

IRRIGATION MANUAL

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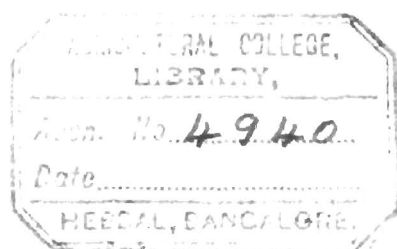
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PREFACE

The *Irrigation Manual* is primarily intended for the use of the Civil Engineering Diploma students in the Polytechnics of India. It had its origin in the notes used for my lectures on "Irrigation" for three years commencing from 1954-55, at Sri Jayachamarajendra Polytechnic, Bangalore. I am now publishing this work at the request of students to satisfy a need for a text-book covering the essentials of this important subject, as I have not come across a work dealing with it at the diploma level.

I wish to acknowledge my indebtedness to *Irrigation* by Col. W. M. Ellis, Late Chief Engineer for Irrigation, Madras, which was our guide not only at College, but through service as Irrigation Engineers. But that work is too advanced for the diploma students and with my experience in the Mysore P.W.D. and, later, as Principal, Sri Jayachamarajendra Polytechnic, Bangalore, I have tried to deal with the subject at the diploma level.

It is not my intention that the *Manual* should merely be an easy guide for the boy who is eager to get through the examination. I expect he will try to understand the subject, by taking the help of his lecturer, wherever necessary. To the lecturers on Irrigation in the Polytechnics, I appeal that they may kindly bring to my notice any defects that they may notice in the compilation of my lecture notes and that they may favour me with suggestions to make the work more useful to the future members of the Lower Irrigation Service in India.

BANGALORE,
18th February 1959.

H. C. K. BHATTA.

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A. Introduction

1. What is Irrigation ?—Irrigation is the artificial supply of water essential for plant growth to soil. In nearly all areas where irrigation is practised, crops get some water from rains during the crop season. Irrigation is essentially a practice of supplementing the natural rainfall for the production of crops.

In few parts of the world is natural rainfall adequate for growing all varieties of crops. In some humid regions, where the annual rainfall exceeds 200 cm., crops are grown satisfactorily every year without irrigation. In other regions where the annual rainfall is about 150 cm., the rains supply all the water during most of the years but only a part in some years when the rain is not distributed properly within the growing season. It is advantageous to supply supplemental water in these regions. In those regions where the annual rainfall is of the order of 100 cm., or where the rain falling in the crop season is inadequate though the total yearly rainfall may be more, irrigation is necessary every year.

2. Wet and Dry Crops.—In South India, the normal rainfall is sufficient for growing millets like ragi and jola, and cotton which are referred to as “Dry Crops” which do not need irrigation. For crops such as rice, called “Wet Crops”, irrigation is essential for growing a good crop except in a limited area close to the Western Ghats called the ‘malnad’. Even when the total yearly rainfall may be enough as in the “Semi-malnad”, it becomes necessary to resort to irrigation over a portion of the crop period

owing to the uncertainty of rain just when the crop needs water. A third variety of valuable crops known as "Garden crops" like sugarcane, areca, plantain and betels, on the other hand, require intensive irrigation throughout the year except in *pucca-malnad* areas. Rainfall has, however, the advantage of reducing the supply of irrigation water in every situation.

3. Indian Irrigation.—The natural assets of our country compare very favourably with those of the other advanced countries. We have plenty of agricultural land of the most fertile quality. We have a large manpower gifted with a high irrigation sense, intelligence and ability. Our mountains and forests hold inexhaustible sources of mineral wealth. The climate of the country is suitable for sustained hard work on the part of the people in all seasons. Our rivers carry large volumes of water over long lengths in all parts of the country and sustain life in a motherly manner. But yet, except in favoured regions like the Sindhu-Ganga plain in the north and a narrow tract along the coast in the south, a deficiency in rainfall causes harm to men and cattle in the first year, and if repeated, results in distress conditions which may be accentuated into famine conditions. That irrigation plays a significant part in the country's agricultural economy was recognised early by our enterprising forefathers who have bequeathed to us an efficient, indigenous irrigation system consisting of tanks, anicuts, channels and irrigation wells to suit the topography and water resources of different parts of the country.

In spite of the fundamental unity of India, there is an enormous diversity in climate, rainfall and topography in the several parts of the country. The climate is hot and the rainfall is practically nil near the Thar Desert while it is intensely cold near the Himalayas. On the one hand,

there are places like Cherra Punji in Assam with a yearly rainfall of 1500 cm. and like Agumbe in the Karnataka with a yearly rainfall of 900 cm. On the other hand, there are large areas in the plains with a rainfall between 50 and 100 cm. There are areas like the Sindhu-Ganga plains which are practically level and other tracts which are hilly and undulating where it is difficult to find a substantial level bit. In between these extremes, the bulk of the country is full of low hills, river valleys and plateaus. There is similar variety in the soils and in crops grown in different parts of the country. Some rivers like the Mahanadi, Damodar and Kosi swell into mighty proportions during floods and overflow their banks causing much destruction to life and property while they are practically dry in the summer months. Irrigation in India has to suit these varied conditions in the nature of the works for the storage and distribution of water to the fields. Indigenous irrigation works in one area are therefore different in type from those in another area. At the same time there is no need for irrigation works where the rainfall is sufficient in quantity and distributed throughout the growing season in accordance with the needs of the crop but such areas are exceptional. In a country where the main occupation is Agriculture, the need for irrigation works of one type or another is nationwide so that cultivation may not be a gamble on fitful rain.

4. Seasons in India.—There are, broadly speaking, three clear seasons. The summer or the hot season lasts from the middle of February to the middle of June, the rainy season till the end of October, and the winter from November to the middle of February. The onset of the seasons varies slightly in the different parts of the country. The early part of summer is Spring and that of winter is Autumn. There are two distinct rainy periods separated by a short summer in the latter part of September. The earlier period

from the 20th of June is the South-West Monsoon and the latter one from the second week of October is the North-East Monsoon. From November to February is a practically rainless period and we have a few showers with thunder and lightning from March to May, which slightly relieve the summer heat. This is in contrast to the seasonal conditions in the temperate zone where all the months may be rainy and to places like Egypt where there is practically no rainfall all through the year and where the country is protected by rivers flowing through them.

5. Multipurpose Projects.—The potentialities of India on the one hand and the varying needs of the developing country on the other are such that irrigation is just one of the uses of stored water. The success of the Tennessee Valley Project in the United States of America has encouraged the multipurpose idea. The objects are: (1) Irrigation, (2) Generation of Electric Power, (3) Flood Control, (4) Navigation, (5) Industrialisation, and (6) Recreation, Fish breeding and water-supply.

The development of these aspects needs careful planning not only during execution but during maintenance over a long range. The requirements for the various purposes are mutually conflicting and they have to be co-ordinated. Irrigation requires a large quantity of water during the growing season and the reservoir must be full at the commencement of the season. On the other hand, flood control requires that the reservoir should be nearly empty so as to store the water and prevent its damaging crops and villages downstream. Both water-supply and power generation need a steady supply of water all through the year, day and night. The same canal system cannot serve both irrigation and navigation. Navigation canals have an uniform cross-section and depth for the passage of boats and a low velocity

of flow which does not impede navigation. Irrigation canals have a gradually diminishing cross-sectional area as the area to be irrigated diminishes, and a velocity which is both non-silting and non-scouring. Special works like locks are needed along navigation canals when their level is dropped while a different kind of 'drops' are needed for irrigation canals. On large rivers like the Mahanadi, Damodar and Kosi, one reservoir may not meet the pressing need for flood control and a number of reservoirs means co-ordination in their working in the interests of irrigation and power generation. More than all, it is a problem to finance these projects and costs have to be allocated to the several uses. The multipurpose idea is as useful and interesting as it is costly and complicated.

6. Irrigation in the Karnataka State.—The surface soil on the table-land is generally hard, the sub-soil is rocky, and the ground slopes away rapidly. The water flowing in the valleys in the rainy season was therefore stored in small reservoirs called tanks. The usefulness of the tanks was felt to such an extent that it was considered an act of religious merit to build them and kings and persons of influence took a pride in building new tanks. This process has gone on for so long and so systematically that practically every drop of water flowing in the minor valleys has been bunded up. The finished pattern is a series of tanks on every water course in which the surplus from one tank is caught up in the lower tank. The area has been explored so thoroughly that it now requires some ingenuity to discover a good new tank site.

The River Channel.—The next type of indigenous irrigation work makes use of the water flowing in the rivers by partially blocking them by anicuts or weirs with massive sections of huge blocks of stone set close and dry. During

floods, the extra water rolled over the anicuts. The water the level of which was raised was taken on one or both the banks by means of channels parallel and close to the river for irrigating patches of land between the channel and the river.

Bhandara.—This is the name given to small anicuts in Dharwar, Belgaum, and the adjacent Maharashtra Districts.

Wells.—Well irrigation or lift irrigation is the third type of irrigation which makes use of the underground reservoir of water. Wells are common in the northern and eastern parts of the Karnataka and in the Mangalore and Karwar Districts on the West Coast. Large wells are sunk at the head of irrigated fields for tapping the sub-soil water which is lifted by a 'Kapile' or 'Pikota' or other device drawn by men or bullocks for watering small areas of land and growing valuable garden crops.

B. Systems of Irrigation

Where the irrigation water is conveyed on to the land by the force of gravity, it is known as "Flow Irrigation" as opposed to "Lift Irrigation" where the water-supply which is at a lower level has to be lifted up by devices like pikotas or pumps.

7. **Flow Irrigation Systems**.—These are of three classes.—

- (a) Perennial,
- (b) Inundation, and
- (c) Flood Irrigation.

(a) In *Perennial Irrigation*, water is supplied continuously in accordance with crop requirements during the 'crop period'.

1 (b) *Inundation Irrigation* is practised on the large alluvial rivers of India with a fluctuating discharge for several months in the rainy season. In most cases, the canal heads are uncontrolled open channels leading from the rivers. The canals come into operation when the rivers rise above the channel beds. But latterly, large size regulators are built to shut out heavy floods, which, if admitted, might damage the channels and submerge the irrigated areas.

An inundation river without a fixed course may erode the regulator and canal bed and silt up the canal bed. A weir may then be built to fix the river course at the head and to raise the upstream water level. Inundation Irrigation is then converted into "Flow Irrigation".

The advantages of Inundation Irrigation are that large areas can be irrigated without much expense and the silt brought by the floods improves the fertility of the land. The main disadvantage is the varying nature of the supply necessitating larger canals and involving wastage of water in good seasons.

(c) In *Flood Irrigation* as practised in Egypt and Sind, flood water is diverted to the higher lands for soaking the area so that crops can be grown on the flooded area after draining the land. Cultivation by this method is precarious unless the supply is supplemented by a few showers at the proper time.

The difference between Inundation and Flood Irrigation is little. In the former, the canal system is inundated by the floods, while in the latter, the flood waters are stored in basins as in Egypt and there is no separate system of canals.

Both the systems are gradually being replaced by Perennial Irrigation.

8. Engineering Science mostly deals with "Flow Irrigation" which is classified as (a) Direct and (b) Tank Irrigation depending on the source of water. In Direct Irrigation, water is drawn from the river without the intervention of any storage work. Anicut irrigation belongs to this class. In Tank Irrigation, flood waters when in excess of immediate requirements are impounded and this storage is made available for use when the river flow gets below the required limit. There is no storage in a Direct Irrigation work. Flow irrigation in South India is almost exclusively developed for the cultivation of rice.

9. Lift Irrigation.—It is practised widely from wells for growing rich commercial crops and is peculiarly adapted to the cultivation of two or three crops in a year.

When rain falls on the ground, (1) part of it is absorbed by vegetation or evaporated from the land and water surfaces, (2) a part "runs off" the surface to produce flood flow and (3) another part soaks into the ground. It is the last portion which forms the source of underground water. Depending on the geological nature of the underground strata, the water infiltrating into the soil may either penetrate deep or get locked in the sub-soil, or may reappear in streams as springs or seepage. Much of the water absorbed by the soil in irrigated lands also percolates into the ground and adds to the underground reservoir of water.

10. Ground Water.—The occurrence of ground water depends on the nature of the geological formations below the ground surface, while the form in which soil moisture is held depends on the amount of water present.

"Hygroscopic water" occurs as a thin film on the surface of soil particles. It is held in the soil with great force and is not capable of movement. It is not available to plants.

Below the "hygroscopic layer" is the capillary fringe in which the smaller quantities of water are held as a film surrounding the soil grains. As the amount of moisture increases, the surface tension of the film decreases and moisture moves more freely in the soil. This water becomes easily available to the plant roots during intervals between irrigations.

When all the pore space in the soil is filled with moisture the soil is 'saturated' with water which enters soils by gravity during rains. "Gravitational water" is that which drains through the soil under the action of gravity. The level of the plane of saturation is called the 'Water-Table'.

The zone below the water-table is the zone of saturation and the water held in it is 'Ground water'. The water-table is not a level surface but follows the ground level. The earth is composed of permeable and impermeable strata occurring in patches. The water-table therefore varies from place to place. It normally coincides with the free surface of lakes and streams. Its position depends on the continued accretion of ground water and therefore fluctuates with annual variations in rainfall. It is naturally higher under the hills than under the adjacent valleys.

For various reasons, the water-table intersects the land surface when 'springs' result. A water-table confined from above by an impermeable layer so that the water is under pressure represents artesian conditions.

Occasionally a body of ground water will be found above a bed of impervious material. Such an anomalous condition is called a 'perched water-table'.

The region between the water-table and the surface of the ground is the 'Zone of aeration' in which there is free exchange of air and moisture. Water in the zone of aeration is divided into three belts: (1) Soil water, (2) Intermediate suspended or Vadose water and (3) the capillary fringe.

11. Soil Constants.—The 'hygroscopic coefficient' is the maximum percentage of moisture which could be taken up in a dry soil in contact with an atmosphere with a definite relative humidity less than 100%.

The 'wilting point' is the moisture at which permanent withering of plants occurs.

The field capacity is the amount of water held in the soil after excess gravitational water has drained away.

The maximum capillary-water capacity is the maximum water constant which a soil retains when subjected to the force of gravity. Since wilting point marks the lower limit for plant growth, 'available water' is the quantity of water represented by the difference between the wilting point and the maximum capillary-water capacity.

The 'moisture equivalent' is the water retained by initially saturated soil when centrifuged under specified conditions.

The 'moisture-holding capacity', or 'maximum water capacity' is the water content when the soil voids are completely filled. It is the quantity of water required to saturate the soil.

12. Wells.—Wells are holes sunk from the ground into the sub-soil water to tap the underground water. Wells have played an important part in all countries from the earliest times both for the supply of drinking water and water for irrigation.

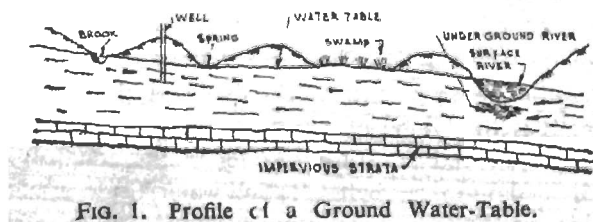


FIG. 1. Profile of a Ground Water-Table.

They can be classified in a number of ways: (1) Shallow or Deep, (2) Ordinary or Artesian and (3) Open or Bore Wells.

Shallow wells are sunk a small depth into a superficial water-bearing stratum. They are supplied by the infiltration of rain or water drained from rivers or channels, or sewers in cities. The numerous wells sunk in villages for domestic water-supply are of this kind. Shallow wells are sometimes sunk in the vicinity of rivers and are supplied by the water filtered through the sands or gravels in the river margins.

The quantity of water derived by a shallow well depends on the depth of the well, the nature and position of the water-bearing stratum in which the well is sunk and the disposition of the impermeable stratum below.

Deep wells are supplied by water percolated and filtered through large masses of the earth's crust. The difference between a shallow and a deep well lies not in the actual depth of the well but in the smaller or greater distance of the source of water which flows into it.

13. Artesian Springs and Wells.—A geological formation which transmits water in sufficient quantity to support wells or springs is called an 'aquifer'. In areas of considerable leakage from confined aquifers, the water-table is found near the surface, and no pressure head is found in a shallow well. If the depth of the well is increased until it enters the aquifer, there is an increase in pressure head but the pressure head does not increase only with depth. The shallow well is in unconfined water with all pressure head spent in percolating upward while the deep well is tapping an artesian aquifer under pressure.

In Fig. 2, A and C are beds of clay or other impervious material and B is a water-bearing stratum. Water will

accumulate in the hollow of the lower impervious stratum until it is pressed upwards against the underside of the upper one by hydrostatic force. If a well be sunk at K, the water will rise to a level determined by this hydrostatic pressure. Such wells are called artesian.

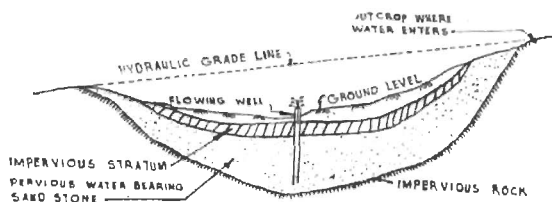


FIG. 2. Diagram showing the Flowing Artesian Well.

14. Wells and Boreholes constitute the principal means of obtaining water-supply from underground sources. They are alike in principle and action.

The dug well consists of a pit entering the aquifer. Such wells are practicable only in unconsolidated material where water is reached at relatively low depths. They should be lined with stone or pre-cast reinforced cement concrete rings to prevent collapse. Their diameter is large, above 6 feet.

Wells may be bored in unconsolidated material by the use of a large auger up to 12 inches in diameter.

Shallow, small-diameter wells may be constructed by driving a well-point and casing into the ground until an aquifer is reached. The well point is slightly larger in diameter than the tubing and the combination is driven with a power ram up to 100 feet in depth and 6 inches in diameter.

The common method of percussion well drilling uses cable tools. A heavy bit is raised and lowered in the hole, crushing the material at the bottom. Sufficient water is introduced into the hole to permit the removal of the crushed

material by a pump or bailer. The cable tools are suspended from an 'A' frame at one end of a portable rig and the reciprocating motion of the drill is obtained by an eccentric crank. A long tube smaller in diameter than the hole and with a flap valve at the bottom is used as a bailer. As the tube is lowered into the hole, the flap valve permits entry of broken material into the tube and prevents the escape as the tube is raised.

In the hydraulic rotary method the hole is drilled by the rapid rotation of a bit on the end of a drill tube. A mud fluid is circulated down through the drill tube and up on the outside to bring the drilling debris to the surface.

The hydraulic rotary process can be used to drill larger holes than the cable-tool method, but the latter method is cheaper for shallow holes, is more portable, and requires less water in operation.

15. Cone of Depression.—If a line of wells is driven and the central one is pumped, a drawdown of the water level in the other wells will result. In homogeneous material with a level water-table, the 'cone of depression' will be symmetrical around the pumped well.

16. Properties of Aquifers.—The porosity of an aquifer is expressed as a percentage by volume of the interstices in the material. The total water content of an aquifer is obtained by multiplying its volume by its porosity. Some of this water is not available for wells but is permanently retained in the aquifer by molecular forces.

The volume of water free to drain from the rocks, expressed as a percentage of the total volume of the aquifer is called the "Specific yield", and that volume which is not available is called the "Specific retention".

The permeability of an aquifer is its availability to transmit water.

17. Lifting Water.—The water required for irrigation is lifted by

- (1) man or animal power,
- (2) mechanical power.

(1) Though lifting by animal or man power is primitive, the method is widely used in Indian villages owing to its cheapness. The following are some of the common methods:—

(a) 'Rāti' or Pulley is the most common type. The pulley is attached to a horizontal beam of wood, stone, steel, or R.C. supported on vertical pillars at the ends of the well. A rope goes round the pulley and carries a pot at the end dipping into the well while it is dragged from its other end.

Sometimes a wheel and axle arrangement is used which gives a small mechanical advantage.

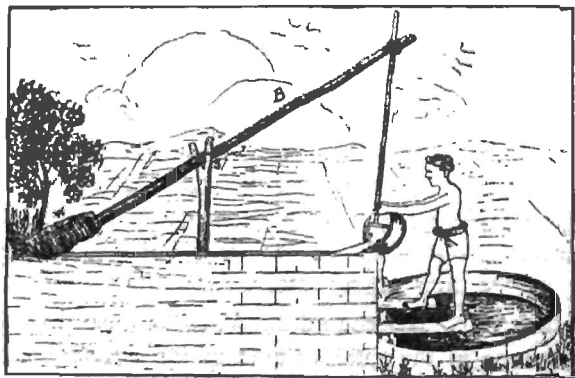


FIG. 3. *Picota* or *Yatha*

- B. Beam to which the Bucket pole (P) is attached.
 S. Pin on which the Beam moves.
 W. Counter-weight at the small end of the beam.

(b) *Basket*.—When the water is shallow, it is lifted by a basket which is swung on ropes by two men standing across the sheet of water.

(c) *Yāta*.—When the lift is about 20 feet, the pikota is a convenient arrangement. A long wooden beam is suspended on a fulcrum at the end of a vertical pole fixed at the bottom firmly in the ground. To the long end of the lever or beam is attached a bucket while the short end is loaded with a counterweight of stones or wood. The man throws his weight on the long end when the bucket fills and the counterweight lifts it.

(d) *Doon*.—When the lift is small—up to 3 feet, a wooden trough, or *Doon* is used which oscillates on a fixed point. One end of the trough is dipped into water and raised. The weight of the filled trough is balanced by a counterweight. The operator stands on a plank in the water, fills the trough, lifts it and lets the water flow into the fields.

(e) *Kapile*.—Two bullocks may be made to walk down a ramp and lift water from an ordinary pulley in a well. The rope is attached at one end to a leather bag or *mote* and at the other to the yoke of the bullocks. When the *mote* comes up it is emptied into a trough leading to the field channel. The bullocks then walk up the ramp to the well. The *mote* can be used for lifts up to 30 feet, effectively.

(f) *The Archimedean screw and the Persian Wheel* are largely used in Egypt. The Persian Wheel is also used in Northern India. It is difficult to lift water over high lifts with the Archimedean Screw but the Persian Wheel is efficient even at heads up to 50 feet. It is the most effective means of lifting water by animal power. These appliances are not used largely in India.

(g) Similarly, the windmill is not also used much in this country. It is common in Holland.

18. Pumping from Wells and Canals.—Pumps may be worked by (1) Stream Engine, (2) Oil Engine, (3) Internal Combustion Engine, or (4) Electrical Power.

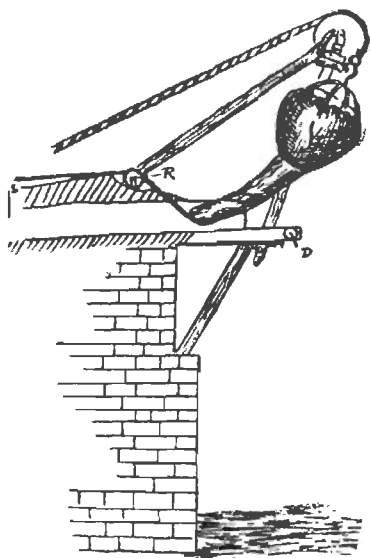


FIG. 4. *Mhote or Kapile* showing the bucket discharging water through its tail.

(After Wood)

The types of pumps used for lifting water are: (a) Air lift pump, (b) Force pump, (c) Pulsometer pump or (d) the Centrifugal pump.

Centrifugal pumps worked by Electricity are being used in India from 1 HP. to several hundreds of HP. for lifting water from 10 feet to 1,000 feet. Single stage centrifugal pumps lift water up to 60 feet, while three-stage pumps like those used at Thippagondanahally for the water supply of Bangalore raise water nearly by 1,000 feet. But irrigation pumps are not worked against heads of more than 100 feet,

while the quantity of water is large. A small quantity of water from the suction end whirled at a high velocity in the impeller of the pump has its kinetic energy converted into potential energy, thus raising the level of water at the delivery end. This type of pump has an efficiency from 50 to 70% and can be used in open wells, tube wells, canals or streams. This is the most economical and efficient method of lift irrigation.

19. Advantages of Lift Irrigation.—Its great advantage is the economy with which the supply is used but it is expensive to the cultivator. There is dispersion in the irrigated area but this is not a defect when large areas are irrigated by the use of electrical pumps from steady sources like rivers or channels. The system deserves to be practised side by side with flow irrigation in areas where there is plentiful supply of ground water which would otherwise be lost underground. In seasons of drought when one year of low rainfall may succeed another, the surface sources may fail but ground water-supply is maintained though the water-table may be lowered. Lift irrigation is entirely under the owner's control and can be used for growing valuable commercial crops. It is peculiarly adopted to the cultivation of two or three crops all round the year and the agriculturist is kept engaged all the time, which is a very important matter in this country.

C. Methods of Irrigation

Irrigation water is applied to the land in four methods:—

- (1) Flooding the surface,
- (2) In furrows, wetting a part of the surface,
- (3) Sub-irrigation, without wetting the surface, and
- (4) Spraying, wetting the surface like rainfall.

20. Flooding.—Where the water is applied without any embankments to guide its flow, it is ordinary flooding.

Dividing the farm into a number of strips separated by low borders is the border method of flooding.

Running large streams into level plots surrounded by berms is check flooding.

Check method adapted to irrigation of orchards is the basin method. A basin is made for each tree into which water is conveyed by flowing through one basin to another.

The objective in each irrigation by flooding is to moisten every square foot of the surface without permitting excessive deep percolation losses through the soil in any part of the field.

21. Furrow Method.—Only a half to a fifth of the area is wetted in this method thus reducing evaporation losses. Nearly all row crops are irrigated by this method. The furrows are run down the steepest slope avoiding inconvenience due to overflowing the banks of the furrows.

22. Sub-Irrigation.—A porous surface soil and moderate slopes with an impervious sub-soil at about 6 feet depth favour sub-irrigation where water is applied directly under the surface. Stoneware pipes with open joints are laid under the cultivated area about 10 feet apart and 3 feet below the surface. The disadvantage of this method is that its expense is great. It can only be used in orchards. The advantage is that there is great economy in water-supply.

23. Spray Irrigation.—It applies water as it falls in ordinary rains and is economical for growing valuable crops. There are two types of spray irrigation. (a) The overhead pipe system and (b) The circular spray system. The former consists of parallel lines of pipe about 50 feet apart, supported on rows of posts 6 feet high, each line equipped with nozzles 3 to 4 feet apart. Each nozzle discharges a tiny stream of water perpendicularly to the pipe line, all streams being parallel. The circular spray system distributes water

from nozzles at an elevation of 4 to 6 feet above the ground, spaced equidistant from each other at the corners of equilateral triangles. They are supplied with water through uprights distributed uniformly all over the field.

24. Rotation of Crops.—This is a matter which has to be settled in consultation with the Agricultural Department. It refers to the kinds of crops to be grown in the area for which irrigation water is supplied in the best interests of the area. In small irrigation works it is usual to grow only rice when it is found that the soil gets exhausted in a few years by the removal of the same kinds of plant food. The growth of rice also causes water-logging and unhealthiness unless the area is drained properly.

Under a large irrigation work it is advantageous to change the crop on the same plot in successive years so that it may recuperate the loss of certain plant foods. Water stored in reservoirs has practically no fertilising silt and the farmer is generally too poor to use a sufficient quantity of balanced manures. A remedy for this state of affairs is to rotate the crops suitably. The system has these advantages:—

(i) The soil will be better utilised by growing shallow rooted crops in one year and deep rooted ones the next year.

(ii) The growth of leguminous crops which increases the soil-nitrogen and its fertility periodically is beneficial to the soil.

(iii) The impoverishment of the soil regarding the particular plant food used up by the continuous growth of just one crop year after year is avoided by changing the crop in a periodic manner.

(iv) The change of crop prevents the growth of the same crop-diseases and insect pests.

(v) The introduction of commercial crops in the rotation system will be to the advantage of not only the soil but of the farmer.

25. 'Phad' System of Crop Rotation.—This is being practised in Nasik and Khandesh Districts in Bombay. The total irrigated area is divided into three or four equal parts called phads. Each phad is further subdivided into a number of plots. Only one crop is grown in one phad in one year while different crops are grown in different phads. In three or four years each crop will be grown in all the phads. Rice, sugarcane and dry crops are rotated in this manner.

26. Block System of Crop Rotation.—This is the system suggested by Bhārata Ratna Sir M. Visvesvaraya for the Visvesvaraya Canal, but it has not been adopted for various reasons.

The irrigated area in each village is divided into blocks of convenient size, and the blocks to equal sub-blocks. The system of rotation proposed is the following:—

Block I	I Year	II Year	III Year
Sub-Block 1	.. Paddy	Sugarcane	Irrigated dry
Sub-Block 2	.. Sugarcane	Irrigated dry	Paddy
Sub-Block 3	.. Irrigated dry	Paddy	Sugarcane

27. Benefits of Irrigation.—Irrigation maintains the prosperity of the land and increases food supply. In this country which is subject to periodical famine conditions somewhere or other, it provides the best security against the effects of drought. It provides employment for persons affected by distress. It increases the value of land and

property and renders possible the cultivation of valuable commercial crops like sugarcane, oilseeds and cotton in place of only dry crops with low yield. The direct return to Government on the capital invested is only a small portion of the indirect benefits conferred on the area since general well-being cannot be measured in terms of money. Water stored in irrigation works may be used for the generation of hydro-electric power. The prosperity of Mandya District is entirely due to the Visvesvaraya Canal from Krishnarajasagara which also provides an unfailing supply of water for the generation of increased power from the Sivasamudram and Shimshapur generating stations. Electricity provides lighting and heating facilities in towns and villages. It promotes the growth of various types of industries.

28. Over-Irrigation.—Irrigation without proper drainage may lead to water-logging and cause efflorescence. In the earlier stages, it provides breeding places for mosquitoes and causes the spread of malaria. A few years back an extremely virulent type of malaria spread over the Mandya area and had to be controlled by elaborate and costly measures. Tanks were breached, water collected in pools was canalised and swamps and stagnant ponds were treated with oil. Pyrethrum extract and D.D.T. were sprayed by the public health department in the houses and the menace controlled effectively.

D. Duty of Water

29. 'Duty' indicates the irrigating capacity of water. It is the relation between the area irrigated and the quantity of irrigation water used. It can be expressed in a number of ways:—

(i) By the number of acres irrigated by an average flow of 1 cubic foot, per second for the whole crop season.

(ii) By the number of acres irrigated per million cubic foot of stored water.

(iii) By the number of acre-feet of water per acre of crop.

The first method is suitable for canal irrigation where the discharges are expressed in cusecs. The second and third methods have relation to storage capacities of reservoirs. The third method is direct in application since irrigated area is measured in acres. In South India, 6 acre-feet are found to be sufficient for an acre of rice cultivation and characterised as an "unit of water".

It is convenient to express the duty as the depth of irrigation water standing over an acre since rainfall is measured in the same manner. It is measured in centimetres of irrigation water though this is supplemented by rain falling during the irrigation season which is considerable. Sometimes duty is stated as the total depth of water including rain.

When duty is expressed in acres per cusec, the supply is referred to the whole of the "crop period" from the time water is first issued for preparing the ground to the last watering before the harvest. It may also be stated with reference to a period shorter than the entire crop period called the "base" which must be specified for defining duty.

30. In a canal system where there is Transmission loss, continually between the canal head and the fields, it is necessary to specify the place of measurement to which duty has reference. Irrigation water is commonly measured at 3 points.

(1) Gross use, at the point of diversion from the source of supply,

(2) Lateral use, where large distributaries are taken from the main canal, and

(3) Net use, at the point of delivery on to the irrigated land.

1 cusec flowing 1 day = 0·0864 million cu. ft.
 = 2 acre-feet nearly (1·983 exactly)
 = 24 acre-inches.

1 acre-foot = $\frac{1}{4}$ cusec flowing 1 day.

$1fd$ = duty in acres per cusec.

 Δ = equivalent depth in inches and

B = Base in days.

 $\Delta = 24 B/d$ and $d \approx 24 B/\Delta$ nearly.

32. The factors affecting duty are:

- (i) The amount and incidence of rainfall,
- (ii) The nature of soil of the canal trace and of the irrigated fields, and
- (iii) The kind of crop grown and the length of the growing season.

(i) If there is a total rainfall of 24 inches during the base period of 135 days and if it is well-distributed over the

favourable rain-stars, it has the effect of reducing the irrigation water by a third of a unit, or of increasing the duty by a third, from 45 acres to 60 acres. But if the total rainfall is less, or if it is untimely, the increase of duty is correspondingly less.

(ii) The loss by percolation depends on the character of the soil; the finer the soil, the less the percolation and the higher the duty. Loamy soils are best suited for crops with artificial irrigation.

(iii) The maximum depth of water is required for rice—48 to 72 inches followed by sugarcane—48 to 60 inches. The longer the growing season, the greater the quantity of water used.

33. In South India, Irrigation is concentrated on the cultivation of rice. On an average the duty for rice works out to 45 acres per cusec with a total depth from 56 to 75 inches, over a crop period of 140 to 160 days, water required in each month varying considerably.

The preparation of the field consists of flooding the land for several inches and ploughing it so as to reduce the surface soil to a slushy state. The rice seed is germinated in separate seed-beds or nurseries and the young plants are transplanted in the prepared field after draining it, being thrust by the hand into the soft slush. A light flooding is given after transplantation. This operation requires a large quantity of water varying with soils and localities from 12 to 18 inches, on the field, and takes 8 to 10 days from start to finish. Transplantation is speeded up or slowed down by the extent of useful rain during the period.

After transplantation, the crop requires enough water to keep the land submerged which is given at intervals of to 8 days. Subsequent to transplantation, water is required

from 4 to 6 inches per fortnight, including rain. The quantity is less during the last 2 months of the crop period. Only one watering is required 3 to 4 weeks before reaping, after which the field is allowed to dry.

The duty for rice cultivation varies with the seasonal rainfall and with the rate of percolation from the irrigated lands. It has therefore to be fixed with due regard to each separate tract to be cultivated. When water is kept standing over large areas for several months, the loss by percolation even in retentive soils becomes considerable and the supply necessary to maintain water standing on the fields has to be increased in proportion to the permeability of the soil. The duties realised in deltaic areas with flat slopes are nearly twice those in the upland reaches, allowing free percolation through the sub-soil back into the river. The mean duties in the Kaveri delta vary from 80 to 110 acres per cusec whereas they are between 20 and 40 acres in the old channels above and below Krishnarajasagar.

While designing new irrigation projects, the monthly variations in duty in the irrigation season for the same crop in the same place have to be taken into consideration.

34. Improvement of Duty.—The following are the methods of improving duty in an existing irrigation system:

(1) Economical use of water by educating the ryots to make them realise the interests of their neighbours.

(2) *Canal lining.*—On embankments and cuttings where the soil is very permeable, certain lengths of canals may be lined with brick or stone in mortar, or with cement concrete to reduce the heavy losses by percolation.

(3) Introduction of the block system or the system of triennial rotation of crops with rice, sugarcane and irrigated dry crops. This system will also improve the fertility of

the soil and prevent dampness and water-logging of the irrigated lands.

(4) Introduction, if possible, of the method of charging for the irrigation water by measuring the actual quantity of water used by each cultivator. There are certain difficulties in applying this system at present.

(5) Unless absolute economy is practised in the use of irrigation water by the ryots, loss by seepage from irrigated lands is inevitable into the natural drainage channels. This water can be used for additional irrigation by constructing small masonry pick-up weirs across the valleys.

(6) The seepage may also be taken into a parallel channel below the main channel and extra land brought under irrigation.

(7) The canal may be closed for 2 days in a week by turns in the several distributaries physically to limit the water let out of the branch channel. The minimum amount of water needed for growing a good crop in the different soils in an irrigation project should be ascertained by studies in experimental farms and by irrigation under controlled conditions in small plots in Research Laboratories.

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A. Rainfall

1. The Hydrologic Cycle.—It denotes the circulation of the water from the sea, through the atmosphere to the land and back to the sea by overland and subterranean routes, and partly by the route of the atmosphere. The energy for operating the hydrologic cycle comes from the Sun. Hydrology is the science concerned with the occurrence of water on the earth.

2. Rainfall.—Moisture is distributed throughout the lower portion of the atmosphere below a level of nearly 6 miles. Rainfall results from the condensation of atmospheric water vapour through an atmospheric disturbance or storm.

‘Rainfall is the source of all water used for irrigation purposes and therefore a knowledge of its amount, character, seasons or periods and the effects produced by it is of primary importance to all those whose duty it is to design, carry out, improve or maintain irrigation’ works-(Mullins).

3. Measurement of Rain.—Rainfall is measured in India by means of rain gauges at a number of stations distributed throughout the country and the daily fall at each station was being recorded in inches and cents. It is now measured in cm. and mm.

A standard rain gauge consists of a cylinder 5 inches in diameter with the base enlarged to 8 inches. Over the cylinder is fixed a funnel provided with a brass rim 5 inches in diameter. The funnel shank is inserted in a glass bottle

which receives the rain water. A graduated measuring glass is supplied with each guage reading to cm. and mm. and holding when full, water equivalent to 1.25 cm. of rain. The rain guage should be set up in a masonry foundation with the level rim 12 inches above the surface of the ground. To secure uniformity all over the country, the rainfall at all Government Stations is measured every day at 8-30 A.M. and recorded as the rainfall on that day. Actually the

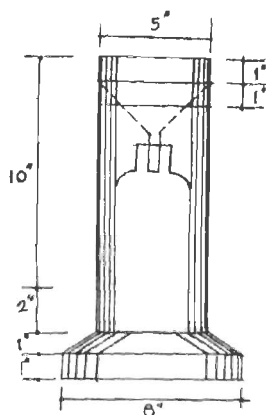


FIG. 5. Symon's Rain Gauge.

rain has fallen in $8\frac{1}{2}$ hours on that date and in $15\frac{1}{2}$ hours on the previous date. The receiving bottle does not hold more than 10 cm. of rain and when the rainfall exceeds this quantity it should be measured several times in the day to prevent the receiver overflowing. In all cases, the last measurement should be taken at 8-30 A.M. and the sum of all the readings during the previous 24 hours added and entered in the register.

The daily rain gauge readings in all the stations are communicated to observatories where they are recorded, totalled up for each month and for each year. Since this informa-

tion is important for Engineers, and the public, it is collected and printed each year in each province, and a consolidated report is prepared by the Government of India.

4. Variations in Annual Rainfall.—Rainfall varies from place to place during the same year and from year to year at the same place. The distribution during the several months also varies at the same place from year to year. It is necessary to have some standard in such cases to which the variation can be referred. Such a standard is the average of 35 consecutive records of yearly rainfall.

Rainfall is more often below the average than above it and so far as water-supply for irrigation is concerned, it is in the years of deficit rainfall that artificial supply is most wanted. In the case of projects for domestic water-supply, the absolute minimum in one year and the minimum of 2 or 3 consecutive years is important as works have to be designed to give enough supply under the most unfavourable conditions. In irrigation projects, it is unnecessary to secure supplies for the whole area normally irrigated under the worst conditions, but it is necessary to secure such supplies in an average bad year which may be taken as the mean of the lowest rainfalls in a number of consecutive years. For preliminary investigations of an irrigation project, the rainfall of an average bad year may be taken as $\frac{2}{3}$ to $\frac{3}{4}$ the mean annual rainfall.

5. Point Rainfall and Areal Rainfall.—Point rainfall is the rainfall at a single station. There may be several stations in a given area. The areal rainfall is the average rainfall of the different stations as rainfall over a large area is not uniform. There are three methods of computing areal rainfall (1) by the arithmetical average of the records in the several gauging stations, (2) by the weighted average according to what is known as the "Theissen method"

and (3) by averaging with the help of isohyetal maps for each storm in the area.

6. Cultivation of Rice.—Rice is the staple food crop in South India and has a crop period varying from 4 to 5 months. The cultivation of this crop is regulated by long practice by the Mahanakshatras through which the Sun moves along the Zodiac. Certain rains are found to be beneficent while others are not. The Bharani from about the 28th April to the 11th May is good, while the next, Krithika is harmful. The field operations are best started in Ardra commencing from the 20th June. The next, Punarvasu and Pushya are the heavy rains during which the transplanted seedlings take root and grow. Svathi from the 23rd October to the 5th November is the last useful shower and the monsoon closes with Visakha by the 18th November. Even today our cultivator watches for these rains for adjusting his work. After Hasta in the second week of October, the direction of the monsoon changes from South-West to North-East. The Eastern and Northern parts of the Karnataka are more favoured by retreating monsoon. The rains in July and August are the tank-filling rains while those in October are the tank-breachers under certain conditions when the tank is already full and when there is a heavy and sudden down pour upstream.

B. Runoff

7. Run off is that portion of the rainfall which is carried away through the surface channels. It is the residual of rainfall after the deduction of losses by evaporation, transpiration and deep percolation.

8. The following is the process of transition from rainfall into runoff.

(1) The rain first saturates the soil surface and trees and plants.

(2) Infiltration into the ground begins and continues throughout the rain.

(3) The depressions in the surface are all filled.

(4) The flow of water over the land commences.

(5) Overland flow reaches the valleys and is drained into the rivers flowing towards the sea.

9. The Factors Affecting Runoff from a catchment area draining into a valley are:—

(a) The amount intensity and distribution of rainfall.

(b) The characteristics of the catchment.

(i) Its area, shape, slope and ruggedness.

(ii) Nature of the soil and the geological formation of the sub-soil and permeability.

(c) Nature of vegetation and cultivation in the catchment.

(d) Pools or ponds or other storage in the catchment.

(e) Temperature and wind.

All these factors are variable so that runoff is a very variable quantity, more variable than rainfall.

The supply of water for “Flow irrigation” depends entirely on runoff. Measurements of runoff or yield from a catchment are neither so common nor so accurate as measurements of rainfall, so that generally, the volume of water discharged by a stream is deduced from rainfall data. But where possible, even approximate gaugings of actual discharges in streams are preferable.

10. Climate and Weather.—They deal with conditions of the earth's atmosphere and have a great influence on

runoff. Weather denotes atmospheric conditions at a particular time or during short periods of time. Climate refers to weather conditions during a longer period of time and includes normal values as well as variations from the normal conditions. Elements of weather include temperature, radiation, sunshine, wind, humidity, fog, rainfall, evaporation and barometric pressure.

Climates have been classified on the basis of (a) Temperature, (b) Rainfall and (c) Prevailing winds. From the view-point of irrigation, the classification based on rainfall is reasonable.

(i) If the average annual rainfall is less than 40 cm., it is an 'Arid' climate.

(ii) If it is between 40 and 75 cm., it is a semi-arid climate.

(iii) If it is more than 75 cm., it is a humid climate.

In an arid climate, a drought is the normal state of affairs, and while calculating runoff from an arid catchment each fall of rain has to be considered as a separate unit.

In a semi-arid climate, a drought occurs at least once a year except in abnormal years.

In a humid, or standard climate, a drought does not occur in ordinary years.

We have all the three classes of climate in India, so that the runoff from each type of climate is equally important.

11. Proportional and Subtractive Methods.—Two methods are employed for deriving runoff from rainfall (a) the proportional method, (b) the subtractive method.

The proportional method gives Runoff/Rainfall, as a percentage which varies for catchments with different qualities and different quantity and intensity of rainfall. Tables are constructed giving percentages of runoff with reference

to (a) the total yearly rainfall and (b) the rainfall in the rainy season, on the basis of the best available observations. Catchments are roughly classified as (1) Good, (2) Ordinary, and (3) Bad, according to their runoff qualities.

The subtractive method is applicable to the standard type of climate where the rainfall is known accurately. The observed rainfall loss consisting of (a) Evaporation, (b) Transpiration and (c) Deep percolation and measured in cm. depth over the catchment is subtracted from a fairly high proportion of the annual rainfall.

$R = 0.94, P = 35$ cm., for England according to Parker where R is the runoff and P , the precipitation in cm.

$R = 0.85, P = 30$ cm. for the Deccan tract in India with its impervious sub-soil according to the adaptation of Parker's formula by C.C. Inglis.

12. Runoff Stated.—Runoff is stated in three ways:

(i) *Like rainfall in cm. over the catchment.*—When the F.P.S. system was in vogue, both rainfall and runoff were measured in inches. Reduced to feet and multiplied by the area of the catchment in sq. feet, the volume of water was obtained in million cu. ft.

The American unit of storage is an acre-foot which is the volume when a foot depth stands over an acre, i.e., $4,840 \times 9 = 43,560$ cu. ft.

In Indian Tank irrigation, a unit is an acre-fathom. or 6 acre-feet, i.e., $43,560 \times 6 = 261,360$ cu. ft.

One million cu. ft. is equivalent to 3.826 units of water.

The unit of volume in the C.G.S. system, or the metric system is a litre or 1,000 c.c. An inch is 2.54 cm. or a foot is 30.48 cm. so that a cu. ft. is 28.32 litres, nearly.

(ii) *In inches per hour over a sq. mile of catchment.*—
This may be converted into cusecs thus:

An inch/hr. = 12 foot/3600 sec. = $1/43,200$ cusec. It is thus a very small unit adopting the unit of length with which rainfall was measured in the old system.

Associating this with the sq. mile which was the unit of measurement for catchment areas, we get

$$1/43200 \times (5280)^2 = 645.3 \text{ cusecs}$$

as the flow per sq. mile with a runoff of 1 inch per hour.

In general, if

M = Catchment area in sq. miles,

r = Runoff in inches per hour,

the discharge from the catchment = $645.3 Mr$ cusecs.

(iii) *In inch depth of water over a sq. mile of catchment.*—
This form is convenient in Tank irrigation. An average coefficient c is assumed depending on the nature of the catchment and the yield is expressed in units/sq. mile.

If, for instance, $C = 1/10$, as in Mysore maidan, the yield per sq. mile in units = C.M.R. where M is the catchment in sq. miles and R = Average yearly rainfall in inches.

$$\text{C.M.R.} = \frac{1/10 \times (5280)^2}{4840 \times 9 \times 6} = 0.89 \text{ unit per sq. mile.}$$

By separating the assumed coefficient of runoff as p , the formula for yield can be made more general as $0.89 p.M.R.$ units per sq. mile. Multiplying it by the catchment in sq. miles, we get the total yield from the area, in units.

13. Runoff from Reservoir Levels.—Where the runoff from a catchment is intercepted by a reservoir as at Krishnarajasagara and at Thippagondanahally and where tables of

capacities at intervals of a foot are prepared by contour surveys of the water spread the coefficient of runoff may be calculated accurately. The daily level of the reservoir is recorded and the draw off from the lake for water-supply to Bangalore, the surplus through the gates and over the masonry waste weir are available at Thippagondanahally. The leakage from the sluices is measured. If the losses by evaporation are also observed by means of floating pans, absorption loss is the only item which has to be assumed. The difference in storage has to be correlated with the corresponding days of rainfall and rainfall in inches over the catchment to deduce the percentage of runoff.

C. Flood Discharge

14. Whereas the total runoff from the catchment in a season measured in million cu. ft. represents the yield from which the storage capacity of a reservoir may be determined, the maximum intensity of flow measured in cusecs determines the design of the surplussing arrangements on which the safety of the reservoir largely depends. The maximum stream flow is caused when a rainfall of great intensity falls over a large part of the catchment at a time when the ground is saturated. Floods are of great economic importance because of their destructive effects on life and property.

15. The usual methods of estimating maximum flood at a given place in a river are:

1. Indications of past floods,
2. Empirical formulæ,
3. Stream gauging.

16. **Indications of Floods.**—When reliable records of flood levels are available for a long time, a few cross-sections of the stream at the high flood level may be taken

and by observing the maximum velocity of flood by means of floats, the corresponding flood discharge may be calculated by $Q = A \times V$. An extra allowance of 10% may be made to obtain the design flood, *i.e.*, the flood for which the surplussing works have to be designed.

It is not desirable to calculate maximum flood discharge from rainfall statistics, on large catchments as they lead to erroneous estimates.

In the absence of flood level records, local enquiries may be made in the marginal villages to get information regarding the highest level to which the water reached in recent years. The various points shown as flood limits must be connected by levelling and compared with one another. As a further test, the discharge calculated on the basis of these levels may be compared with rainfalls recorded on those occasions to get a fair estimate of the maximum flood discharge.

In the case of small catchments, the information regarding high floods is likely to be smaller. The forecast of maximum flood is then made from the maximum intensity of rainfall recorded at places in the catchment during short periods from 1 to 10 days and by assuming a suitable coefficient of runoff from the nature of the catchment.

17. Empirical Formulæ.—Many formulæ have been evolved for estimating maximum flood flows which are applicable to regions where they were developed. Formulæ involving drainage area only, are of the form $Q = C.M^n$ where Q is in cusecs, C is a coefficient, M , the drainage area in sq. miles and n , an exponent smaller than unity.

18. Ryves' Formula is $CM^{\frac{1}{4}}$. It is applicable to South Indian maidan catchments where the annual rainfall is less than 100 cm., with the following values for C : (1) 450 for flat tracts, (2) 560 for tracts with greater slopes at distance

of 15 to 100 miles from the coast, and (3) 675 for limited areas near the hills.

The coefficient for a particular valley must be settled after taking into consideration (a) the importance of the work, (b) the nature of the catchment and (c) the rainfall and its maximum intensity in short periods.

Steep or rocky catchments with small absorption and high runoff require a high coefficient and the coefficient for flat absorbent soils may be reduced. The shape of the catchment has an effect on the intensity of flood. A large catchment shaped like the fan in which the tributaries join the main river near to one another will yield a larger flood than a long narrow catchment shaped like a fern. But when the rain storm producing the flood travels down the valley, a narrow catchment may also give a high flood.

If there should be a great difference in the rainfall over different parts of a large catchment, the flood discharges of the several parts should be considered separately.

19. Dickens' Formula is $CM^{\frac{1}{4}}$ in which the value of C varies from 150 to 1,000. For catchments in Bombay with their impervious trap sub-soil, the average coefficient is taken as 825. The formula is applicable to Dharwar and Belgaum but when applying it to Raichur and Gulbarga, the coefficient has to be reduced suitably. It is well suited to the semi-malnad parts of Mysore with an annual rainfall from 100 to 200 cm. or even 250 cm.

20. Later Bombay Practice.—Whiting and Beale of the Bombay P.W.D. have modified Dickens' formula and prepared curves based on the measured discharges of maximum floods up to 100 sq. miles. Runoff tables are given in inches per hour and the maximum flood discharges are also plotted graphically with reference to the area of catchment. The runoff varies from 1.17 inches per hour for

20 sq. miles to 0.25 inch per hour for 1,000 sq. miles in Whiting's curve. In Beale's curve the runoff is 0.25 inch per hour for 1,600 sq. miles. Whiting's curve gives slightly higher figures than Beale's curve.

The corresponding coefficients in Dickens' formula vary from 1,600 for 20 sq. miles to 1.012 for 1,600 sq. miles. Both the values are higher than the maximum of 1,000 suggested by Dickens.

These curves may be used for obtaining maximum flood discharges in the malnad tracts of Karnataka State where the annual rainfall is between 250 and 375 cm.

21. Inglis' Formula.—River gauging was started in 1902 by Beale in Bombay and the results of a quarter century of work were set down by Sir C. C. Inglis in 1929. Velocity was measured by floats and areas calculated by taking soundings. Gauge readings were taken both at the beginning and end of velocity observations and the area corresponding to the average gauge depth was multiplied by velocity to obtain the discharge.

(1) Mean discharge, (2) Area and (3) Velocity were plotted against gauge depth for each gauging station and curves prepared. Chart 'A' was prepared showing the relation between the area of catchment and flood runoff in cusecs. A similar Chart 'B' showed the relationship between the area of catchment and flood runoff in inches per hour. Charts were also prepared for each gauging station showing runoff in mill. cu. ft. year by year. Annual rainfall for periods up to 70 years starting from 1855 was plotted and finally Chart 'C' showing the relation between runoff and rainfall in inches was plotted.

Inglis divided catchments into 2 types: (1) Fan or multiple catchment and (2) Fern or normal single catchment.

From Chart A, Inglis evolved the formula $7,000 A^{\frac{1}{2}}$ for fan-shaped catchments which gives higher figures than Beale's curve. The exponent $\frac{1}{2}$ was taken from *The Hydro-Electric Handbook* by Creager and Justin of America.

For fern-shaped catchments, Inglis' formula is $7000 A^{\frac{1}{2}} - 240 (A - 100)$ where A is the catchment area in sq. miles.

From Chart B, the runoff in inches per hour was worked out as $1085 A^{\frac{1}{2}}$ for fan-shaped catchments,

$$1085 A + 37.5/A - 0.375 \text{ in inches per hour.}$$

From Chart C, runoff was worked out as

$$0.85 P - 12 \text{ inches,}$$

where P = precipitation in inches on the lines of Parker's investigations for 'ghat' catchments.

For non-ghat catchments, runoff

$$R \text{ is } P - 7/100 \times P,$$

where P is rainfall in inches.

Inglis' formula is suited for *pucca-malnad* catchments of Karnataka with an annual rainfall of over 375 cm.

22. Hydraulic Formulæ.—They state the laws of discharge and velocity. $Q = A \cdot V$, where Q is in cusecs, A in sq. ft. and V in feet per second.

Flow in open channels is a function of the slope S, of the water surface and r , the hydraulic radius which is the ratio between the area of cross-section and the wetted perimeter. An experimentally determined coefficient is introduced for evaluating energy losses due to friction and turbulence caused by friction in the channel bed. $V = C\sqrt{r \cdot s}$ is Chezy's formula.

The coefficient C is given by the formula of Ganguillet and Kutter, Swedish Engineers.

$$C = \frac{41.65 + \frac{1.811}{n} + \frac{0.0281}{S}}{1 + \left(41.65 + \frac{0.0281}{S}\right) \cdot \frac{n}{\sqrt{r}}}$$

The Chezy formula is used with values of C computed from this formula.

Manning, a British Engineer, evolved a formula

$$V = 1.486 r^{\frac{2}{3}} s^{\frac{1}{2}}$$

eliminating n , the coefficient of friction.

Both the formulæ may be used for calculating discharge in river channels because of their simplicity.

D. Stream Gauging

23. The most satisfactory determination of the runoff from a catchment is by measuring the discharge of the stream draining it. The discharge is the product of the cross-sectional area of the river and the velocity of flow. A discharge measurement determines both these factors.

24. **Gauging Station.**—A gauging station is the place where discharge measurements are made. The gauge is the instrument for observing the stream level. Gauges are of 2 kinds: (1) non-recording and (2) recording. A common form of non-recording gauge is (a) the staff gauge consisting of a graduated scale firmly attached to a support in the water with its zero below water, (b) In the float type, a float which rises and falls with the water is attached to one end of a chain and a counterweight at the other end to keep the chain taut, (c) In the weight type, a weight

attached to one end of a chain is lowered until it comes in contact with the water surface and the level is read by the marker at the other end, (d) Hook gauges of various designs are used where greater accuracy is required.

25. Recording Gauges.—There are two types of recording gauges. (a) The float type and (b) the pressure type.

(a) The essential parts of a float gauge are: (1) a float in the stilling box on the bank which rises and falls with the water surface, (2) a drum with a record sheet on its surface rotated by the action of the float through a chain and (3) a clock which moves a pen axially along the drum. The pen is in contact with the record sheet and produces a curve as the drum rotates and the pen itself is moved along the drum.

(b) The pressure gauge consists of a 'compensator' or an air-tight bulb connected by copper tubing to a pressure chamber which actuates a pen-arm in contact with a circular chart which is revolved by clock work. The compensator is fixed in the bed of the stream below low-water and its bottom is the zero of the gauge. The weight of the water above the compensator compresses the air in the copper tubing and the pressure chamber. The movement of a diaphragm on the latter actuates a pen-arm in contact with the record sheet.

26. Flow in a Channel.—Two methods are used for determining the flow in a channel: (1) the weir method and (2) the velocity-area method.

(1) *Weir method.*—The weir method involves the use of a formula containing 3 factors: (1) the length of the weir, (2) the depth of flow over the weir and (3) a coefficient depending on the type of weir. The most commonly used is that of Francis, without end contractions: $Q = C.L.H^{3/2}$

where $C = 0.31$ to 0.33 , L = Length of weir in feet, H = depth of flow over the weir in feet, Q is in cusecs.

The cross-sectional area of the channel is obtained by taking soundings below the water level at intervals of 20 feet. The use of weirs is restricted to irrigation canals and cannot be recommended for river gauging.

(2) *Velocity-area method*.—It involves the measurements of cross-sectional areas and velocities of flowing water. The area is determined by measurements of width and depth at cross-sections to show the shape of the river-bed and give the cross-sectional area. Velocity may be measured (a) by the slope method, (b) by floats and (c) by a current meter.

27. Slope Method.—(a) In the *Slope Method*, (1) the mean cross-sectional area A , (2) the surface slope S of the stream and (3) the coefficient of roughness n are determined,

$$(i) Q = A.V.$$

$$(ii) V = C\sqrt{rs}.$$

$$(iii) C = \frac{41.65 + \frac{1.811}{n} + \frac{0.0281}{S}}{1 + \left(41.65 + \frac{0.0281}{S}\right) \cdot \frac{n}{\sqrt{r}}}$$

in the F.P.S. System.

Errors in applying this method are due to errors in measuring all the 3 factors which may amount to 20%.

28. Float Method.—(b) The *Float Method* is more direct and more accurate than the slope method. Surface floats are generally used but surface velocities must be reduced to mean velocities by coefficients varying from 0.75 in shallow streams to 0.95 in deep rivers.

If surface velocities are observed at points distant L feet from each other across the river, and if d_m is the mean depth corresponding to the width $L/2$ on each side of the point where V_s is the surface velocity, $Q = P.L.d_m.V_s$, where p varies from 0.75 to 0.95.

The area can be calculated by Simson's rule and the discharge by Harlacher's method.

29. Sub-Surface Floats.—Sub-surface floats are (a) the double float, (b) the twin float and (c) the velocity rod.

The *double float* consists of a copper ball 3 inches in diameter attached by a thin wire to a disc which floats on the surface. The ball is weighted so that the disc is just above water. It directly records the velocity of the film of the water in which the ball floats. Its accuracy is affected by the action of the current on the disc and the connecting rod.

The *twin float* is similar to the double float but has two balls instead of one. The balls can be adjusted to any required depths from each other or from the surface. The lower ball is loaded so that the required distance between the two balls is maintained.

Velocity rods.—*Velocity rods* are stout tin tubes an inch in diameter loaded at the foot with a small piece of iron. They are closed at the ends and float vertically with a small length extending above the water. The velocity of the rod is less than the mean velocity past its own depth and the immersed length of the rod should be 0.94 of that depth to measure the velocity correctly. They have the advantage of recording the mean velocity in one operation.

30. Current Meter.—(c) The *current meter* is an instrument used to measure the velocity of flowing water by means of a rotating element which is so constructed that,

when placed in the water, the revolutions of the cups in a unit of time will have a definite relation to the velocity of water. By placing a current meter at a point in a stream and ascertaining the revolutions per second, the velocity of the water at that point can be determined from the calibration of the meter.

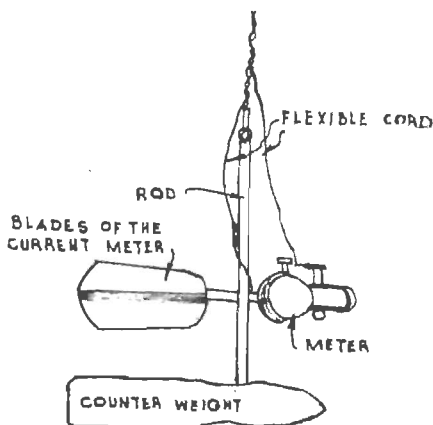


FIG. 6. Current Meter.

Current meters are of 2 types: (a) the direct action and (b) differential meter. The Off and Haskell direct action meter and the small Price differential meter are in general use. The Price meter is regarded as a universal meter and is equipped with a pentahead, a device for indicating every fifth revolution. The meter may be used for velocities from 1 to 15 feet per sec. in depths from 1 to 50 feet, from a boat, bridge, cable or by wading.

The velocity of flowing water is measured at several points in a cross-section and each velocity is multiplied by the corresponding partial area to obtain a partial discharge. These partial discharges are combined to give the total discharge.

Current meters are 'rated' by drawing them through still water at known velocities. Each meter must be rated before use.

When a river is gauged throughout the rainy season for different depths by the current meter, the station rating curve is drawn giving the relation between gauge reading and the corresponding discharge. Once a suitable curve is obtained, it is easy to convert any previous gauge reading into quantity of flow in cusecs.

E. Percolation, Evaporation and Transpiration

31. Rain water passes into the ground by "percolation" or gradual flow between and through the particles of the soil. The term "Absorption" is commonly used as interchangeable with percolation. But percolation refers to the movement of ground water under the action of gravity while absorption is retention of water in the pores of the soil by capillary attraction.

Seepage denotes percolation into or from soil. Soakage is only percolation into the soil, not from it.

The effect of percolation is to diminish the runoff from rainfall and to equalise it over the year.

32. **Soils.**—Soils may be divided into three classes for considering percolation: (a) Sound Rock, (b) Grit Soils and (c) Plastic Soils.

Stone is practically impermeable and percolation can take place only through cracks or fissures in it. Grit soils like sand are an aggregation of particles of hard material which do not become soft in water and which do not adhere to one another when moist. They are freely permeable. Plastic soil particles adhere to one another when moist or to particles of intermixed grit. They are only slightly permeable

when wet. Soils in nature are a mixture of grit and plastic soils.

The designs of irrigation works such as earthen bunds, weirs, regulators and masonry dams should aim at stopping all percolation through or under the works or at least of restricting it within the limits required for the stability of the works.

33. Loss by Absorption.—Loss by absorption is the total loss caused on irrigation by percolation and seepage. In a reservoir this loss depends on the nature of sub-soils in the basin and the pressure head of the stored water. In a canal, it varies as the wetted perimeter and with the nature of the soils passed through.

34. Loss in Transit.—Loss in transit is the decrease in the flow of the channel from the source of supply to the irrigated fields. It comprises (a) absorption and evaporation from the head of the main canal, through the branch and distributary channels and (b) the field channels and (c) wastage at the outlets or other masonry works and by over-saturation in some parts due to irregular distribution of water. The average wastage under (a) is about 25%, (b) is 20% and (c) is 25% so that the water applied on the land is only 30% of that supplied at the head of the canal. While designing new canals, a general allowance of 8 to 10 cusecs is made per million sq. feet of wetted perimeter, depending on the nature of the soils.

35. Evaporation.—Evaporation is the process of change from liquid into vapour. It is rainfall returned as water vapour from land and water surfaces to the atmosphere.

Evaporation is measured by means of a shallow pan floated on the body of water whose evaporation rate is to be ascertained. The pan is filled with water up to a certain

mark and filled up daily to the same mark. The quantity of water required for replenishment after making allowance for rainfall is the measure of evaporation from the surface of the pan and is expressed in inches of depth.

The evaporation from the water surface of a large reservoir is slightly less than that observed in evaporation pans.

The following table gives the monthly evaporation from Krishnarajasagara in inches.

Month	Inch depth	Month	Inch depth	Month	Inch depth
January	4·675	May	8·000	September	4·675
February	4·000	June	5·950	October..	5·950
March	5·000	July	4·675	November	4·675
April	7·000	August	4·675	December	4·675
				TOTAL	63·950

The annual evaporation from small irrigation tanks is of the order of 72 inches.

36. Transpiration.—Transpiration is the vaporisation of water from the breathing pores of vegetable matter. It is transpiration which maintains plant life on the earth since water is as necessary to plants as it is to animals. Transpiration is the process by which water vapour escapes from the living plant, leaves and enters the atmosphere. The source of water-supply for plant is soil moisture absorbed through the root system.

Transpiration ratio is the ratio of the weight of water absorbed, conveyed through and transpired from a plant during the growing season to the weight of dry matter produced exclusive of roots.

Consumptive use.—The consumptive use or evapotranspiration is the quantity of water in acre-feet per cropped acre per year absorbed by a crop and transpired or used directly in the building of plant tissue, together with that evaporated from the crop-producing land.

37. Total Evaporation.—Total evaporation is the sum of water lost from a given land area during a specific time by (1) transpiration from vegetation and building of plant tissue. (2) by evaporation from land and water surfaces and (3) by interception by trees. It is essentially the total precipitation *minus* runoff, corrected for change in volume of storage within the basin and for sub-surface leakage. It is fairly constant in amount during long periods of time.

38. Silt and Scour.—Silt is the non-floating solid matter carried forward by water flowing in rivers and canals and may be contrasted with drift which includes floating substances.

Silt is divided by Parker into two classes: (1) Bed silt which is rolled rather than carried forward along the bed of the river.

(2) Suspended silt, which being light, is carried forward in suspension, as turbid water.

The larger and coarser particles form the bed silt, and the finer particles are carried in suspension. Bed silt is more gritty and sandy while suspended silt is clayey.

In a canal, the silt content is due to (a) Original silt derived from the reservoir, or river supplying the channel and

(b) Derived silt eroded by the water from the banks or the bed.

Silt in tanks is both bed silt and suspended silt conveyed by the feeder valleys conveying runoff from the rain over the catchment.

Alluvial plains in the lower reaches of rivers are formed by deposition of silt when the river velocity is checked by the flat slope. Rivers flowing through alluvial plains have very tortuous courses and where a river outfalls into an open sea, there is a 'bar' of sand which partially obstructs the outfall channel.

39. The proportion of silt which can be conveyed by moving water depends on the velocity of water and the specific gravity, size and shape of the particles moved. It is important to design and construct earthen channels with such bed falls and cross-sections that they will neither be obstructed by the deposition of silt nor injured by the erosion of their beds or marginal slopes. According to Kennedy's theory, this critical velocity is a function of the depth $V = c.d^m$ and $c = 0.84$ and $m = 0.64$ for the Punjab area where Kennedy's observations were made $V = 0.84 d^{0.64}$.

40. The silt deposited in a canal is removed by:

- (1) Grading, sectioning and aligning the canal.
- (2) Flushing the canal.
- (3) Silt escapes and traps, and
- (4) Excavation, which is far too costly.

Large reservoirs also get silted up with the result that their effective capacity is reduced in course of time. A provision for possible silting must be made in the calculation of reservoir capacity. The silt carried in the rainy season may be excluded from the reservoir by means of scouring

sluices slightly above the deep river-bed which discharge the heavily silt-laden water at high velocity.

41. Soil Erosion.—The development of the soil and its subsequent erosion are a part of a long cycle of geological processes. The Irrigation Engineer is concerned with the depletion of agricultural land by these processes. It is in the uppermost layers, the top soil, that the complex processes of soil formation have been most active. This is the fertile layer furnishing the bulk of the plant food for crops.

Of the two agents of erosion, water and air, water is the more important. Erosion by water is of 2 kinds: (1) sheet erosion and (2) Gully erosion.

Sheet erosion is the process of slowly removing soil from all parts of the sloping surface of a field. It is caused by storm runoff developing a number of rills which scour the surface with minute channels which are eradicated by tillage operations.

Gully erosion is the development of relatively deep channels from rills not eradicated by tillage and become too deep for permitting normal farming operations. Conditions conducive to gully erosion are steep slopes, main water courses much lower than the surrounding ground and high rainfall intensities.

The protection against soil erosion is effected by vegetation through the binding force of the root system, the retardation and detention of runoff by plants and the reduction in runoff through the increase of infiltration.

42. Soil Conservation.—Measures by which erosion on agricultural lands can be controlled involve not only the more effective handling of cropped lands but a change in the use of lands not suitable for farming. Lands not suited for cultivation because of their steep slopes must be with-

drawn from farming and substituted by a protective perennial cover which reduces the volume and velocity of runoff. Crop rotation is used on cultivated lands to prevent the continuous planting of row crops on the same lands and to improve the quality and condition of the soil. Low terraces up to 18 inches high are made on rolling lands to reduce the velocity of runoff and to divert it to safe outlets. Bench-like terraces are used on steeper lands giving the hill side a step-like appearance. Cultivation is made on furrows coinciding with land contours to increase infiltration and reduce runoff. Gully control is effected by diverting runoff from the head of the gully, stabilising and protecting the sides with vegetation and preventing undercutting by small structures. The protection of forests and idle lands involves the prevention of the abuse and destruction of the natural cover.

A. Bund

1. **A Tank.**—A tank is formed by throwing an earthen “embankment” of trapezoidal section across a defined stream or depression. It is a small reservoir to store the rain water flowing in the valley. Water is drawn out to the rear of the bund by an opening in the bund called the “sluice” on one or both the flanks which is regulated by a suitable plug on the upstream side. A “channel” behind the bund from the sluice leads the water for being applied to the fields or “achkat” or “ayacut” for the purposes of irrigation. When the valleys feeding the tank bring in more water than can be stored in the tank as in the rainy season, the surplus is discharged into the main valley in the rear back again by a device located in the flanks called the “waste weir”. The weir acts as a safety valve and prevents damage to the bund by diverting the surplus which would otherwise flow over the bund and cause erosion or breach.

2. **Definitions.**—The crest level of the waste weir is the “Full Tank Level” and the level corresponding to the spillage over the waste weir while discharging the maximum flood in the stream is called the “Maximum water level”.

The “capacity” of the tank is the quantity of water available between the sill of the lowest sluice and the Full Tank Level.

“Duty” in irrigation is the relation between the volume of water used and the area irrigated, and is expressed as so many acres to the cusec. The ‘Base’ of the duty is the period of supply during which the flow is continued.

The tract of land draining into a valley is termed its "catchment basin" and the area of this tract is the "catchment area". The ridge dividing the catchment of one valley from that of another is termed the "Water shed" or "Ridge line" or "Divide".

"Runoff" is that part of the rainfall which flows off the catchment into the drainage lines.

A tank which yields an assessment of not less than Rs. 300 from its *achkat* is called a major tank. A Minor Tank has an assessment of less than Rs. 300 from its irrigable area.

"Construction" is the creation of a new tank. "Restoration" is bringing back to an efficient condition, a tank which from neglect or accident has fallen into disuse or become less efficient than before. "Improvement" is increasing the capacity of a tank or rendering it more useful for purposes of irrigation.

"Maintenance" means works of petty repair required to keep a tank at the standard of safety that may be fixed for it after its construction, restoration or improvement.

3. Bund.—The *top width* of the trapezoidal bund varies from $4\frac{1}{2}$ to 8 feet depending on its height, and where it carries a roadway, the width is increased from 12 to 18 feet.

The *side slopes* vary from $1\frac{1}{2}$ (horizontal) to 1 (vertical) to 2 to 1 in front and from 2 to 1, to 3 to 1, in rear depending on the nature of the material and the height of the bund. The upstream face is naturally saturated with water but it receives support from the water pressure on it and from any revetment or backing with which it may be faced. The down-stream slope depends mainly on two factors. The first is the "Angle of repose" of the material freely heaped up into a conical shape, in the dry state, and the second is

the "Saturation gradient" in the rear slope when the bank is full up to the M.W.L.

4. Saturation Gradient.—Earthen bunds holding water become saturated up to a surface starting at the water line on the front face and sloping downward through the bund. The actual plane of saturation through a bund is somewhat irregular, changing with the water level and with the soil. In a homogeneous bund standing on a similar foundation, the saturation gradient due to a steadily maintained water level would be a straight line and the steeper the inclination to the horizontal, the greater the resistance to percolation. The slope of the plane of saturation is called the "saturation gradient" or sometimes the 'hydraulic gradient' of the bund. If perforated pipes are sunk in various places across profiles of the bank, the level at which water stands in such pipes gives the line of saturation.

5. Design of Bunds.—As a first approximation, the rear slope should not be made steeper than the angle of repose for dry earth, since it is not expected to be saturated with water.

The angles of repose of the common materials are noted below:—

1. Dry Gravel .. 35° or 1.4 to 1.
2. Dry Earth .. 40° or 1.2 to 1.
3. Moist Earth .. 45° or 1 to 1.

To fix the rear slope finally, the front slope is taken as $1\frac{1}{2}$ to 1 and a saturation gradient of 3 to 1 or 4 to 1 is marked starting from the M.W.L. of the tank depending on the nature of the soil. With a top width of $4\frac{1}{2}$ to 8 feet, the rear slope is so designed as to keep this saturation gradient well within the profile. A margin of 1 to 4 feet may be left between the gradient and the surface of the bund.

In practice, the following dimensions may be adopted for repairing and restoring medium-sized tanks.

	Depth of bed below F. T. L.	Top width	Front Slope	Rear Slope	Free board
1	5' to 10'	4'	1½ to 1	2 to 1	3'
2	10' to 15'	5'	1½ to 1	2 to 1	4'
3	15' to 20'	6'	2 to 1	3 to 1	5'
4	Over 20'	9'	2 to 1	3 to 1	6'
5	For carrying a roadway	12' to 18'	Do	Do	Do

6. Types of Bunds.—Earthen bunds may be formed of material disposed in several ways in the profile of the work. The following three types are common in this country:—

Type A .. Of homogeneous material throughout the section.

Type B .. A 'hearting' of plastic material supported by a 'casing' of more gritty material.

Type C .. A bund with an impervious core wall of puddle or of masonry.

Type A.—The tank bunds of South India are mostly built of Type A and of soil dug from pits in the immediate vicinity of the section and carried by head loads to the bund. There are thus great varieties of soils used for tank bunds.

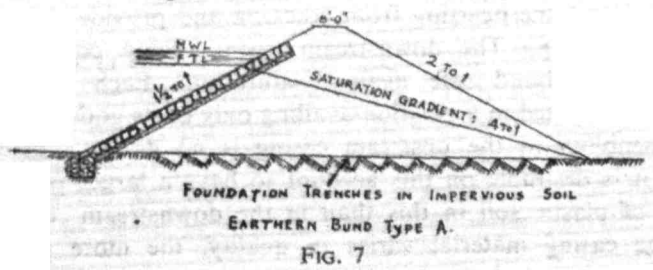


FIG. 7

Type B.—This type is largely used in Bombay Presidency and deserves to be adopted in Chitradurga, Dharwar and Belgaum in the Karnataka State, where the soil available for constructing the bank is on the surface black cotton soil to a depth of 2 to 4 feet, overlying decomposed rock with more grit than plastic material.

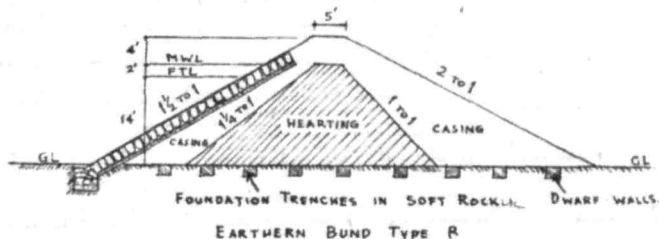
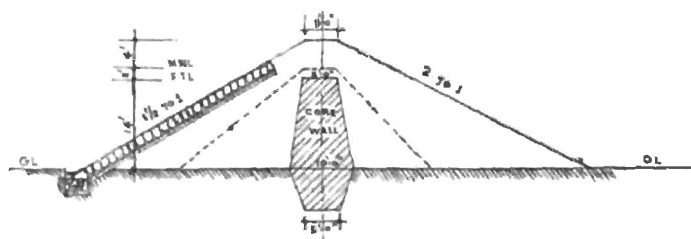


FIG. 8. Foundation of Bund in Soft Rock.

The surface of the bund made all of black cotton soil will crack when it becomes dry and when the tank is empty, these cracks are likely to extend into the heart of the bund and in the event of the rapid filling of the tank, may become a source of danger. There is also the fear of the front face slipping in when saturated. On the other hand, a bund formed throughout of the gritty sub-soil would leak freely and much water would be lost by percolation, which might attain a velocity endangering the safety of the bund. If, however, the plastic soil is used for 'hearting' and grit for 'casing', the former will stop percolation while the latter will protect the hearting from cracking and provide support from slipping. The downstream casing, being permeable, makes the bund safe against saturation. Each material is thus disposed in a position availing only of its good points. Permeability in the upstream casing is no doubt a defect and it is desirable on this account to have a larger proportion of plastic soil in this than in the downstream casing. If the casing material varies in quality, the more plastic

soil should be placed upstream, and for the rest of the section, the more impervious casing should be close to the hearting, and the most permeable on the downstream facing composing the toe of the dam.

Type C.—This type differs only in degree from Type B, the comparatively thin puddle wall taking the place of the 'hearting'. The bulk of the material on either side of the core wall, particularly on the water side, should be of the most impermeable material conveniently obtainable, and the wall used as a water-tight bulk-head to intercept any percolation which passed through the upstream thickness of the bund.



Earthen Bund Type C
FIG. 9

The clay hearting, of Type B, combined with a puddle trench, as in sketch, is another combination, suitable where good puddle clay is not available in sufficient quantities.

7. Core Walls.—Core walls of rubble masonry are not suitable for common use in small tanks as they need a rocky foundation and as they are costly and difficult to build properly in out-of-the-way places. They may be adopted in high bunds, particularly at the stream crossing, to form a stop wall below the foundation level of the bund, the top of the wall being carried to a sufficient height into the bund to guard against the 'creep' of water past the wall.

8. The Seat of the Bund.—The surface of the ground at the selected site should be dug or ploughed along the length of the bund and vegetation and roots removed. If the soil is fairly impervious, trenches as in Fig. 7 should be dug to form a good joint with the new earth to be laid on it.

Where the foundation is soft rock, it should be trenched as in Fig. 8 and if it is hard rock, a serrated joint may be made between it and the earth by a series of dwarf walls similar to the trenches in Fig. 8.

When the surface is of permeable soil overlying an impermeable stratum, the former may be either be altogether removed, and the bund situated on the latter, or, one or more trenches dug down to a foot or two into the impermeable stratum, and filled with the best available puddle, to form stop walls. In exceptional cases, a masonry wall may be used as a stop wall. When the permeable soil goes down to a great depth, the stop wall, whether of puddle or masonry, must be of such a depth as to prevent percolation attaining such a velocity as to endanger the solidity of the foundation by piping, and to prevent excessive loss of impounded water.

In connection with the seating of the bund, it should be considered if it is necessary to secure the foundation or outer slope of the bund from settlement or slipping by the introduction of drains under the outward slope, considering the permeability of the foundation soil.

9. Testing the Impermeability of a Tank-Bed.—If the surface soil in the bed of the proposed tank is not more pervious than the sub-soil, pits should be dug in various places in the bed to a depth of about 3 feet, which should be filled with water and kept full for 24 hours, and the rate per hour at which the water surface in each pit falls should

then be observed to compute the loss from combined percolation and evaporation per unit area.

If the surface soil is pervious but the sub-soil can be relied on, the pits should be taken down to sub-soil and the sides lined with clay so that all percolation loss will take place through the bottom of the pits. The rate of percolation should be observed as above.

10. Testing Soil for the Bund.—A test tank should be marked, 10 feet square, with a top width of 3 feet, side slopes of $1\frac{1}{2}$ to 1 and of depth 3 feet, with a free board of $1\frac{1}{2}$ feet. The tank should be kept filled with water for a week to within $1\frac{1}{2}$ feet from the top, any percolation loss being made good at intervals. The actual slope of the percolation plane may then be observed. The qualities of the soil as regards slipping when saturated may be observed by gradually cutting back the outer slope.

The quality of a plastic soil, or of a mixture of plastic and gritty soils to resist saturation by water, may be tested by making up balls of soil, 4 or 5 inches, in diameter, after damping and kneading in the hands until it becomes a soft plastic mass. The balls may be immersed in *still* water $1\frac{1}{2}$ to 2 feet deep and left there. A good clay soil will resist saturation for days and even for weeks.

11. Junctions in Bunds.—Junctions in earth work should be levelled in section and joggled in plan, as shown in Fig. 10. The former have to resist tendency to leaks and the latter to slips.

Junctions to a depth of more than 20 feet should not be made in one season. Deep junctions of this kind should only be necessary while closing a flood gap in a new bund or closing the breach in an old one.

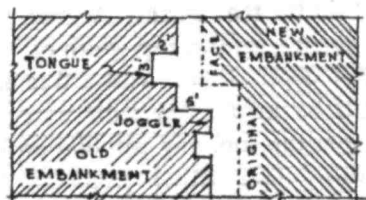
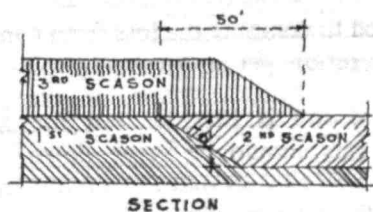


FIG. 10. Junction of Earth Work.

Where an old bund has to be widened by the addition of earth, the old slope should be cut into steps with the

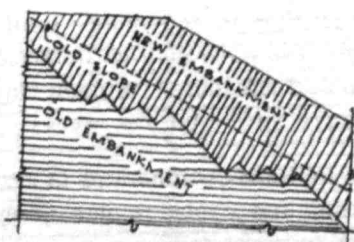


FIG. 11. Junction of New Bund with Old.

treads sloping slightly towards the centre of the bund, and when throwing on the new soil, the surface of the old work

should be wetted so that the new earth may adhere properly to the old.

12. Drainage.—Drainage is necessary to remove water which may be met with in the heart of the bund due to percolation or the wetting of the rear slope by rain, or along the junction of the base of the dam with the ground, or through the strata underlying that base. It is effected by rough stone drains built under the outer slope of the bund to provide a path through which percolation water can find ready exit.

These drains are merely trenches filled with rough stone and protected from being choked with soil washed into

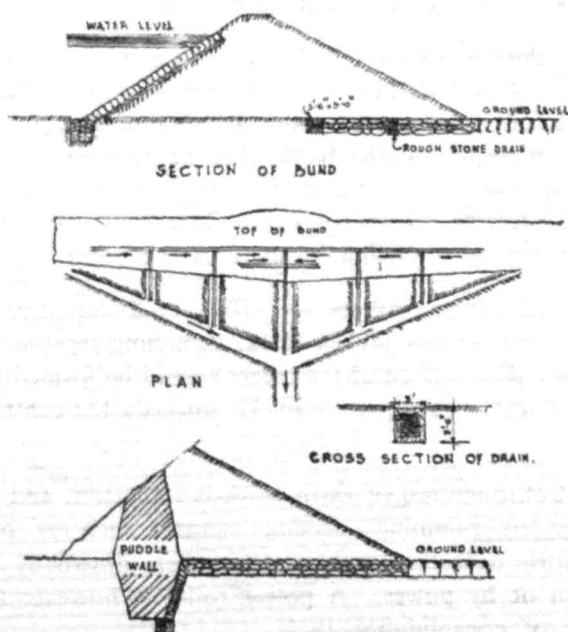


FIG. 12. Drains in an Earthen Bund.

them from the bund, by being enclosed in casings of broken stone or pebble or quarry chips to form a strainer. The water from the drains must be led away from the toe of the bund.

Figure 12 shows a plan and section of a typical system of drains. The drain *a a*, parallel to the length of the bund, is given a bed fall towards the outfalls generally following the fall of the ground. Drains *CC* at right angles to the bund are also provided with a suitable bed fall and when clear of the bund are led away in regraded open channels. When the sub-soil is permeable and a puddle trench is put in, a drain *E* along the downstream lower edge of the trench is desirable. Such a drain is connected at intervals with the outfall drains by drain shafts *e c*.

13. Borrow Pits.—It is desirable to make most of the pits in the tank-bed as these will silt up more rapidly than those on the outer side. The position and limits of depth of pits with respect to the bank must be decided before commencing the work. No pits should be excavated nearer than 30 feet to the toe of either slope. All clods must be broken up before being brought to the bund.

14. Execution of Work.—(1) The bund should be formed with layers 6 to 9 inches thick, stretching right across the section. The surface of the layers should be formed so as to slope inwards at about 1 in 12 towards the centre of the bund.

(2) Consolidation of earth work is important and may be effected by ramming, driving animals such as buffaloes and cattle over the layers, or by rollers drawn by animals or men or by power. A power roller is however the best means of consolidation.

(3) On no account must a bund be made of less than the full width with a view to widen subsequently; this leads to the formation of a joint which is likely to start a slip.

(4) When puddle is used, it must be thoroughly worked up into a plastic mass with as little water as suffices to bring it to this state, and when placed in the work, it must be well incorporated with the layer below. Care must be taken to prevent the puddle in the trench becoming dry, as this will cause cracks.

(5) Where practicable, the bank should be given time to settle before the revetment is built; the settlement of a new tank may cause the revetment to bulge.

(6) The following allowance for shrinkage has to be made to start with so that no addition of earth will be needed to bring the bund to the designed dimensions after settlement.

1/12 or 1 inch per foot, in firm compact earth

1/8 or 1½ inches „ in ordinary loose soil and

1/6 or 2 inches „ in black cotton soil.

15. Closure of the Dam.—On account of the largeness of monsoon floods, it is not practicable to divert them outside the bund during its construction by bye-wash channels and they have to be passed through the bund in a gap left for the purpose. The closure of a high dam is a difficult work as it ought to be effected in a single season and well before the commencement of the monsoon.

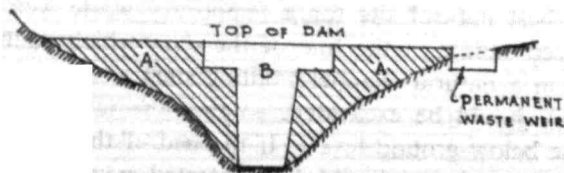


FIG. 13 (A). Usual Closure.

In small tanks, it may be effected as shown in Fig. 13 A but it is preferable to carry out as in Fig. 13 B. In the latter

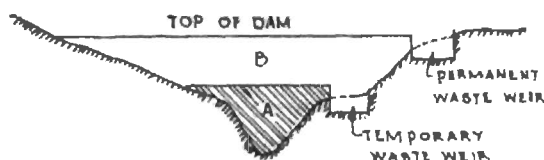


FIG. 13 (B). Revised Method of Closure.

arrangement, the bund is raised in continuous level stages, commencing with the one at its lowest base at A and working up gradually to that at the crest at B. To provide for the passage of flood water, a temporary waste weir is made at a suitable place in the longitudinal section to discharge the maximum flood with safety. The advantages are obvious:

(i) The junction between A and B is of minimum height minimising the difference in settlement.

(ii) A is at first subjected to infiltration only from a shallow reservoir which cannot penetrate it far and keep it green. All the upper part, except the low closure of the temporary waste weir, can be allowed to consolidate before water comes against it slowly and evenly. Only the low closure of the temporary weir will have to be done in one season.

(iii) The construction of 'A' will form a small reservoir which will be useful for works and saves water charges.

The best natural site for a temporary waste weir is at a depressed saddle in the line of the dam which confines the floods in a natural channel; failing such a site, an artificial channel should be excavated, so that the tail flood surface may be below ground level. If the bed of the draft channel should be soft, it should be protected near the bund by pitching or by masonry curtain walls. The discharging

capacity of the temporary weir should be a little greater than that of the permanent one, since the bund will be in an unfinished state.

16. Revetment.—The water faces of a bund are usually protected from erosion by rough stone revetment which acts to some extent as a retaining wall, so that a revetted slope will stand at a steeper slope than a non-revetted one.

The thickness of revetment must be decided in each case on its merits. $1\frac{1}{2}$ feet thickness of rough stone with a packing of 6 inches thick broken stone, quarry rubbish or gravel is common, though in small tanks with low revetment, a foot thickness is sufficient.

The top of the revetment is carried to 1 foot above M.W.L. of a tank. The foot of the revetment should be carried 2 to 3 feet below the foot of the protected slope and should be widened out at the toe to distribute the pressure over a wide area.

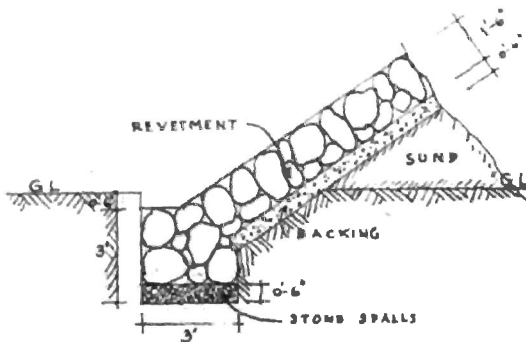


FIG. 14. Revetment.

There should be a large proportion of through stones in the revetment with due regard to bond, and the interstices between stones should be wedged tight with spalls driven in with a hammer.

The courses should be horizontal. At sluices and at places where people have to get down for drinking and washing purposes, revetment should be built in steps.

For tanks in a country of black cotton soil where lead for stones is great, turfed 3 to 1 slopes are frequently adopted but the turf dries up in winter and the slopes are washed down to 1 in 4 by waves in the rainy months. It would be better to provide jelly packing 9 inches thick or to plant 'nanal' grass close on the slopes.

In malnad tanks, 2 to 1 earth slopes properly turfed will stand as they are generally kept moist and are overgrown with evergreen shrubs and plants.

17. The causes of failure of earthen bunds are:—

- (1) Percolation.
- (2) Slipping.
- (3) Overflow, and
- (4) Erosion by waves.

(1) *Percolation*.—Failure by percolation is due to water which enters either through the bund or under its foundations carrying with it particles of soil, thus forming tunnels which once formed, rapidly enlarge forming cavities into which the bund sinks, thus enabling the water to overtop it and carry it away by erosion.

The cause may also be 'piping' due to the insufficient length of the path of percolation and a resultant percolation velocity too high for the soil to sustain. Inadequate profile or want of an effective stop wall to obstruct the path of percolation would be the reason for such a failure.

More frequently, failure by percolation is attributable to the opening through the bund, or under its base, of a passage for water either by cracking of the soil or by the

burrowing of animals, or by roots of trees which have decayed. The precautions to be taken against this contingency are:—

(i) To construct the bund of material not liable to crack under the conditions to which it will be subjected.

(ii) To cover the upstream slope of the bund with a layer of material through which the burrowing animals will not penetrate.

(iii) To keep both the slopes and the top of the bund clear of trees; or whenever a tree dies or is cut down, to open the bund and cut out the roots, filling in the hollow with good earth, well consolidated.

(2) *Slipping*.—A slip in the earth is due to the earth standing at a slope steeper than that at which it can be retained by the combined friction and cohesion between the particles composing it.

Both friction and cohesion are affected in plastic soil by the amount of moisture absorbed; in dry plastic soil, both are considerable and while a small quantity of water increases adhesion, excessive absorption reduces both cohesion and friction, and a saturated plastic soil is likely to become slush. Plastic soils are also liable to crack when dry.

Grit soils, on the contrary, are little affected by moisture; they have little cohesion but friction is considerable and increases with the size of the particles. These soils are not liable to crack when dry. Pure grit is freely permeable and the larger the particles, the greater the permeability.

A mixture of plastic and grit soils would be a suitable material for an earthen bund as in Type B, with a plastic hearting and a gritty casing. Natural mixtures of plastic and grit soils are frequently used for bunds of all types.

(3) *Overflow*.—If the tank level rises to such a height that it flows over the top of the bund, failure will speedily occur by erosion. This must be guarded against by suitable surplus works.

(4) *Erosion by Waves*.—Failure by erosion may take place by the water face of the bund being cut into and eaten away by the action of waves, or erosion by wind and rain so that the slope becomes so steep that the upper layers slip, reducing the top width and causing failure by over topping. This is prevented by revetment or pitching of the water face or by turfing or growth of shrubs.

18. Closing of Breaches in Tank Bunds.—(1) All water should be cleared out of the bottom of the breach and all soft mud removed, to provide a proper foundation. Loose soil should be removed from the ends of the bund on either side of the breach.

(2) It will sometimes be necessary to introduce a fresh puddle wall in large tanks which should be located at or above the centre of the embankment. The top width of the puddle wall is 2 feet and the top level, a foot above the M.W.L. The cross-section of the foundation trench and of the puddle wall is sketched in Fig. 9.

The following precautions are needed during construction:—

(i) The puddle should not be made too wet; it should be well tamped;

(ii) The battered faces of the wall should be prevented from drying up, by making up the earth work simultaneously, keeping it well watered and tamped;

(iii) The ends of the puddle wall should be carried into sound non-porous soil and a sound joint effected.

(3) After the scour has been filled in, the soil for the new filling should be thrown evenly over the whole area, commencing at the front and rear toes of the slopes and working towards the centre, taking care that the filling in of the cuts at either end, keeps pace with the rest of the work.

(4) When all the soil is not good, selected earth should be used for the front of the bund, while sandy or gravelly soil will do well for the rear portion.

(5) The reconstruction of the revetment can, with proper consolidation of the earth work, be carried on simultaneously with breach filling. Where unrevetted, the slopes should be turfed.

(6) Extended scour on the water side need not be filled in, but such scour in the rear should be carefully filled up to ground level and consolidated.

(7) The top of the bund at the newly-filled in breach should be made higher than the old part of the bund by about $1/16$ th height of filling, to allow for settlement and should be gravelled so that rain water may not soak in.

19. Silting in Tanks.--The silting up of tank-beds is a natural phenomenon as the flood waters are muddy and in old tanks, the problem of the raised tank-bed with vastly reduced storage is causing anxiety about the safety of the bund and about the sufficiency of water to the irrigated fields.

In Mysore, where the tanks are in series, the upper tanks get silted up more rapidly and their capacities are reduced, making the *achkat* suffer from inadequate supply. The floods pass over the waste weir more frequently and when the independent catchment of a lower tank brings in floods simultaneously the bund is in danger of being overtopped at the low places, in a year of unprecedented rainfall, if

the waste weir is not of sufficient discharging capacity. The number of tanks so breached may be alarmingly high and even larger than the number restored particularly in the malnad area where the tanks are in the interior and inaccessible even for inspection, not to speak of improvements. This factor is responsible for the phenomenal fall in the irrigated area under tanks in Shimoga, Chickmagalur and Hassan Districts.

Though the problem is so serious and so much talked about, an effective remedy is not yet discovered so that the situation continues to worsen steadily, year after year. The vastness of the problem and the un-spectacular nature of the work come in the way of concerted Government action.

Whatever may have been the circumstances under which the ryots did not take interest in their village tanks in the past, it now behoves the representatives of the people and the Voluntary Service Organisations to take to this quiet constructive work with the enthusiastic co-operation of the ryots. A start may be made in the Community Centres.

20. Prevention of Silt.—(1) Silt should be removed by hand and carted for use as manure by the *achkatdars* for their fields.

(2) Suitable plantations may be raised in the upper margin of the tank to arrest further accumulation of silt and to reduce the silt-carrying capacity of water.

(3) The excavation of a trench in the upper margin above the M.W.L. of tank may prevent the heavier silt from getting into the tank.

Dredging by mechanical means has been suggested, but its cost is prohibitive even in maidan districts where the tanks are served with motorable approach roads.

21. Hydraulic-Fill Dams.—An earth dam in the construction of which the materials are transported on to the dam by water and distributed to their final position in the dam by water is a Hydraulic-fill dam.

If the materials are transported and dumped within the dam section by other means but some of the material is moved to its final position by the action of water, it is a semi-hydraulic-fill dam.

The hydraulic-fill method deposits the material from flumes or pipes near the faces of the dam. The larger particles stay there and the finer particles move towards the centre. Thus the toe and faces of the dam are more pervious and allow water to drain out from the interior of the dam. But in semi-hydraulic-fill dams the toe and faces consists of material dumped by lorries.

22. Rock-fill Dams are used in remote locations where the cost of cement for a concrete dam would be high, where suitable materials for earth dams are not available, where rock can be quarried near the dam site but where the foundation at the dam site may not be suitable for a masonry dam.

A rock-fill dam has 3 parts: (1) the dumped rock-fill, (2) an upstream rubble cushion of stone bonding into the dumped rock and (3) an upstream impervious facing resting on the rubble cushion.

The water pressure on a rock-fill dam is resisted by the weight of the rock. Rock-fill dams are cheap to construct, but they depend on the downstream facing for their stability for, when that is injured, the pressure of water will disintegrate the hearting and carry away the stone blocks.

Composite rock-fill and earth dams are adopted for bigger heights up to 400 feet.

23. Use of Machinery.—All the earth work on tank projects was carried out in the past in this country with the help of manual and animal labour. Since the beginning of this century, earth work involving long leads and large quantities is being conveyed by light tramways using tipping wagons of 1 cu. yard capacity. Consolidation on deep embankments is being done by steam road rollers. But conditions have changed fast in some parts of the country since the Second World War. Projects are planned on a large scale and the speed of earth work possible by trollies hauled by manual labour is considered too slow and Earth Moving Equipment is being increasingly used.

But before deciding to use machinery on tank projects, three points have to be considered.

(1) How does the overall cost of doing the work with machinery compare with the cost by the traditional method ?

(2) What is the proportion of the cost that goes outside India when machinery is used ?

(3) Cannot the job be done with trollies and steam road roller at all within a reasonable time by planning the work properly ?

Except in places where the wages for labourers are high and where facilities exist for attending to the repairs of the machinery, the cost of using machinery is not much less than that by manual labour in most parts of the country until the machinery can be made in the country. India can ill-afford the luxury of building ordinary tanks with the help of imported Earth Moving Equipment. The cost of spares, depreciation, repairs and fuel will make their use prohibitively costly on moderate sized tanks at present. Where the quantity of the earth work is large and where time is of importance, we cannot possibly do away with the use of machines. One great advantage is that the work

can generally be finished in one working season so that arrangements for the passage of floods in the working season and the final closure of the bund will be avoided. But to get the best out of the equipment, the various operations must be planned, all delays avoided so that the machinery is not kept idle, the drivers and operators must be specially trained, repairs to units must be effected at a local workshop, and there should be proper supervision to co-ordinate the different processes. The size of the equipment must be chosen judiciously with reference to the local facilities for transportation, personnel available and availability of Electric Power.

24. Two processes are executed by mechanical equipment on a tank project: (1) Excavation of earth and conveyance to bund and (2) Consolidation on the bund.

Equipment for excavation and conveyance is of 3 types: (a) Excavators, (b) Combined excavating and hauling units and (c) Hauling units.

(a) An "Excavator" moves earth while the main unit is stationary and its range is limited to the reach of the boom which carries the bucket. They are built to take a variety of interchangeable attachments or "front-ends" to move earth under different conditions.

A "shovel" is used to dig at or above operating level and to load into hauling units. The boom of the shovel is not interchangeable with other attachments.

A "hoe" is meant for digging trenches and foundations below operating level and disposing of the material like a shovel.

A "drag line" is used for digging at or below operating level and loading into hauling units standing below, at, or above the operating level.

A "clam shell" excavates below, at, or above the operating level and loads into hauling units.

Excavators are mounted on crawlers and are built in several sizes with bucket capacities from $\frac{3}{8}$ to 6 cu. yards. Smaller machines upto $\frac{3}{4}$ cu. yard capacity are available on rubber tyres also.

(b) *Combined excavating and hauling units* do their work while the entire unit is on the move.

Tractors.—The operation range of tractor of the crawler and pneumatic tyred type is 200 feet one way haul when fitted with dozers. When fitted with front-end loaders, the range is 100 feet. Loaders are built with $\frac{1}{2}$ to $\frac{3}{4}$ cu. yard capacity.

Scrapers are built from 3 to 32 cu. yards capacities.

Motor scrapers with a range of 500 to 7,000 feet, one way, are built in sizes from 5 to 30 cu. yards.

Four wheel tractors with self-loading scrapers are built with an operating range of 500 to 1,000 feet one way, with a capacity of 10 cu. yards.

These machines operate without loading devices.

(c) *Hauling Units.*—Hauling waggons are pulled by the power units of the motor scrapers or by four-wheel truck type tractors. They are usually of the 2-wheel type. Waggon may be of the "bottom," "side" or "rear dump" type, built in sizes from 6 to 45 cu. yards.

While scrapers are self-loading, waggons, trucks and front-end dumpers must be loaded by a separate unit, by means of shovels, hoes, or draglines or clam shells mounted on tractors.

Another method of loading waggons and trucks is by the belt loader and elevating grader, pulled by one of the large-

sized crawler tractors. The belt loader digs the soils in a thin layer, transfers it to an inclined belt and spills it over the sides of the waggon or truck which move with the belt loader.

25. Prime Movers.—Engines for motivating modern earth moving equipment are internal combustion engines of the spark-ignition type running on petrol, or of the compression-ignition type running on diesel oil. Spark ignition types also use mixtures of petrol and power alcohol or kerosene with a compression ratio from 4:1 to 10:1. Compression pressures vary from 100 to 200 lb./sq. inch. Engines work on the four-cycle principle with carburettors.

Compression ignition engines suitable for two-stroke operation use high speed diesel oil with compression ratio from 1:13 to 1:18 and compression pressures from 400 to 700 lb./sq. inch.

26. Land-Clearing Equipment.—This is based on the bulldozer idea with special blades and attachments. One is the tree-dozer with 2 movements for tree-felling. First a blade is inserted underneath the roots and then a controlling arm pushes the tree as it is dug out.

Irrigation projects are located in out-of-the-way places where roads do not exist and the construction of access roads is the first step in starting the work.

Where shovels are used, the hauling units would be rubber-tyred self-propelled waggon with bottom or side dump. Heavy duty tractors are used for pulling the loading in "tough going". Bull-dozers do the cleaning up around shovels.

Where motor scrapers are used, tractors are used as loading helpers and bull-dozers for keeping the borrow pits and loading sites in condition.

A Crawler-tractor can maintain its own haul roads with a bull-dozer or by means of a Scraper's cutting edge.

27. Rolling.—Rolling is the usual method of compaction but rollers vary as the soil conditions vary. The flat roller drawn by steam or diesel engines is the common type but the layers of earth must be limited to 6 inches. The previous layers must be picked and watered before fresh material is added and rolled with watering.

Sheep-foot Rollers are generally associated with the use of mechanical equipment on an earthen bund. The light type can give a bearing pressure of 350 lb. per sq. inch while the heavy type gives 1,600 lb. In practice, the number of 'passes' from 5 to 20 is specified, based on experience with the particular soil. Soil layers vary from 6 inches to 1 foot. Sheep-foot rollers can be varied in weight by having their drums filled with water or sand.

Grid Rollers.—This is a round drum, the rolling surface of which is built of metal rods that form a grid. The rods act as tampers on the confined soil effecting a higher degree of compaction on silt and silty material. Sheep-foot rollers, on the other hand, are satisfactory for consolidating loams or clays.

Grid rollers are not in common use in South India. Pumps have to be used for supplying water for watering the layers of the earth in the embankments.

28. The Advantages of Earthen Dams.—The popularity of earthen dams is due to the following reasons:—

- (1) They do not need solid and expensive foundations;
- (2) They can be built by unskilled labour and peculiarly adapted to the employment of famine labour;
- (3) The construction materials are close at hand.

(4) This is the cheapest type and can be built to any height by using suitable precautions.

(5) They can be raised from time to time to meet the demand for more water.

B. The Outlet

29. The outlet or the sluice is the work by means of which the water in the reservoir is passed safely through the bund for being utilised for the irrigation of the *achkat*. It consists of:—

- (1) A front head or forebay with regulating arrangements;
- (2) Tunnel or pipe or barrel through the bund; and
- (3) A rear head.

The usual types of forebay are sketched in Fig. 15. The front head should be raised to a sufficient level to carry a platform at or above M.W.L. for regulating the sluice. Where a head wall is provided, it must be connected with wing walls of suitable length and slope to retain the earth slopes of the bund.

The regulating arrangement for moderate-sized tanks consists of: (a) an upper orifice, (b) a lower vent at sill level, and (c) gibbet stones to support one or more horizontal stones with holes in them as guides for the plug or gear rod.

The upper orifice is usually circular and the lower one rectangular. The dimensions of upper orifices are calculated to give the required discharge for irrigating the total *achkat* with a minimum head of 2 feet, assuming a duty of 40 acres to the cusec.

Cylindrical plugs are in use with round or square heads with plug rods or chains fixed to them, but this does not permit of varying the discharge according to the extent of cultivation or the necessities of the season.

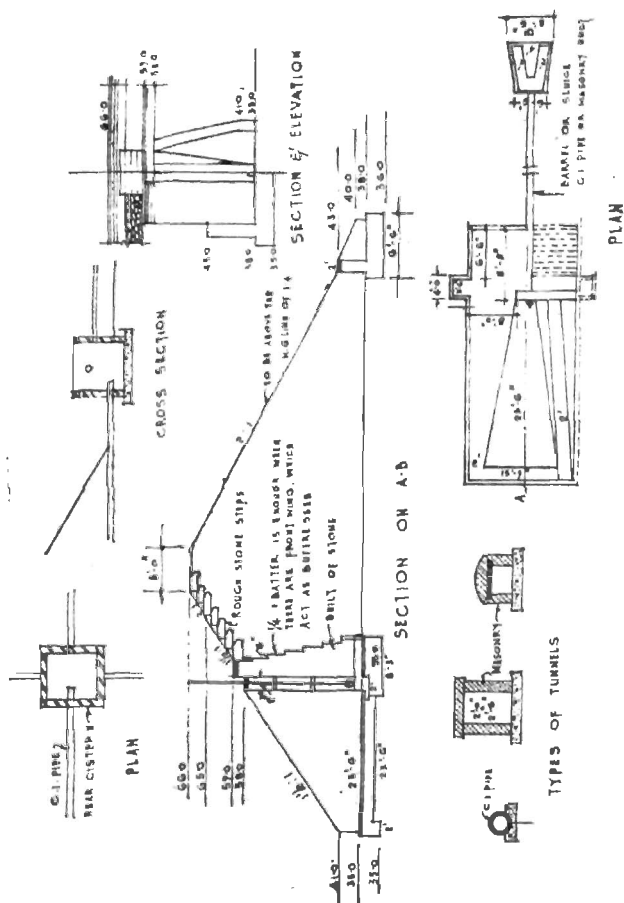
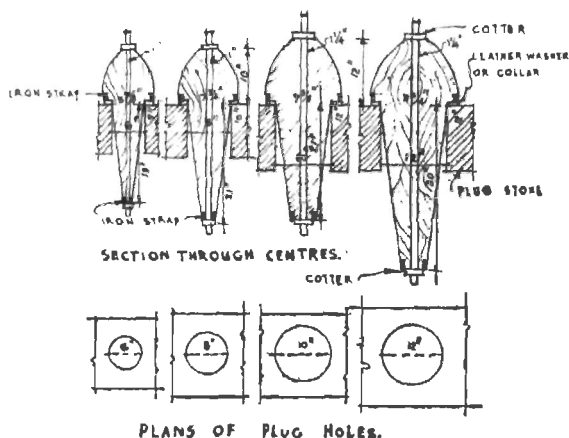


FIG. 15. Design of a Tank Sluice,

The area of opening is, on the other hand, regulated by a long conical plug the wider end of which has the same diameter, allowing for clearance, as the vent or 'plug hole'. As the plug is raised, the orifice is opened more and more and when it is raised clear of the plug hole by an amount not less than the diameter of the hole, the full discharge of the orifice is obtained.

30. **Plugs.**—Details for plugs for tank sluices are shown in Fig. 16. The standard sizes are 4, 6, 8, 10 and 12 inches diameter, this being the size of hole in the stone, the largest



rod is long, it should be passed through holes in guide beams at fixed intervals. The top end of the rod passes through holes at about 3 inches interval in a flat iron on the regulating platform through which an iron pin is passed to keep the plug suspended at whatever height it gives the suitable elevation.

Where the plug holes exceed 10 inches in diameter, it is advisable to provide two or more holes and plugs in the plug platform stone, while for deep sluices of over 25 feet depth, two or more plug stones at different levels, each with separate plug give greater facilities for easy regulation.

The lower rectangular vent at sill level is meant for operation when water in the tank is less than 4 feet over the sill of the sluice and at other times, it is kept closed by a stone slab. In larger tanks, this vent may be covered by flat shutters working in grooves and regulated by screw spears.

31. Shutters.—Flat shutters should not be used for tank sluices except in the low water vents unless they are fitted with screw gear, and even when so fitted, they are undesirable except when large quantities of water have to be passed.

For heads of over 25 feet, special balanced valves or shutters moving on rollers are usually provided.

32. Sluice Tower.—A tank sluice with a tower head is sometimes more economical than the head wall type owing to the saving on the heavy wing walls; there is also less danger of failure by cracking or bulging. On the other hand, the length of barrel is greater. Tower heads are generally placed in the water slope of the bund, saving the expense of a bridge leading from the bank to the regulating platform. There is, however, the disadvantage of making it impossible, while the tank has water, to

have access to the portion of the barrel upstream of the wall.

33. The Barrel.—The barrel may be of the following types:

- (1) Masonry tunnel, slabbed or arched over.
- (2) Earthenware or concrete pipes.
- (3) Cast iron pipes.

Tunnel.—The masonry tunnel is the most favoured type and may be built of stone, brick or concrete with a section either rectangular or like an inverted ovoid. The dimensions of the culvert will be determined by the maximum quantity of water it is intended to convey but not less than $2 \times 2\frac{1}{2}$ feet (high). The velocity through the barrel should be limited to 15 feet per second under the condition of the plug being fully open with the water level at F.T.L.

Earthenware Pipes.—It is not desirable to use earthenware or cement pipes where the depth below F.T.L. exceeds 8 feet as the fracture of the pipe or the leakage of the joints may result in a breach, while examination or repairs cannot be carried out without cutting open the bund.

Cast Iron Pipes.—It was not formerly, economical to use Cast Iron pipes for sluices but with the manufacture of these pipes at Bhadravati, their use is getting to be common up to a diameter of 30 inches.

34. Foundation of Sluice Culverts.—It is important to secure a uniform foundation soil throughout the length of the culvert so that the settlement may be uniform. The width and thickness of the concrete slab should be settled with reference to the foundation soil and the weight and nature of the earth above the culvert. If there is a puddle trench, the crossing is a point of weakness and the puddle wall may be stopped on either side of the sluice and a masonry stop wall substituted.

35. Super-Structure.—The sluice culvert is generally covered with masonry arch work, of stone, brick or concrete. When the bank is not high, stone or R.C. slabs may be substituted for the arch. Where stone slabs are laid, a layer of lime concrete 6" to 1' thick should be laid over them to prevent leakage.

The length of the barrel has to be designed to suit the cross-section of the bund.

A sluice is always regarded as a weak point in a bund owing to the possibility of water under pressure to creep along the joint between the masonry barrel and the earth. Where the head over the sluice exceeds 20 feet, two or more stop walls may be built along the upstream two-thirds of the barrel to form an obstruction to creep.

36. Rear Cistern.—In small tanks, the barrel discharges into the irrigation channel and is connected to the bank by suitable wing walls. But in larger tanks, it usually delivers into a rear cistern (Fig. 15). Where several small channels take off at different levels, this form is convenient, as notches and vents in the cistern can be built to deliver the water at the appropriate levels. The rear wings and returns of sluices are sketched in Fig. 15 and are similar to sluices in canal distributaries.

37. Level of Sill.—The following points should be noted in deciding the sill level:—

(i) The level of the highest and lowest fields and the mean level should be ascertained and the sill fixed between the mean and the highest levels.

(ii) Space should be left below the outlet sill for the accumulation of silt in the tank up to about 10% of the gross capacity.

The heading of information which is generally recorded in the design of a tank sluice is shown in the following table:

1. Area to be irrigated	Acres
2. Discharge required at....duty ..	Cusecs
3. Area of plug hole	Sq. feet
4. Diameter of plug hole	Inches
5. Sill of lower vent	R.L.
6. Level at which required discharge can be secured through the plug hole ..	R.L.
7. Level at which required discharge can be secured through the lower vent ..	R.L.
8. Bench mark to which levels are referred	Sill

38. Repairs to Sluices.—When a sluice is complained to be leaky, the first step should be to examine the front head and tunnel. If, after examination and repairs to this portion, the leaks continue, the bund may be cut, allowing $\frac{1}{2}$ to 1 slope on either side. The excavated stuff should be placed some distance beyond the edges of cutting to prevent slipping of sides by the load at the top. Temporary fencing must be put up to prevent accidents to men and cattle till the refilling is completed.

The sluice barrel will then be examined and the necessary repairs executed. To prevent creep of water along the new cut, three masonry collars, 3 feet long may be added on either side of the tunnel, one at the front head, one in the middle and one at the rear head. All debris, brick bats and stone chips should be removed from the sides of the tunnel and the space between the face of cut and the tunnel carefully filled in with good clay put in layers and consolidated.

While filling the cut in the bund, dentals should be made on each side to grip the new earth with the settled bund. The filling with the excavated stuff should be made in thin layers, concave towards the centre, watered and tamped to prevent cracks at the junction.

The rear of the sluice must be drained and if there should be any tendency for the formation of pools, rear revetment should be provided for the sluice filling.

39. Computation of Tank Capacity.—The gross capacity of a tank is the quantity of water impounded up to F.T.L. while the effective capacity is that between the sill of the lowest sluice and the F.T.L. The capacity is computed from the contour map of the water-spread area.

If A_1 is the area of water-spread at sill in sq. feet, A_2 the area at F.T.L. and d the difference in level between sill and F.T.L., in feet, $Q = \frac{1}{3} (A_1 + A_2 + \sqrt{A_1 A_2}) d$. This formula can be applied to know the quantity of water between any two given intervals and the smaller the contour intervals, the greater the accuracy of computation.

When great accuracy is not required or when it is not possible to ascertain the water spread at levels below the F.T.L.; the following formula may be used:—

$Q = \frac{3}{4} \frac{(d_1 + d_2)}{2} A$ where Q is in cubic feet, $d_1 =$ Av depth of tank below F.T.L. along the valley in feet.

$d_2 =$ Av depth of tank below F.T.L. in the front of bund (along a line parallel to bund in the tank-bed.

and $A =$ Area of water spread at F.T.L. in sq. feet.

40. Yield from a Catchment.—The total amount of water which runs off the surface owing to the rainfall on it is known as the 'Yield' off the catchment and is what matters to the Irrigation Engineer. It depends upon:—

- (1) The size and shape of the catchment.
- (2) The nature of the surface; whether hilly, sloping or flat, barren, grass-clad or cultivated; treeless or wooded; impermeable or porous; wet or dry; free from, or containing minor reservoirs and swamps.
- (3) The extent, intensity and frequency of the heavy falls of rain; and the position of the area with respect to the monsoon storms.

The proper replenishment of a reservoir must be assured in the many lean years rather than in the few years of scarcity. To determine the sufficiency of yield during a series of years, working tables have to be prepared under estimated conditions. Starting from a full reservoir, the estimated replenishment, draw-off, losses by absorption, and evaporation, consumption and net storage should be tabulated month after month, for 3 lean years in succession and the capacity of a proposed reservoir increased or reduced with reference to the tables.

The tank need not be designed to hold the full average yield of the catchment, as it can fill $1\frac{1}{2}$ to 2 times its capacity, as its contents are drawn upon for irrigation during the rainy season. The yield from a small catchment is derived by applying a suitable coefficient of runoff to the annual rainfall in inches over the catchment area during the seven wet months from May to November. This is assumed as 75 to 80% of the average annual rainfall. For instance, if the average annual rainfall in an area is 35 inches, 80% of it or 28 inches is the runoff-producing rain. If the coefficient of runoff is 15%, the depth of runoff is $28 \times 15/100$

or 4.2 inches. Spread over 1 sq. mile, the quantity of water in sq. feet = $5280 \times 5280 \times \frac{4.2}{12}$ cu. feet.
 = 9.6 m. cu. feet per sq. mile.

If the catchment area is $2\frac{1}{2}$ sq. miles, the yield is 9.6×2.5 = 24 m. cu. feet.

C. Waste weir

41. The Waste Weir.—The object of the waste weir is to provide a passage for the discharge of floods so that the water in the reservoir may not rise higher than a safe height below the top of the dam.

42. Length of Weir.—The length of the weir must be such that the estimated maximum flood discharge can be disposed of with a depth of spillage of 2 to 5 feet. The effective storage capacity is limited by the F.T.L. but the area submerged and the height of revetment and bund are dependent on M.W.L. which is F.T.L. *plus* depth of spillage. On the one hand, in order to restrict the dimensions of these, it is desirable to reduce the depth of spillage but on the other, this increases the length of the surplus escape to pass a given discharge. The actual dimensions have to be decided by a study of the comparative estimates and the local conditions in each case.

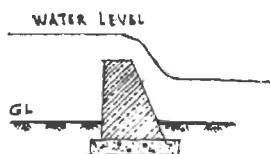
43. Classification of Waste Weirs.—Waste weirs may be classified in a number of ways.

(1) In respect of their locations whether (a) in the flanks or (b) in a subsidiary valley;

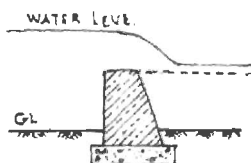
(2) By their hydraulic features whether (a) a drowned channel, (b) a drowned weir or (c) a clear overfall weir; and

(3) By their constructional details whether the masonry apron below is:

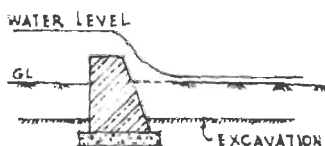
- A. horizontal;
- B. sloping;
- C. Whether the sloping apron is of rough stone;
- D. Whether the apron is stepped or provided with a water cushion.



① CLEAR OVER FALL



② DROWNED WEIR



③ DROWNED CHANNEL

FIG. 17. Types of Tank Waste Weirs.

(1) *Flank Weir*.—The *Flank Weir* in continuation of the **embankment** is the more common type. To prevent the discharge outflanking the dam, a wing wall is required on

the upstream side and a lining wall on the downstream side of the embankment.

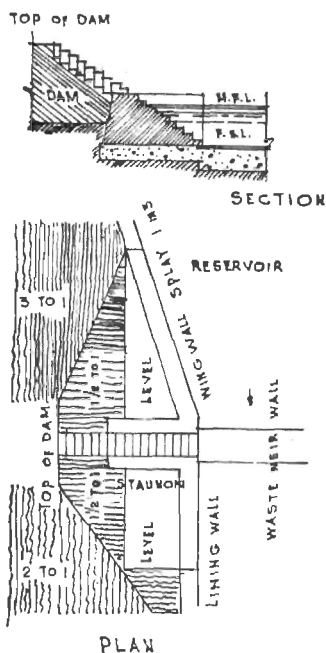


FIG. 18. Flank Weir.

Saddle Weir.—*Saddle weirs* separated from the dam by high ground discharge into a separate valley joining the main valley lower down but to obviate the outflanking of the weir itself, hard rock foundations are necessary.

(2) *Drowned Channel.*—(a) *The drowned channel* is merely an excavation with the weir crest at bed level and the tail channel falling therefrom. If it is feared that a natural crest will be scoured, a masonry weir may be provided. This form is suitable where the soil in the tail channel is soft.

(b) *Drowned Weir*.—The crest of a *drowned weir* is below the surface of the tail channel when fully discharging. This form will be needed where the levels necessitate it and where the tail channel is soft, since its discharging capacity is less than that of the clear overfall type.

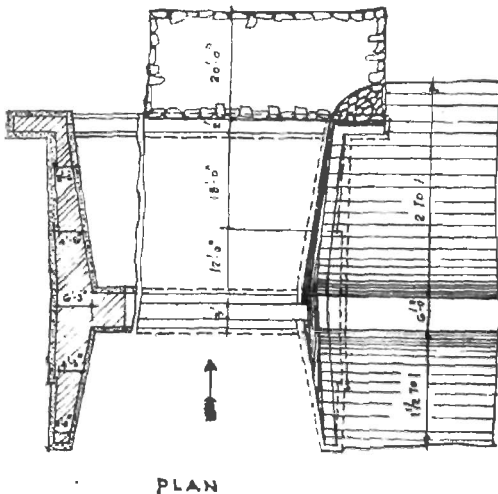
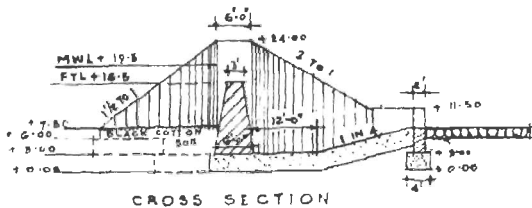


FIG. 19. Weir Type "A" with Water Cushion.

(c) *Clear Overfall Weir*.—The *clear overfall weir* has its crest always above the high flood level in the tail channel. This type causes heavy strain on the weir wall foundations

as well as on the tail channel. The weir must be founded on solid unfissured rock or deep into fissured rock. If

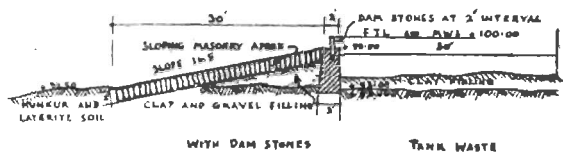


FIG. 20. Tank Waste with Dam Stones.

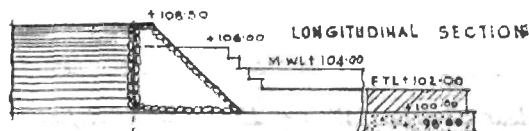
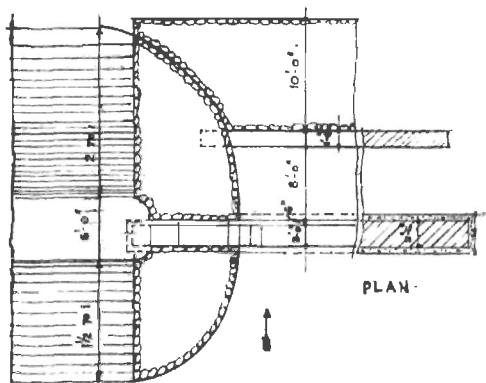
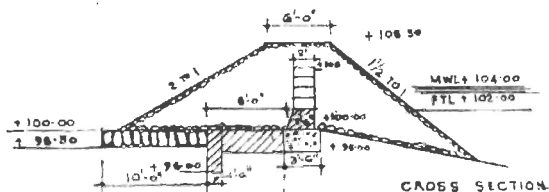


FIG. 21. Tank Waste—Weir Type C.

the soil in the tail channel is not sufficiently hard, an ordinary apron may not suffice and a water cushion will be necessary with curtain walls across the tail channel lower down. Its discharging capacity per foot length being the maximum, it is suited for hard weir sites with restricted length.

(3) The third classification is merely with respect to the details of the apron and four types are specified above.

As regards foundation soil for tank weirs, almost every class of soil has to be built upon, but rarely is the foundation sand.

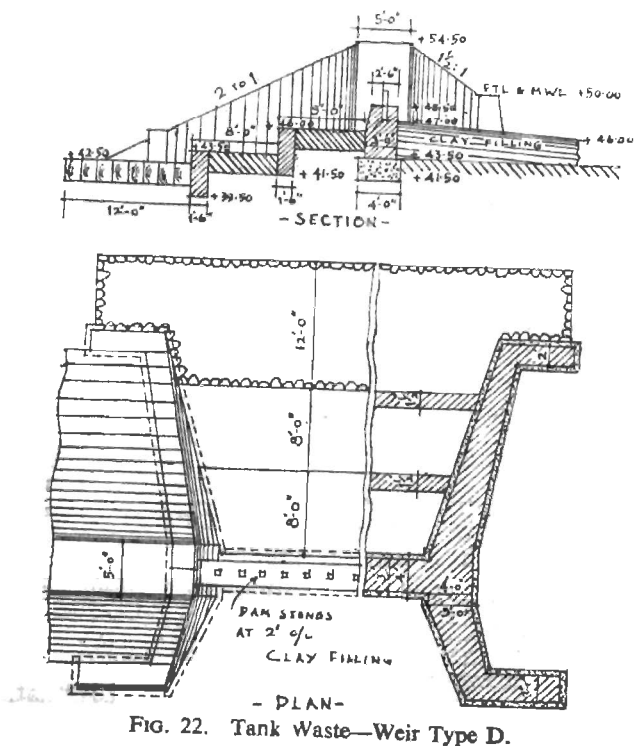


FIG. 22. Tank Waste—Weir Type D.

Where the soil is rock or where water cushion is brought into use, or where the vertical drop does not exceed 2 to 3 feet, Type A is the most suitable.

Where the fall is greater than 10 feet, and the soil soft but only slightly pervious, Type A with water cushion may prove more economical than Type D.

44. Bank Connections.—Bank connections are designed to secure a good joint between the masonry weir and the bank, and the length of the line of creep must be sufficient to guard against percolation round the flanks of the work.

There are two types of bank connections:—

- (i) Abutments and wings,
- (ii) Core walls.

Type (i) securely protects the end of the bank from the rush of water while the wings serve as guide walls to the current on approach and exit from the weir and should be adopted in high weirs.

When surplus escapes are on the extreme flank of a tank, it is unnecessary to have any bank or bank connection at the outer end. The weir wall or flush escape floor may be stopped where the natural ground is at F.T.L. and the continuation protected from scour by flush pitching.

45. Flush Escapes.—A flush escape may be constructed where, for a sufficient width at either flank, the natural ground surface is at F.T.L.; and the ground falls away in rear, giving a suitable tail channel. This type is common with the smaller tanks in South India.

Where these escapes require no pavement or protective works, they are termed “natural escapes” or “natural waste weirs”.

A common practice which has been resorted to for centuries in South India is to fix dam stones in the crest wall of weirs or along the floors of flush escapes. The stones are 6 inches square and $2\frac{1}{2}$ to 3 feet long and are built into the masonry wall at intervals of $1\frac{1}{2}$ to 2 feet and project above the weir crest to a height of 1 or 2 feet, their tops being at M.W.L. (See Fig. 20).

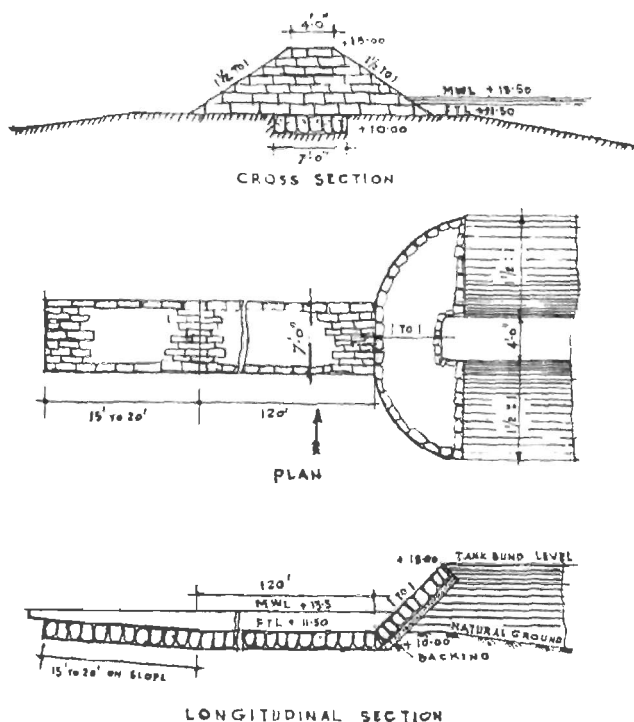


FIG. 23. Flush Escape.

It is dangerous to depend on sluices for disposing of the surplus unless an efficient and trustworthy establishment

living at the work site is maintained for working them. This is not practicable for any but large works.

Occasionally sluice vents or notches fitted with shutters form a part of weirs and are used for irrigation in place of separate irrigation sluices.

46. Approach.—It is important to provide a free and unobstructed approach to all surplus works. The width of the approach should not be less than that of the weir and the bed level half a foot lower than the weir crest.

47. Draft.—The disposal of the surplus after it passes the weir requires careful consideration. The effect of floods passing down a tail channel is generally to deepen, but not widen it. The deepening usually results from the cutting back of the bed where the soil is soft. Even hard material may thus be removed rapidly. This action is termed the “retrogression of levels”. Where it occurs, it should be stopped by the construction of curtain walls across the channel.

48. Outfall.—The best outfall for a tail channel is back into the stream on which the reservoir is constructed, and as near to the bund as practicable without injuring the works.

49. Site of Weir.—For the fixing of the exact site of a surplus weir in the best place and for the detailed design of the work and its approach and draft channel, it is necessary to have detailed information regarding the levels on the selected site. Trial pits along the centre line of the weir and downstream should be dug to ascertain the nature of the surface and sub-soils.

50. Tanks in Series.—The tanks in South India are built across the valley lines, the tanks on one drainage forming a series receiving the surplus of the upper tanks and surplussing into the lower ones. Considerable economy of

water is obtainable from this grouping since the seepage from the wet cultivation is also caught by the next lower tank.

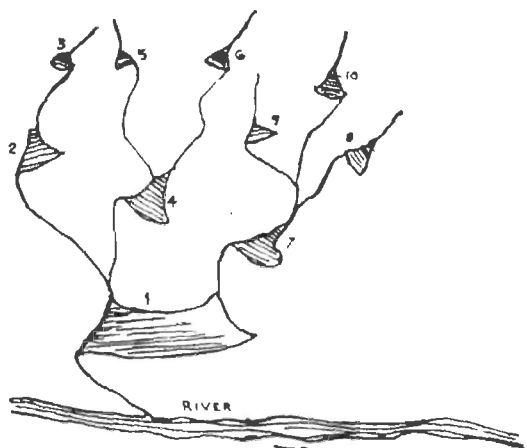


FIG. 24. Groups of Tanks.

The catchment of a tank in a series may be classified as follows:—

- (i) the 'Free' catchment which is the area which only drains into the tank;
- (ii) the 'combined' catchment which is the area of the whole catchment above the tank.

The difference between the 'combined' and 'the free' catchments gives the area of the catchment intercepted by the upper tanks. Each tank receives all the runoff from its free catchment and the balance left after the upper tanks have been filled, from the intercepted catchment. In such a case, $Q = cM^{\frac{1}{2}} - c_1M_1^{\frac{1}{2}}$,

where c is the usual coefficient in Ryve's formula,
 M is the total catchment, independent and intercepted,

c_1 is another coefficient, usually $\frac{1}{8} c$ and

M_1 is the intercepted catchment.

The above formula should be used only where the upper tanks are provided with adequate surplussing works and kept in a proper state of repair.

The system is scientific and convenient to work, but there is one difficulty. A breach in the upper tank exposes the series of tanks below it to risk of failure. The provision of suitable breaching sections at a well-selected place overcomes this defect. The failure of the section which is meant to breach does not empty a tank but supplements its surplussing capacity to the security both of the particular tank and of those below it in the group.

The waste weir of a tank should be capable of passing the estimated maximum flood discharge from the catchment.

51. Length of a Weir.—(i) The high flood discharge from the catchment is obtained by (a) $cM^{\frac{3}{2}}$ where the whole catchment is independent and by (b) $cM^{\frac{3}{2}} - c_1M_1^{\frac{3}{2}}$ where the catchment is partly intercepted. Q is in cusecs and M in square miles.

This is equated with the flow over the weir, $Q = 3.1 L.H.^{3/2}$ where L is the length of weir in feet and H is the depth of spillage over the weir in feet. For moderate-sized tanks H is assumed between 2 and 5 feet.

After calculating Q and assuming H , the length of weir L in feet can be easily calculated. It is important to distinguish between the indices $\frac{3}{2}$ and $\frac{3}{4}$ in the two separate formulæ.

52. Flood Absorbing Capacity.—In cases where the difference between the F.T.L. and M.W.L. is high, the capacity between these levels must be filled up before the weir can

discharge at its full capacity. This capacity to absorb floods may be so great, compared with the flood discharge that a reduction of the waste weir capacity may be made possible. But no allowance is usually made for this and it acts as a factor of safety.

53. The Waste Weir Wall.—The weir wall has a trapezoidal section with the front face usually vertical. The batter is thrown on the rear side.

Its top width varies from 2 to 3 feet depending upon:

- (a) the depth of spillage and the velocity of flood;
- (b) the specific gravity of the masonry—whether stone or brick work;
- (c) the height of the weir; and
- (d) the coefficient of friction of the masonry.

The bottom width is between $0.67 H$ to $0.7 H$, in which

$$H = H_1 + d,$$

where H_1 is the height of the weir from foundation and d is the depth of spillage.

This section is quite sufficient for ordinary tank weirs, but in high weirs, the section should be tested for stability graphically.

54. Water Cushion.—Where the drop in the tail channel is great or where the soil is soft, a stepped apron or a water cushion is necessary to deaden the impact of falling water on the soil. A water cushion is preferred as it is safer while not being much costlier.



FIG. 25. Stepped Water Cushion.

55. Feeder Channels.—It is common now-a-days for the ryots to complain that water in their tanks is inadequate for the *achkat*. Apart from the silting up of the tank bed and defects in the distribution system, even the supply from the feeders may be found to be poor. In such cases, the possibility of augmenting the supply may have to be considered. Since our tanks are all in series, the matter has to be decided after some investigation.

New feeders to an existing tank may be of 3 types:—

- (i) A channel taken from a perennial source like a rivulet;
- (ii) Diversion of a jungle stream in the neighbouring valley into the tank during the monsoons;
- (iii) Adding gathering ground to the tank taking it away from the adjacent valley.

(i) is not practicable except under large canal system where there is generally no scarcity for water; (iii) is no desirable as that would deprive another tank of its supply.

(ii) is possible, if levels permit. If the stream is a small one, an earthen bund may be thrown across the *halla*, but if it is big, a masonry overflow dam may be necessary. The cost of the channel determines its practicability even if levels permit.

Where a channel is possible, its design is on the lines of irrigation channels given the quantity of water to be conveyed and the bed slope permissible. The only point to be kept in mind is that, as the discharge in the feeder is being added to with increase of catchment, its section has to be increased suitably.

D. Preparation of Tank Projects

56. Fieldwork.—A preliminary reconnaissance is necessary before starting a detailed investigation. If there are com-

peting schemes, rough alternative estimates based on common assumptions as to the nature of the foundations and the character of the designs have to be prepared before making the final choice.

All levelling operations should be connected to permanent *Bench Marks*, one at each extremity of the bund about 6 feet above the proposed weir level.

The existing waste weir of the tank should be taken as + 50.00 above *datum*. When the weir is not of masonry, the level upto which water is stored must be taken as the weir level.

The *right and left* of a Surveyor when he stands in the bed facing the bund are, by convention, taken as the right and left portions of the valley.

The magnetic *North* point should be noted with reference to the bund and correctly transferred to the drawings.

The general survey of the *catchment* basin is taken from topo sheets on which the watershed line of the basin may be marked.

The survey work to be done on the field may be classified:—

A survey of the tank *bund*, of the contours of water-spread (a) at present weir level, and (b) 2 to 3 feet above it, and of the wet cultivation below the tank.

The sinuosities of the bund, the position of sluices, waste weirs and their discharge channel should be noted.

The *longitudinal section* of the bund is then taken, marking the level of the top and toe of revetment, and of the highest point of the bund. When the weir is a natural one, the longitudinal section of the bund should be prolonged on to the ridge line across the discharge channel.

The *cross-sections* of the bund are next taken, one section being taken at the sluice. The sections must show the levels not less than 50 feet on each side from the centre line of the bund and at right angles to it. The section at the sluice should show the levels of the upper and lower orifice, the floor level of the front and rear cisterns.

Weir.—The longitudinal and cross-sections of the waste weir and of the approach and draft channels are taken starting from the bed of the tank and extending 100 to 150 feet beyond the face wall of the weir to show any sudden falls in the channel.

For the survey of the *water-spread*, a traverse starting from and closing on the two Bench Marks on either bank should be run on a line between the F.T.L. and M.W.L. and offsets taken right or left to the two contours.

A *contour* taken at the sill level of the *sluice* may be taken when the water has gone sufficiently low to permit a man to fix bamboos at intervals of two chains.

Two bed sections are normally required, one parallel to and a short distance from the bund, sufficiently far to avoid old borrow pits and another down the centre of the valley.

Measure the *maximum depth* of water up to weir level excluding channels and pits. The mean depth of tanks whose depth varies from 4 to 10 feet, will be from 0.45 to 0.55 of the maximum depth, the lower figure being taken if the valley is steep.

The wet cultivation is readily available from the revenue survey maps of villages.

The general description of the catchment may be noted to determine the percentage of runoff from it. The porosity of the bed and the nature of the soil and sub-soil at

the waste weir site and draft channel should be noted by taking a few trial pits in the waste weir valley.

57. Office Work.—The following are the plans, based on surveys, to accompany a tank restoration project:—

(1) An *index plan* which is a tracing from the topo sheet noting the location of the tank, its catchment area and the important places in the vicinity.

(2) *Achkat Plan*.—A tracing of the revenue survey maps of the villages of which the lands are commanded, marking the bund, sluice channels, waste weir and draft channel. The permanent Bench Marks should be marked on this tracing. The F.T.L. contour should be marked to find out the survey numbers which will be submerged. The *achkat* may be coloured green.

(3) *Contour plans* showing contours (1) at sill level of sluice, (2) at F.T.L. and (3) at M.W.L., with bed sections.

(4) *Details of Works*.—Plan, Longitudinal Section and Cross-Section of the bund. The position of the cross-section, sluices and weirs to be shown in the plan.

(5) Plan, Longitudinal section and cross-section of the sluices.

(6) Block level plan of the waste weir, approach and draft channels;

Longitudinal section and cross-section of waste weir showing works in the draft channel. Scales are to be noted clearly on plans (3) to (6).

The following statements should be attached:—

- (i) Survey and *constructive details* in the standard form.
- (ii) Table of *capacities* at different levels, computed from (3) above.

- (iii) *Achkat statement* compiled from (2) above and *muchalikas* from ryots.
- (iv) *Detailed estimates* duly countersigned by the Deputy Commissioner.
- (v) *Report* describing the works and explaining the estimate provisions noting the hydraulic calculations for sluice and weir and the financial aspect.
- (vi) *Working tables* to determine the suitability of the capacity of the tank to irrigate the proposed *achkat*.

58. Execution.—When plans and estimates for a work are sanctioned and forwarded, the following procedure should be adopted during execution.

(1) The plans and estimates should be subjected to check with reference to the site. The accuracy of the areas, dimensions of masonry works, levels of bund, weir and sluices must be verified.

(2) Bench Marks should be checked and bed grade stones planted and numbered after leaving a margin from the bund at intervals of 100 feet for facilitating check measurements of earth pits.

(3) The site of each cross-section, shown in plan must be planted and numbered along the C.L. of bund or on stones well beyond the toe of the bund. The profiles at each cross-section must be set up with reference to some fixed line which may be:—

- (a) the proposed centre line of the bund, after restoration;
- (b) the rear toe line; or
- (c) the front toe line which must first be set out with easy curves. The front and rear toe lines must be lock-split by a 'V-shaped drain' 6 inches deep.

(4) *Profile Marking*.—The marking of the profile at each section is next done as follows:—

Set up the level and find out the R.L. of each cross-section stone along the centre line. The further marking is done with the slope boards, lining string, spirit level and 12 feet rod or levelling staff. Suppose the existing top level at a cross-section is 51.50, and ultimate sanctioned level 55.00: Top of revetment 53.00; Front slope $1\frac{1}{2}$ to 1, rear slope 2 to 1. Top width 6 feet.

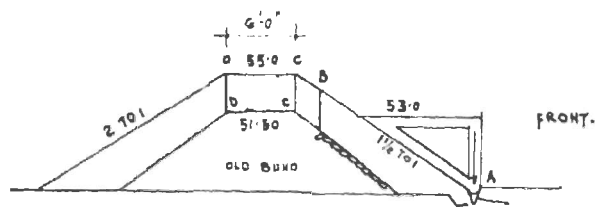


FIG. 26. Extension of Old Bund.

Point A is on the lock split front toe line opposite the cross-section, marked by a firm peg. Tie one end of a long lining string to the peg and place a $1\frac{1}{2}$ to 1 slope board with its vertical face truly vertical. Then take up the lining string so as to touch the face of the hypotenuse. This line will show the face of the revetment but its top level remains to be fixed. For this purpose, place the staff on the cross-section point, C (51.50) and transfer 53.00 to the string by the teak wood rod and spirit level. Drive a long stout peg B with its top to touch the string and tar it. The front slope can be marked by planting another peg C, 2 feet above and 4 feet horizontally from B (slope above top of revetment = 2 to 1). From C, mark the top width of 6 feet horizontally and drive another peg D directly above E. Now adjust 2 to 1 slope board with its vertical side coinciding with D. The hypotenuse is the rear slope and

where it touches the ground will be the rear toe at the cross-section.

This process may be repeated in as many cross-sections as may be required. It is preferable to set up these profiles regularly in earth work, 1 or 2 yards wide, but these permanent sections may be set up after the revetment work is completed so as not to prevent the carting of stones for revetment work.

After the profiles are set up, further work may proceed according to the specifications already given.

59. Regarding Projects—General.—The character of the communications required to develop irrigation, depends on the size of the scheme.

Local materials alone have to be used on construction.

The co-operation of the following departments is necessary to make an irrigation project a success:—

(i) Revenue and Survey Departments regarding land acquisition and the acceptability of the project by the ryots so that the collection of the promised contribution and water rates may be easy.

(ii) Agricultural Department regarding the nature of the soil, the suitability for the various crops, and the artificial manuring that may be necessary.

(iii) Public Health Department regarding the effect on the health of the villages as a result of wet irrigation, and the precautions like spraying with D.D.T., to prevent the spread of malaria.

(iv) Forest and Geological Departments for the resources necessary for the execution of the project.

Tight estimates may induce every one concerned to be as economical as possible while inflated ones, though savings may be shown on them, may have the opposite effect.

A. River Weirs

1. River channels are drawn on one or both the banks from head works constructed across the stream. Headworks are of two types: A. Diversion works, B. Storage works. Diversion works are associated with direct flow irrigation from anicuts while storage works are connected with reservoirs. Some reservoirs, however, receive supplies diverted into them from anicuts.

Diversion Work.—

A diversion work consists of:—

- (i) A weir or anicut across the river;
- (ii) A canal taking off from one or both sides of the anicut;
- (iii) A head sluice or regulator for admitting water into the canal;
- (iv) Scouring Sluices; and
- (v) Flood banks to prevent the outflanking of the anicut by flood.

An anicut is a rough stone or masonry barrier wall across the river by means of which the water level upstream is raised to the crest of the anicut before any of it can pass down the river.

Its height must be sufficient to maintain the water level above the work which will give a command of the area to be irrigated and a depth of water in the canal to carry the required supply.

Since water is discharged over the crest of the weir, protective works like aprons and water cushions are necessary below the weir unless the outfall channel runs in hard rock to prevent the erosion of the river-bed by the weir discharge. A certain depth of tail water is usually standing against the downstream face of the weir which deadens the impact of falling water on the river-bed.

The height of diversion weirs is generally limited to 30 feet when the object is to raise the water level and divert it from the river into canals. On the other hand, a number of masonry weirs combine the functions of diversion and storage while others act as storage dams, a part of their length being utilised as surplus escapes for passing off excess flow which cannot be stored. Some of these overflow dams are built to heights up to 600 feet.

The elementary profile of a gravity weir is a triangle with the apex at the maximum water level, the triangle top being cut off at the crest of weir. The base of this triangle would be $H/\sqrt{\rho}$ where H is the depth of the base below the maximum water surface and ρ the specific gravity of material. The upstream face of the economical profile would be vertical but it is sometimes desirable to keep the downstream face steeper than admitted by a vertical upstream face when the balance of the batter is provided in that face.

If the height of the wall be H , in Fig. 27 and the depth of water passing over it be d , the base b of the trial section will be $\frac{H+d}{\sqrt{\rho}}$ in the elementary triangle ABC. The water pressure on the upstream face is shown by the shaded portion of the triangle ABE, BE being equal to $(H_1 + d)/\rho$ while the piece cut off, the top has a base $AG = d/\sqrt{\rho}$ and a water pressure d/ρ . The back pressure of the tail water is represented by the triangle KCM equal to D/ρ .

2. Middle Third Rule.—In the theory of stability which relates to the distribution of pressures along any joint in masonry structures, it is assumed that the change of pressure along any horizontal joint is uniform and that the curve of pressure distribution along a joint is a straight line. The mean intensity of pressure along a joint of unit width is the total pressure divided by the length of the joint, and under the assumption made, the maximum or minimum intensity of pressure must be at the extremities of the joint. Under such conditions, S_1 = mean stress and S = max. or min. stress, then $S = S_1 (1 \pm 6c/b)$ where b is the length of the joint and c is the distance from the centre of this length to the centre of pressure. In particular, if the line of resultant pressure passes through one extremity of the middle third

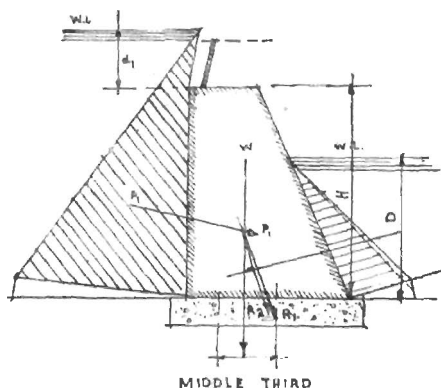


FIG. 27. Stability Diagram for Weir.

of the base, $c = \frac{1}{6}b$ and $S = 2S_1$ (max.) and nil (min.). As masonry is unsuitable for withstanding any considerable tensile stresses, the centre of pressure of masonry works must fall within the middle third of every horizontal joint. The most economical condition under which this condition is fulfilled is when the line of resultant pressure under the

condition of greatest stress cuts the base at the outer extremity of the middle third.

3. Maximum Stress.—As the depth of flow over the weir and the discharge increase in a weir, the tail water level also increases. The difference of level between the head and tail water surfaces becomes smaller. The stress on the weir is therefore not the maximum when the water level above the work is maximum. On the other hand, the difference between the water levels up and downstream of the work is maximum when the head water is at crest level and no water is passing over the weir. In high weirs, however, the downstream water surface is far below and the condition of maximum stress is induced when the head water is at the highest level.

4. Weirs on Impervious Soil.—When a weir is founded on an impervious soil and built of water-tight masonry the weight of the masonry is not affected by variations in the water level above or below the weir and the moment of resistance to overturning is practically constant. Therefore the condition of maximum stress on such a weir is either when (1) the head water is at crest level or (2) when the tail water is at maximum flood level. The weir must be so designed that its moment of resistance about the downstream extremity of the middle third of its base must not be less than the overturning moment under conditions (1) or (2). When there are no weir shutters, condition (2) is always that of maximum stress.

5. Bhadra Anicut.—The Bhadra Anicut is situated in N. Lat. $13^{\circ} 46'$ and E. Long. $75^{\circ} 43'$, near the village of Gopala, close to Bhadravati. The site is at the head of a small rapid and is rocky, located at the close of the upland reach of the torrential river. Its length above the anicut site is 110 miles and the catchment area which is entirely in the malnad is

1083 sq. miles. The calculated flood discharge at Bhadravati bridge is 1,20,000 cusecs and deducting 2,000 cusecs as the discharge between it and the anicut site, the flood discharge at the anicut site would be 1,18,000 cusecs.

Ryve's formula with a coefficient of 640 gives

$$640 \times 1083 = 69,120 \text{ cusecs.}$$

This formula is applicable to maidan catchment with an annual rainfall of 30 to 40 inches and is not suitable for the hilly catchment above Bhadra Anicut.

A coefficient of 625 in Dickens' formula gives nearly 1,18,000 cusecs at the anicut site.

The coefficient corresponding to this discharge in the formula C.M.[‡] would be 2,850. Here again, the coefficient of 5,000 suggested by Creager and Justin is high while Inglis' coefficient of 7,000 is very much on the high side.

The width of the river at the bed is 830 feet while it is 905 feet at the highest flood level of RL 1985.34 before the construction of the anicut. The crest level of the anicut 900 feet long is RL 1978.75 and its average height is 9.34 feet. The maximum H.F.L. after the construction of the anicut will be RL 1990.25 or 11.5 above the anicut crest and the level of tail water during the highest floods is RL 1985.34 or 6.59 feet above the anicut crest. The anicut will be completely submerged during high floods.

The section provided for the anicut is a trapezoid with the upstream face vertical and the downstream face battered at 1 in 3. The top width of the anicut is 6 feet which is built of coursed rubble masonry in surki mortar.

It is verified from the stability diagram for the anicut that the centre of pressure cuts the middle third of the base 13 feet 6 inches below the crest of the anicut; well within the middle third and that the anicut section is ample.

The section of the weir is found by trial assuming a suitable crest width and the base width of the elementary profile modified from considerations of flotation and crest width. The modified section is tested for stability mathematically or by graphical methods and the profile settled finally.

6. Weirs on Sand.—Weirs on sand or permeable soils are generally low weirs for diversion of water rather than storage. They vary in height from 10 to 15 feet and are drowned weirs, subject to submergence. Such works are of great length and magnitude. It is necessary to protect the foundations of the masonry weir from damage by masonry aprons downstream and sometimes also upstream of the wall.

Failure of Sand Foundation.—Sand is incompressible and is a good foundation for masonry works if it can be protected against erosion and disturbance. Sand foundations under a weir can be disturbed (a) by percolation and (b) by scour or erosion.

Downstream Apron.—The apron downstream of the weir destroys by impact some of the velocity due to the head over the weir wall when there is an overfall and transfers the locality of the scour in the river-bed from immediately below the weir wall to downstream of the tail of the apron. The width of the apron should therefore be sufficient to ensure that any scour caused below the weir shall be sufficiently far removed from it as to leave the soil on which it stands unaffected. The downstream section of the apron is made of loose stone or blocks of concrete which will adjust to any slipping of the sand by forming a gradually sloping side to the cavity caused by the scour. The greater the velocity and turmoil of the water at the toe of the apron, the greater the width and depth required for the loose stone of the tail apron or "talus".

B. Types of Weirs

7. Bligh has divided weirs on sand foundations into three types:—

- A. Vertical Drop on to a horizontal floor.
- B. Slopping masonry apron, and
- C. Rock-fill sloping apron.

He subdivides Type A into two types— A_1 where the surface of the apron is at or below Low Water Level, and A_2 , where the apron is partly or wholly above Low Water Level.

The body wall of a weir sometimes requires protection from scour on the upstream side in addition to a downstream apron. Such scours are due to cross-currents, any current running in a direction parallel to the face of the work causing deep scours. These currents are most likely to be formed where the weir is situated across a course of the river which is crooked or obstructed by shoals or where the main current is not parallel to the axis of the river or the weir not at right angles to the current. Protection against scour above a weir is given by a flat or sloping rough stone apron and where cross-currents are developed either above or below the weir, they are checked by "hanging groynes".

Hanging groynes are groynes of rough stone run out from the work at right angles to its length, the top level being 2 to 3 feet below the masonry crest level if upstream and at about low water level if downstream of the weir.

8. Types of A_1 and A_2 Compared.—The main apron would require additional thickness for Type A_1 to resist the drop of water over a greater height. If the weir had been Type A_1 with the upper main apron surface at Low Water Level, the underside of the aprons would be lower than for Type A_2 and the balancing levels would be obtained by plotting the

full weight of masonry apron and water overlying it on the base. It is therefore an advantage to have the apron above Low Water Level.

There is a smaller vertical fall on to the apron in Type A₂. On the other hand the raising of the floor restricts the water way over it and increases the velocity at the downstream toe of the work.

9. Weir Type B.—The only difference between a weir of Type A₁ and Type B is that the apron behind the weir is sloping in Type B, instead of being level as in Type A. Type B is suitable for weirs below which the river-bed slopes steeply, since water would be led down smoothly down the slope. A horizontal apron could also be used but it has to be built in steps with cross-walls where the water level is dropped, or preferably a water cushion may be used to deaden the impact of falling water.

Owing to the increase of velocity down the sloping apron in Type B, there is a tendency for scour in the river-bed at the end of the apron which has to be filled by stone blocks. This increases the maintenance changes every year.

10. Types A and B Compared.—There is no drop in the water from the weir in Type B as the sloping apron starts from the weir crest, while a sheer drop in Type A causes greater stresses from falling water on the apron which has to be made thicker to resist the impact.

If the river-bed is moderately hard, soft rock or gravel and the drop from the weir crest to the apron level does not exceed about 5 feet, Type A is preferable. It is also suitable for weirs on poor soil where the height of overfall does not exceed 3 feet. Where Type A is adopted over sandy bed the underside of the main apron must be kept about the Low Water Level to reduce the uplift on the apron and the height of overfall.

Type B is desirable on a sandy foundation if the height of overfall is considerable.

11. Water Cushions.—A water cushion apron is a modification of Type A weir, the main apron being a cistern of water into which the water passing over the crest falls, the impact on the masonry being reduced by the cushioning effect of the depth of water over the apron.

If the soil in the river-bed is sand, a water cushion is rarely suitable as the floor of the cushion has to be laid at a considerable depth below low water level. This entails expense in dealing with water during construction in a permeable river-bed. The water cushion is desirable when the river-bed soil is only slightly pervious. The effective weight of the floor of the cushion and of the depth of water must be sufficient to resist uplift.

If the river-bed is of rock not hard enough to resist the impact of the falling water, the apron may be dispensed with and a low subsidiary weir may be built across the river below the main anicut forming a protective water cushion immediately below the main weir.

12. Weir Type 'C'.—This type of weir is common throughout South India across rivers and streams of all magnitudes, resting on all kinds of foundations, rock, sand or clay. It is a barrier or bund of rough stone with a flat downstream slope and the width of the barrier varies with the nature of the river-bed. The more easily the bed is eroded or undermined, the wider the barrier.

In some old works the crest stones have been tied together by iron clamps and there is a face wall of rubble or brick in mortar which serves to prevent percolation. Such a wall will altogether stop leakage on an impermeable foundation but will check it in permeable foundations. In modern

works, there are one or more curtain walls provided parallel to and downstream of the body wall dividing the width of the apron into compartments. These walls check percolation and localise damage.

There is no impervious apron in Type C and the percolation under the body wall is free to escape through the interstices between the stones of the bund. The flow is somewhat obstructed if these interstices have been filled with quarry rubbish or stone chips but this line of flow will offer less resistance than through sand. There is an upward percolation through the spaces between the stones which raises the sand off the bed and deposits it in the interstices of the stones so that the stone settles into the sand. More stone must be added and the surface of the apron made up as it settles and this makes the maintenance of this type of apron more expensive. Gradually the interstices of the lower layers of stone become filled with sand and settlement of the sand movement will cease and if the apron is of sufficient width, it will not differ materially from a Type B weir.

It is economical to grout the interstices between the surface stones of the sloping downstream apron when the settling has stopped as this saves maintenance costs by reducing displacement. On the other hand, grouting makes the surface layer liable to uplift by making it impermeable and the amount of uplift depends on the slope of the apron surface and on the nature and density of the silt accumulated between the stones in the lower layers.

Except where stone and unskilled labour are cheap and plentiful and skilled labour and mortar are expensive, Type C is not suited for original construction because the quantity of stone required not only for construction but

for filling up sinkage is very large. Types A and B will be found more suitable for fresh constructions than Type C.

The apron slopes of Type C weirs may be about one in six for coarse grained sand foundations and 1 in 8 on less pervious foundation. The slope is sometimes given in the form of steps and it is usual to have a drop of 1 to 3 feet immediately below the body wall with an apron of solid masonry.

The width of the downstream apron for a Type C weir may be the same as for Type B. The upstream apron depth will depend on the conditions of the site and the liability to cross-currents.

13. High Coefficient Weir.—When the sheet of water spilling over a sharp-crested weir leaves the face of the weir, there is a danger of the formation of a partial vacuum under the sheet, called ‘nappe’ which causes extra overturning force on the weir. The general form of the nappe is indicated in Fig. 28. If the area below the lower nappe is filled with masonry or concrete, there will be no change in the discharge

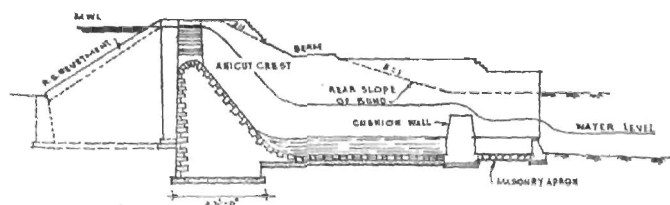


FIG. 28. Cross-Section of Tunga Anicut (High Coefficient Weir).

or in the shape of the nappe. This shape of crest is called the “Standard dam crest”, and the falling nappe exerts no pressure on it as long as it follows the shape of the nappe. As the shape of the nappe varies with the head over the crest, any one shape of the rear profile can suit only one

depth of flow called the "design head". The coefficient of discharge C in $Q = C.L.H.^{3/2}$, also varies with the actual head on crest.

The coefficient of discharge for a broad-crested weir is about 3.1. For a vertical front face and an aerated nappe, the coefficient increases with the head ratio $= \frac{\text{Actual head}}{\text{Design head}}$ from 3.1 to 3.9 until the actual head equals the design head and decreases for higher heads upto 3.6 for a head ratio of 2.

On the other hand, for a vertical water face and adhering nappe, the coefficient is increased to 4.0 for a head ratio of unity and to 4.5 for a head ratio of 2. That is, for higher heads, the coefficient of discharge is increased and for this reason such an overflow dam is called a 'High Coefficient Weir'.

Overflow dams are now built up to 700 feet height and the velocity of water at the toe of the dam is too high for the best of foundations. The scour caused in the river-bed owing to the impact of falling water has to be prevented by special devices. One of them is a "bucket" which deflects the sheet of water in a horizontal direction.

14. Hydraulic Jump.—Another common method of preventing erosion below a weir is to cause a hydraulic jump to be formed beyond the bucket on a horizontal floor as far as possible at all discharges. A hydraulic jump is produced when a shallow stream moving with a high velocity strikes water of sufficient depth. It consists of an abrupt rise in the water surface in the region of impact between the rapidly moving stream and the more slowly moving water accompanied by a great tumbling and the production of a white foamy condition. Under suitable conditions this standing weir remains steadily

in one position and dissipates the energy of water flowing over dams.

Water cushions.—The base of a high waterfall in nature is protected from further scour by the formation of a deep pool, in which the impact of the falling water is absorbed by the water in the pool. This design is imitated by the Engineer in forming the water cushion by the construction of a weir.

The depth of the pool has to be decided with reference to the nature of the foundation to be protected. The length of the downstream cross-wall should not be less than the width of the channel downstream.

If the fall in the tailwater channel is considerable, a series of water cushions may be built in cascades.

Apron.—In most of the tank weirs, a water cushion would be too costly and not needed where the material is fairly hard. The foundations may then be protected by a paved apron. Part of the energy of the surplussing water is destroyed by the friction of the apron surface. In all cases, the slope of the apron should be carried beyond the probable limit of scour and should end in a toe wall carried deep to prevent the danger of undermining.

15. Siphon Spillways.—The discharge per lineal foot of an overflow weir can be increased considerably by constructing the weir as a siphon. The velocity of flow through the siphon is increased as a part of the vacuum head which is 34 feet theoretically, becomes available. The operating head causing the flow through the siphon is the difference in level between the water-surface at the inlet and the centre of the outlet.

The siphon spillway is a device for discharging water through a closed conduit on the principle of the siphon.

The discharges of siphons may be calculated from the equation of flow in short pipes.

An advantage of this form is that it reduces the difference between the full tank level and maximum water level. This prevents the submersion of valuable land and reduces land acquisition charges in a reservoir project. It is automatic in action. The height of the dam for a given capacity may be reduced. The length of the weir may be lessened to a third of the overflow weir length. There are no moving parts to go out of order and no machinery to manipulate.

The Action of the Siphon.—When the upstream water level in the reservoir rises above the crest of the siphon, it falls over the crest and fills the downstream cistern. When the downstream level is above the cistern crest, the downstream leg of the siphon is sealed and the entrance of air from the outlet into the barrel is stopped. A further rise of water in the reservoir brings the water level to the mouth of the siphon and prevents the entrance of air from the upstream side also. The water following over the crest carries some air from the siphon barrel, making the pressure inside the siphon subnormal. This induces a greater flow into the siphon until all the air is driven out and the barrel is filled. The siphon is now primed.

When the water in the reservoir goes below the mouth of the siphon, air enters the siphon and breaks the siphonic action. The siphon is deprimed.

Several improvements are made for reducing the priming head, and for increasing the discharge through the siphons.

16. Two New Types of Siphon—(1) Volute type and (2) saddle type have been developed in Mysore by Sri. V. Ganesha Iyer, Superintending Engineer (Retired).

The volute siphon is a dome with a funnel within and a bent pipe taken down the funnel leaving an annular space all round, to pass the discharge through the dam. The pipe is vertical where it takes off from the funnel and is bent through a right angle. It becomes horizontal where it leaves the dam. Volute siphons are located at intervals in the body of the masonry dam like sluices.

17. Action of the Volute Siphon.—When the water in the lake rises above the full reservoir level, it spills over the circumference of the lip of the funnel, winds along the volutes, and drops into the barrel at the throat whirling with a spiral motion and forming a water seal. The air in the siphon confined between the inlet and the water seal is then evacuated. As the suction becomes more powerful, more water enters and creates a greater vacuum until the siphon is primed and flows full. The reverse action takes place when air enters through the deprimers during depriming.

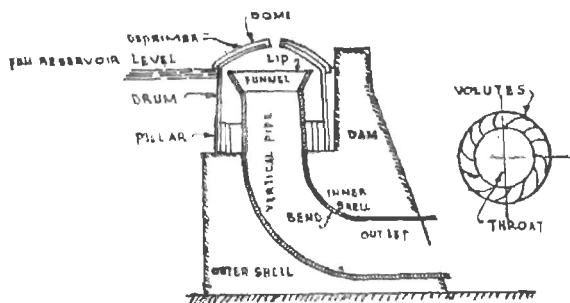


FIG. 29. Volute Siphon.

Two dome siphons of 6 feet diameter for surplussing 2,000 cusecs each are installed in Markonahally. Eleven volute siphons of 18 feet diameter for discharging 10,000 cusecs each are built into the Hirebhaskar Dam on the Jog Project.

Six saddle siphons are functioning efficiently in Hesar-ghatta tank weir under a head of 10 feet. Each siphon discharges 1,620 cusecs.

Details of design of the volute siphons are worked out by experiments with models at the Hydraulic Experimental Stations like the one at Krishnarajasagara.

C. River Regulators

18. A regulator is essentially a bridge spanning the waterway to be regulated in the openings of which shutters are installed capable of adjustment so as to regulate the extent of waterway left free for the passage of water. The bridge platform may either be adapted for public traffic on a platform from which the gear for regulating the shutters can be operated.

Regulator shutters are made of wood or steel and fitted to move in vertical grooves in the piers and abutments of the regulator superstructure; they are raised or lowered by means of winches or screw gearing, operated from the bridgeway above.

Regulators are occasionally made across rivers or streams to serve the purpose of weirs without causing the obstruction which is usual with a solid weir. These are termed "Open Weirs" or "River Regulators", or 'Barrages'.

It is usual to construct a regulator on one or both flanks of a solid weir for scouring the river channel feeding the canal head and keeping the deep river channel in regime. Such regulators are termed "Scouring Sluices".

Regulators across the heads of canals through which the irrigation water is passed to the canal systems are termed "Head Regulators" or more frequently "Head Sluices"

The roadway of regulators or sluices is, as in ordinary road bridges, carried by masonry arches or by beams of wood, steel or reinforced concrete. The relative cheapness of masonry in South India, and the stability resulting from the extra weight of masonry arches makes this form of construction almost universal.

The spans of the bridge openings of a regulator are limited by the width of shutter to be installed; the general introduction in recent years of shutters moving on rollers have resulted in great increases in the width of shutters, and, in correspondingly larger spans for regulators. This has led to the more extensive use of regulators in place of weirs for river diversion works. Before the introduction of rollers, the width of regulators was restricted to 10 or 12 feet, and as bridges with such small spans must cause great obstruction in the waterway of rivers, weirs with or without crest shutters were almost universally employed for river diversion works. The spans of shutters for river regulators are now up to 40 feet and bridges with such spans do not cause undue obstruction to river discharge.

19. Design of Regulators.—When the shutters of a regulator are not raised clear of the ventway, any pressure of the water on the shutters is transferred to the piers or abutments which must be designed to resist such pressure without sliding, or the development of tension or undue intensity of pressure in any joint, or in the foundation soil.

Unless the work is founded on hard rock, a floor is necessary which must be designed of weight and strength sufficient to resist displacement by impact and the impervious portion of it must be sufficiently long to prevent 'piping' and sufficiently thick to withstand uplift.

Bligh has, as in the case of river weirs, proposed empirical rules of design for regulators based on successful

practice as ascertained from existing works, and these rules form a convenient basis of design but each design must be verified for stability of piers and weight and length of floor.

(1) Thickness of piers should not be less than $\sqrt{s}$ where s is the span in feet. It varies from $\frac{1}{3}$ to $\frac{1}{2}s$ and should be larger in great depths of water for small spans. The proportion may be increased to 0.4, where the work is subject to a large difference of head up- and downstream, as in the case of a head regulator.

(2) The thickness of segmental masonry arches at crown $= 0.45\sqrt{R}$, where R is the radius of the intrados of the arch. For spans over 30 feet, increased thickness towards the abutments will be necessary.

(3) Thickness at springing of abutments may be fixed according to Trautwine:

$2 + 0.2 R + 0.1 r$, where r is the rise of the arch.

Back slope of abutment $= \frac{\text{Span of the arch}}{25 \times \text{Rise of arch}}$ i.e., $0.04 \frac{s}{r}$.

(4) The springing of arches should not be below the M.F.L. upstream. The points of suspension of the shutters must be at such a height that the shutters when fully raised have their lower edges above M.F.L. When the regulator is not used for traffic, this consideration alone will determine the height of the piers and the platform.

(5) The length of the piers is fixed by the necessary width of roadway and by the requirements of stability. Where a public road is combined with the regulator and is carried by a masonry arch, the length of pier required to give adequate roadway is sufficient to provide stability under all conditions of flow. When the structure is restricted to use as a regulator or when a roadway is a light one carried

on beams, the lengths of piers have to be increased above what would suffice to carry the bridge, in order to fulfil the requirements of the regulator as regards stability.

(6) The design of floorings and aprons follows the rules given for solid anicuts.

20. Scouring Sluices.—These are placed on the flank of the weir on the side of the river from which the canal offtakes. Their functions are: (a) to maintain a well-defined channel in front of the head-sludge by scouring the silt deposited in front of the gates and (b) to prevent bed silt entering the canal.

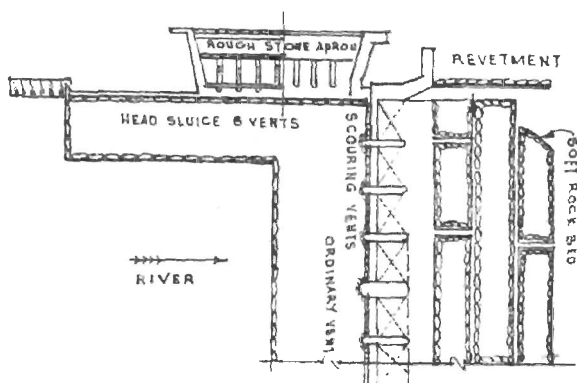


FIG. 30. Regulator.

The sill of scouring sluices should be as low as practicable and generally at about the deep bed level of the river.

Large spans and unobstructed path of flow are an advantage. Any arch or platform over the piers should be above flood level and the working platform should be accessible under all conditions of flood.

The head sluices must be closed whenever the scouring sluices are open, but it is not possible to close the head-

sluices unless there has been heavy rain in the irrigated area. Thus even when there is a flood in the river, it is sometimes desirable to keep the scouring sluices closed. In order to make this possible, there must either be a face wall between the piers of the scouring sluices from the crest level of the scouring sluice shutters to M.F.L. or water must be allowed to pass over the crest of shutters. It is not usual in Madras to allow water to spill over the tops of lift shutters but there are advantages in allowing this in the case of scouring sluice shutters.

Where there is no face wall to the scouring sluices the crest level of the sluice shutters should be at least as high as the crest of the weir or weir shutters.

The scouring sluices pass great volumes of water at high velocities and the kinetic action on the floor of the apron below is likely to be greater than on the floor and apron of the weir.

The width W_g of the main floor of the scouring sluices is affected by the crest level of the weir shutters or of the weir crest when there are no shutters because the sluices will be worked when water is being maintained at this level.

21. Head Sluices.—A head sluice is designed to pass all the water required for irrigation at moderate velocities while excluding bed silt as far as practicable and capable of excluding from the canal any water which may not be required. The work has to be built across the canal head to a level above that of the maximum flood and of sufficient stability to support water standing at M.F.L. against its upstream face when there is no water in the canal below it.

22. Requirements of Head Sluices.—It is only by drawing water from the film of water nearest the surface that bed silt can be excluded and water passed at a low velocity.

The problem of silt is not so acute in Mysore and Madras as in Northern India and the head regulators of the old systems in Northern India where these points have been lost sight of have had to be remodelled in recent years. The drawing of the surface water at low velocity involves great width of vantage and arrangements to vary the level at which water is taken into the canal to suit the varying levels of water in the river. Whether the extra cost is justifiable by the reduction of silt troubles and savings in future expenditure for silt clearance has to be considered in each case of remodelling.

23. Head Sluices in South India.—The sill of the head sluices in South India is $1\frac{1}{2}$ to 3 feet higher than the sills of scouring sluices and head sluice shutters are designed in 2 or 3 tiers working in separate grooves to admit water at different levels. The lower shutters are closed as long as it is possible to pass the irrigation supplies through the upper ones. The vents are not more than 10 feet wide and there is no advantage in having large span shutters for head regulators.

24. Sites for Head Sluices.—The head sluice should have its face in line with the river margin so that the scouring sluices may draw directly across the front of the sluice. If it is placed away from the margin of the river a short distance down the canal cut, a recess will be left out of the draw of the scouring sluices from which silt deposit cannot be scoured and eddies and swirls are formed which shift the silt off the river-bed whence it is carried into the canal. In such a case, the scouring sluices should be located in the leading channel just above the head sluices and at right angles to them.

25. Foundations and Flooring of Head Sluices.—Head sluices are subjected to greater statical heads than the connected weir or scouring sluices and where the foundation

soil is similar, uplift and percolation are greater under the head sluice floor which must be designed suitably. The draw of scouring sluices will operate across the face of the head sluices and undue scour must be guarded against by a curtain wall or apron upstream of the face of the head sluice.

As water is not passed with undue velocity on the downstream side of the vents, protective aprons of head sluices have only to resist piping or uplift and need not be so expensive or so heavy as in the case of river regulators or scouring sluices which are subjected to high velocities and uncontrolled magnitude of flow.

26. Head Sluices of Minor Canals.—Many minor channels have their heads in streams where there is no anicut to prevent water falling below a convenient level in the river and the head sluice constitutes the only permanent headwork. The sill of such a sluice is made to suit the bed of the irrigation channel which is at the lowest level from which the irrigated area can be commanded. Where the bed of the stream is sandy, the sill is generally above the deep bed of the stream as a precaution against excess of silting and to get a command of higher land. The irrigation supplies are naturally fluctuating in such channels and the head sluices must be designed for supply at a low duty to make up for the intermittent nature of the supplies.

27. Regulating Shutters and Gates.—The simplest type of shutters for operating regulators and sluices is the 'needle shutter'. These shutters are vertical planks of wood about 6 inches wide with the lower ends resting on the sill while the upper ends are supported by a platform beam. The disadvantage of this type is that the shutters take much time and labour to operate and that the leakage past the planks is much. Horizontal planks sliding in grooves are also used on weir crests and in regulators and manipulated

by the help of rings fixed at the ends of each plank for small spans up to 6 feet.

Regulator shutters of spans larger than 6 feet are made of wood or steel and fitted to move in vertical grooves in the piers and abutments of the raised. They are raised or lowered by means of winches or screw gearing, operated from the bridgeway above. This type is used for operating head sluices and scouring sluices in weirs.

28. Open Weirs.—Before the introduction of rollers, the width of regulators was limited to 10 or 12 feet. Open weirs were not possible on rivers until spans up to 40 feet could be built in the river-beds. Such large spans do not cause obstruction to river discharge. These gates are also operated by screw gear in barrages.

Sometimes, the shutters in head sluices of anicuts are built in tiers. The Dhavalesvaram head sluice on the Godavari is in three tiers, but the upper gates cannot be lowered. Gates in two tiers are very common in the Kaveri delta. The shutters work in separate grooves, the lower shutter as it rises, picking the upper one. The object of this is to save headway and diminish resistance to motion of the shutters made of wooden planks.

29. Crest Shutters.—Shutters of the falling type are erected on the crests of masonry anicuts to obstruct the waterway as little as possible and to hold up as much of the total head as is feasible. The height of the solid obstruction is thus reduced by the height of the shutters reducing afflux during floods and the velocity of flow over the rear apron. Crest shutters on the Godavari anicut at Dhavalesvaram are 2 feet high while those on the Vijayavada anicut across the Krishna River were 3 feet high. The latter anicut is now being remodelled.

Crest shutters may be lift shutters where bridges exist at the sites of weirs when the shutters moving in grooves in the upstream ends of bridge piers may be lifted by the gearing arrangement being worked from the top of the bridge.

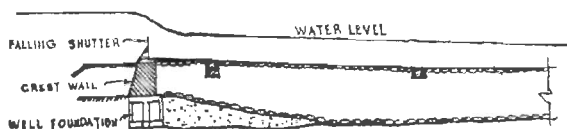


FIG. 31. Krishna Anicut at Bezvada.

Another type of falling shutter is attached to the weir crest by hinges along its lower edge. Each shutter is held up by a strut which can be released on the approach of a flood by one man.

Large span lift shutters of open weirs are hung on chains from an overhead platform on which winches for raising and lowering are installed. The winches are mounted on travelling carriages so that one set may work several vents in succession. It is more usual to instal a separate fixed gear for each vent. These shutters are made of a frame of built-up girders or rolled steel beams with a facing of sheet steel. They are moved on rollers 6 to 8 inches in diameter either running on axles which are fixed to the shutter, or mounting in a cage separate from the shutters which runs on the surface rollers. The latter type with 'free rollers' is known as Stoney's patent type. The undersluice gates in Krishnarajasagara dam are of this type. The type is repeated at Thippagondanahally, Markonahally and Hirebhaskar. They have now replaced rollers running on fixed axles altogether.

30. Screw Geared Shutters.—A very large number of gates of spans up to 12 feet working under moderate heads of

water are operated by screw gear, the force being transmitted through a single screw shaft or spear, attached to the centre of the gate at the top. On the upper end of this shaft, a square screw thread is cut for the length necessary to secure the required travel of the shutter and a screw nut to fit this thread is housed in a box fixed to the sluice platform in which the nut can revolve freely. The nut is revolved by means of a spanner or capstan head, to lift or lower the shutter.

Small sluices may be of cast iron, faced with gun metal, moving between similarly faced cast iron frames and guides. Larger sluices may be built with frames and plates of steel.

31. Gates and Valves.—Gates are used in canal structures and valves in pipe distribution systems. Gates differ from valves in the passages they provide when fully open. Valves contain elements that move within the waterways and partially obstruct the flow when the valves are opened. Gates are constructed so that unobstructed waterways are available during full openings.

5

1. Masonry Dams.—A masonry dam is a wall which holds up water on the upstream side while the downstream side is free from water to any appreciable extent. Masonry dams are of two types: (*a*) the non-overflow type and (*b*) the overflow type. An overflow dam is not very different from a weir but the heights up to which an overflow dam is built are higher and its conditions of stability are different so that the distinction is necessary.

Non-overflow dams may be divided into 4 classes:—

- A. Gravity dams of masonry or cement concrete,
- B. Arched dams,
- C. Buttress dams, and
- D. Reinforced concrete panel box dams.

In gravity dams, the water pressures are resisted by forces brought into action by the weight of the dam only. In arched dams, the pressures are resisted in the same manner as an arch sustains the load which is placed on it. In a buttress dam, the space to be dammed is divided into a number of spans by buttresses or piers and panels are constructed consisting of horizontal arches or reinforced concrete beams to hold up water between the buttresses. In the former case, the dams are called "Arched Buttress dams". A reinforced concrete dam consists above the ground of a series of buttresses of triangular section, supporting a flat slab of reinforced concrete on the water face.

2. Stability Conditions.—A masonry dam must be designed so as to be safe against failure by (1) Overturning, (2) Rupture from tension, (3) Sliding, and (4) Crushing.

Conditions 1 and 2 are satisfied by the "Middle Third Rule", when the centre of pressure falls within the middle third of every horizontal joint and base. The most economical condition under which this is fulfilled is when the line of resultant pressure R under the conditions of the greatest stress cuts the base at the outer extremity of the middle third. Condition 4 is also satisfied in this position provided the maximum compressive stress developed which equals twice the mean stress is within the safe compressive stress the particular quality of masonry can bear. This condition gives the shortest base and a series of horizontal joints and therefore the most economical section of the dam when it obtains both when the reservoir is empty and when it is full. In the former case, there is no water thrust on the upstream face of the dam and the centre of pressure will be on a vertical line through the centre of gravity of that part of the dam which is above the joint considered. The water thrust on the upstream face is maximum when the reservoir is full and the centre of pressure is shifted to the greatest extent downstream.

The minimum profile, which fulfils the middle third condition with reservoir empty, when acted on by its weight only, is an isosceles triangle or a right-angled triangle. In the former case, the centre of pressure is on the central line of the figure while in the latter case, it passes through one extremity of the middle third of each horizontal joint. Similarly, the minimum profile fulfilling this condition when subjected to maximum water pressure is a right-angled triangle with the water face vertical. A right-angled triangle therefore satisfies both the conditions of 'empty' and 'full' reservoirs when the water face is on the vertical side of the triangle.

(3) Liability to sliding at a joint depends on the friction and cohesion between the surfaces meeting in the joint. The cohesion between mortar and stone is considerable but very varying and is neglected in computation being allowed to remain as a factor of safety against failure at a joint by sliding. The coefficient of friction between dry masonry surfaces is taken as $\tan \alpha$, between 0.67 and 0.75 where α represents the angle of friction varying from 34° to 37° with the vertical. Condition 3 requires that the resultant R of the horizontal thrust of water and the vertical weight of the dam *plus* the vertical component of water thrust above the joint must not be inclined at a greater angle with the vertical than 34° to 37° . If smooth joints are avoided and if an irregular surface is maintained at each level with a number of stones projecting from it so as to improve the resistance of the joint to sliding, a dam which fulfils the other conditions of stability will also fulfil this one.

The minimum width of base required for such a profile is derived as follows:

Let

H = Height of dam,

P = Total water pressure on the water face,

W = Weight of dam resting on the base b and

ρ = Specific gravity of masonry.

Then

$$P = \frac{1}{2}H^2 \text{ and } W = \frac{1}{2}H \times b \times \rho$$

from which

$$H = \frac{2W}{b\rho}$$

$$\frac{P}{W} = \frac{b/3}{H/3} = \frac{\frac{1}{2} H^2}{H.b.\rho.}$$

$$b^2\rho = H^2 \text{ and } b = \frac{H}{\sqrt{\rho}}.$$

This "Elementary profile" of the dam fulfils the following conditions. The resultant pressure at all times falls within the middle third of the base. When the reservoir is empty, the centre of pressure due to the weight W of the dam passes through the upstream extremity of the middle third and when the reservoir is full with water level at the apex of the triangle, the resultant of P & W passes through the downstream extremity of the middle third. The profile satisfies conditions 1 and 2. With the precautions in construction to avoid smooth joints, condition 3 is also satisfied normally. Condition 4 depends on the height of the dam.

Masonry dams or weirs are usually built of stone masonry or cement concrete with stone jelly as aggregate. The weight of the material compared with that of an equal volume of water is its relative density ρ or specific gravity. If the weight of 1 cu. foot of water, w , is taken as $62\frac{1}{2}$ lb. or $1/36$ th the ton, weight of masonry per cu. foot is $w\rho$. The specific gravity of ordinary stone masonry is taken as $2\frac{1}{4}$, when its weight per cu. foot is $1/16$ th ton. Specially dense masonry built with large-sized rubble and a low percentage of mortar has a specific gravity of $2\frac{3}{4}$ while good brick work has a specific gravity of 2 which is nearly $1/18$ th ton per cu. foot.

3. Limit of Depth of Elementary Profiles.—The limit of height H_r of a dam designed on the basis of the elementary profile can be computed if the maximum safe compressive stress on masonry per sq. foot, λ is determined from the formula $WH(\rho + 1)$ which reduces to $H/11 \cdot 1$ when $\rho = 2\frac{1}{4}$.

$$\lambda = H_r \cdot w(\rho + 1) \text{ from which } H_r = \frac{\lambda}{w(\rho + 1)}$$

if $\rho = 2\frac{1}{4}$ and λ is in tons per sq. foot, $H_r = 11 \cdot 1\lambda$.

The limit of stress allowable in stone masonry dams λ depends on the quality of masonry and varies from 6 to 12 tons per sq. foot involving a range of height of 66 to 132 feet. The tendency is towards allowing higher stresses in recent years.

For purposes of design, masonry dams may be classified as (1) Low, (2) High and (3) very high dams.

For low dams, the basis of design may be that of the "Elementary profile" where the lines of resultant pressure, reservoir empty or full are kept within the middle third of each horizontal joint and as close as possible to the outer extremities of this zone.

For high dams, that part of the dam below the limit depth of the low dam must be designed so that the centres of pressures, reservoir empty or full, shall fall nearer to the centre of the base than the extremities of the middle third. From Fig. 32 it will be seen that the nearer these are to the centre

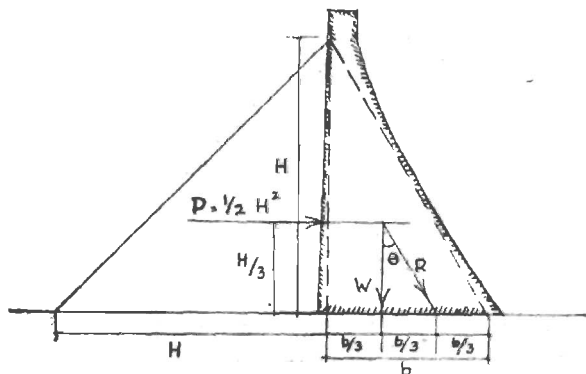


FIG. 32. Elementary Profile.

of the base, the nearer the maximum stress approaches the mean stress. Even in high dams, the section up to

the limit depth of the low dam up to 11.1λ is designed on the basis of the elementary profile of the low dam. The real determining factor in high dams is not resistance to compression but to shear. The ordinary theory gives the maximum intensity of shear as inclined at 45° to the maximum intensity of compression and half that maximum. The maximum shear is therefore equal to $101.5 H$ lb. per sq. foot

$$\left\{ \frac{62.5 (2\frac{1}{4} + 1)}{2} H \right\} = \frac{w \cdot (\rho + 1)}{2} H$$

approximately and accurately

$$\frac{q}{2} \sqrt{(1 + \tan^2 \theta)},$$

where θ is the angle between the resultant and the vertical, q = maximum intensity of compression pressure, reservoir full.

Very high dams are getting more common latterly and ordinary stone masonry will not suit the very high stress developed on the bases. They will have to be treated by special methods. In such dams, the intensity of compression stress becomes the limiting factor in the lower portions.

4. The Elementary Profile: Crest Width.—In actual practice, a dam must be provided with a crest of definite width and not terminate in the apex of a triangle. Bligh gives an empirical rule for the crest width of a dam as $a = \sqrt{H}$. This may be taken as the minimum permissible but the width needed in any particular case depends on the nature of traffic required to pass over the dam, the nature of the crest shutters and such other special requirements. The face line of the crest is kept vertical for a depth of 10 to 15 feet depending on the height of the dam when the batter for the rear face is started. The outline of the elementary profile thus becomes pentagonal.

5. **Free Board.**—The interval between the full reservoir level and the maximum water-level in a reservoir depends on the depth of spillage over the open waste weir if there is one. This interval can be reduced by the adoption of automatic gates or siphon spillways. The crest of the dam has to be raised above the computed maximum water-level to allow for the action of waves by an extent which depends on the 'fetch' or the longest straight length from the dam face of water surface exposed to the wind.

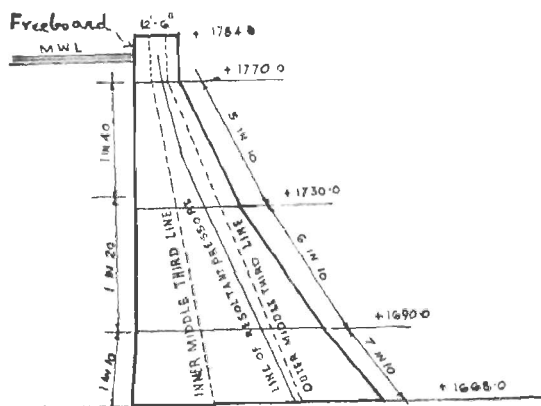


FIG. 33. Hirebhasgar Dam (Cross-Section).

Stephenson's rule for finding the height h of a wave with a fetch F miles is given by the equation

$$h = 1.5 \sqrt{F} - \sqrt[3]{F} + 2.5$$

This formula is modified by Molitor as

$$h_w = 0.17 \sqrt{vF} - \sqrt[3]{F} + 2.5.$$

where wind velocity v in miles is included. For F greater than 20 miles, the formula may be simplified as

$$h_w = 0.17 \sqrt{vF}.$$

Generally the 'free board' above the maximum water level should be greater than the height of waves found out by this formula and should not be less than 4 feet in any case.

The water-level for purposes of stability calculations is generally taken as the M.W.L. though, sometimes, it is considered as the level of the highest wave or as the crest of the dam.

6. Profile for a Low Dam.—The addition of a crest having a certain width and height above water-level has the effect of altering the position of the lines of resultant pressure from those of the elementary profile bringing that with the reservoir full towards the centre of the base but throwing that with the reservoir empty, a little outside the middle third. The former increases the stability slightly and needs no modification in the rear face. The latter is adjusted by giving a small projection to the lower part of the front face outside the elementary profile in the form of a continuous batter, starting from some point below water-level. The final profile is given in Fig. 32 assuming the specific gravity of masonry as $2\frac{1}{4}$ and the limit of safe compressive stress on masonry as 10 tons per sq. foot with upstream water surface at the crest level according to Tudsbery and Brightmore.

In practice, the upstream water-level is somewhat lower than the crest owing to the provision of a free board above the maximum water-level. In this case, the width downstream of the vertical from the front face at water-level would be, as usual $H'/\sqrt{\rho}$ where H' is the depth of water. This width would be slightly less than that in Fig. 32 where $H = H' + \text{free board}$. As a result the upstream projection of the base has to be increased slightly to get the theoretical profile for counterbalancing the weight of the top. This

would mean a greater variation from the elementary profile or from Fig. 31 when a trial section may be made and tested for stability mathematically or graphically.

7. High Masonry Dams.—The design of a ‘high dam’ is complex. There is a limit to the maximum compressive stress on masonry, but it is not desirable to increase the base width indefinitely in order to reduce this stress and it is not usual to have a slope flatter than 1 to 1 in the rear face.

A theoretical profile is first arrived at, similar to the “Elementary profile” for a low dam in which a suitable base width is obtained which limits the compressive stress in masonry while avoiding all tension, corresponding to the given depth of water, and the weight of the dam profile per foot length. Such a profile is given by Tudsbery and Brightmore for heights from the limit of the Elementary profile of 110 feet up to about 150 feet. This theoretical profile has to be modified to suit the practical requirements of any particular case to obtain the “trial section”.

This trial section has to be tested for stability by two methods: (1) the method of moments and (2) the graphical method.

In both the methods, the trial section is first divided into a number of slices or laminae by drawing horizontal lines at intervals of about 20 feet. The weights of these laminae are calculated. A vertical line is chosen upstream of the face and the moments of all the loads on the dam including the horizontal thrust on the upstream face and the vertical weights of each of the dam slices are taken round this vertical axis. The position of the centre of pressure and the intensity of resultant pressure at the bottom of each slice are derived and tested for the middle third rule and safe compressive

stress at the bottommost section. If the stress is too much, a revised section is assumed and tested similarly.

8. Graphical Method.—All the forces acting on a gravity dam can be represented graphically by lines representing the point of application, the magnitude and the direction of the forces. For example, the forces acting on the Elementary profile of a masonry dam are represented by a triangle from which the centre of pressure, and the intensity of pressure on the base can be determined graphically.

The methods of Graphic Statics are applied to the slices of the high masonry dam in testing the stability of the dam.

(i) The forces acting on the dam are drawn in the force diagram, which is in two bits. The vertical line gives the weights of masonry slices and the horizontal one gives the pressures of water in magnitude and direction from which their resultant can be determined easily.

(ii) A suitable pole is chosen to the right of the force diagram and the polar diagram drawn only for the vertical forces.

(iii) Now the centres of gravity of the dam slices are found graphically on the dam profile, when verticals dropped through them represent the slice weight.

(iv) To find the centre of gravity of the whole dam section, a funicular polygon is drawn below the dam profile with links drawn parallel to the corresponding rays of the polar diagram.

(v) The position of the resultant-reservoir empty is given by the centre of gravity of the dam from (iv) and its magnitude from the force diagram of weights.

(vi) The direction and magnitude of the resultant-reservoir full is given on the force diagram for each slice.

The points where the corresponding resultants cut the base line of the laminae are the centres of pressure-reservoir full. The 'line of pressure' is drawn by joining these centres of pressure. The total resultant acts on the horizontal base but the stress on a plane at right angles to its line of action can be determined graphically. If this is within the middle third of the base, there is no tension in dam masonry.

(vii) Given the point of application of the resultant on the base, its direction and magnitude, the distribution of stress from the heel to the toe of the dam may be represented graphically.

Thus, the maximum stress on masonry, and the presence of absence of tension in any part of the dam are obtained graphically. If the maximum compressive stress is within safe limits, the trial section is suitable.

Otherwise, the dam profile has to be modified and tested again by the graphical method.

As between the analytical method of taking moments and the graphical method, the latter is preferable since any mistake can be made out by the lines not meeting at the proper places.

9. Very High Dams.—The design of very high dams is beyond the scope of this manual.

10. Arched Masonry Dams.—An arch dam is a curved dam that carries a major portion of its water load to the abutments by arch action. It differs from a gravity dam in that the horizontal pressure is transmitted to the rock forming the sides of the gorge spanned by the arch, rather than to the bottom of the dam. Its stability depends not on its weight but on its crushing strength, though the weight

assists in reducing arch pressures. The pressure is transmitted through each horizontal ring to the abutment.

Arch dams are of two types: (1) the massive arch, where a single curved wall, nearly vertical, spans the full width between the abutments and (2) the multiple arch, with a number of inclined arches supported on piers or buttresses.

As in the case of the gravity dam, the economic profile of an arch dam would be a triangle with the apex at water-level. The upstream face is generally vertical. The minimum top width would be about 4 feet.

The Boulder dam in America of the gravity-arch type is the most notable in the world, 726 feet high above its foundations.

11. Buttress Dams.—A buttress dam consists of a number of buttresses or piers, dividing the dam into a number of spans. Panels consisting of (a) horizontal arches, (b) reinforced concrete beams and slabs or (c) steel beams and plating are inserted between the buttresses to retain water. It is like a bridge on its side, with arches vertical and forming the vertical face of the dam and piers or buttresses laid horizontally. They are called "Arch buttress dams" where the space between the buttresses is spanned by arches. The arch in a buttress dam is generally built with its axis inclined to the vertical at 30 to 45°. The spacing of the buttresses will depend on the depth of water and the nature of foundations.

Buttressed concrete dams are of four types: (1) Flat slab or Ambursen, (2) Multiple-arch, (3) Multiple dome and (4) Round-head.

Bed-rock foundations are desirable for building buttressed concrete dams. They are easier to build at sites with

gradually sloping abutments than at canyon sites with precipitous side-walls.

12. Deck Dams.—The more recent types of buttress dams have a relatively thin facing supported by buttresses so as to secure true structural action. A hollow type of gravity dam consisting of a series of concrete buttresses of a trapezoidal shape with slab faces resting on them is called a "Deck dam". They are suited to overflow as well as non-overflow conditions. The upstream face is inclined because of the small volume of concrete so that a portion of the water pressure may be utilised to provide a safe sliding factor.

13. Reinforced Concrete Flat Deck Dams.—Dams and weirs of the panel and buttress type are now built of reinforced concrete up to a height of 100 feet. They consist of an upstream deck sloping at 45° supported by buttress piers at short intervals. A thin covering is placed over the ends of these piers on the downstream side for the water to roll down in the case of a weir. The whole thus forms a hollow box with a number of partition walls.

Considerable reinforcement is required for a flat deck which may be replaced by arches.

6

Distribution Works

1. Channels.—The water let out from the head sluice of an anicut or reservoir for the irrigation of the achkat is conveyed by channels to the fields. The channels of a large distribution system may be classified as follows:—

(i) Main canal, taking the supply direct from the head-works;

(ii) Branch channel, taking off from a main canal;

(iii) Distributaries, taking off from a main or branch canal.

The above three classes of channels are maintained by Government, and

(iv) Field channels or watercourses are small channels, running from outlets in Government channels, constructed by the cultivators through their lands for distributing water to individual fields.

2. Main Canal.—A main canal is generally designed as a contour channel, falling at the slope needed to produce the required velocity of flow. As the line of flow of surface drainage is at right angles to the ground contours, such a channel cuts across the drainage lines of the country traversed. The line is sometimes shortened by crossing the spurs in deeper cutting and the valleys in higher bank than the general average. The considerations which would guide the selection of the best of the alternative routes in such a case would be:

(a) Cost of earth work and land acquisition by each route;

(b) Additional cost of the cross drainage works necessary if the contour line had been followed; and

(c) The value of the head saved by taking the shorter route.

In the case of direct flow systems, the first reach of the main canal is generally so deep in cutting that no land can be commanded and the canal is carried along the most economical line to the point where it commences to command the land. From this point, it is continued along the falling contour.

3. Branches and Distributaries.—Branches and distributaries are aligned as water-shed or side-slope channels so as to carry the water throughout the irrigated area at levels giving adequate command without obstructing the drainage of the area. They should take off from the main canal where it crosses the watershed of a spur.

A watershed channel is one aligned along any natural watershed so as to command all the ground up to the next valley on either side without any interference with cross drainage. This is the best alignment for a distribution channel.

A side-slope channel is one aligned at right angles to the contours of the country traversed and not on a watershed or on a valley line. Such a line would be parallel to natural run-off of drainage and would avoid intercepting any cross drainage. Its course must follow the shortest route to the nearest valley and such a channel will lie along the line of steepest possible slope. Except in very flat areas, only the smaller of the distributary channels should be so located.

Contour channels are carried on slopes, the main watersheds of which are not commanded. Such channels irrigate

only on one side and themselves form the boundaries of the irrigated area.

4. Watercourses.—For paddy cultivation, the fields are surrounded with low bunds, the slopes of the ground being terraced. It is unnecessary to provide a separate field channel for each field and all that is required is to group together fields which drain from one to another, the highest field from each group being supplied from the field channel and the others by field to field flow.

Field channels are laid out as contour channels without making special provision for cross drainage, for they are so small that they suffer little from the effects of cross drainage while the heading up caused by the low banks of these channels is insufficient to submerge and damage the wet crops. The drainage can be passed through temporary cuts made in the field channel.

The field channels should, if possible, be laid along the field boundaries. Separate channels should be made for low and high lying lands. Independent outlets and field channels should be provided for different villages; but a multiplicity of outlets results in great wastage of water.

Field channels are worked by internal turns and their capacity must be fixed accordingly. Large quantities of water are required for individual fields for transplantation and it is necessary that capacity to provide for this should be given to the tail reaches of these channels.

Balancing of Depth of Cutting

5. Main Canal.—When the spoil from the excavation of a canal is equal to that required to form banks of minimum standard free board, top width and slopes, the canal is said to be in balancing depth of cutting.

Where the cutting is deeper than the balancing depth, the extra spoil should be used in widening rather than raising the banks. In all cases, even when the water-level in the canal is below ground surface, the spoil from the excavation must be formed into regular sections, the top being given a slight slope to drain away from the canal.

6. Width of Berms.—The standard slopes of cutting being 1:1 and those of the spoil bank being $1\frac{1}{2}$ to 1, the distance to the centre of bank from the centre of channel would increase or diminish as the depth of cutting became smaller or greater. In order to make the centre lines of the bank parallel to the centre lines of the channel, it is necessary to introduce the expression $d/2$ as a part of the standard width of berms.

Berms are frequently omitted in small distributing channels running through valuable lands and may also be omitted from lengths, running wholly or almost wholly in bank.

Typical sections of channels with and without berms. are given in Figs. 34 for a channel (1) Entirely in cutting, (2) Partly in cutting, (3) Entirely in bank.

7. Standards of Canals.—The following statement gives the standard widths and heights of spoil banks, widths of berms and of lands to be acquired in South Indian practice.

Q = Discharge in cusecs

d = Depth of excavation in feet

1 to 1 = Channel cutting slopes

$1\frac{1}{2}$ to 1 = Spoil bank slopes

8. Side Slopes.—Although side slopes are excavated in ordinary soil at 1:1, the silt deposit on the edges and berms, gradually converts the slopes to about $\frac{1}{2}$ to 1. The discharges in the canal are, therefore, computed with an assumed side slope of $\frac{1}{2}$ to 1. The actual estimates for

	Main and Branch canals		Major distributarie			Minor distributaries	
	Above 1000 cusecs	1000 to 300 cusecs	300 to 150 cusecs	150 to 25 cusecs	25 to 10 cusecs	Below 10 cusecs	
Top width of widest bank	15'	6'	6'	6'	4'	3'	
Free board	3'	3'	3'	2'	1½'	1½'	
Berm width	$6 + d/2$	$4 + d/2$	$4 + d/2$	$2 + d/2$	$2 + d/2$	$2 + d/2$	
Land width to be acquired clear of banks	15'	Half the height of bank above ground subject to a minimum of 5 feet Height of bank plus 5'					
(1) When canal is in cutting deeper than balancing depth	Height of bank plus 15'						
(2) When canal is in cutting less than balancing depth							

excavation and the designs are, however, made for 1 to 1 slopes.

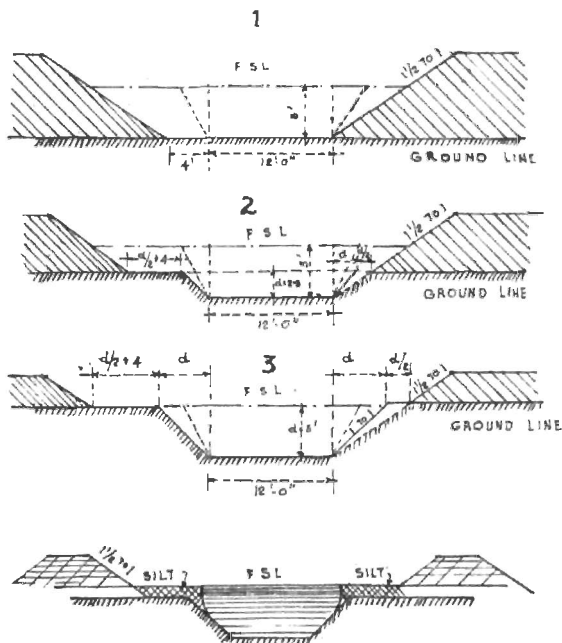


FIG. 34. Typical Sections of Canals.

1. In Full Embankment.
2. In Partial Cutting.
3. In Full Cutting.

9. Bends.—Bends in canals should be set out as circular curves the minimum allowable radius being 20 times the bed width of the canal.

10. Velocity.—The velocity in each canal should not be less than the critical, non-silting velocity according to Kennedy's formula— $V_0 = 0.84 d^{0.64}$, where V_0 is the critical velocity and d is the depth of the canal in feet, which should not be less than 2.5 f.s.

The maximum velocity must be limited to that which the margins and bed soil of the canal are capable of withstanding without erosion which should not exceed 5 f.s.

11. Best Discharging Section.—The best discharging section, *i.e.*, the channel, which for the same cross-sectional area, and shape, passes water with the greatest velocity will be the channel of greatest hydraulic mean depth.

12. Bed Width/Depth.—For a channel with side slopes $\frac{1}{2}$ to 1, the best discharging section would have the ratio

$M \frac{\text{Bed width}}{\text{depth}}$ as 1.236 or nearly $1\frac{1}{4}$.

The value of M which entails the least loss by absorption is, on the other hand 4, considering the absorption per unit of time as varying with the area of wetted surface and depth overlying this surface, for channels having the same surface fall, cross-sectional area, and coefficient of rugosity $N = 0.025$. Side slopes are assumed as $\frac{1}{2}$ to 1.

13. Design of Channels.—In practice, it is the quantity of canal discharge and not the channel slope and cross-section which is fixed, and the variables are the surface slope and the proportion of bed width to depth. The problem is to design a channel which will carry the fixed discharge with a non-silting and non-scouring velocity with the smallest absorption losses consistent with economy of construction.

(1) In the case of a canal in which the relation between velocity and depth is given by Kennedy's equation, the absorption is minimum for values of M (Bed width/Depth) between 1.5 and 3.2, and for values between 1.2 and 5, the loss by absorption is not more than 5% over the minimum.

So far as absorption is concerned, any value between 1 and 5 may be given. A low value of M is advantageous

to minimise evaporation losses, but where absorption is considerable, these losses are relatively small.

(2) When a canal is carried in balancing depth of cutting, the quantity of excavation varies but little for values of M between 1 and 5. The greater the value of M , the greater the area of land occupied.

(3) For any ordinary values of M , for channels of the same capacity, running in balancing depth of cutting, the level of the full supply water surface above or below ground level varies but little. For a channel which carries 2,000 cusecs, the level of full supply would be 3 feet above the ground surface for Kennedy channels in balancing depth and banks of standard dimensions but it is undesirable to carry the canal so much as 3 feet out of soil. Therefore, for the larger channels, deeper excavation than the balancing depth would be aimed at.

Similarly, for the smaller channels, it will be necessary to carry the water at a higher level, so as not to sacrifice command of land.

14. Coefficient of Rugosity.—A channel with flat, regular curves, smooth bed and slopes, free from weeds would have a low coefficient of rugosity in Kutter's formula. For small channels upto 50 cusecs, though a coefficient of 0.200 may be realised, it is desirable to assume 0.0250, and for larger channels, to assume 0.0225.

The shallower the channel, the flatter the slope with non-silting velocities, and so, when aligning a channel in very flat country, it is necessary to make it wider and shallower than would otherwise be desirable or economical.

In carrying a channel at a slope less than the ground slope, it is better to have a small value of M so as to reduce the height of 'drops'.

Where reservoirs are the sources of supply, there is no danger of silt, and channels may be given velocities subject to non-scouring limits. The practice will then be the reverse of that necessary with a "Kennedy (non-silting) Channel" and channels may be deep in proportion to their width.

It is desirable to avoid cutting into very hard, or very porous strata when a shallow cutting may become necessary. If the soil is unsuitable for making good banks, the channel may be taken as much within the ground as possible without sacrificing command.

Where there are no good reasons to the contrary, the values of M should be kept within the limits between $1\frac{1}{2}$ and 5.

15. Drops.—Where the fall of the ground surface is steeper than the canal slope, the longitudinal section of the canal will be divided into reaches by a series of drops. In such cases, command must be obtained immediately above each drop; and for some distance upstream of this.

The spacing and, therefore, the heights of the drops are fixed by the convenience in the location and spacing of outlets, and the requirements of command. Given the height and spacing of the drops, and the capacity, bed-fall and standard section of a channel, the most economical depth for the channel bed in each reach with reference to the ground

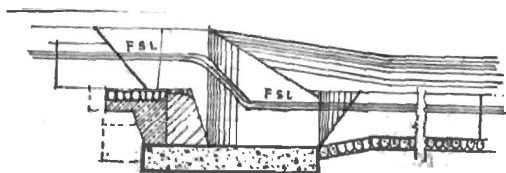


FIG. 35. Cross-Section of Canal Drop.

levels must be ascertained. This depth will be fixed by the depth of the channel bed just above each drop with reference to ground level.

16. Capacity of a Canal.—The duty of water, the allowances for absorption, and the system of distribution are the 'data' from which the carrying capacity of channels must be computed.

For paddy cultivation, the continuous flow system is practised; but there are many systems in which the regulation is effected by turns and in such cases, the carrying capacity must be increased to provide for the non-supply during the closure periods, or of restricted supply. The capacity must then be computed, working up from the tail end to the head of channel.

17. Command.—The level at which a distributary can be carried is the factor which fixes the area of the land to be irrigated. Sometimes, the channels are so designed that the full command of all land is attained when three-fourths of full supplies are carried. In any case, it is undesirable to include in the achkat, any land which can be commanded with difficulty.

In most soils, marginal scour will not occur with mean velocities of less than 2.5 feet per second and velocities upto 3.5 feet per second are safe in gravel and soft rock.

Channels may generally be designed with a proportion of bed width to depth, M , between $1\frac{1}{2}$ and 5.

It would be desirable to have a higher proportion of M , at the head of a reach, to make a fewer changes of channel depth.

18. Change of Capacity.—Change of capacity in a canal is effected by:

- (1) reduction of width,
- (2) reduction of depth,
- (3) reduction of the velocity of flow.

(2) and (3) react on each other and where depth is altered, grading is also altered to give the critical velocity due to the new depth.

Capacity is commonly reduced by changing the bed width just below an off-take. As the bed width becomes narrower, M becomes smaller and after a time, change of depth is necessary to give a more suitable value of M .

Changes in depth may be made at regulators, or at drops, and in a section where there are neither drops nor regulators, by reducing the bed fall below the surface fall for the required length.

When passing through a country with steep cross-slope, the channel is often carried with a bank only on the lower side of the slope. The drainage taken in must be disposed of at suitable sites. Such a channel will have a M.W.L. higher than F.S.L. and the bank must be given a free board above the M.W.L.

When the channel crosses a valley in bank, the depression is filled up to F.S.L. forming a tank, and full supply can be passed in the beginning of the irrigation season only when this tank is filled. Also, the submerged area is not available for cultivation, losses by percolation and evaporation are increased. The regime of the channel is interfered with by the silt brought by the surface drainage and the maintenance charges are high. On the other hand, these tanks function like storage reservoirs whenever a drop in F.S.L. takes place just below the tank. The capacity

of the tank between F.T.L. and F.S.L. of channel will then become available for steadying the flow in the length of channel below.

19. Mile and Furlong Stones.—Mile and furlong stones set in concrete foundations are planted on the downstream side. The mileage is measured along the axis of the main canal, commencing from the head works. For the branch canals, the mileage is started from the channel head sluice or regulator.

In large canals, a furlong is divided into six grades and grade stones with a distinctive shape are fixed firmly.

In the smaller channels, the number of grades per furlong is 2 to 3.

Masonry Works

The masonry works on a canal system consist of:—

(i) Works for the regulation and distribution of water, such as Regulators, Regulating Notches, Dividing Dams, Drops, Rapids, Irrigation Sluices and Pipe Outlets.

(ii) Communication works, *e.g.*, Foot bridges, Cart bridges, Road siphons, Cattle crossings.

(iii) Cross drainage works like Aqueducts, Siphon Aqueducts, Super-passages, Siphons, Level Crossings, Inlets, Outlets, Surplus and Scouring Sluices.

1. **Regulators.**—Regulators are built just below the off-take of a branch channel or distributary. Piers of masonry are built on a floor at bed level, at intervals across the canal and provided with grooves in which are installed movable shutters by means of which the water-level above the work may be regulated and the total discharge distributed to the channels taking off immediately above it.

2. **Regulating Notches.**—Trapezoidal notches are built on the channel floor which are not fitted up with planks or shutters, which automatically regulate the discharge above and below the work. They are termed "Regulating Notches".

3. **Dividing Dams.**—Small regulators consisting of a masonry wall with one or more rectangular notches regulated by planks inserted in grooves in the sides of the notches are called Dividing dams. When a channel divides up into two or more parts, a dividing dam is built across the channel

at the point of bifurcation. It is designed as a regulating notch automatically to subdivide the whole flow of the channel in the required proportion.

When there are no off-take sluices immediately below the bifurcation, rectangular weirs or notches with clear overfall are built where the necessary head is available. When the overfall is clear, the discharge of each notch is proportional to its crest length. This is a simple method which is recommended as it is easily understood by the ryots. Where one or both of the notches are submerged, this simple proportion no longer holds good. In such a case or where there are off-take sluices immediately above the dividing dam, the division should be made by trapezoidal instead of rectangular ones and these should be calculated for the actual conditions of submergence of each notch.

4 (a). Canal Drops.—Where the natural slope of the ground over which a canal is carried is greater than the bed fall of the canal, the difference is adjusted by constructing vertical 'drops' or 'falls' at suitable intervals.

A drop is merely a vertical step in the channel bed, the drop wall being a retaining wall forming the riser, the aprons and wing walls forming protective works to prevent the drop wall from being under-scoured or outflanked, or the channel eroded by the turmoil or velocity due to the drop.

4 (b). Rapids.—A rapid or a chute is a lined canal placed on a steep grade with an inlet and an outlet. It takes the place of a series of drops in a ridge—distributary. It is more economical where the ground slopes very steeply.

5. Irrigation Sluices.—A sluice carries water from the channel through culverts or pipes under one of the channel banks into a branch channel or distributary or watercourse. At each end of the culvert, suitable wings are provided to

retain the earth slopes, to form effective bank connections and to stop percolation. At the upper end is a head wall fitted with a shutter, moving in grooves, built in the masonry by means of which each vent can be closed or opened or the opening regulated.

The size of the sluice vents must be sufficient when fully open, to pass the water required for the full supply of the channel it feeds, under the available head or the difference of level between the water surface in the parent channel and the level of the distribution channel when at F.S.L.

6. Width.—The width of the culvert may be greater than that of the vent, to pass the full supply at a velocity not exceeding 5 feet per second. A masonry culvert should not be built of a smaller section than $2 \times 2\frac{1}{2}$ feet.

7. Shutters.—The heads on distribution sluices being comparatively small and not subject to great variation, the plug type of shutter is not used. Flat shutters, raised and lowered by a stem attached to the centre of the shutter and long enough to project a little above the regulating platform of the sluice when the shutter is resting on the sill are used. The stem or spear is usually an iron bar and the shutter is raised or lowered by screw gear. A single screw spear may be made to operate 6 to 8 feet spans of shutters.

Wooden spears without gears may be fixed to shutters up to 4 feet span for small sluices where regulating establishment is not maintained.

8. Pipe outlets.—Now, that C.I. pipes from 3" to 30" are manufactured at Bhadravati, the barrels of sluices may conveniently be made of these pipes with masonry head and tail walls of sufficient length to retain the channel slopes. The pipes must be laid on light concrete foundation to prevent unequal settlement and consequent leakage. It is

desirable to enclose the C.I. pipes with well-kneaded puddle so that the junction of the embankment and the pipe casing may be perfect.

9. Head Wall.—The upper face of the head wall should be placed at a distance equal to half full supply depth from the intersection of the channel bed and side slope. The top of the head wall must be at F.S.L. When the top walls are provided with wings, the wings are stepped to suit a canal side slope of $\frac{1}{2}$ to 1.

10. Tail Wall.—The tail walls of pipe outlets should be built upto F.S.L. in the field channel and the length should conform to the cross-sectional dimensions of that channel.

11. Head.—The head available under conditions of full supply at pipe outlets varies owing to alterations in bed levels caused by silt clearance and the cross bunding of the channel to suit internal turns, and it is, therefore, assumed that the water-level in the rear of the pipe is $\frac{1}{4}$ foot above the mean level of the highest field irrigated by the sluice.

12. Discharge in Pipe Outlets.—Pipe outlets are short pipes with cylindrical entry.

$$D = 0.438d^2 \sqrt{\frac{d}{0.36(4.17d + L)}} \times \sqrt{h}$$

where D is discharge in cusecs,

h is total head including friction in feet,

L is length of pipe in feet,

d is diameter of pipe in inches.

Communication Works

13. Bridges.—The bridges across canals are built on the same principles as Road Bridges but the difference is that, in a

canal bridge, the flow to be passed, the section of the stream, the velocity and levels of water are controlled and defined.

There is no advantage in having large spans for canal bridges. The waterway under the bridge should be equal to that of the canal as it is undesirable to head up water above the bridge. Deep foundations are generally not required. Flooring is unnecessary, but bed pitching round the piers and abutments for a width of about 5 feet is desirable. In small bridges, this amounts to the provision of a pitched floor throughout the span.

Cut waters should be provided both upstream and downstream for piers and half-cut waters for abutments which may be semicircular, rather than pointed.

If it is an arched bridge, the springing will be at the F.S.L. of the channel.

'Mantap' type of bridges with stone slabs is common on the older channels and can be built entirely with local materials and labour.

Reinforced concrete Tee-beams with decking slabs over masonry abutments and piers have now become popular, owing to facility in construction, using fairly large spans.

Road and foot bridges are frequently combined with weirs, regulators, falls, regulating notches and sluices.

14. Road Siphon.—As the F.S.L. in irrigation channels is generally above the level of the country traversed, the construction of arched bridges with springing at F.S.L., entails heavy approaches involving expense and inconvenience to traffic. This can be avoided by siphoning the channel under the roadway. Such structures are called "Road Siphons" which are not so common in Mysore and Madras.

15. Cattle Crossings.—The approaches to fords and cattle crossings consist of ramps on each side of the canal. The ramps commence at bed level where the side slopes of the canal intersect the bed and are carried up at a uniform slope to above the F.S.L. of the canal.

The slope of the cart crossings should not be steeper than 1 in 12, and should be paved, metalled or gravelled.

16. Retaining Walls.—In a wall which retains earth, the earth side is termed its 'back' and the outer side, its 'face'. The greatest stability for a retaining wall is secured when the batter is on the face. The batter in a dam is similarly given on the downstream side, the side remote from the thrust.

Sometimes batters in brick walls are given in offsets on the 'back' to prevent steps giving lodging to seeds of shrubs which may take root and damage the masonry. In South India, retaining walls are mostly built of random rubble and it is not necessary to resort to this method which involves additional masonry for equal stability to the extent of 10 to 15% depending on the face batter varying from $\frac{1}{4}$ to $\frac{1}{2}$ and the height of wall from 10 to 25 feet.

With a retaining wall of height H , top width 2 feet, built of masonry with specific gravity 2.1, to retain earth of specific gravity 1.8 and angle of repose $1\frac{1}{2}$ to 1, the base width may be taken as $0.4 H$ with a vertical face. But with a face batter of 1 in 8, the width may be reduced by 2 feet upto a wall height of 20 feet. When the wall is built of random rubble and all the batter is on the face, the base width may be reduced to $0.35 H$.

The base width of a surcharged retaining wall where the depth of earth is as much as the height of the wall, the base width may be taken as $0.4 H$ for a random rubble wall and $0.45 H$ for a brick wall.

Retaining walls with sloping tops are common for wing walls in irrigation works retaining earth slopes. Their tops are generally designed with uniform slope corresponding to the slope of the earth to be retained.

Cross-Drainage Works

The cross-drainages intercepting a canal may be disposed :—

(a) by passing the canal above the drainage,

(b) by passing the drainage above the canal,

(c) by taking the drainage into the canal.

17. Aqueduct.—Under (a), when a canal is carried over the drainage without dropping the bed level of the drainage, the work is called an aqueduct.

18. Siphon Aqueduct.—When the bed level of the drainage is dropped where it passes under the canal, so that the drainage is passed through an inverted siphon, the work is termed a siphon aqueduct.

19. Aqueduct and Drop.—There is an intermediate type between the aqueduct and the siphon aqueduct, where the drainage is dropped just above the crossing and the halla-bed cut out from this point at a gentle slope, there being no rise in the bed downstream of the work. This type is really a combined aqueduct and drop.

20. Super-passage.—Under (b), if the drainage is passed over the canal without dropping the bed level of the canal, the work is called a super-passage.

21. Siphon.—Where the irrigation water is passed through an inverted siphon below the drainage, the work is termed a siphon.

22. Level Crossing.—Under (c), the drainage is taken into the canal, by a 'level crossing', or an inlet and outlet or an 'inlet' only.

23. Types of Aqueducts.—Aqueducts and siphon aqueducts which are the most common type on a contour canal, may be classified according to the nature of the sides of the aqueduct into three types:—

Type I: where earthen banks with earthen slopes form the sides of the aqueduct.

Type II: where a retaining wall takes the place of the outer slopes of the earthen banks.

Type III: where masonry walls form the sides of the aqueduct, dispensing with earthen banks.

Type I, is most suitable for small spans upto 10 feet of culvert waterway; Type II for medium spans from 10 to 50 feet; and Type III for larger spans.

24. Aqueducts.—The level of the canal bed at the crossings of the drainage is fairly fixed, and the headway over the drainage bed can be increased by putting the crossing lower down, or diminished by taking it higher up.

It is desirable to limit the length of the aqueduct to the width of the stream and the contraction of lineal waterway from abutment to abutment must not be more than 25% owing to the width of the piers. The total area of waterway is fixed by the high flood discharge in the halla divided by the maximum velocity allowable through the openings.

The design of abutments, piers and arches of an aqueduct is similar to the design of road bridges with this difference that the load of water to be borne by the arches in large aqueducts is far greater than that on road bridges. Latterly, reinforced concrete Tee-beams with decking are replacing cutstone arches, and R.C. flume walls are used in place of masonry flume walls or earthen banks.

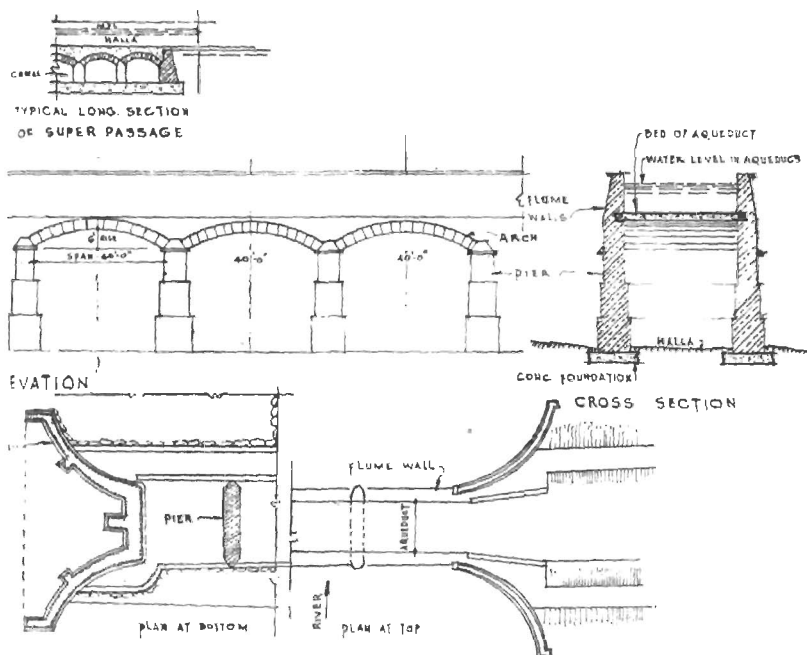


FIG. 36. Typical Detail of an Aqueduct.

25. Siphon Aqueduct.—The objection to the siphon aqueduct type is the risk of the blocking of the culverts by silt rolled along the bed of the halla for want of headway between the underside of the culvert and the bed level of the drainage downstream. This headway must be less than 3 feet or half the height of the culvert, whichever is less. The more nearly the siphon aqueduct approaches the combined aqueduct and drop, *i.e.*, the less the rise in level of bed on the downstream side of the drainage culverts, the better.

Whether to provide an aqueduct or a siphon aqueduct depends on the relative size of the canal and of the drainage. In the case of a large river, an aqueduct is generally necessary

as the lowering of the river-bed by a drop would be costly. On the other hand, the siphoning of the drainage will be preferable to carrying the canal in heavy banks when the drainage is small and a siphon aqueduct is indicated.

26. Bank Connections.—Connections between the masonry aqueduct and the earthen slopes of the canal retain the earth slopes and act as stop walls to percolation so as to allow no line of percolation steeper than 1 in 5.

A double set of wings is required in an aqueduct, the canal wings between the masonry sides and the canal banks, and the drainage wings retaining the earth slopes forming the guide walls entering and leaving the work and to act as stop walls to lengthen the path of percolation from the canal above.

In order to design the wings properly, it is necessary to make a plan showing the canal banks, stream margins and earth slopes.

27. Uplift on Siphon Culverts.—The floors of the siphon culverts are subjected to uplift: (a) from the subsoil water in the halla bed, and (b) from the canal.

(a) is a maximum when there is no drainage flow and when there is no water or silt standing on the siphon floor, but when the subsoil water is at the halla bed. It is a static head measured by the vertical height from the subsoil water to the underside of the apron.

(b) is a maximum when the canal is full and no water is flowing in the drain, and may be measured by a certain proportion, 60 to 75% of the difference of level between the canal F.S.L. and the bed of the drain, and the greatest uplift is developed in the centre of the length of the flank culverts.

The thickness of the floor would be the combined uplift head divided by the specific gravity of masonry, and where the thickness is found to be more than 3 feet, the apron is relieved in the following way:—

(1) By springing inverted arches between the footings of culvert piers so that the uplift load due to subsoil water is carried by arch action and the whole weight of the superstructure carried by the piers is thus employed in resisting uplift.

(2) By providing a puddle apron in the canal bed up and downstream of the aqueduct, to reduce the uplift due to the canal water at this point, by increasing the line of percolation.

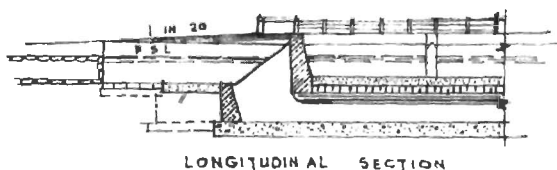
In the case of aqueducts where the halla level is not dropped, there will be no net uplift on the floor due to subsoil water, but only the uplift due to the canal water has to be provided for.

When the tail water-level below a siphon is higher than the under-surface of the culvert coverings, uplift is exerted on the culvert floor, which is counteracted by water in the aqueduct. But the worst condition when the canal is empty has to be considered in design. The pressure on the roof of the culvert is defined by the hydraulic gradient through the culvert barrel.

When the bed of the tail channel is 3 feet or more, below the underside of the culvert covering, it is unnecessary to make special provision for uplift of the culvert covering, but where the tail channel is at a higher level, the covering of culverts has to be made thicker solely for this cause. Alternately, reinforced concrete culvert coverings capable of taking bending stresses, anchored through the piers of the culverts, have to be provided for reducing the cost.

Surplus sluices can easily be installed in one of the side walls of Type III aqueduct. Vents through the side walls closed by screw geared shutters moving in grooves and worked from the roadway above, would constitute an effective surplus sluice, the outfall being into a water cushion formed by the counter slope of the siphon, while the drain forms a ready-made surplus channel.

28. Super-passages and Siphons.—The general principles of designs of super-passages and siphons are similar to those of aqueducts and siphon aqueducts. In both cases, the design of the two sets of wing walls and earth connections must be made by plotting the earth slopes on a plan of the banks.



LONGITUDINAL SECTION

FIG. 37. Siphon.

The respective level of the canal and drainage beds will generally determine which waterway is passed underneath the bed of the other. Where the difference in level is small, it is advantageous to siphon the channel.

In a super-passage, the canal will sometimes approach the work in full cutting, but in siphons, part of the channel water will be above ground, and the drainage is carried in a masonry or reinforced concrete trough of adequate dimensions.

A road bridge is generally combined with a super-passage on the downstream side of the canal.

In a siphon, it is desirable to pass water with considerable velocity to minimise silt troubles, and in all such cases, several alternative designs must be worked out to decide on the safest and the most economical type.

29. Level-Crossings.—When a canal is taken across and through a drainage, at about the natural level of the drainage bed, it is called a “Level-Crossing”. It is effected by the construction of a regulator, or weir, with falling sluices across the drainage just below the place of crossing.

A level-crossing is similar to a diversion weir across a river, with a regulator on one flank. It is the canal which generally carries the greater part of the water which is supplemented by the flow in the drainage. During floods in the cross drainage, the level in the channel may go above the F.S.L. of the channel, when a regulator is necessary across the infall to prevent flood water and silt from being carried upstream into the canal. This is similar to the head sluice in ordinary river head works.

Many level-crossings are effected with solid anicuts, but they should be provided with efficient scouring sluices, on both the flanks. This type is common in crossing large drainages where the cost of a siphon or siphon aqueduct would be prohibitive.

30. Inlets and Outlets.—Cross-drainage water can be disposed of, by an inlet which admits the drainage water on one side of a canal and an outlet which passes out an equal quantity of water on the opposite side. A level-crossing is a form of inlet and outlets with sills at the canal bed level.

Inlets are merely openings in canal banks admitting up-land drainage water into the canal.

Unless the drain entering a canal by an inlet is banked, it is necessary to carry the canal at the side of an inlet in such a depth of cutting that the F.S.L. is at or below the ground as, otherwise, the canal water would submerge land on the upstream side.

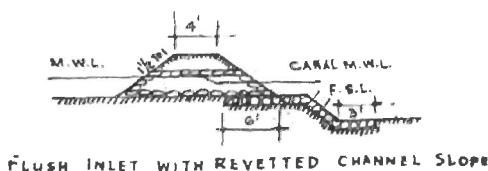
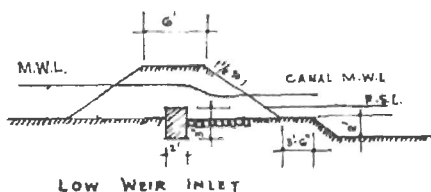


FIG. 38. Flush Inlet.

Surface inlets are generally arranged so that the difference of level between the M.W.L. in the drain above the work and that in the canal shall not be more than 1 to 1½ feet.

31. Outlets.—Outlets are surplus works which may be flush escapes, weirs or sluices and may be built in connection with and opposite to inlets, or as independent surplus works. They may be surface outlets, capable of discharging water from a level above the F.S.L. of the canal, or surplus sluices which can discharge water down to their sills.

Surface outlets may be weirs or flush escapes with or without crest shutters and of design similar to tank weirs. The sites of such works are fixed by the facilities for the disposal of the surplus water, after leaving the outlet.

Sluices used as surplus escapes must be designed to suit the level, bed fall and nature of soil of the outfall channel.

Surface outlets and surplus sluices may be combined by making sluice openings in the walls of surplus weirs.

The objection to the disposal of cross-drainage by inlets and outlets is the problem of silt, growth of weeds and the consequent spread of malaria. A scouring sluice should be provided in the vicinity of any inlet which brings in large quantities of silt, with its sill at about the bed level of the canal.

32. Scouring Sluice.—It is necessary to have a scouring sluice in the first reaches of a main canal to dispose of the silt that may enter through the head sluices.

33. Escape Sluices.—Escape sluices having no connection with cross-drainage works are provided in long canals to enable water to be reduced before this could be effected by the regulation of the head sluices. If the sills are at, or below canal bed level, these works are also useful for scouring purposes and for quickly draining the canal when closed for repairs. Escape sluices should be placed at suitable intervals throughout the lengths of long canals, sites being chosen above reaches in which liability to leaks is great. They are chiefly required after heavy falls of rain in the irrigated area, when cultivators block the pipe outlets to their field channels, when breaches may occur unless the canal water-levels can be promptly lowered.

The lengths of canals carried in single bank form continuous surface inlets and outlets of suitable spacing and capacity must be provided to dispose of the water so received.

A scouring sluice should be provided in the vicinity of any inlet which brings in large quantities of silt into the

canal. The sill level of such a work should be at or below the canal bed, and the bed of the outfall channel must also be below the canal bed and so graded as to carry away the silt from the canal bed. When the water entering a channel through an inlet is disposed of by a weir or flush outlet, scouring vents should be provided for silt disposal.

34. Catch Drain.—Where a considerable quantity of surface drainage is intercepted by a contour canal, a catch drain, graded to lead the water to the nearest cross-drainage work should be provided and it should be located clear of the outer toe of the canal bank.

35. Tail Escapes.—It is desirable to build a weir at the tailend of a canal with sill at F.S.L., to enable excess water to be surplussed without causing a breach, or it should be cross-bunded, the top of the bund being lower than the channel bank so as to form a breaching section.

A. Drainage Works

1. In South India, the incidence of rainfall in the irrigation season is such that the irrigation supply in a short time, is less than what is precipitated by heavy rain. It is, therefore, essential in laying out an irrigation system to see that natural drainage courses are reserved for drainage purposes and not interfered with more than is absolutely necessary and that adequate cross-drainage works are constructed whenever a channel has to be carried across a line of natural drainage.

2. **Marshes.**—In flat localities, as in the achkat of Sulekere in Chennagiri Taluk, there are considerable areas of which the natural drainage is bad, and where water stagnates forming marshes. Such land in its undrained condition is unfit for wet cultivation where the soil is of the black cotton variety even in a year of normal rainfall, while artificial drainage will make the land capable of taking wet crops even in a year of abnormal rainfall. The improvement of drainage by enlargement of existing and construction of new drains must therefore form a feature of new irrigation projects.

The difference between the level of the water into which a drain discharges and of the land to be drained may be termed the drainage head, and, where this is sufficient, a drain should be graded to secure a non-silting velocity of flow.

3. **Weeds.**—The fluctuating quantities of flow in drains and the restricted fall in flat areas not only lead to deposit

of silt but also the growth of weeds and grasses, which is difficult to control and which obstructs discharge. It may become necessary to exclude from a drainage scheme, any specially low lying blocks, the inclusion of which would have the effect of unduly restricting the slope of the drain.

The surface of the water in drains which traverse the lands draining into them must be lower than that of the lands and although the earth from the excavation may sometimes be spread out in low lying places, it will generally have to be disposed of in spoil banks on one or both sides of the drain; it is necessary to leave numerous openings in such spoil banks to serve as inlets for the drainage of the fields. The banks should be set back from the edges of the drainage cuts by leaving wide berms to provide a clear waterway for the disposal of exceptional floods.

Where the level of ordinary floods is sufficient to submerge cultivated lands by backing up of water through the drainage outfall, it is necessary to build a sluice across the outfall with shutters which are automatic or hand-operated, which are closed to prevent the river water backing up the drain, whenever the water-level at the outfall is higher than that in the drain, and opened when these conditions are reserved.

4. Irrigation and Drainage.—The primary functions of irrigation and drainage channels are antagonistic, the former requiring water to be carried at a higher level than the lands irrigated, and the latter requiring the water-level to be below that of lands drained. Yet, there are a number of old channels which act in both the capacities.

5. Cross-Bunds.—The primitive method of using the same channel for both drainage and irrigation is to cross-bund the channel for irrigation purposes, and breach the bunds when the channel has to be used as a drain. The method

involves excessive labour and is open to abuse. In small channels, effective regulation may be secured by masonry regulating notches, or by regulating dams. In larger channels, weirs with notches or sluices in their body are used for the same purpose which surplus large quantities of water after heavy rain.

B. Regulating Shutters

6. Numerous varieties of shutters—hand-operated and automatic are used to regulate the flow of water in modern irrigation works but only those types used in the old irrigation works in South India will be briefly described here.

7. Crest shutters used on the top of masonry anicuts are of two types:

(1) Falling shutters which fall flat on the weir crest when not in operation.

(2) Lift shutters which are operated from a platform above the weir crest.

8. **Falling Shutters.**—Their use is not common in Mysore but, on the Krishna anicut at Vijayawada and on the Godavari anicut at Dhavalesvaram, Fouracre's falling shutters 10 feet wide by 3 feet high were used. Those on the Krishna anicut are now removed during the recent remodelling works. Each shutter is maintained in the upright position by 3 tie-bars. When water rises to a level at which the moment of the water pressure about the points of attachment of the tie-bars is enough to overturn it, the shutter falls flat on the weir crest.

These shutters have been very helpful in temporarily raising the storage in the anicut when the irrigated area is badly in need of water at the close of the irrigation season.

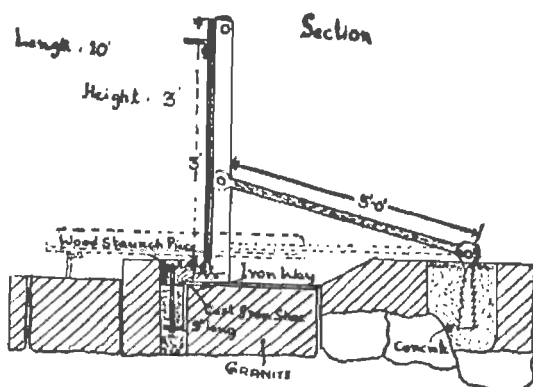


FIG. 39. Falling shutters used on the Krishna anicut.

9. Lift Shutters.—These are used everywhere on open weirs, regulators or sluices from spans varying from 10 to 40 feet. They are usually hung on chains from an overhead platform on which winches are installed for lifting and lowering.

The shutters are constructed of a frame of built-up girders, or of rolled steel beams with a facing sheet of steel. The larger ones move on rollers 6 to 8 inches in diameter, either running on axles fixed to the shutter, or mounted in a separate frame which runs on the surface of the rollers.

Rollers running on fixed axles were used when large span shutters were first used in the Kaveri delta, but latterly, the system of free rollers called "Stoney's type" after the inventor, are used in Krishnarajasagara, Thippagondanahally, Markonahally and Hircbhasgar. There is differential movement in this type between the roller cage and the gate. The movement of the gate is half that of the rollers, and for this reason, the roller cage is suspended from a pulley block through which is suspended a wire rope, one end of

which is fixed to beams of the lifting platform and the other to the moveable counterweights. The span of reservoir shutters is limited to 10 feet and the height of the gate is increased to get the necessary waterway. Several improvements are effected in this type latterly.

The advantages of a gate moving on rollers is that they reduce the effort to move the gate. If the effort required to move the old type sliding gate is 1, the effort to move a gate moving on fixed rollers is $1/50$, while it is $1/75$ for the Stoney's type of free rollers. Nowadays, the free roller system is adapted for gates above 10 feet span.

10. Drop Shutters.—This type is not common in South India but Ashford's patent drop shutter is largely used in North India. It is attached to the weir crest by hinges along its lower edge. The shutter is held up by a tie rod or a strut which can be released by the operating staff. The shutters are 3 to 4 feet wide.

11. Screw-gear'd Gates.—Sluices, shutters or gates up to 10 feet span working under moderate heads of water are usually operated by screw gear. The force is transmitted through a screw shaft attached to the centre of the gate. A screw thread is cut on the upper end of this shaft for the length needed to secure the travel of the shutter, and a screw nut to fit this thread is housed in a box which is fixed to the sluice platform above. This nut is revolved by a spanner or capstan head to lift or lower the shutter. The threaded length of the spear should be kept lubricated by a heavy oil and free from rust. Double shutters, sometimes in 2 or 3 tiers are lifted by a single screw in the Krishna and Godavari deltas.

12. Needle Shutters.—Small regulators and sluices are regulated by vertical planks 6 inches wide, the lower ends of which rest on the sill while the upper ends are supported

by a platform beam. One or two beams are given to support the needles in the middle, if they are long, between the platform and the sill. A step in the floor of the sluice up to the sill forms a ledge against which the lower ends of the planks rest.

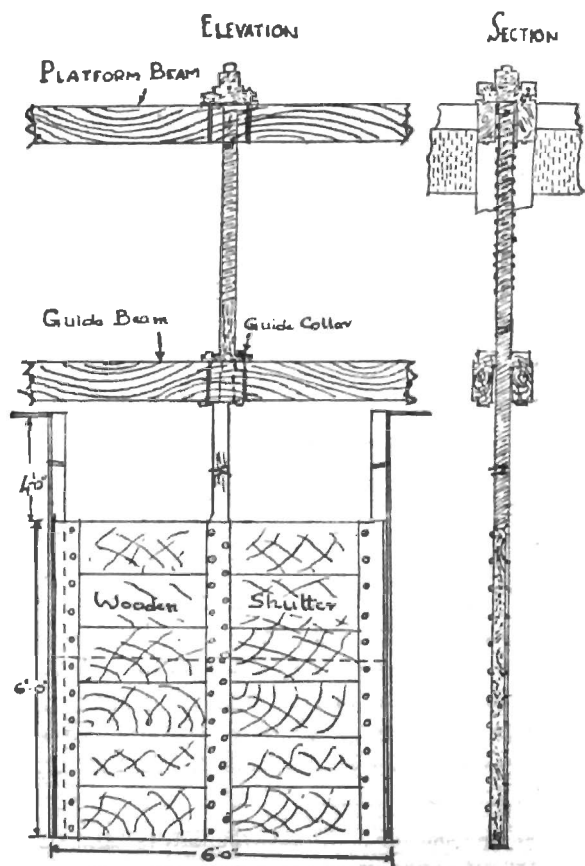


FIG. 40. Screw-gear Shutter.

Such planks are called 'needles'. They can be manipulated under considerable heads in depth of water up to 10 feet. Their disadvantages are that the operation takes much time and labour, and that the leakage through the needles is much.

13. Horizontal Planks.—Sliding planks in grooves or in grooved piers are also used on weir crests, in regulators and in escape sluices, up to a height of 3 or 4 feet. They are manipulated with the help of rings fixed at the end of each plank. These are more difficult to handle than needles except in shallow water and are not desirable in works subjected to floods.

C. Navigation and Irrigation

14. The topography of the Mysore plateau is not suitable for combining navigation with irrigation. But the large irrigation canals from the Krishna and Godavari anicuts are used with great advantage for navigation also.

The requirements for the one conflict with those for the other. For navigation the waterway must be uniformly wide and deep for the passage of the largest boats in use. The supply of water in the canal must be sufficient in all seasons to maintain the required depth. The velocity of flow must be low enough to make navigation easy. On the other hand, in an irrigation canal, the width and depth get reduced as the irrigated area is reduced. The widths and depths in the tail reaches are too small for boats to pass. It is generally kept closed in the off-season to conserve water in the storage reservoir. The velocity of flow should be high enough to be non-silling. It is thus clear that irrigation requirements have to be compromised if the canal is to be made navigable. It has to be made wider and deeper in the lower reaches than necessary. It has to flow at full depth at all times and the velocity has to be limited to 2 feet

per second. What is more important, the source of supply has to be unfailing as in a deltaic perennial river with a vast catchment area. In sloping country, the navigation canal has to be taken in a series of level reaches with vertical steps in the form of locks at intervals. The minimum width of a navigable canal is 20 feet and the minimum depth 4 feet. These provisions naturally increase the capital cost, but yet where water is available, a navigation canal is advantageous.

The following are the additional works needed on a navigable irrigation canal.

15. Canal Bridges.—Canal bridges have to be provided with large spaces with sufficient waterway. The minimum span in the deltas is 25 to 30 feet. The headway in the centre of the span for a width of 12 feet should be at least 11 feet. Girder bridges are more convenient than arched bridges.

Bridges have to be constructed across lock channels wherever there are bridges across lock weirs.

16. Canal Locks.—A lock has to be built where there is a drop in the canal. A canal lock is a short chamber cut off from the canal at the site of the drop. One end of the chamber opens on to the higher reach above the weir and the other end to the lower reach below. The chamber is closed at each end by water-tight gates. It is filled with water by sluices until the water level in the chamber is the same as that in the upper reach when the upper gate is opened and the boat enters the lock chamber. The upper gate is closed and with the lower gate also kept closed, the chamber is emptied until its water-level is the same as that in the lower reach. The lower gate is then opened when the boat moves into the lower reach. The process is reversed when the boat has to get up from the lower to the upper reach.

The sluices to fill or empty the lock chamber are either openings in the sheeting of the lock gates above the sill beams, or culverts passing through the masonry side walls behind the lock gates. The operating gear for the gates is of the rack and pinion type. The time for filling or emptying the locks is from 3 to 5 minutes.

The lock gates are made in pairs, pivoted on a vertical axis in a recess in each abutment. The combined width of the two gates is greater than the width of the lock so that the outer edges of the gates meet in an angle pointing upstream.

The lock chamber is in 2 sizes: (1) 150 feet long and 20 feet wide, (2) 105 feet by 15 feet in the Krishna and Godavari deltas.

17. Lock Weirs.—These are combined regulators and drops across the lines of navigation.

The drop in an irrigation canal extends from side to side of the canal and the approach and draft are straight on the axis of the canal.

But in a navigation canal, where a lock is introduced, it is necessary to have an approach for boats separated from the approach to the fall, or the lock weir.

The works may be arranged in three ways: (a) weir across the canal line and the lock on a diversion, (b) lock on the canal line and weir on a diversion, (c) both lock and weir may be side by side, the approach to the weir being separated from that to the lock by a row of fender piles.

Lock weirs regulate the depths of water above the lock for navigation and pass the water needed for irrigation to the reach below.

18. Fender Piles.—The piles may be of wood or of reinforced concrete, well-strutted and their heads joined by a wailing piece at about the full supply level of the canal. They are placed 8 to 12 feet apart. They are particularly needed across the faces of any off takes from the canal where the draw causes difficulty to navigation.

19. Tow-path.—All large canals are provided with continuous inspection paths and the extra cost for tow-paths is generally small. They can be taken along the edges of the berm where the canal berms are above F.S.L. Otherwise they have to run on the top of the bank. Light foot bridges are necessary to carry the tow-path across cuts leading from inlets or to off-takes.

20. To use an irrigation canal for navigation is not possible in the upland reaches of a river, but in the flat alluvial and deltaic tracts, navigable canals have many indirect benefits to the people. Cart-traffic is generally not possible in the rainy season except on metalled roads. Stone and other building materials are not easily available in this region for the construction and maintenance of irrigation works, roads and buildings. They can be conveyed at low cost over long distances on the canals. Agricultural produce can also be moved to the markets much cheaper than by road. These advantages are being enjoyed on the navigable canals in Krishna and Godavari deltas for a long time and, with the irrigation facilities, are responsible for the great prosperity of the area.

APPENDIX I

CENTRAL BOARD OF TECHNICAL EXAMINATIONS,
MYSORE

SYLLABUS FOR IRRIGATION I

II YEAR DIPLOMA EXAMINATION IN CIVIL ENGINEERING

Rainfall and its measurement—Catchment area—calculation of probable discharge from the catchments—Intercepted catchments and their discharges.

Duty of Water.—Measurements and calculations—Duty of different crops.

Tank Irrigation.—(a) Minor tanks, Major tanks and dams—Selection of site, location of weirs and sluices—Disposal of floods—Storage capacity.

Restoration of Minor Tanks.—Increasing the capacity of tanks and prevention of silt.

(b) *Head Works.*—Purpose of head works—Selection of site for head works—Elementary principles underlying the design of head works.

(c) *Distribution.*—Channels, description and classification of channels—Alignment of channels—Channels in different soils—Feeder channels—Difference between feeder and irrigation channels—Distributaries—Prevention of seepage.

(d) *Cross-Drainage Works.*—Regulators, aqueducts—Superpassages—Level crossing—Inlets and outlets—Syphons—Relieving weirs.

Lift Irrigation.—Source of supply—Subsoil water and water-table—Methods of raising water from wells.

APPENDIX II

CENTRAL BOARD OF TECHNICAL EXAMINATIONS, MYSORE

SYLLABUS FOR IRRIGATION II

III YEAR DIPLOMA EXAMINATION IN CIVIL ENGINEERING

Tank Works.—Elementary principles of design and execution of new projects. Duty of water. Determining capacities for storage and irrigation.

Lift Irrigation.—Sources of supply, subsoil water level. Cone of percolation, movement of subsoil water, quantity of water available, the moat and its use, entry of water to a well, artificial moat, drainage cones, classes of wells and area protected by walls.

Canals.—Duty, methods of improving duty. Finding the required discharge. Longitudinal and Cross-sections of main, branch and distributary; bed slopes of main canals; disposition of spoil banks. Brief notes on high embankments. Approximate losses of percolation and evaporation. Use of discharge tables and charts. Measurement of water modules. Block systems.

Head Works.—Selection of site for head works and main weirs. Brief description of types of weirs; height of weirs and afflux. Tendencies of weirs to fail. Foundations for weirs. Undersluices. Lifting sluices. Talus below weirs. Purpose and description of groynes below weirs. Afflux. Embankments. Canal head regulators. Temporary bunds.

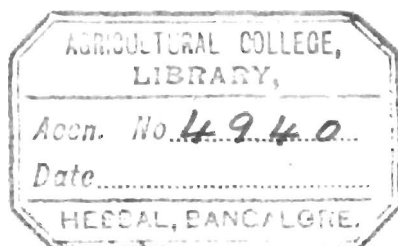
Drainage Crossings.—Principles of design and description of drainage crossings such as Superpassages, Level crossings, Aqueducts, Inlets, Outlets and Relieving weirs.

Regulating Works.—Description of regulators.

Falls.—Purpose and description of falls. Type falls. Object of raised crest. Purpose and description of notched falls, rapids, escapes, bridges, mills, plantations, water-courses and outlets. Earthwork in distributaries and consolidation of banks.

Drainage Works.—Importance of draining irrigated area. Silt tanks.

Training Works.—Purpose of straightening channels. Temporary training works. Methods of influencing current.



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<i>Page</i>	<i>Line</i>	<i>for</i>	<i>read</i>
15	24	<i>Add (Fig. 4) after "effectively"</i>	
16	2	Stream	Steam
39	8	<i>Add "and" after "Catchments,"</i>	
39	9	<i>Add "for fern-shaped catchments" after "hour"</i>	
90	Fig. 20 letterpress	Tank Waste with Dam Stones	Tank Waste Weir with Dam Stones —Type B.
108	29	13"46'	13° 46'
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133	1	$\frac{\frac{1}{2}H^2}{H.b.p.}$	$\frac{\frac{1}{2}H^2}{\frac{1}{2}H.b.p.}$
135	8	<i>Add "approximately" after the Equation</i>	
135	9	$\frac{q}{2} \sqrt{(1 + \tan^2 \theta)}$,	and $\frac{q}{2} \sqrt{(1 + \tan^2 \theta)}$, accurately
138	2	Fig. 31	Fig. 32

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