam stones are laid at 1m interval.

No of dam stones = {(L/Z)-1} = [()-1] = 15 numbers

2. Weir Details

Top width of weir, a= 0.55 √ h+√H] = 0.55[√3.075+√0. 5]

a = 1.35 m ≈ 1.4 m

Base Width of weir, b = $\frac{H+h}{\sqrt{\delta}-1} = \frac{3.0 + 0.5}{\sqrt{2.3}-1}$

b = 3.135 m ≈ 3.2 m

Providing the foundation concrete of 0.6m thick and 0.3m projection on either side, therefore the total base length = 0.3+3.2+0.3 = 3.8m. Also provide protection work such as wing walls, returns and abutment on either side of the tank suitably also provide stepped apron up to 10m on d/s with two steps and 4m talus will be provided at the end of apron.

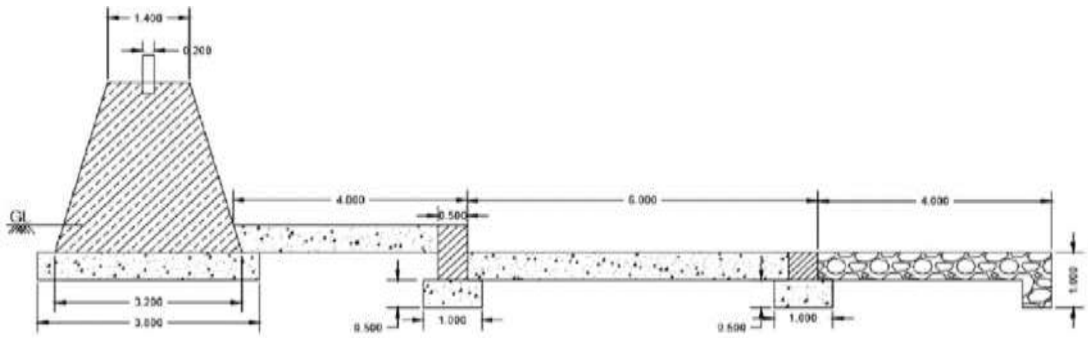

Fig 4.2: C/S of the Waste Weir

4.11 Earth Work Calculations: For Filling

Chainages (m)	Cross section Area (m^2)	Average c/s Area (m^2)	Length (m)	Volume of Earthwork(m^2)
0	0.20	-	-	-
20	1.8597	1.02985	20	20.597
36.6	8.945	5.40235	16.6	89.67901
66.6	6.3845	7.66475	30	229.9425
86.6	6.912	6.64825	20	132.965
106.6	3.905	5.4085	20	108.17
126.6	1.727	2.816	20	56.32
146.6	0.0010	0.864	20	17.28
166.6	5.208	2.6045	20	52.09
186.6	5.998	5.603	20	112.06
206.6	7.076	6.537	20	130.74
226.6	3.787	5.4315	20	108.63
246.6	6.8955	5.34125	20	106.825
266.6	4.1317	5.5136	20	110.272
286.6	4.9865	4.5591	20	91.182
306.6	5.825	5.40575	20	108.115
326.6	5.9275	5.87625	20	117.525
346.6	3.5940	4.76075	20	95.215
366.6	2.1625	2.87825	20	57.565
386.6	4.8215	3.492	20	69.84
406.6	6.1975	5.5095	20	110.19
426.6	4.97	5.58375	20	111.675
446.6	9.192	7.081	20	141.62
466.6	7.862	8.527	20	170.54
495.6	4.237	6.0495	29	175.4355

Total=	**2524.47 m³**
Earth work required = Total + 15% compaction	**2903.15 m³**

4.12 Determination of Irrigable area (CCA)

The yield of storage tank has been found to be 2628.193m3 Assuming 10% for evaporation losses and 15% of conveyance loss i.e. 25% as total loss in the tank storage capacity.

Volume of water available for irrigation is

V= x 2628.193 m

3= 1971.144 m3

Assuming the commercial crop, take the Delta 0.3 m

Volume of water = Area x Delta

Area = Volume of water/ Delta

Area = 0.197/0.3

Area = 0.65 Hectares ≈ 0.7 Hectares

Hence the total Irrigable area by this tank is 0.7 hectare’s.

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