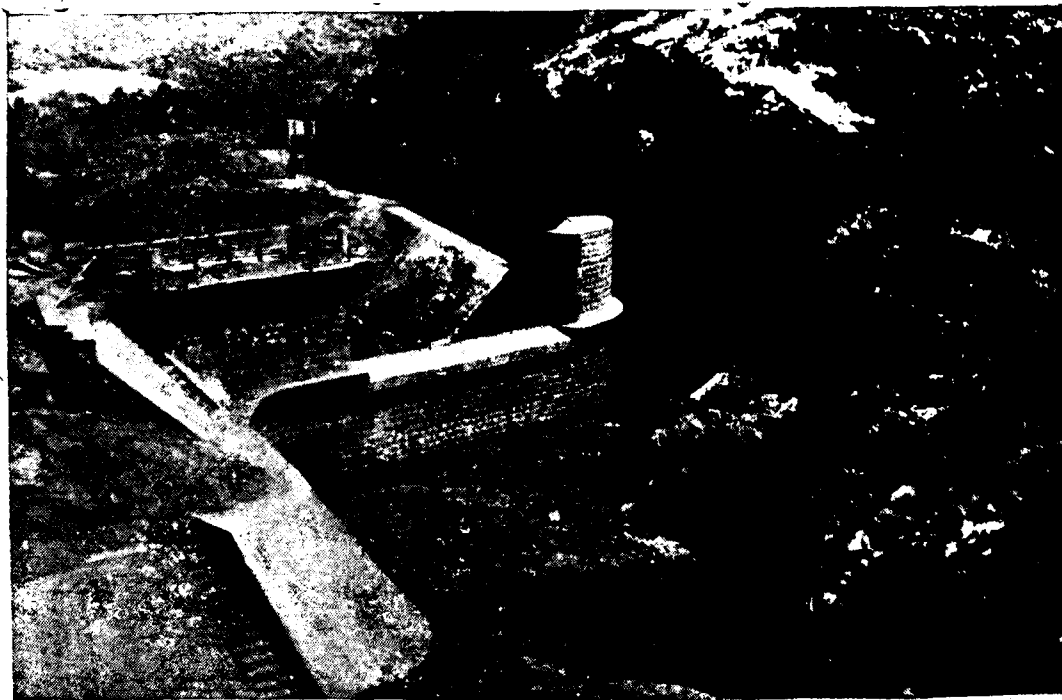


The
NEW IRRIGATION ERA





Parambikulam-Aliyar Project—A view of contour canal.



Parambikulam-Aliyar Project—
A view of Nallar crossing—Contour canal at mile 22/0.

THE NEW IRRIGATION ERA

Published by the Irrigation Branch of the Madras Public Works
Department, Madras-5, South India.

(For Departmental Circulation only)

Vol. X

JANUARY 1966

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The New Irrigation Era

1966 - January
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IRRIGATION CHIEF RETIRES



Sri K. V. Ekambaram, B.E., I.S.E., M.I.E. (Ind.), M.Am. SOC. C.E.,
Chief Engineer (Irrigation), Madras.
Retired on 2nd June 1965

Born in the year 1907, Sri K. V. Ekambaram was a distinguished student throughout his studies in the Engineering College, Guindy, from 1925 to 1929, securing the first rank in the College winning the College gold medal besides the University gold medal and prizes. After undergoing practical training in the Darwah-Pusad Railway construction near Yeotmal, he was taken into the Indian Service of Engineers as a full member from August 1932 and began his services as Assistant Executive Engineer in Madras State. As Executive Engineer, he was in charge of many important works including repairs to Agniar Aqueduct, cyclone damages to breached tanks in Chingleput District, Air raid shelters and radar stations in Visakhapatnam and so on. He was sent on deputation to the United States in 1946 and he was acclaimed to be the first Indian Engineer to have completed one full year with the Bureau of Reclamation and he returned to India soon after independence after visiting works and engineering laboratories in United Kingdom, Amsterdam in Holland, France, Switzerland and Egypt. His experiences have been brought out as a Government publication in 'My deputation to the United States of America'.

As Superintending Engineer he was in charge of Ramapadasagar investigation, Tungabhadra Canal circle, Tungabhadra Headworks circle

and Designs Circle, Madras. Then he went as the first Chief Engineer of Kosi Project in Bihar where he served from November 1954 to November 1957.

On return he worked as Chief Engineer of the Highways Department for a year. Then he joined as Chief Engineer, Irrigation, in February 1959. Under him major breach closing works in Cauvery in 1961, Sathanur II stage, Gomukhi, Manjalar and Chittar Pattanamkal projects were taken up. As the water resources of Madras State had almost been exhausted, a good bit of his attention was devoted to Planning and execution of Minor Irrigation Works.

Sri Ekambaram when he retired was the last officer of the distinguished service of the Indian Service of Engineers in Madras State. He was the Chairman of the Madras Centre of the Institution of Engineers in 1960-61, Vice-president of the Central Board of Irrigation and Power for the year 1964 and still continues to be in the consultative committee of the Srisailem Hydro Electric Project of Andhra Pradesh.

With such a distinguished record of service for the last 33 years, he retired in 1965 finally at the age of 58. We wish him long life and happiness in his retirement.

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R.C.C. PILE CUT OFF FOR THE ANICUT ACROSS PONNIAR AT SORNAVUR VILLAGE IN SOUTH ARCOT DISTRICT

BY

* SRI A. V. ALWAR, B.E.

1. The Scheme.

At the instance of the Government of Pondicherry, the Government of Madras in G.O. Ms. No. 2955-P.W. Department, dated 2nd November 1962 have approved the scheme of "constructing an anicut across Ponnai river in Sornavur village of South Arcot District at a cost of Rs. 17.56 lakhs and thereon, the detailed plans and estimates have been technically sanctioned for Rs. 17.56 lakhs by the Chief Engineer (Irrigation).

The Scheme contemplates the construction of an anicut of 1,000 feet length across Ponnai river at a place 1,000 feet above the existing Bangaru channel head sluice, including necessary scouring vents at the left flank of the anicut and the construction of a new head sluice capable of discharging 206 cusecs. The existing Bangaru voikkal would be connected to the proposed head sluice by excavating a link channel for a length of about 1,500 feet. The scheme would be complete with the provision of suitable flood banks on either side of the river.

The proposed anicut, replacing the sand dam (Korambu) usually formed by the Pondicherry Government involving considerable recurring expenditure, is to be the last among a chain of anicuts and reservoirs across the Ponnai river which has got a total ayacut of about 77,000 acres in Madras and Pondicherry States. This anicut is intended to give assured supply to the ayacut of 6,053 acres under Bangaru voikkal comprising 4,778 acres in Pondicherry State and 1,275 acres in Madras State. The Government of Pondicherry as the major beneficiary have come forward to bear the entire cost of the

scheme. As the proposed anicut site lies within Madras State, the construction has been ordered to be taken up by the Madras P.W.D., as a full contribution work.

2. General Principles of Design (Weirs on Permeable Foundation).

The proposed anicut at Sornavur is of a unique type as far as the Madras, P.W.D., is concerned since, for the first time in its history, a departure has been made from the conventional design of anicuts with body walls founded on wells and the present design is based on the theory, principles and rules enunciated in the "Design of Weirs on Permeable Foundations" by Dr. A. N. Khosla, Dr. N. K. Bose and Dr. E. McKenzie Taylor, published by the Central Board of Irrigation as publication No. 12.

A synopsis of the theory and principles involved in the present design is given below :—

(i) Theory of seepage flow in sub-soil.—

Until recently we were considering "Bligh's Creep Theory" for the design of weirs on permeable soils, according to which, the length of the path of flow of the sub-soil water had the same effectiveness in reducing the uplift pressure, whether it was along the horizontal or vertical. Lane's "Weighted Creep Theory" sought to amend Bligh's Creep Theory to some extent. As per these theories, in all weir designs, if the value of "C" the percolation co-efficient, was less than the safe value assigned by Bligh for the given class of sub-soil, the design was considered safe. While some of the weirs designed on this principle stood, some others failed.

In the nineteen hundred and thirties, when intensive field researches were undertaken by Khosla at the Upper Chenab syphons in the Punjab, consequent on their failures, it was observed that the pressure gradient lines derived from Bligh's Creep Theory were entirely different from those given by actual observations at site. During this period, the Central Board of Irrigation decided about the necessity to enunciate a simple, rational and correct method of design of weirs on sandy beds and to this end Khosla was commissioned to undertake full scale proto-type experiments in Punjab weir. Simultaneously intensive laboratory researches were also carried out at the Punjab Irrigation Research Institute on hydraulic and electrical models to correlate and compare with the results obtained from the field as a result of which the following theory was evolved.

Water in its flow through the sub-soil under a weir or other hydraulic structure loses head gradually along its travel, till at the exit, the head is reduced to zero. Thus, at each point under the weir floor, the water has a certain residual pressure gradient. Hence the weirs will have to withstand two forces, viz., (1) the residual pressure which will tend to lift up the floor if its weight is less than the upward pressure of water at that point and (2) the pressure gradient or the force of water acting along the direction of flow. If at the end, where the water emerges from the sub-soil, the upward force of water is in excess of the restraining force of the sub-soil, the surface soil will be lifted up followed by progressive disruption of the soil further down. This may result in undermining of the foundation soil and ultimate failure of the structure. Hence the two important factors to be considered in weir design are :—

(a) Residual or uplift pressure on the weir floor and ;

(b) Exit gradients.

For a given foundation profile of a weir, the uplift pressure must be kept as low as possible consistent with safety at the exit, so as to keep the floor thickness at a minimum and the exit gradient must not be allowed to exceed a certain safe limit, generally 1/5 to 1/7.

(ii) General Principles of Design :—

The Khosla's method of Independent variables evolved on the basis of the above theory offers a complete and accurate solution both in respect of uplift pressure and exit gradients for any complicated foundation profile of a weir. The solution is supported by data from prototypes in the field, models in the laboratory, both hydraulic and electrical and by mathematical theory.

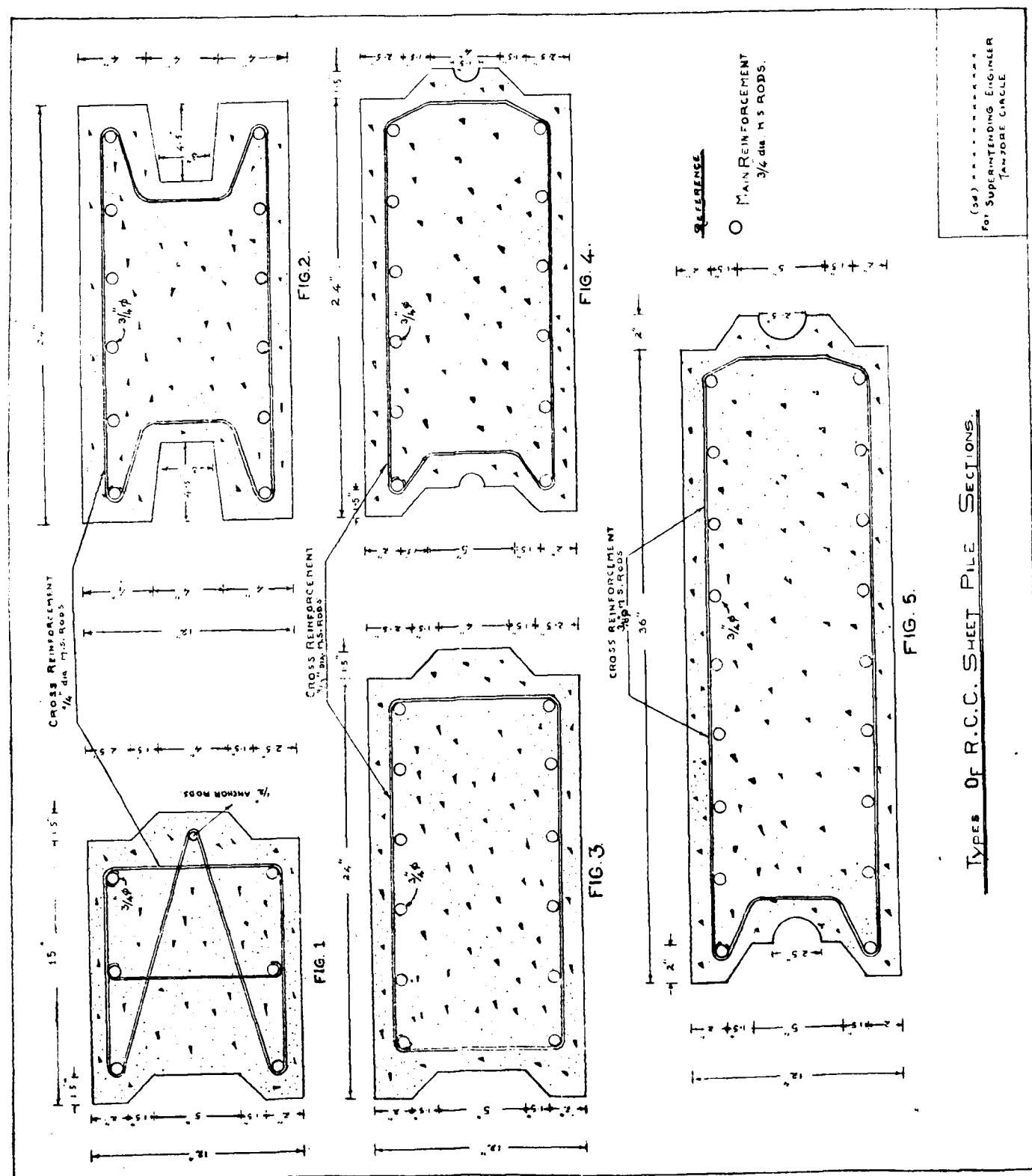
The uplift pressures will be counteracted by a suitable thickness of floor at different points and the exit gradient taken care of by a suitable depth of pile line at the downstream end of the floor. The depth of the downstream pile line is governed by two factors, viz., (a) that its bottom is at or below the normal scour depth and (b) that it gives a safe exit gradient with a suitable length of floor.

Scour can take place both on the upstream and downstream so that, a pile line at the upstream end of the floor is also essential, just as such at the downstream end, to prevent failure by slipping of the sub-soil into the scour holes by simple earth pressure. In the case of scours occurring on the upstream end of the floor, the flow of seepage water will tend to keep the sub-soil in position, as its force will oppose slipping due to earth pressure. But at the downstream end, this force of seepage water will assist and accelerate slipping due to earth pressure as the two forces will act together. The depth of the upstream pile line is governed by the depth of scour only, whereas, that of the downstream one by the depth of scour and exit gradient. Under normal conditions, the bottom of the scour upstream will be higher than the bottom of the scour downstream by the amount of the afflux and the sheet piles can be driven to the corresponding depths on the upstream and downstream sides. The necessity for the piles at the upstream and downstream end is thus obvious. Their function is to protect the work against undermining of the foundation soil.

3. Types of Cut-offs :—

Khosla advocates the use of interlocked steel sheets piles as cut-offs on the upstream and

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TYPES OF R.C.C. SHEET PILE SECTIONS

downstream end of the floor in preference to wells, as the gaps between the individual wells can rarely be blocked effectively all the way from top to bottom. On the upstream side the use of wells will be less objectionable, as their function is to restrain the earth from slipping into scour holes, although, if the gaps are not properly blocked, some slipping can occur. On the downstream side, the use of wells is definitely risky as undermining of the sub-soil can set in through any of the gaps, if they remain open at any place along the depth, as in this case the earth pressure and the forces of sub-soil water act together.

The special interlocking steel piles of the type required to form the cut-off are not indigenously manufactured and hence will have to be imported from abroad. With the present stringent foreign exchange position it would be very difficult to procure them. To get over this difficulty, the original design for the anicut at Sornavur sanctioned for Rs. 17.56 lakhs by the Chief Engineer (Irrigation) provided for concrete walls of 2 feet thick at the upstream and downstream end of the floor, to be built with plain concrete in cement mortar 1 : 2 : 4 using 3/4" size broken stone with 20 per cent replacement by surki. It was apprehended, that as long as the water table in the river bed is fairly high, it might not be possible to resort to this type of plain concrete cut-off, as this type of work involves open excavation with attendant dewatering for depths exceeding 20 feet in the unconfined sandy bed of the river which looked to be highly impracticable. Subsequent experience at the field during the last two seasons have also strengthened this view, as this type of work (plain concrete cut-offs) could be attempted only during four to five months in an year, when the water table is sufficiently low. Hence an alternative which could be worked during major portion of an year, except when the river carries some considerable discharge, had to be thought of and it was decided to try R.C.C. sheet piles for this purpose.

4. (i) Choice of R.C.C. Piles :—

The R.C.C. sheet pile thought of first was of the conventional type of size 15" x 12" with a male and female groove to act as guides between

piles (Figure 1). There was the inherent danger in adopting this type of piles for a cut-off, due to the piles not being close enough. In other words, such a cut-off formed with R.C.C. sheet piles with possible gaps in between the joints would be similar to the one formed of wells with imperfectly blocked gaps in between, discussed in paragraph 3 above and had to be rejected on the same grounds. The shape of the piles had to be modified in such a way, that when the piles are driven adjacent to each other in a row, an impermeable cut-off could be achieved. An experimental "H" shaped pile was the first result of this line of thinking.

(ii) The "H" shaped piles were of size 24" by 12" with grooves at both ends (Figure 2). The width of this type of pile was increased from the original 15" to 24" which would reduce the total number of joints, ultimately helping to reduce the quantum of sub-soil flow through the joints under the anicut profile. The grooves at both ends were provided, with the idea of effectively plugging the hexagonal hole, made up of the two grooves of the adjacent piles, by slipping in R.C.C. planks after clearing the sand from the holes. Some six experimental piles of this shape and size were manufactured. After allowing for proper curing time, the pile was hoisted and placed in position by the help of derrick posts (Tripod), guy ropes, winches and wire ropes. Then it was tried to sink the pile by the conventional high pressure water jetting method. As this did not help in sinking the pile, compressed air supplied through nozzles was applied at the bottom of the pile in addition to the nozzles supplying high pressure water. Even this improvisation did not help in sinking the pile, as the sand disturbed at the bottom of the pile by the high pressure water and compressed air settled back again in the same place, as there was no medium of running water in the river bed to carry it away, facilitating the pile to sink by its self weight. A technique had to be evolved by which the sand under the bottom of the pile could be gradually brought out as to enable the pile to sink by its self weight. This could be achieved by a novel process of applying high pressure water jet, compressed air through independent 3/4" diameter pipes ending in nozzles within

a 3" diameter casing pipe, placed on the faces of the pile, 2 numbers on each face. Inside each 3" diameter casing pipe, water and an air jet were worked and this resulted in the sand getting stirred and disturbed by the former, while the latter helped to blow it out through the top opening of the same casing pipe. As the sand under the bottom of the casing pipe and consequently under the piles got displaced the pile sank by its self weight. By this method one pile could be sunk to its full depth of 16 feet in about 3 to 4 hours. After driving some piles in a row, an attempt was made to remove the sand from the grooves in between the piles, so as to make them water tight by plugging them with R.C.C. locking slabs and cement mortar by applying the water and air jet directly inside the grooves. This process was not fully successful, as during the course of blowing out of the sand from the inside of the grooves, it was observed, that the sand from both the upstream and downstream side of the joint started slipping in through the gaps at the joint by earth pressure, rendering it impossible to have the grooves completely removed of all sand. An alternative method of inserting the locking slabs simultaneously with the blowing up of sand was also not quite effective, as the locking slabs did not go right up to the bottom of the pile, indicative of the fact of some slight shift in alignment and tilting between the adjacent piles. The above experiments with the "H" shaped piles indicated the necessity for (a) some sort of guide between the piles as available in the male and female groove type; and (b) Some innovation for plugging the joints effectively to achieve an impermeable cut-off.

4. (iii) To find out their behaviour while sinking, a few R.C.C. sheet piles of size 24" by 12" with plain male and female grooves (Figure 3) were manufactured and sunk by the process explained earlier in sub-paragraph 4 (ii). It became evident from their performance during sinking that this type of piles not only afforded good guide between each other but also made it possible to achieve close joints between the piles.

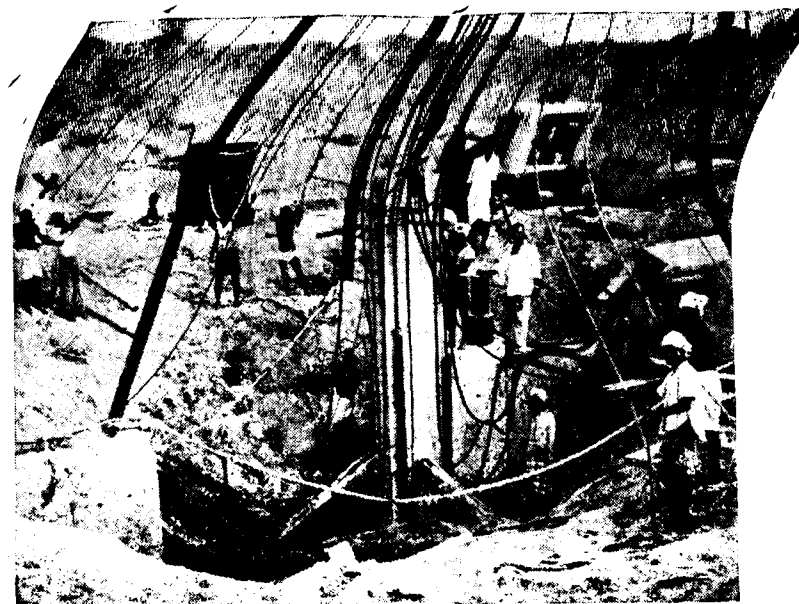
4. (iv) Though the type of piles described in sub-paragraph 4 (iii) above satisfied condi-

tion (a) stated in the concluding sentence in sub-paragraph 4 (ii), condition (b) stated therein remained unfulfilled. To satisfy this condition, a further innovation of a 1½" diameter semi-circular hole cut at the middle of the male and female grooves, in the type of piles described in sub-paragraph 4 (iii) above, was introduced (Figure 4). The trouble with this type of piles was, that, in the formed hole of 1½" diameter obtained at the joints between the piles, a pair of water and air jets could not be operated simultaneously for want of enough space and hence the entire sand from inside the formed hole could not be removed. Hence this had to be further improved. Consistent with the dimensions of the male and female grooves, the largest formed hole that could be accommodated which would facilitate the operation of a pair of water and air jets without much difficulty, was finally decided upon as the one with 2½" diameter.

4. (v) The final size and shape of the R.C.C. sheet pile adopted for the cut-off was 36" by 12" with the conventional male and female grooves, with 2½" diameter semi-circular formed holes cut at the middle of the male and female grooves (Figure 5). It would be noticed here that the width of the pile has been further increased from 24" to 36" in this type. The important factor influencing this decision had been, to have consistent with the possibility of handling and hoisting, as wide a pile as possible with a view to reduce the number of joints between the piles to the barest minimum, thus reducing to the minimum the possibility of seepage flow of sub-soil water through the joints.

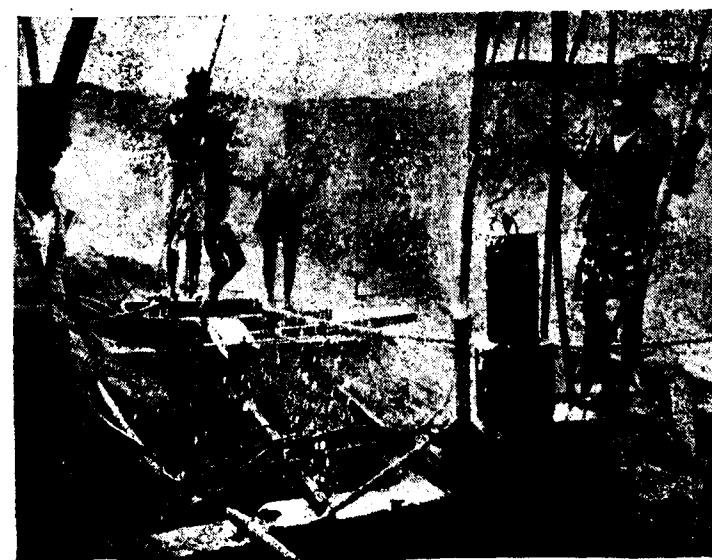
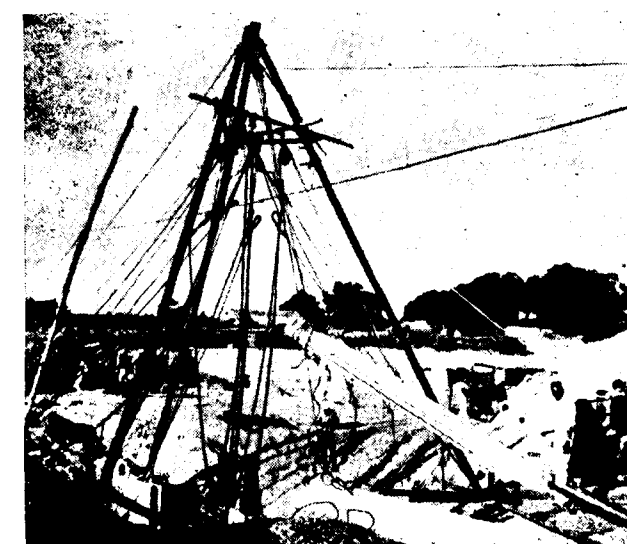
4. (v) (a) Casting of Piles.—It is highly advantageous to cast the piles as near to the place of driving as possible, to avoid the extra cost involved in moving the piles, if they are to be manufactured at a particular moulding platform and conveyed to the place of driving. Further, as each pile weighs about 3.3 tonnes, its movement over long distances would not only be costly but also very cumbersome and damages to the grooves cannot be entirely precluded. To obviate all this possible difficulties, the river bed itself is levelled up, watered and rammed well, over which 16 gauge M.S.

R.C.C. PILE CUT-OFF FOR THE ANICUT ACROSS PONNIAR AT SORNAVUR VILLAGE.



R. C. C. pile for the cut-off placed in position for driving.

R.C.C. pile being hoisted in position with the help of pully Blocks and Tripod.



The Grooves in between the R.C.C. piles are cleared of sand by the application of water and air jet before grouting.

sheets or G.I. sheets are spread to form the moulding platform. Over this the mild steel grill is placed, on either side of which the two halves of the country wood mould representing the two sides of the piles are placed and held in position by steel brackets slipped from top at regular intervals. Cement concrete of proportion 1 : 2 : 4 using $\frac{3}{4}$ " broken stone is then laid and after allowing for proper curing the piles are ready for driving.

4. (v) (b) Driving of Piles.

The pile is hoisted and held in position with the help of derrick posts (tripod), guy ropes, winches and wire ropes. After checking the plumb line and alignment, 3" diameter pipes at the rate of two Nos. for each face of the pile are placed and high pressure water jet and compressed air are applied into the pipes through independent $\frac{3}{4}$ " diameter pipes provided with nozzles at the ends. By the same process of displacement of sand from the bottom of the pile as discussed in sub-paragraph 4 (ii) above, the pile sinks down by its own self weight. After a number of piles have been driven thus in a row to the required depth, the sand from the $2\frac{1}{2}$ " diameter holes formed in between the piles is

removed by first directly applying a water jet and compressed air through $\frac{3}{4}$ " diameter pipes and then through a 2" diameter pipe inserted therein. After the sand is completely removed from the hole, cement mortar 1 : 3 is poured into the hole through the 2" diameter pipe and tamped well by a sufficiently long tamping rod, extracting the 2" diameter pipe gradually. The top few feet is grouted without any pipe insertion and tamped till refusal from top. By this process completely water tight joints are achieved resulting in a continuous impermeable cut-off, which is our object.

The above procedure is successful and effective as long as the water table is at or slightly below bed level. Under such favourable conditions this method is reliable and recommended for similar works. But in dry condition; when the water table goes down at a rapid pace, we meet with serious difficulties and the progress is very much retarded.

5. Economics of R.C.C. Pile Cut-off.

The cost of forming the R.C.C. sheet pile cut-off is detailed below :—

(i) The cost of R.C.C. sheet pile for one cubic foot :—

	Rs.	P.
Cost of R.C.C. 1 : 2 : 4 for the pile excluding reinforcement	5	00
Cost of steel including fabrication at 12.2 lb./1 cft. rate at Rs. 57 cwt.	6	20
Cost of sinking including grouting the formed hole in between the piles	5	50
Total for one cubic foot	16	70

(ii) The cost of one number of R.C.C. sheet pile including sinking and grouting the formed holes is worked out below :

Size of sheet pile	= 16'-0" × 3'-0" × 1'-0"
	= 48 Cft.
Cost for 1 cubic foot	= Rs. 16.70.
Cost for 48 cubic feet	= Rs. 801.60.
Therefore cost of one pile	= Rs. 802.

(iii) The cost of one running foot of R.C.C. sheet pile :

Cost for 3'-0" width of pile	= Rs. 802.
Width of pile : 3'-0"	
Cost for 1'-0" width of pile	= Rs. 267.33.
Therefore cost per one Rft. of cut-off	= Rs. 268.

This would compare very favourably with the cost of a cut-off formed with interlocking steel piles. The double advantages of (1) the economy in cost; and (2) the ease with which they can be manufactured at site according to

the requirement, weigh in favour of the aforesaid R.C.C. sheet pile operation in preference to steel inter-locking piles which will have to be imported under great difficulties.

COLLECTOR WELL

BY

* SRI J. WALTER, B.A., B.E., M.I.E. (COLORADO), M.I.E. (INDIA).

The Collector Well is a circular well about 15' in diameter going down as far as nearly 100'. Horizontal pipes and screens through the aquifer, lead into this well. The water from the aquifer is collected through these horizontal pipes into the well and hence it is called "Collector Well". Before going into the details of the collector well, a brief review of the background of the ground water exploration in our State is made.

A Ground Water Cell has been formed in our State in the Public Works Department on 1st March 1965. Prior to the formation of the Ground Water Cell, several Central and State agencies have been engaged in investigation and exploitation of Ground Water in our State. So far, the drilling work necessary for this purpose, both for exploration and production has been done by hand-boring sets and a few power drills, both of the percussion and rotary types. These tube wells go vertically down. The dimension of the tube wells varied from 3" to about 18". The large diameters however, are very rare and few.

There has been a growing realisation of the vast potential of ground water unutilised especially now when we have run short of surface waters. While several fears and guesses exist about the exploitation of ground water with reference to the available potential, scientific opinion seems to be that in our State, we have sufficient ground water that can be utilised without detriment to the ground water storage, i.e., scientifically, ground water exploitation can be attempted in our State to the extent rain fall in the upper region can reach to recharge the aquifers. A geological map of Madras State is appended (Fig. 1).

Several agencies and departments, both from the centre and State have been exploring and

developing ground water projects and *ad hoc* Projects. But time is opportune now for a systematic development of a grid of ground water projects through out the entire State depending upon the Geo-Hydrological aquifer formations. The ground water project for Madras State have necessarily to be multi-purpose in character serving drinking water supply, Industrial use and for Agricultural and other purposes. So far, we are used to the vertical tube wells of small diameters. A good tube well can at best yield about half a million gallons per day. Therefore large numbers of tube wells will be necessary to meet the full needs of our State. One of the serious bottlenecks for driving tube wells is the non-availability of good power drills which are at present imported into our country. Hence a study of the collector well deserves some considerations. A general description of the collector well has been attempted in the following paragraphs.

A drawing showing the plan and section of a typical collector well is appended (Fig. 2).

The principles underlying abstracting water from the ground by the collector well are :—

1. The exposure of a large area of water bearing strata to horizontal perforated pipes. The number and area of these pipes can be increased or decreased as requirements demand.

2. Selective removal of fine materials such as sand etc., which are an impediment to free flow of water from the aquifer formed near about the screen pipes and the formation of more permeable ground or gravel pack adjacent to the screen pipes, so that each installation consists of a series of radiating drainage channels emptying into a central shaft.

* Superintending Engineer, Ground Water Cell, Madras.

3. Access to and independent control of each radial pipe permitting easy backwashing or flushing of every screen pipe separately. On account of these features and because of the horizontal collecting pipes which are located below the draw down level of the ground water, a good collector well can be used to develop a water supply scheme at a particular site of a quality and quantity that may not be economically obtainable by conventional method.

Due to the decrease in the actual pumping head, low maintenance charges and permanence of the productive capacity favour the adoption of collector wells over the conventional type of wells.

The practicability of going in for a collector well depends primarily on the Hydro-geological conditions of the site and the quantity and the characteristics of the water required.

A completed collector well essentially consists of a central cylinder of monolithic concrete caisson about 13 to 15 feet inside diameter with 18" thick walls. This cylinder is sunk by excavating the inside earth and the bottom is plugged and sealed with concrete after the well has been sunk to the required depth. About 4' above the top of this plug 6" or 8" diameter perforated pipes are driven into the aquifer horizontally through the precast holes in caisson to form a radial pattern. Each screen pipe is surrounded by a bed of gravel on the inner end of these pipes and a gate valve with a stem extending to the pump house is fixed. The length and size of the perforated radial pipe or screen pipes as they are generally known, depends, on the quantity of water required and they can be to a length of even 300 feet. The use of vertical wells as a source of water supply to the large cities and industrial plants is not economical since a number of units would be required and the operation of each well must be supervised which results in a large maintenance cost. If physical conditions are suitable for the construction of a good unit, one collector well can yield as much water as can be obtained by a dozen or more vertical wells located in the same ground because of the much larger screen area.

Concentration of the collection of water from fewer points is far easier and more economic since the acquisition of necessary land is low and simplification of feeder pipe lines and their connections with the mains.

The design of the collector well and their method of construction permits choice of the geological formation and depth from which water is to be collected. The extraordinarily slow and uniform rate of filtration that can be achieved, results in thorough bacteriological purification. Experience has revealed that in some cases the water is softer than that well at the same location. This is either to the draw down being less, a smaller proportion of water extracted from the deeper strata or because of slower velocities through the ground.

The quantity and quality of water coming from each screen pipe can be measured and tested, which simplifies the control of proper rates of filtration and pumping. It is also possible to backwash or blow out such pipes and thus maintain the effectiveness of the natural filter. The water is supplied at all times from the wells which have water tight walls with no chance of even accidental contamination lesser than even in the case of filter plants.

Maintenance and Operation Facilities—

For the efficient operation of any mechanical equipment easy accessibility is important and a collector well installation provides full facilities for inspection, maintenance and reconditioning of all the vital parts. Repairs and replacement of parts can be carried out without interruption of service of whole installation for a long time.

Each screen pipe can be regulated or shut down separately without interfering with the production of the other collecting elements.

Pumping Economy—

Saving in the pumping cost is obtained by full utilisation of the natural head pressure available.

The large screen area in contact with the water bearing materials results in correspondingly

lower velocity of the water through the screen openings and through immediately adjacent ground therein. The extension of the collector pipes and radial drains away from the shaft also reduces the approach velocity of the water through the aquifer with the result of only a small loss of head and the level of the water in the shaft is only slightly below that of the water in the ground.

The usual draw down curve (piezometric line) produced by every vertical well is completely eliminated. This may mean a basic gain of many feet in the total pumping head.

Operation During Flood.

At flood stages there is larger head of water over the collecting screens and hence the capacity of the installation is very great. The quality of the water could not materially change since the actual rate of infiltration corresponding to consumption rate is not altered and always remains under complete control.

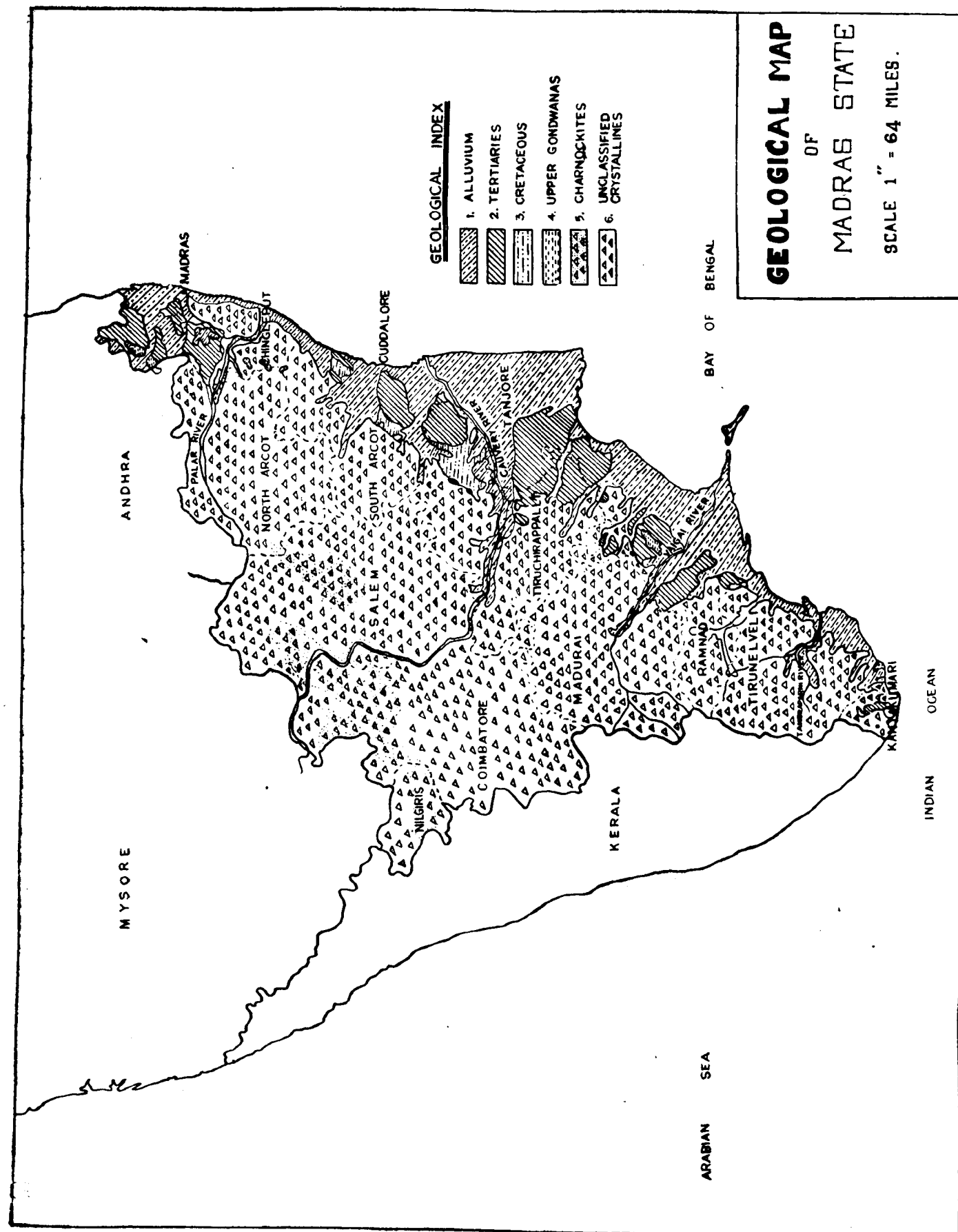
In comparison with conventional wells, the collector wells are more permanent by the use of reinforced concrete and heavy non-corrosive metal alloys. Full access to the interior of the well makes inspection easy and when screen pipes finally deteriorate new screen pipes can be pushed through the extra portholes left in the walls of the caisson.

The above points are the advantages of going in for collector wells in preference to the vertical wells especially where large quantity of water supply is required. There are a few patents of collector wells largely used in America, patented by Ranney is known as Ranney System. Mon-Paul Classic is carrying out water supply schemes in India in collaboration with Mr. R. J. Shah and has constructed one well near Baroda with amazing results. By adopting this system only very little foreign exchange is required and most of the equipments are available in India.

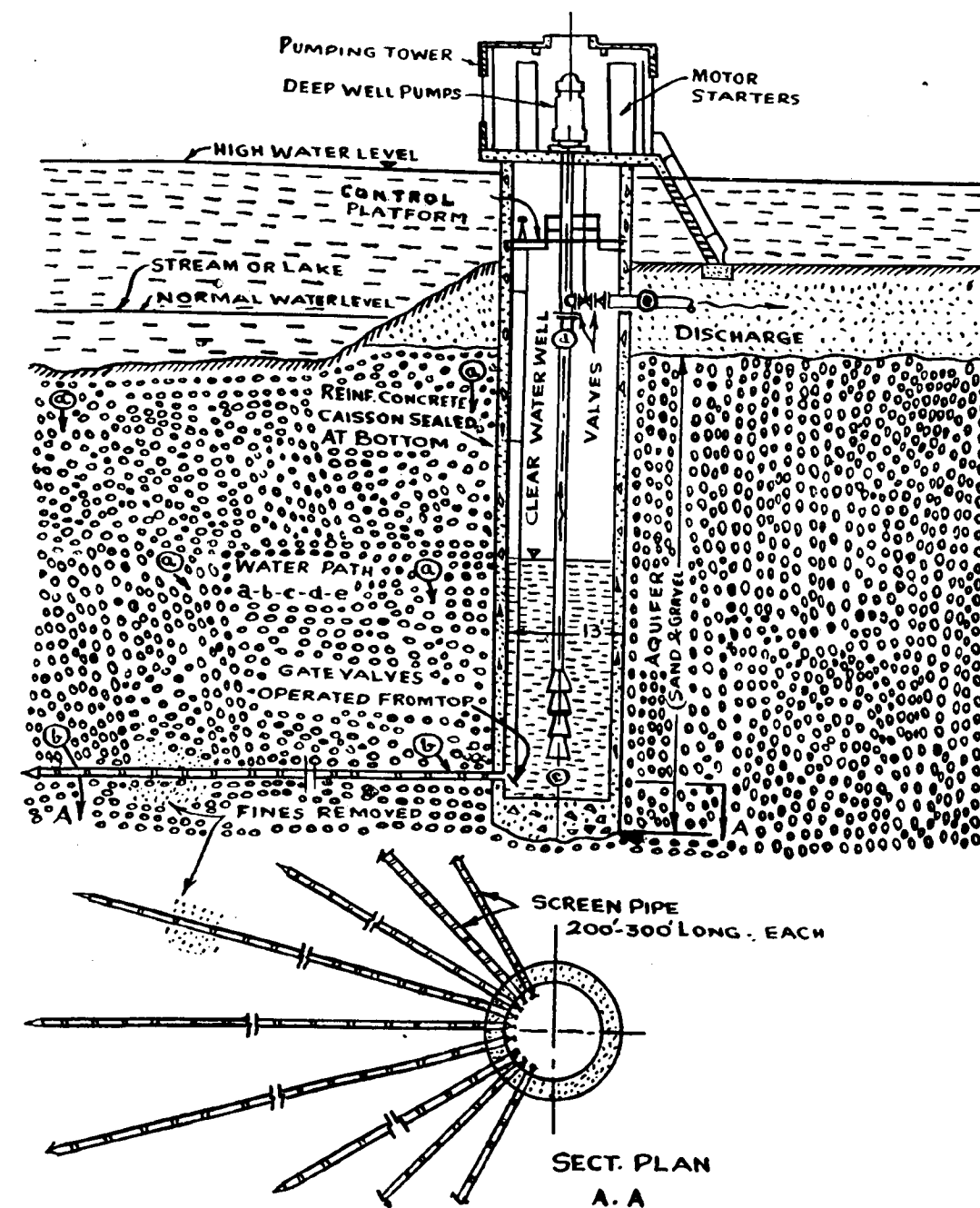
It may be seen from the above description of the collector well, that many of our small rivers like Courtalayar have on their banks ideal locations for development of collector wells.

The readers are requested to offer their valuable comments on the collector well. Information regarding probable suitable locations for collector well for investigations and queries relevant to any problem in ground water in our State are most welcome.

- References.*—1. Ground Water Hydrology by D. K. Todd.
2. Minerals of Madras State by Sri M. K. N. Iyengar.
3. Water-supply Engineering by Babbitt and Dolland.



A COLLECTOR WELL.
FIG. 2.



EFFICIENT USE OF IRRIGATION WATER

BY

* SRI R. SANKARA IYER, B.SC. (ENGG.)

1. Introduction.

In India, a large number of irrigation works, both major and minor, are being constructed so as to bring additional areas under irrigated farming and increase food production. In Madras State, our major rivers have almost been fully tapped, and special emphasis has been laid on constructing minor irrigation works for tapping surface flow and subsoil water. These works are being built at a huge cost, and there is always a heavy demand for extension of irrigation facilities to more areas. To make the best use of the available water and to reduce irrigation costs, it is imperatively necessary that wastage of irrigation water is reduced to the minimum and simultaneously, the yield of crops per acre is raised substantially.

2. Nature and extent of losses of irrigation water.

From observations on the Bari Doab canal in North India, it was found that the losses including evaporation, in utilisation of the irrigation water were as follows :—

	PER CENT.
(1) Loss in main and branch canals.	20
(2) Loss in major and minor distributaries.	6
(3) Loss in field channels . . .	21
(4) Loss due to irregular distribution, deep percolation, etc., in fields.	25

Thus, only 28 per cent of the water is actually utilised for plant growth.

The percentage of losses will however vary with the temperature and humidity in the region, nature and porosity of the soil and the underground water-table conditions. For channels in lighter soils in the upland areas of Madras

State, losses from 10 to 40 per cent are to be expected between the head of the irrigation system and the fields irrigated, in addition to the losses due to excessive irrigation on the fields.

3. Irrigation losses in channels.

The main and branch canals and distributaries are the property of the Government, while the field channels are the property of the farmers and have to be constructed and maintained by them. In areas where losses of water in main and branch canals will be excessive, the canals are usually lined with cement concrete during construction by the Government. In old irrigation system also, remodelling and lining of channels are taken up, wherever such works will yield additional benefits at reasonable cost.

The loss by seepage in field channels is large, and as there are a large number of such small channels, and they are owned by the ryots, it will not be practicable or economical to provide the conventional linings to minimise seepage in them. It has therefore become necessary to devise cheaper methods for lining field channels, and research work in this direction utilising alkathene, water-proof mud plaster, asphalt, etc., is being carried out in some of the irrigation research institutes, especially in North India. The council of Scientific and Industrial Research, New Delhi has published a pamphlet in November 1963 named "Water Proofing of Small Irrigation channels". The technique recommended consists in applying over the sides and bottom of the channels an intimate mixture of mud plaster containing a small percentage of bituminous mixture, in a thickness of about half an inch and allowing the plaster to dry.

4. Irrigation losses in fields.

The function of irrigation is to supply water to the root zone of plants in proper quantities

at proper times to meet the requirements for transpiration and the unavoidable evaporation from the soil surface, which are collectively termed evapotranspiration and represent the consumptive use of water by the crop.

For the efficient use of water, sound land and water management are essential. To provide necessary moisture for growth of plants, the soil moisture in the root zone should be above the wilting point. The maximum amount of moisture that the soil can retain is the field capacity, and therefore the aim of irrigation is to saturate the soil in the root zone to field capacity. It is generally found that for healthy and vigorous growth of plants, irrigation water has to be applied when 50 per cent of the available moisture in the root zone has been utilised. If a depth of water greater than that required to saturate the soil in the root zone up to field capacity is applied and if the substratum is porous, the excess will go down into the subsoil water table. If the soil has an impervious substratum, excessive application of water is likely to create water logging and salinity and alkalinity problems. Too much water also leaches away plant nutrients and reduces soil aeration.

The irrigation water requirements vary from region to region, depending on the soil characteristics, nature of crop and climatological factors. For best results, irrigation water has to be applied to the proper depth at as long an interval as possible without detriment to plant growth. If the field is irrigated more frequently than is necessary, evaporation losses and the cost of irrigation increase. Even in the case of paddy which is grown under swampy conditions, it is observed from experiments conducted in Madras State that maximum production is achieved with a constant depth of standing water of two inches on the field. The water requirements of crops are not also uniform throughout their period of growth. There are certain critical periods during the life of a plant, when any shortage of water considerably affects the growth of the crop and its yield adversely. For some crops it is also found necessary to provide a short period of stress in its early stage so that a deeper and extensive root system

is developed. Under the scheme of research on minor irrigation and water use, research is being conducted at various research institutes in India, to study the optimum requirements of irrigation water and other aspects of water use.

In advanced countries, tensiometers and gypsum blocks are used by many of the farmers to determine the soil moisture content and to regulate irrigation needs. In India, however, such instruments cannot be brought to extensive use at present, and hence, certain simple field moisture tests have been evolved, so that the farmer can judge when he should irrigate his land. For this, the soil sample is taken from the depth where most of the plant roots are growing, by means of a shovel or soil auger, and is squeezed in the hand. For example, in the case of a medium textured soil, if it is too dry to form a ball and is powdery, irrigation has already been delayed; if it is moist enough to form a ball but crumbles when rolled in the hand, it is time to irrigate; and if it forms a durable ball and is somewhat plastic, the time is not yet ripe for irrigation.

5. Efficient cultural practices.

Water is most efficiently utilised when the maximum production is achieved for every acre inch of water supplied on the field. The agricultural aspects deserving special mention are :

(1) Soil fertility—The transpiration ratio, that is, the ratio of the amount of water consumed to the amount of dry matter produced by the crop, generally decreases, with the higher fertility status of the soil. Balanced application of required quantities of different nutrients results in efficient use of water due to increased production even though the irrigation requirements per acre will increase with greater application of fertilisers.

(2) Levelling or grading of the field—For achieving as much uniformity in application of water as possible, fields must be correctly levelled or graded to even slopes and surface smoothened. In an uneven land, there will be accumulation of water in depressions, leading to waste and at the same time, more water is required for irrigation, so as to wet the higher

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parts. The stand of the crop and its growth will be adversely affected by such non-uniform irrigation in spite of there being a larger application of water.

(3) Weed control—Weeds consume a large quantity of water, the loss varying from 15 to 50 per cent under different conditions. The growth of weeds results in quick depletion of soil moisture and in reduction of space required for the normal growth of the crop.

(4) Optimum stand—Plants should be uniformly spread over the field and should be at the optimum spacing for the particular crop, so as to ensure healthy growth and reduction of evaporation losses and of weed growth.

(5) Mulching—Loss of water by evaporation from the soil surface can be considerably reduced by proper mulching operations. Adoption of suitable soil and water conservation measures also will help reduce the quantity of irrigation water to be supplied to fields.

6. Irrigation methods.

The practice widely prevalent with Indian farmers is to apply water to the fields by wild flooding. This causes non-uniform wetting of the field and consequent loss of water. The most economical condition in watering a field is when water is applied simultaneously to an equal depth throughout the field. As this not practically feasible, different irrigation methods have been evolved so as to ensure as much uniformity in wetting the field as possible, and to suit different soil characteristics, land slope, nature of crop, etc., conservation irrigation on the farm involves the use of irrigation and cropping methods that best fit the particular soil, slope, crop and water supply.

The best method of irrigation is an improvement over the wild flooding system and is the method mostly commonly adopted. The border strip method is specially adopted in the case of moderately porous soil and smooth land sloping 2 or 3 inches in hundred feet, when a fairly large irrigation stream is available. Furrow irrigation is commonly adopted for cultivated row crops and for orchards and vegetables. It has been estimated that there is a saving of water by 10 to 15 per cent in the furrow system, as compared to the bed system. For good

results, the size of the border strips or, the length of furrows, have to be adjusted to suit the size of irrigation stream, nature of soil, land slopes, etc.

The field channels have to be maintained in good condition by the farmers. Ryots usually handle water for irrigation by diverting from the field channel by temporary earthen bunds. This procedure does not permit of sufficient control and regulation of supplies, and it is necessary to introduce progressively the use of farm irrigation structures like check-gates, turn-out boxes, siphon tubes, etc., which not only help in quicker and better irrigation but also reduce labour costs. Also, a suitable turn system of irrigation for the area under a canal outlet, if properly evolved and implemented, will ensure judicious and equitable distribution of water among the beneficiaries.

7. Conclusion.

There is large scope of economy in the use of water in existing irrigation systems, and if this economy is effected, it is possible to extend irrigation facilities to a sizeable additional area with the available water. Even in canal irrigated areas, supplemental irrigation with subsoil water tapped by open wells, tube wells or filter point wells, wherever feasible, will not only reduce the consumption of water from the canal system, but also help in early raising of seedbeds, overcoming temporary shortages of water and raising an additional crop on the land using underground water.

The problem of achieving the desired economy and increased efficiency in the use of irrigation water, has to be tackled both from the engineering and agronomic angles. While the necessary engineering works upto the canal outlet can be executed without difficulty by the Government, the conservation problems beyond the outlet, that is, in the field channels and the fields, have to be understood and solved by the farmers, who have to be given the necessary education, and technical guidance in this direction. Irrigation and agricultural officers and subordinates should be conversant with these problems, and be in a position to render necessary advice and assistance to the farmers.

A NOTE ON DESIGN OF HEAD SLUICES AND CANAL REGULATORS

BY

* SRI S. SHANMUGAM, B.E., M.SC.

Introduction.

The sliding type gate is the usual control structure in irrigation channels. At present, for designing the ventway the value of C_d in the equation $q = C_d A \sqrt{H}$ is variously assumed from 5 to 6 and the ventway is arrived at. The effect of submergence due to the downstream water level is not taken into considerations in such designs. By such an omission there is a likelihood of the design discharge being not realised. In this note a design procedure giving due consideration to the submergence derived after experiments is detailed.

Discussion and calculations.

The observations from the scale model were put down in the form of standard plots of upstream depth Y_1 Vs downstream depth Y_2 for different discharges, each chart being for one gate opening one such is shown in Fig. 1.

These plots bring out an interesting fact. For a given gate opening a certain upstream depth is required to drive a certain discharge, e.g., to drive a discharge of 0.944 c.f.t. through a gate opening of 0.15 feet. This depth is independent of downstream depth upto 0.248' after which it rises. In other words the ventway gets submerged for the given condition when the downstream depth is 0.248'. This

is called the modular limit. For tailwater depths lower than this the ventway is a free discharging one and the discharge depends on only the upstream depth and for depths higher than this the ventway is called submerged and the discharge depends both on the upstream and downstream depths. From these plots corresponding values of discharge, gate opening, upstream depth and downstream depth were taken for purposes of further calculations.

Free condition.

For free condition the coefficient C_d in the equation $Q = C_d B G \sqrt{2gY_1}$ Where B —Width of the channel, G —gate opening

was computed and a plot of C_d vs $\frac{Y_1}{G}$ is given in figure 2.

This figure can be used straight away for designing a sluice which is discharging freely into tail channel, the floor being plain and horizontal.

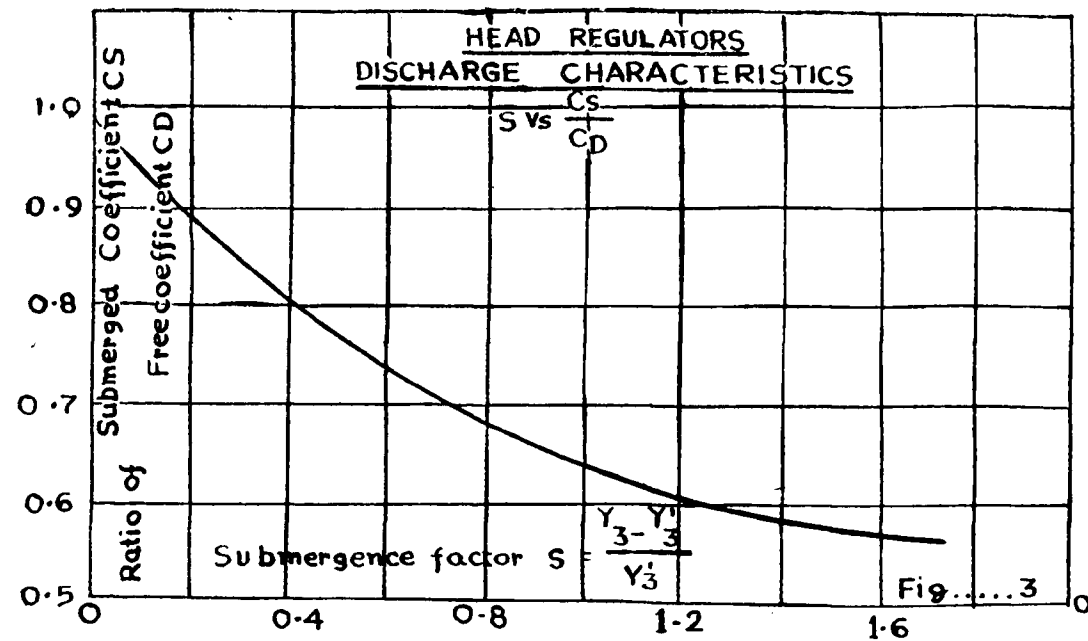
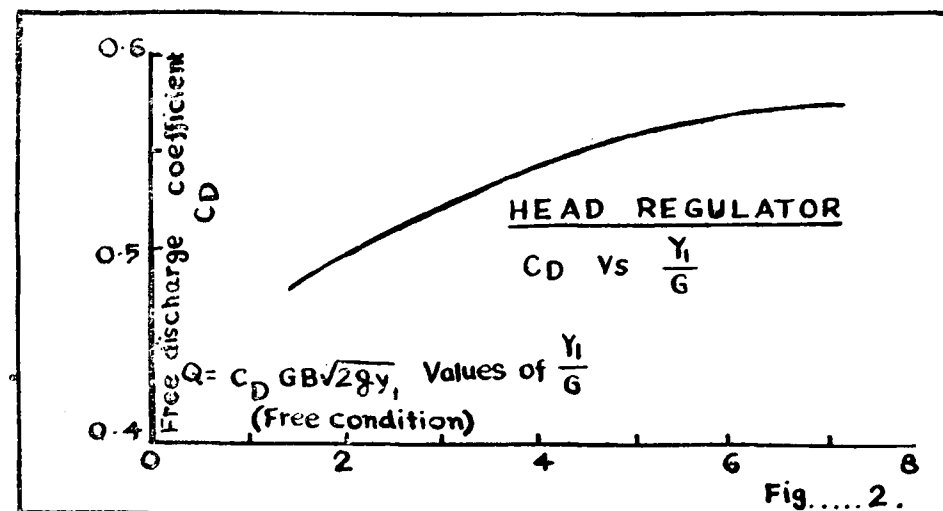
Drowned condition :

For drowned condition also the coefficient C_s was obtained taking the driving head to be the front water depth Y_1

i.e. $Q = C_s B G \sqrt{2gY_1}$.

To indicate the degree of submergence of the jump a factor called the submergence factor "S" is used. This factor for the notations used in this paper is $Y_2 - Y'_2 / Y'_2$ where Y_2 is the actual tail water depth, Y'_2 is sequent (or conjugate) depth as calculated by using the usual hydraulic jump formula taking the depth and velocity of the jet issuing from the gate opening as those of the incoming jet for the jump (normally termed as V_1 and d_1). The plot of C_s/C_d Vs. 'S' is shown in figure 3.

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It can be seen that a mean curve can be drawn for the plot C_s/C_D vs. S . A plot of Y_3/Y_1 vs. S with Y_1/G as the variable parameter is indicated in figure 4.

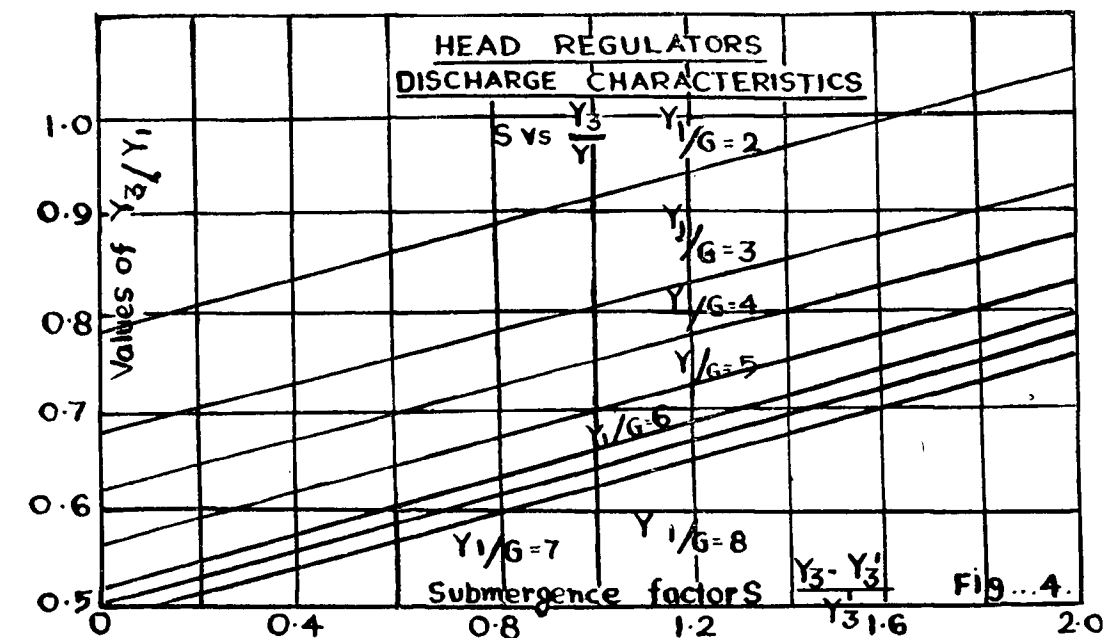
Combining these two figures along with the graph C_D vs Y_1/G for free condition a design chart has been prepared and is shown in figure 5. The method of using this design chart is indicated by the worked out example below.

Example.

It is required to find the ventway for a discharge of 30 Cfs/Ft. run in a canal given $Y_1 = 8'$ and $Y_3 = 7'$ Assume a gate opening of 4'0".

$$Y_3/Y_1 = 0.875 \quad Y_1/G = 2.00$$

Entering the design chart (fig. 5) with these two values we obtain $S = 0.70$ and for $Y_1/G = 2.0$, $C_D = 0.51$



For this value of S , $C_s/C_D = 0.727$

So that $C_s = 0.727 \times 0.51 = 0.3708$

Discharge $= C_s G \sqrt{2g Y_1}$
 $= 0.3708 \times 4.0 \times 8.02 \times \sqrt{8}$
 $= 34.4$ Cfs.

Assume a gate opening of 3'5"

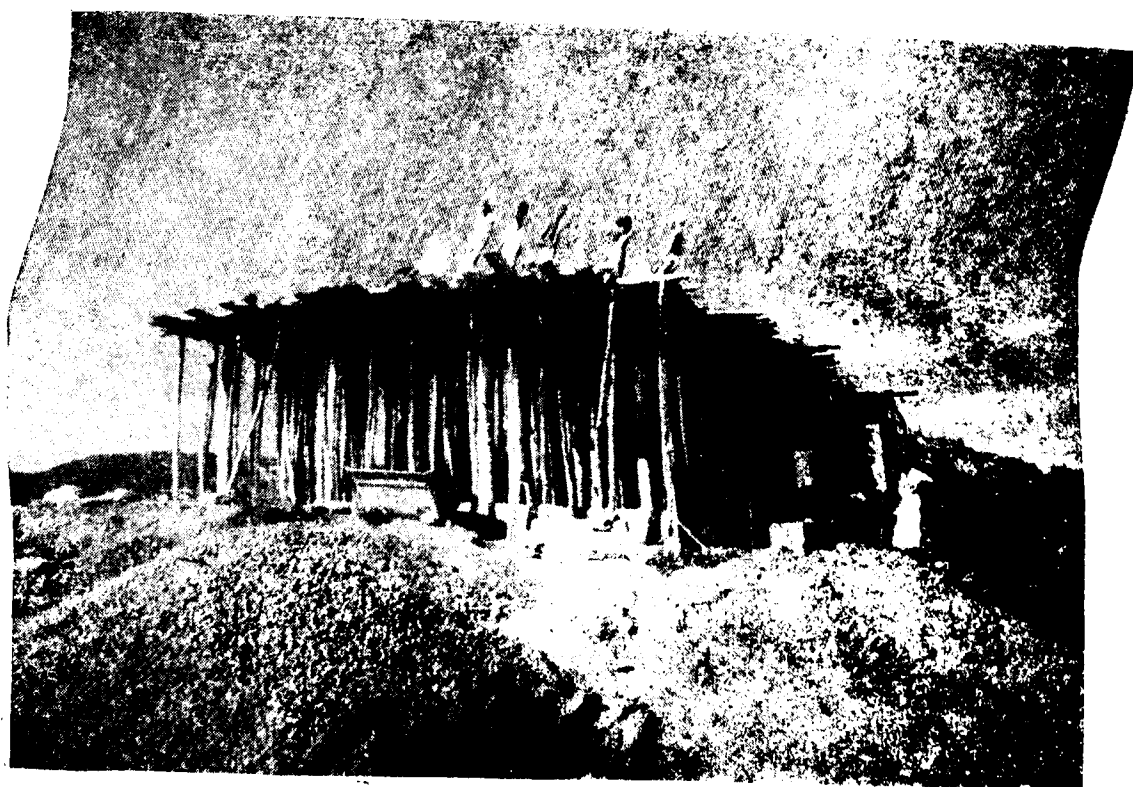
$$Y_1/G = 2.28 \quad Y_3/Y_1 = 0.875$$

$C_D = 0.518$, $S = 0.88$ $C_s/C_D = 0.665$

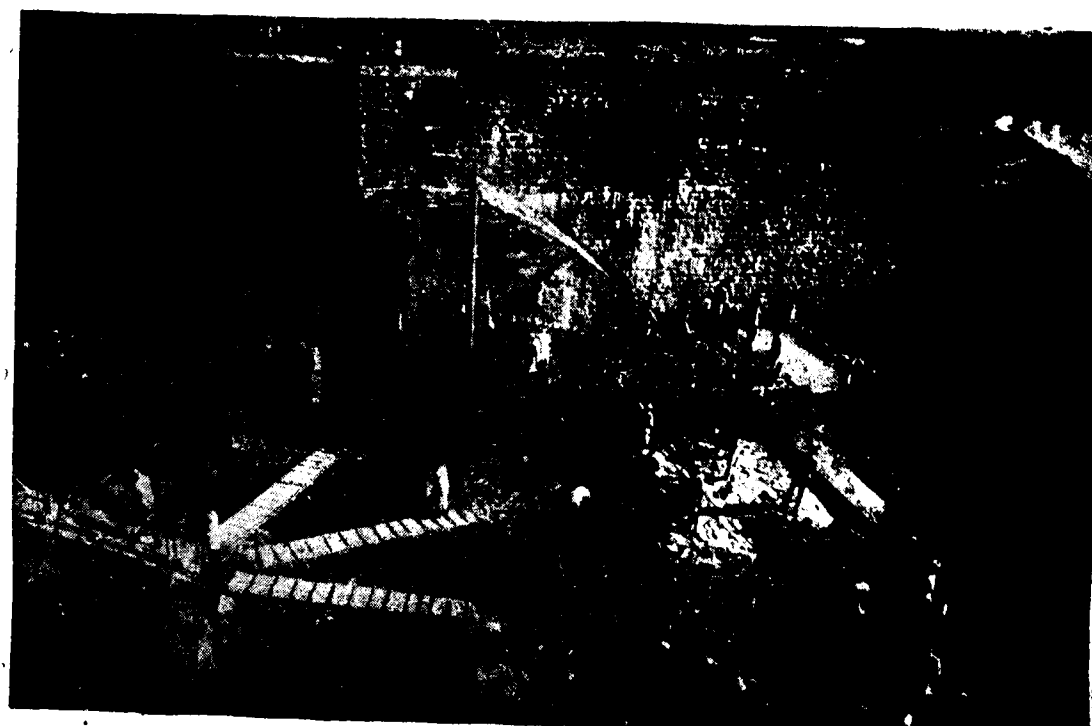
$\therefore C_s = 0.665 \times 0.518 = 0.344$

Discharge $= 0.344 \times 3.5 \times 8.02 \times 8 = 27.4$ Cfs.

The ventway to be provided for, should be between 3'5" and 4'0". As it is customary to provide integral value for ventway a vent of 4 feet high can be provided.



Pavilion on right flank—Centering work in progress.



Masonry Dam—Masonry in spillway portion.

NOTE ON DESIGN ASPECT OF REINFORCEMENT AROUND GALLERY

BY

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1. Introduction.

1.1. Normally, the galleries are left in the structure of the dam after completion and they may run longitudinally, transversely or on a slope for various purposes such as drainage of the interior of the dam, inspection, providing space for drilling and grouting the foundation, and serving as access to and room for mechanical equipment used in the operation of gates in the spillway and outlet works, etc.

1.2. Different size of these formed openings are adopted in practice. Horse-shoe type gallery or rectangular galleries with suitable fillets of size varying from $6' \times 9'$ to $4' \times 7'$ are in practice depending upon the purpose of the gallery. The standard size of the gallery that is adopted in our State is exhibited in figure 1.

1.3. Provision of an opening in the dam always cause disturbance in the path and magnitude of stresses in the structure resulting in redistribution of stresses and concentration of unit stresses at the sides of the opening.

1.4. The reinforcements are to be provided sufficiently to take up the induced stress concentration at the immediate region of the faces of the openings. An attempt has been made in this article to describe the design procedures for provision of reinforcement for the gallery to take up the above mentioned stresses.

2.0 Various methods of computing the stresses around gallery.

2.1. The stresses around gallery are computed by the following two methods namely :—

(1) By mathematical treatment assuming an equivalent circular opening.

(2) By experimental co-efficients deduced from Photo elastic interferometer study for a standard gallery $5' \times 7'$ size, conducted by U.S.B.R.

2.2. Mathematical Treatment.

If a thin plate, subject to a uniform unidirectional primary stress is pierced by a hole, concentrations of unit stresses will occur at the sides of the hole, parallel to the primary stress and a transverse stress of opposite sign, will occur at the opposite side.

For an elliptical hole laid flat to the direction of the primary stress, as exhibited in figure 1 (A), the following equations are given :—

$$P_{o-d} = P_1 \left(1 + 2 \frac{a}{b} \right) \quad \dots \quad (1)$$

$$\text{and } P_{o-f} = -P_1 \quad \dots \quad (2)$$

where—

P_{o-d} is the unit stress at c or d

P_{o-f} is the unit stress at e or f

P_1 is the unit stress if there was no hole and $2a$ and $2b$ are diameters of the ellipse, normal and parallel, respectively to the direction of the stress.

For an inverted ellipse as in figure 2, the equation (1)

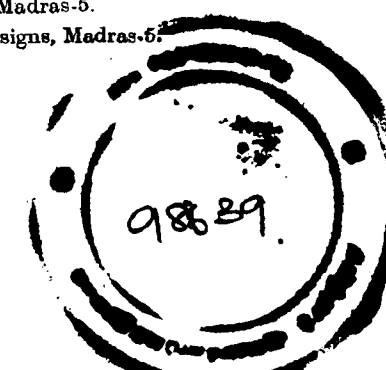
$$\text{becomes } P_{o-d} = P_1 \left(1 + 2 \frac{b}{a} \right) \quad \dots \quad (3)$$

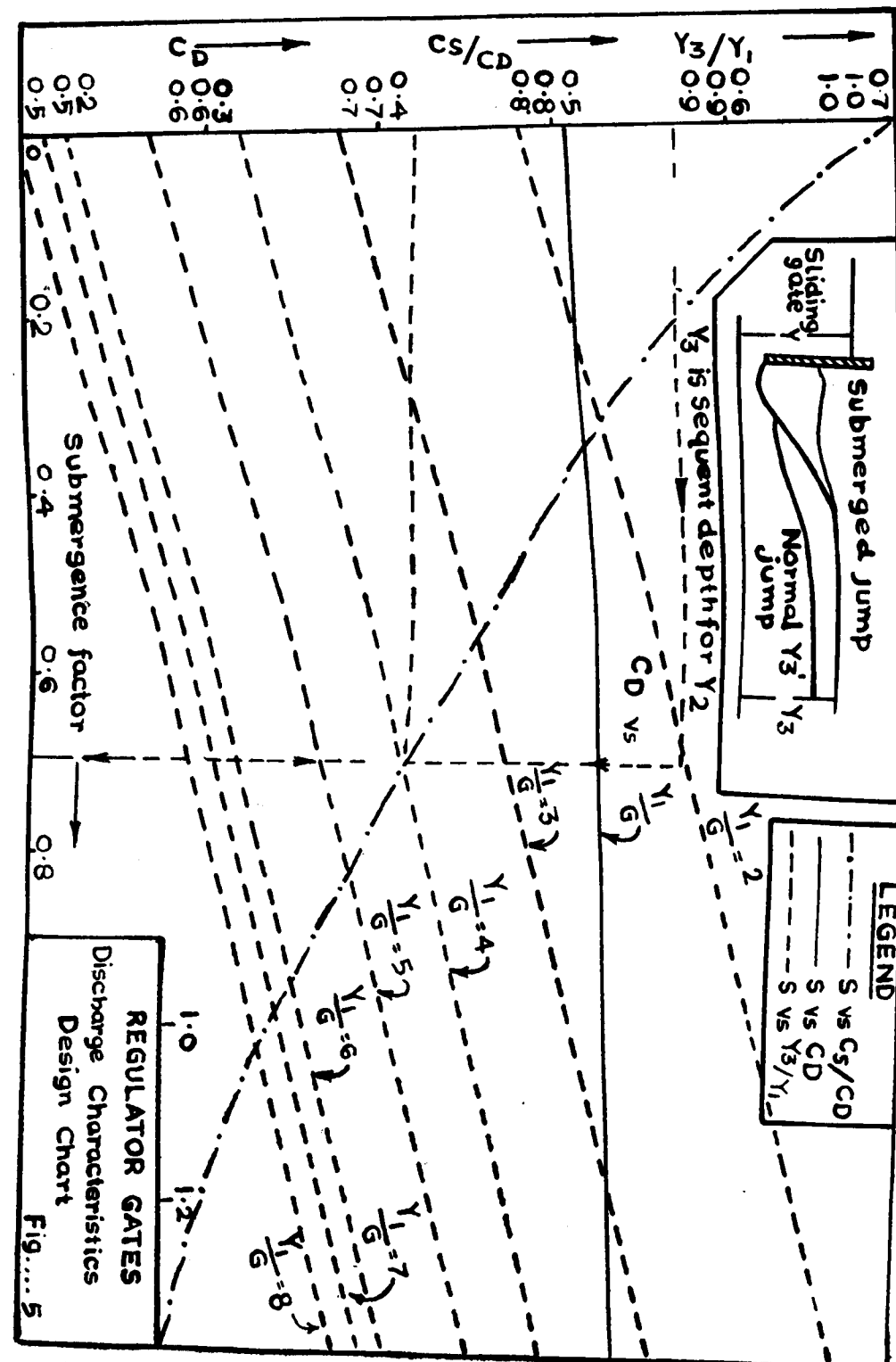
If the hole is circular, a and b are equal and $P_{o-d} = 3P_1$

2.3. These stress concentrations are purely local and reduce rapidly with increasing distance

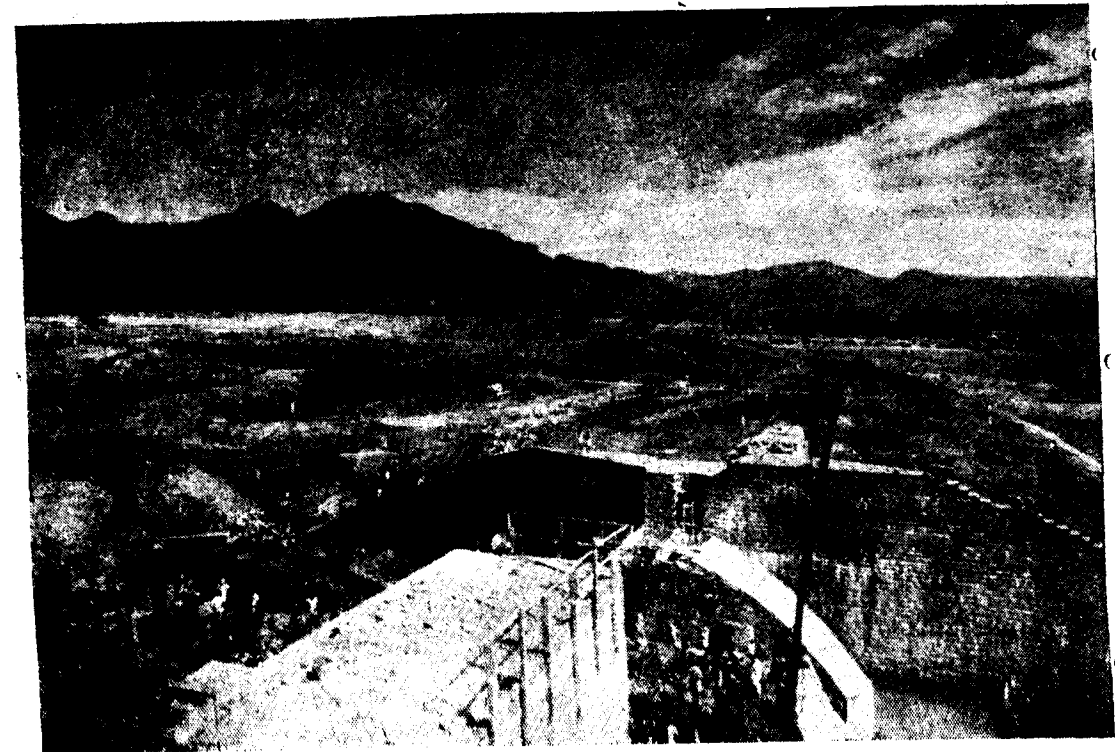
* Superintending Engineer (Designs), Madras-5.

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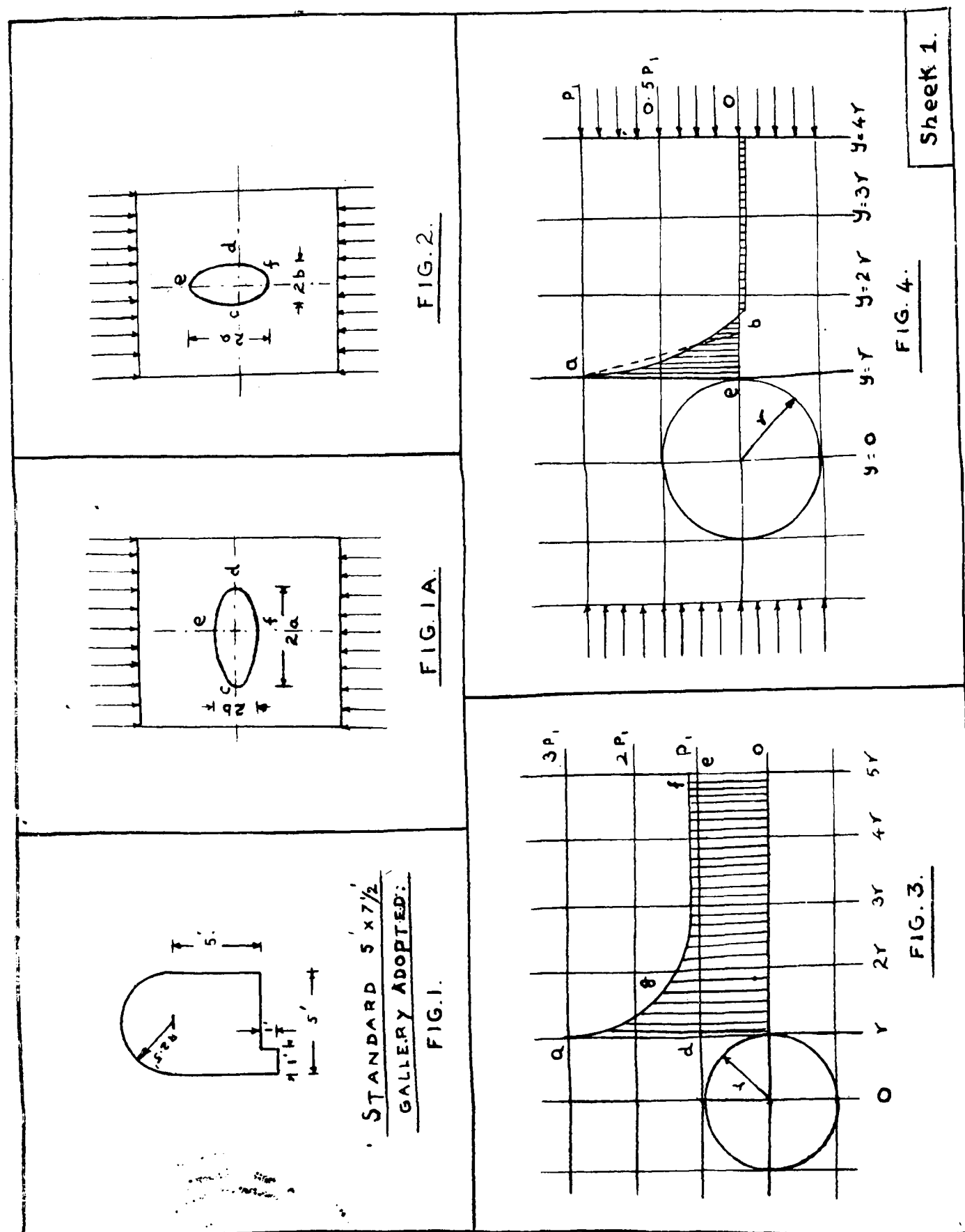
GOMUKHINADHI PROJECT



Panoramic view from left flank



Masonry Dam from right flank.



Sheet 1.

from the face of the hole. The distribution of the side stress for a circular hole in an infinite plate is given by the equation.

$$P_x = \frac{1}{2} P_1 \left(2 + \frac{r^2}{x^2} + \frac{3r^4}{x^4} \right) \dots (4)$$

where r is the radius of the hole and x is the distance from the centre of the hole. The value of p_x remains greater than p_1 to an infinite distance but beyond $x=2r$, the difference is small.

Equation 4 is plotted in figure 3. The area between the stress curve a—f and the line d—e must equal $p_1 r$.

The distribution of the transverse stress for a circular hole in an infinite plate is given by the equation.

$$P_y = 0.5 P_1 \left(\frac{r^2}{y^2} - 3 \frac{r^4}{y^4} \right) \dots (5)$$

where y is the vertical distance from the centre of the circle. The equation (5) is plotted in figure 4. The stress reverses at about $y=1.7r$.

If point b represents $y=1.5r$, the line a—b is a good practical approximation of the stress diagram. For this case, the stress at the surface is of opposite sign from the primary stress.

If the hole is rectangular sharp local stresses occur at the corners.

3.0. Longitudinal openings in dams.

3.1. Consider a circular gallery 'g' as exhibited in figure 5. An enlargement of a rectangular block cut from the sluice of the gravity dam is exhibited in figure 5 (a). This containing hole, is oriented with the directions of principal stresses at the position of the centre of the hole if the hole did not exist. The systems of transverse principal stresses are indicated. If these stresses are assumed to be uniform for indefinite distances away from the opening, two sets of opposing stresses similar to the theoretical stresses in the thin plate as discussed early, will result.

Thus unit compressions at c and d will be approximately.

$$P_{c-d} = 3 p_1 - p_2 \dots (6)$$

where p_1 and p_2 represent the first and second principal stresses.

At e and f, the stress will be

$$P_{e-f} = 3 p_2 - p_1 \dots (7)$$

which becomes tension where p_1 is greater than $3p_2$.

Equation (6) and (7) are merely convenient approximations of the true stress condition and apply approximately only to circular holes at a considerable distance from a boundary.

3.2. Openings parallel to the axis of the dam are of frequent occurrence and are usually rectangular or horse-shoe shaped rather than circular. They are usually located relatively close to the face of the dam or to the foundation. Principal stresses across them are not constant. Hence accurate, exact and absolute prediction of the stress concentrations around them cannot possibly be produced by any known equation.

3.3. Hence, in order to make rectangular and horse-shoe shaped holes also, susceptible to mathematical analysis like circular or elliptical holes, the following standard assumptions are generally in practice, whereby equivalent circular or elliptical form is assumed for such openings.

(1) Equivalent circle method wherein the total area of the gallery is equated to a circle.

(2) Enveloping circle method wherein the standard gallery is considered to be equivalent to a circle, which would just circumscribe the gallery.

(3) The rectangular or the horse-shoe shaped gallery may also be treated as equivalent to an ellipse. In the first case, the total area of the standard 5' x 7'—6" gallery of area 35.82 square feet is considered equivalent to a circle of radius 3.4' whose area is also equal to 35.82 square feet. Then the calculations for

stresses, etc. will be proceeded on the basis of a circular hole, for which equations are available.

In the second case, as seen in fig. 6, a standard 5'x7'6" gallery is enveloped by a circle of 4'5" radius and this assumption has been so far practiced in all the P.W.D. projects in our State. Obviously for the reason that the circle is of greater radius which would give more reinforcement for the same gallery, being on safer side.

In the third case, the horse-shoe shaped or rectangular gallery is treated as equivalent to an ellipse in line with the recommendations made in "Engineering for Dams" Vol. II by Creager and Justin P. 423.

The height and breadth of the gallery may be assumed as 2a and 2b of an ellipse and the available formulae for an elliptical opening may then be adopted in this case.

Of the above three assumptions, the enveloping circle method may be adopted for Longitudinal openings, as it would give the maximum value of reinforcement of all the three assumptions and the same is to be finally averaged with the values obtained by the experimental method of photo elastic studies.

3.4. Conditions for the analysis of stresses.

It is generally known that the maximum stresses occur near the heel of the dam under reservoir empty condition and near the toe of the dam under reservoir full with uplift condition and at the middle portion under the reservoir full without uplift condition. The longitudinal galleries are invariably located close to the heel of the dam and hence the critical stresses may either occur in empty condition or in the reservoir full without uplift conditions. However the approximate location of the gallery with respect to the base of the dam would decide any two possible conditions under which critical stresses are likely to occur. Generally for drainage galleries located near the upstream face of the dam, stresses are analysed for the separate conditions, namely, reservoir empty conditions and reservoir full without uplift

conditions and reinforcements are calculated for both the conditions and larger value of the two is taken into account.

3.5. Transverse opening in dams.

The transverse openings in the dams are studied by considering longitudinal sluices of the dam parallel to the first principal stresses. Let a-b in Fig. 7 represent a circular opening and let a view h-i of a portion of the face containing the opening be represented by fig. 8. The opening will be elliptical in this view considering a slab of unit thickness while p_1 is the only stress acting as shown, stresses at c and d are computed from equation (1) and at e and f from equation (2).

At interior points, the slab may be taken parallel to the first principal stress the second principal stress acting normal to the face of the slab introduces an additional variable. Because the critical stresses occur at the unloaded faces, the effect of these stresses may be ignored in the approximate solution.

3.6. Conditions for the analysis of stress.

Since the transverse spacing usually run through the entire reach of the section, stresses will have to be calculated for all the three conditions, namely—

- (1) Reservoir empty,
- (2) Reservoir full without uplift and
- (3) Reservoir full with uplift and at selected points along the section, stresses are calculated for all the three conditions and the reinforcements are calculated for the maximum stress obtaining at each point.

3.7. Assumption of equivalent section.

In the transverse openings, only sections parallel to the direction of the first principal stress are considered while the effect of the second principal stresses are ignored. In this process the height of the opening will get multiplied by " $\sec \phi$ " where ϕ is the angle of direction of the first principal stress, while the width remaining the same. Therefore the gallery

section is considered equivalent to an elliptical opening with $7\frac{1}{2}' \sec \phi = 2a$ and $5' = 2b$ as for a standard gallery and the usual formulae are applied for evaluation of the necessary reinforcements at selected points along the section.

3.8. Reinforcement for tension.

The deformation of steel under usual unit stresses exceeds the tensile strain limit of concrete and hence adding steel to take tension will not eliminate cracking but it should control cracking and prevent its indefinite spreading. The steel should be concentrated as close to the face of the opening as convenient. Generally the reinforcements are placed at 6" distance from the face of the opening as adopted in our projects.

The total tension for which the reinforcements are to be provided may be calculated from the following formula, based on the assumption that the tension reduced to zero in a distance equal to one-fourth the width of the opening.

$$T = 0.25r P_{o-f} \dots \dots \dots (8)$$

where r is the radius of the circle or half the width of ellipse.

(Note. $P_{o-f} = 3P_2 - P_1$ in the case of longitudinal openings where P_1 and P_2 are the two principal stresses.

$P_{o-f} = -P_1$ in the case of transverse openings where P_1 is the major principal stress.

Hence area of reinforcement.

$$= \frac{0.25r P_{o-f}}{f_s} \text{ Sq. in. } \dots \dots \dots (9)$$

where $f_s = 16,000$ lb. per sq. in., allowable stress in steel.

3.9. Reinforcement for compression.

The compressive stresses at the sides of circular openings in dam are often ignored since they are always less than the permissible stress. However if the principal stresses are high in the case of a high dam and the permissible stress in the masonry is also very limited, reinforcement for compression will then be necessary. If it is assumed that the steel used for this purpose cannot be strained beyond the permissible strain limit for concrete in compres-

sion, the working unit stress cannot exceed ' n ' times the working stress of the concrete, where ' n ' is the ratio of the modulus of elasticity for steel to that of concrete.

Experience indicates that shrinkage and "flow" of concrete appreciably increase the stress in the steel and current specification allow full stress value in longitudinal bars in columns. It may therefore be assumed permissible to use normal working stresses in compressive reinforcement around holes in dams.

In fig. 3 the total excess compression is represented by the area between the curve a-g and the line d-e, its total amount is $T = rp_1 \dots$ (10) ' r ' is the radius of circular hole. In the case of an ellipse, ' r ' is taken as half width transverse to P_1 . Most of the excess compression is concentrated near the face of the hole.

An appreciable reduction in the amount of compression reinforcement is effected if an overstress is permitted in the concrete adjacent to the opening. Generally allowance of 50 Per cent over stress in compression is in practice.

3.10. Computation of principal stresses.

Principal stresses will have to be computed at a point or points where the centre of the opening lies. Regarding the standard 5'x7'6" gallery, the centre is assumed to be located at a distance 3'75" above the base as shown in Fig. 9.

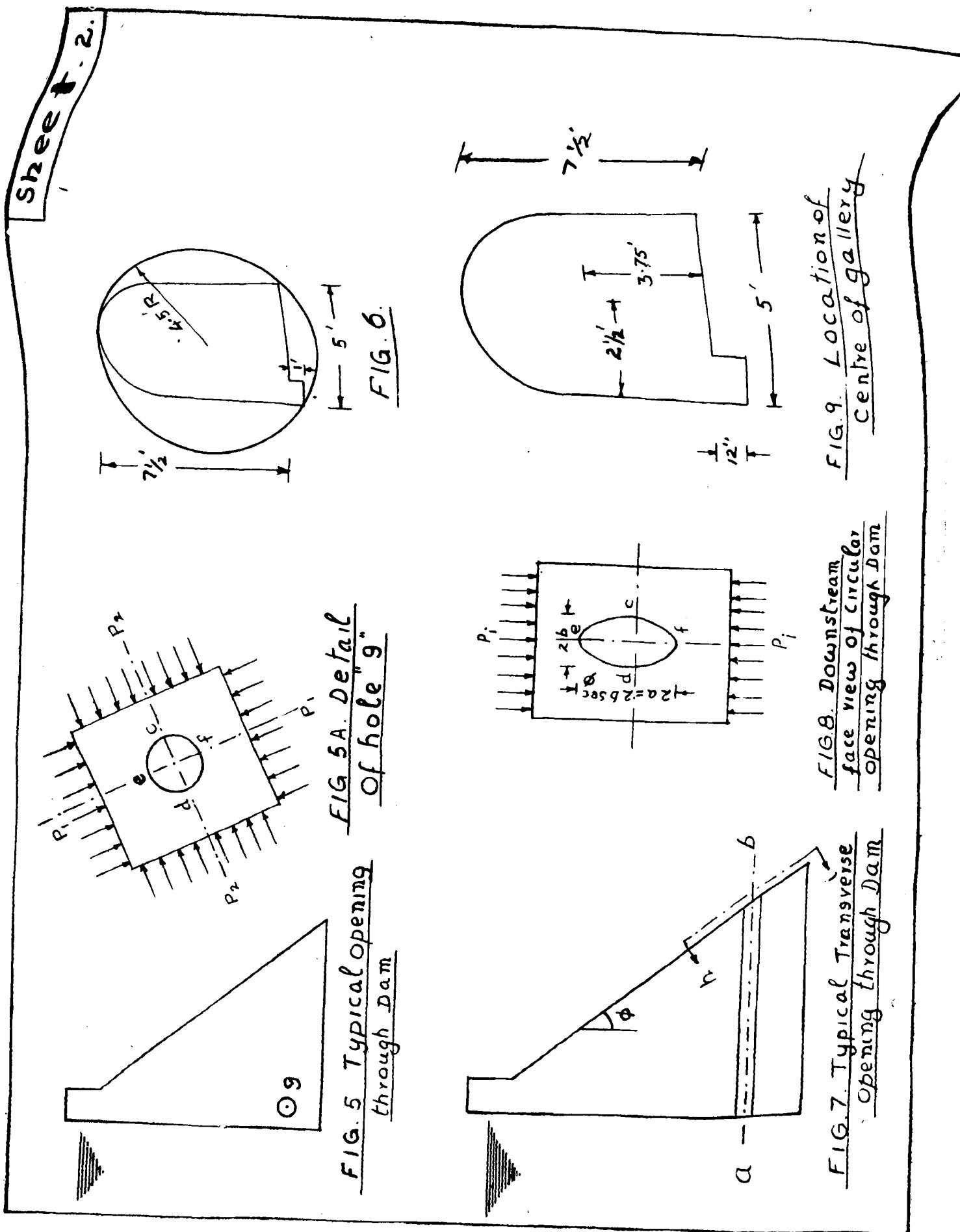
As already discussed, depending upon the necessity, the stresses may have to be computed for all the three conditions or two only as the case may be.

Before actually venturing to enunciate the formulae, it is considered necessary to dwell on the significance of the symbols employed therein.

Refer Fig. 10.

Any normal force on a Plane is denoted by an appropriate suffix to α (Alpha)

i.e., α_z means stress on plane, Z i.e., usual toe and heel stresses.



Any shear is denoted by an appropriate suffix to, i.e., T_{zy} means shear on horizontal or vertical plane, Any additional suffix to the above merely indicates the position.

$$\begin{aligned}\alpha_{zu} &= \alpha_z & \text{at upstream end} \\ \alpha_{yd} &= \alpha_y & \text{at down stream end} \\ T_{zyu} &= T_{zy} & \text{at upstream end and so on.}\end{aligned}$$

The plane of an axis is that which contains the X and Z axis.

3. II. Formula used.

(1) The normal stresses on horizontal plane at the downstream and upstream ends of base section are known by stability analysis.

These are called α_{ZD} and α_{ZU} and assuming a linear variation α_Z can be computed at any point 'y' from the downstream end of the base section.

$$\alpha_Z = a + by \quad \dots \quad (11) \quad \text{where } a = \alpha_{ZD} \text{ and } b = (\alpha_{ZU} - \alpha_{ZD}).$$

(2) The shear stress is assumed to vary parabolically. A general equation to give the value of T_{zy} at any point at a distance 'y' from the downstream end of the base is given as, $\alpha T_{zy} = a_1 + b_1 y + c_1 y^2 \dots (12)$ which is a parabolic equation.

where

$$a_1 = T_{zy} D = (\alpha_{ZD} - P^1) \tan \phi D \quad (13)$$

$$b_1 = -\frac{1}{T} \left(\frac{6 \sum V}{T} + 2 T_{zyu} + 4 T_{zyD} \right) \quad (14)$$

$$c_1 = \frac{1}{T^2} \left(\frac{6 \sum V}{T} + 3 T_{zyu} + 3 T_{zyD} \right) \quad (15)$$

and P^1 = tail water pressure

V = Resultant horizontal force above the section

$\tan \phi D$ = Downstream slope of the dam.

$\tan \phi u$ = Upstream slope of the dam.

(3) Normal stress on vertical plane α_y

$$\alpha_y = a_2 + b_2 y + c_2 y^2 + d_2 y^3 \quad \dots \quad (16)$$

$$\text{where } a_2 = \alpha_y D = a_1 \tan \phi D + P^1 \quad \dots \quad (17)$$

$$b_2 = b_1 \tan \phi D + \frac{\sigma_1}{\sigma_2} a_1 \quad \dots \quad (18)$$

$$c_2 = c_1 \tan \phi D + \frac{1}{2} \frac{\sigma_1}{\sigma_2} b_1 \quad \dots \quad (19)$$

$$d_2 = \frac{1}{6} \frac{\sigma_1}{\sigma_2} c_1 \quad \dots \quad (20)$$

(4) Magnitude of principal stresses

$$\propto P_1 \text{ and } \propto P_2$$

$$\propto P_1 = \frac{\alpha_z + \alpha_y}{2} \pm \sqrt{\left(\frac{\alpha_z - \alpha_y}{2} \right)^2 + (T_{zy})^2} \quad 21$$

If $(\alpha_z - \alpha_y) > 0$, use positive

If $(\alpha_z - \alpha_y) < 0$, use negative and

alternative sign gives $\propto P_2$, which is perpendicular to $\propto P_1$.

(5) Direction of principal stress ϕP_1

$$\tan 2 \phi P_1 = \frac{-T_{zy}}{\left(\frac{\alpha_z - \alpha_y}{2} \right)} \quad \dots \quad (22)$$

by applying these formulae and adopting other conditions already mentioned, the total tensile and excessive compressive forces can be evaluated around the gallery.

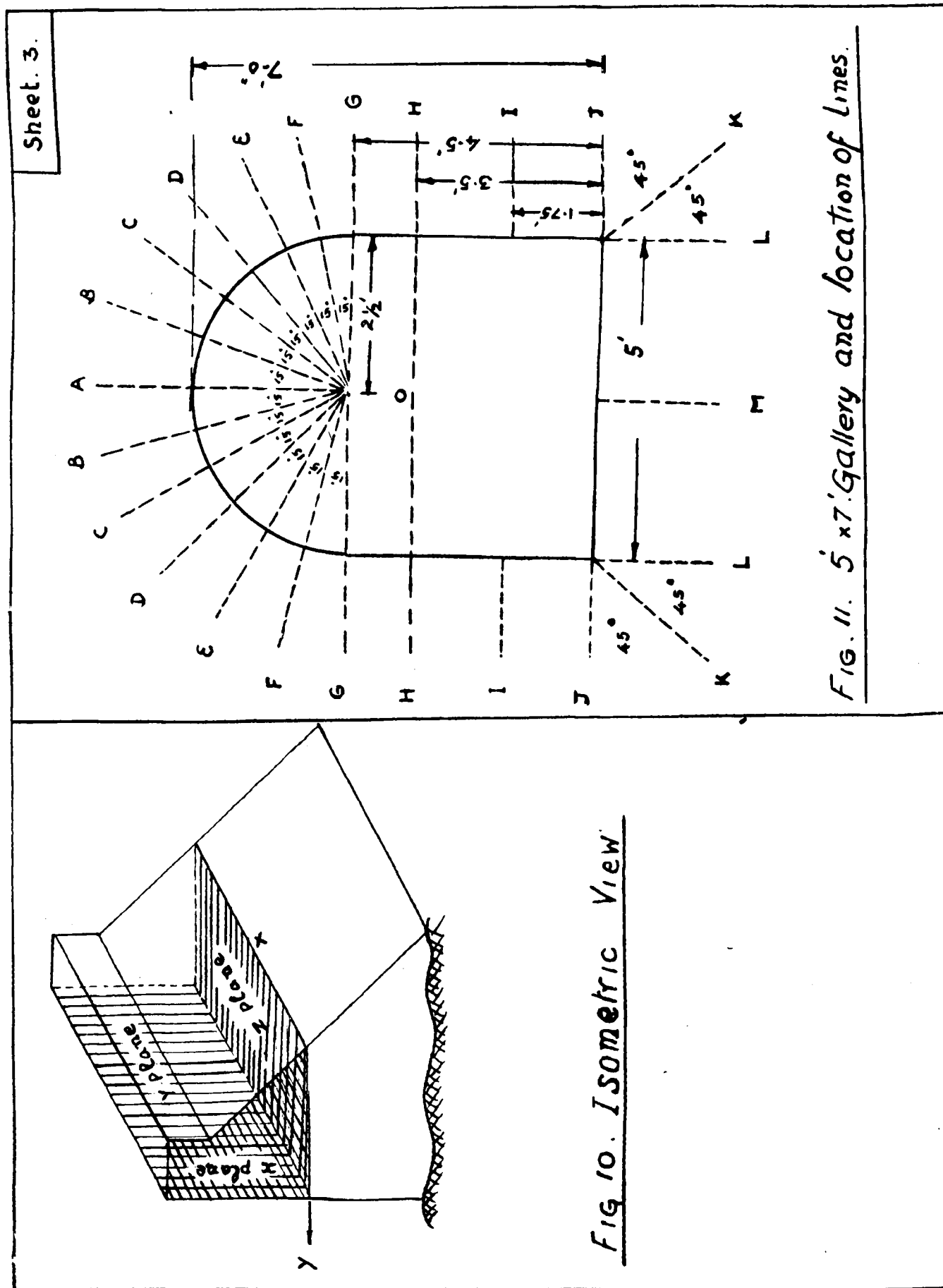
4.0. By empirical co-efficient deduced from photo-elastic interferometer study for a standard gallery $5' \times 7'$ size.

The U.S.B.R. has conducted photo elastic interferometer study to determine the distribution of stresses around a standard gallery of size $5' \times 7'$ and stress co-efficient to calculate the induced stresses (compression or tension) in the material and the gallery opening.

The stress co-efficients are available at several points on various lines, radiating from the gallery as depicted in Fig. 11. These co-efficients are expressed in terms of unit load acting on the external boundaries of the model.

The gallery with its centre at point O was considered to be situated in an infinite elastic medium and subjected to a uniform stress field composed of the stresses initially present at point O.

These stresses immediately surrounding the gallery will not be in error if the gallery is located at least $1\frac{1}{2}$ times the width from a finite boundary.



It is observed that the maximum compression occurs along 'J' line under reservoir full conditions and maximum tension occurs along M line under reservoir empty conditions.

These stresses are—

α_n = Stress normal to the designated lines.

α_t = Stress tangential to designated lines.

T_{nt} = Shear stress along designated lines.

Method of calculation.

Stress co-efficients are available for unit stress of α_z , α_y and T_{zy} , and multiplying the corresponding co-efficients by the values of α_z , α_y and T_{zy} , stresses normal and Tangential to the designated lines and the shear stress along that lines are obtained.

These stresses can be computed at points 0, 0.5, 1, 2, 3 and 4 ft. away from the face of the gallery along any particular line. After calculating these stresses, maximum principal stresses at each point are calculated with the result that the total tension area excessive compression area along a particular line can be determined and suitable reinforcement provided accordingly.

Following is the extract of the co-efficients, furnished by U.S.B.R. in "stress around a gallery determined by the photo elastic interferometer—No. 12" for J and M lines.

M. line Distance from gallery.	α_n			α_t			T_{nt}		
	α_z	α_y	T_{zy}	α_z	α_y	T_{zy}	α_z	α_y	T_{zy}
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
0'	-1.00	2.72	0	0	0	0	0	0	0
0.5'	-0.53	2.09	0	-0.03	0.20	0	0	0	-0.72
1'	-0.22	1.86	0	0	0.27	0	0	0	-0.82
2'	0	1.87	0	+0.21	0.25	0	0	0	-0.88
3'	+0.15	1.50	0	+0.37	0.14	0	0	0	-0.92
4'	+0.21	1.40	0	+0.47	0.10	0	0	0	-0.96

After computing α_n , α_t , T_{nt} for each point in the line, principal stresses will have to be calculated and the total area of the tension will then be determined, based on which the reinforcement is calculated.

Similarly co-efficients for J line are given below :—

J line Distance out from gallery.	α_n			α_t			T_{nt}		
	α_z	α_y	T_{zy}	α_z	α_y	T_{zy}	α_z	α_y	T_{zy}
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
0'	3.20	3.40	7.00	0	0	0	0	0	0
0.5'	1.82	-0.05	1.95	0.26	1.35	1.55	0.05	0.42	0.84
1'	1.44	-0.22	1.29	0.24	1.13	1.46	0.04	0.56	0.91
2'	1.32	-0.18	0.71	0.21	0.95	1.12	0.04	0.42	0.92
3'	1.22	-0.10	0.42	0.18	0.89	0.77	0.04	0.28	0.92
4'	1.22	0.02	0.29	0.10	0.87	0.47	0.04	0.22	0.91

A similar procedure is adopted and the excess compression calculated in this case.

5.0. Computation of reinforcement around gallery.

By employing the formulae, enunciated in paragraph 3.11 and applying other conditions evaluated in paragraphs 3.4, 3.6, and 3.7, the total tensile and excessive compressive forces can be calculated around the gallery by the process of mathematical treatment. This marks only one part of the calculation, because the results so obtained will have to be compared with the values obtained by the photo-elastic studies and only the average of the two values is taken into account, generally, for adoption. This is because, of the greater radius of circle assumed for the opening the process of mathematical treatment, which would give larger reinforcement and the smaller size of gallery of only $5' \times 7'$ as against $5' \times 7\frac{1}{2}'$ size assumed for photo elastic studies, giving smaller amount of reinforcement and the average of the above two values is expected to represent a reasonable, stable, economical, aesthetic and enduring quantity of reinforcement for a standard gallery of $5' \times 7' - 6"$ size adopted in our State.

5.1. Cross reinforcement.

Generally one-fifth value of the main reinforcement is provided as the cross-reinforcement.

5.2. Spacing of reinforcement and cover.

A minimum cover of 6" from the concrete surface to the centre line of reinforcement bars around galleries, shafts and other openings in mass concrete is usually specified. The spacing of bars is usually not less than 6" and not greater than 12".

6.0. Conclusion.

An attempt has been made in this article to explain the procedure to be followed in the design of reinforcement around gallery openings in the dam proper. Detailed and exhaustive information on any of the aspects discussed can be had from the bibliography appended at the end and a typical calculation for the design of reinforcement around longitudinal gallery located at a hydraulic depth of 280' in a non-spillway block is enclosed as an appendix.*

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2. Stresses around gallery—Photo elastic interferometer study U.S.B.R. No. 12.
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4. Nagarjunasagar Specification and Drawing.
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6. Design Manual for Lower Bhavani Project.

* NOTE.—The appendix will follow in the next issue.

PARAMBIKULAM-ALIYAR PROJECT



View of surplus Regulator - Thirumurthi Dam.



Sethumadai canal—Vagiri crossing and approaches to the aqueduct at mile 0-1-330.



Contour canal at mile 17/0 to 21/0—View from Navamalai Tunnel inside at entry.



Contour canal—Direct view of aqueduct across Chinnar near mile 9/7.

USE RATE FOR EARTH MOVING MACHINERY

BY

* SRI K. RAMACHANDRAN, B.E. (HONS.), A.M.I.E. (IND.).

1. *General.*—Engineering has been variously described as “the art of organizing men and of directing and controlling the forces and materials of nature for the benefit of the human race” and as “the conscious application of science to the problem of economic production,” and “as a means of obtaining desired results with economy”. The Engineer’s approach to the solution of problems has been so successful that his field of activity has broadened to the extent that his success now depends as much upon his ability to cope up with *economic* aspects as with physical aspects of the situation. Charles F. Kottering of the General Motors research fame has stated “we have only begun to knock a few chips from the great quarry of knowledge. We know almost nothing about everything”.

have realized this aspect and are trying out various alternatives in attempts to improve the situation.

It is necessary to emphasize cost control at the present time for two reasons (1) large scale execution of works departmentally has started only recently and (2) execution of works by machinery is a new field to us. In foreign countries large scale works are undertaken by specialised and well equipped contractors or firms but in India, the execution of mechanised construction is done departmentally and the risk of cost spiral is real and of large magnitude. Corrective retrospective action can be taken against any labourer who does not do his share of work but the cost of operation of any machinery cannot be recovered. So it is all the more necessary that a system of rapid and detailed costing is maintained as mechanised jobs.

So economical execution of works is one of the fundamental requirements of Engineering skill. For the Engineer to exercise control of costs, he must keep in touch with the actuals of cost and production in adequate detail to judge if there is wasteful and questionable expenditure in any field of his activity. Every job must have its own set of control statistics to enable everybody to judge from day to day the actual performance and cost with reference to any standards prescribed. The mechanisation of project is a new sphere in which we Engineers are not fully oriented. It should be realized that we are attempting to adopt technique in this country with a background that is 50 years behind the times in relation to other countries that we try to follow. Prior to World War II we had very little machinery and with the advancement of science in every field, a time has come that we should also obtain high efficiency in such new fields. Every one of us

This necessitates a correct appraisal of cost which is an elementary valuation. Valuation is a very old art. Engineering valuation is the art of estimating the value of specific work where professional Engineering knowledge and judgement are essential. The term estimate is used in Engineering valuation in the sense in which it is employed in general engineering practice, that is, to indicate a carefully considered computation of some quantity, the exact magnitude of which cannot be determined. The estimate represents the true magnitude as closely as it can be determined by the exercise of sound judgment based on appropriate computations and is not to be confused with offhand approximations that are little better than outright guesses.

Engineering valuation entails a knowledge of the fundamentals of value and of the cost,

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service lives, and operating characteristics of the components and so it is essential that we understand what are meant by the service lives, depreciation, etc., in working out the use rates.

The question of fixing the use rate for several types of machinery, etc., used on project works, in several parts of India which were at variance with one another, was taken up by the Government of India, Ministry of Irrigation and Power and they had appointed several committees such as "construction plant and machinery committee" and "the rates and cost committee". With a view to ascertain and study the actual costs of various projects executed or under execution and find out economical ways for the same. It is known that the committee consisted of eminent Engineers and accounts personnel and they have produced their reports for guidance of project officers.

2. *Estimating service lives.*—The pricing and costing of a machine whether on an original cost, replacement cost or reproduction cost basis, is followed by a procedure of adjusting for depreciation. Machines which have been used are not generally worth as much as the identical machines if new. Property in service has given up some of its potential usefulness; therefore, in appraising such property consideration is given to the fact that the potential useful service of the property has been decreased. The factor almost universally adopted in measuring the consumption of the usefulness of depreciable asset is the potential total usefulness of the asset. The unit most frequently used for this purpose is the time unit of one year. But other units of use such as hour, mile, ton, etc., are used occasionally. The best procedure in estimating service lives is to study a past experience, and then to adjust the results when informed judgement indicates that the probable future experience will produce estimates of service lives significantly different from those arrived at by analysis of experience. In economy problems, patterns of the future depreciation of all manner of

assets must be predicted. This is a difficult task. When a machine depreciates through use a prediction must be made of the extent it will be used. The service life of a machine depends upon the useful service of the machine or in other words, the retirement of the machine.

The conditions which lead to the retirement of a machine include the following :—

A. *Physical condition*—

1. Accident.
2. Catastrophe.
3. Deterioration from time.
4. Wear and tear from use.

B. *Functional situations*—

5. Inadequacy.
6. Obsolescence—
 - (a) Economic.
 - (b) Style and mode.

C. *Situations unrelated*—

7. Termination of the need.
8. Abandonment of the scheme.
9. Requirement of public authority.

A. The physical conditions of a machine which are causes of its retirement may be explained as follows :—

1. Sudden physical damages may be caused by miscellaneous accidents such as explosions, collisions, falls, failures or breaking of the machinery by extraneous sources.

2. When sudden damage from disasters such as fires, floods, etc., amounts to destruction, the machine is to retire thereby. When the damage is partial the cost of repairs or reconstruction is weighed against cost of replacement in deciding whether or not to retire the machine.

3. Physical decrepitude is that physical disablement of property which develops and increases during service in spite of expenditure for repairs and other maintenance. Physical decrepitude may be caused by

physical deterioration from rusting and other chemical processes, the mechanical effects of freezing other temperature variations or gradual decay or in other words by "elements". If depreciation is caused by elements, the rate at which deterioration progresses must be established.

4. Wear and tear from friction, impact vibration, fatigue, etc., is caused by normal use.

B. *Functional situations.*—Even more difficult are the predictions that seek to determine when a machine will become obsolete because of new inventions and development of improved devices and new needs or inadequate owing to unlooked for demand.

The service life of a machine is that period of time or service extending from the date of its installation after purchase to the date of its retirement from service. Service life ends, nevertheless, with the act of retirement. This is the most important fact in cost accounting because the date of ending the service life is that date which controls certain aspects of depreciation accounting and is the date of minimum value to the owner. The economic service life is that the machine is retired when it is more profitable to use other machines or at least it is no longer profitable to use that particular machine. Much has been written concerning the service life of an equipment. Compilations summarising the depreciation of many types of equipment in many different situations are available. Unfortunately such data are only of limited value as a basic for predicting the service life of a particular equipment. Tables giving the

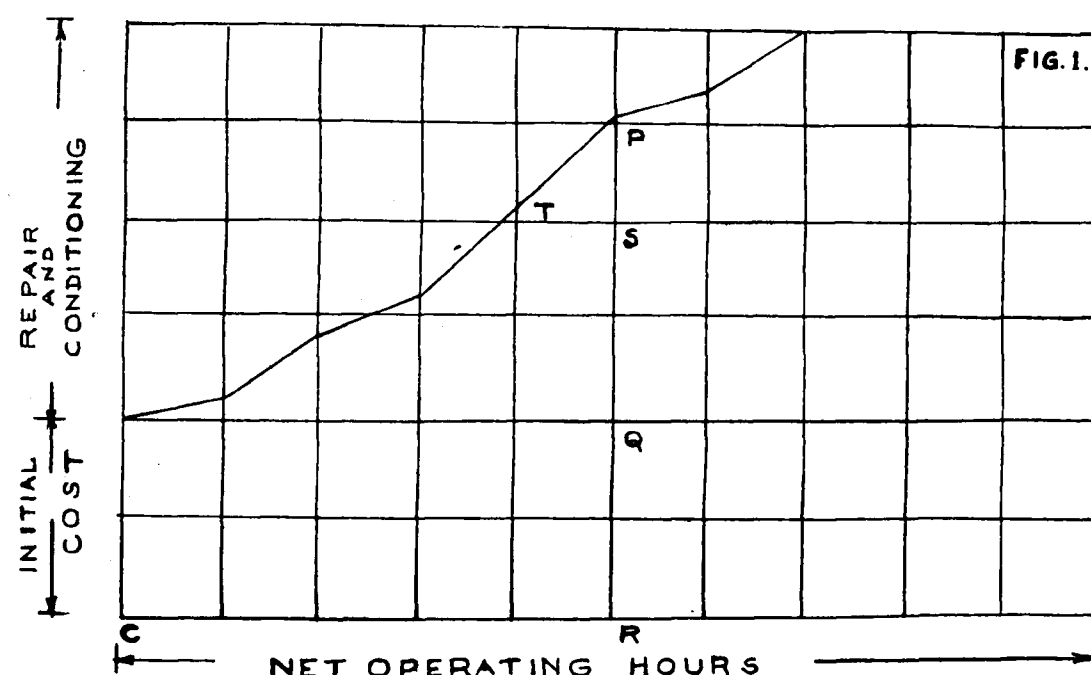
information have been prepared by people of various degrees of competence and ability. In any event they are largely based on judgement.

The estimates of useful life are for new machines only. In applying them, consideration should be given to salvage values, to that portion of the service life already expired and to that portion of the cost previously recovered or recoverable through prior depreciation deductions or other allowances. Salvage value is the net sum, actual or estimated, over and above the cost of removal and sale, realised for a property retired from service. Salvage value and scrap value are identical when the property retired from service is scrapped for the value of its materials, in lieu of additional further serviceable use of the property. Experience shows that it is economical to repair in order to maintain and extend their usefulness. Some repairs are of a current nature and minor in extent. Others are periodic and extensive and expensive. An extensive periodic repair is not usually contemplated until it becomes necessary to extend the life of the unit of equipment in question. Before an expenditure for major repairs is made to extend the service life of a machine, analysis should be made to determine if the needed service might be more economically provided by other alternatives. In some classes of equipment current repairs increase with age. Maintenance may be slight at first but increases at an accelerating rate with lapse of time. Thus a point is ultimately reached where it is more economical to replace than to continue maintenance. The following analysis is recommended :—

A	B	C	D	E
Year.	Cost of maintenance for year.	Sum of maintenance cost to end of year. ΣB	Cost of year of service. Cost + C.	Average annual cost of service to end of year D/A .
(1)	(2)	(3)	(4)	(5)

Whenever the least average annual cost occurs, it may be taken as the life of the machine. Thus the optimum life is reached when the hourly cost of major repairs at that particular stage equals the hourly cost of purchase plus the average hourly cost of major

repairs spread over the life. This was adopted in the Norris Dam project from which the following graph is quoted:—



$$\frac{PS}{ST} < \frac{PQ}{CR} + \frac{QR}{CR}$$

< Average hourly cost of major repairs + Hourly rate of depreciation. This theoretical analysis is still incomplete because it does not take into account the financial effect of the dislocation of work due to long hours spent on repairs. It is enough to visualise that the optimum life will be lower than that worked out by any such study.

3. *Depreciation.*—Broadly speaking, depreciation is the lessening in value of physical assets with the passage of time. Good care and maintenance may slacken the pace somewhat, but the end result is the same. It is characteristic of all physical assets, with the possible exception of land to decrease in value with the passage of time. Since 1934, literature often quotes the following definition from the Lindheimer vs Illinois Bell Telephone case, "Broadly speaking, depreciation is the less, not restored by current maintenance, which is to all factors causing the ultimate retirement of the property. These factors embrace wear and tear, decay, inadequacy and obsolescence". The Federal

communication commission has defined as follows. "Depreciation, as applied to depreciable telephone plant, means the loss in service value not restored by current maintenance, incurred in connection with the consumption or prospective retirement of telephone plant in the course of service from causes which are known to be in current operation, against which the company is not protected by insurance and the effect of which can be forecast with a reasonable approach to accuracy. Among the causes to be given consideration are, wear and tear, decay, action of the elements, inadequacy, obsolescence, changes in the art, changes in demand, and requirements of public authorities. In 1943 the National Association of rail road and utilities commissioner's committee on depreciation stated, "Depreciation is the expiration or consumption in whole or in part of service life or utility of property resulting from the action of one or more of the forces operating to bring about the retirement of such property from

service: the forces so operating include wear and tear, decay, action of the elements inadequacy, obsolescence and public requirements: depreciation results in a cost of service". This definition is more in line with the current practice. The American Institute of Accountants in speaking of depreciation accounting give support to the above definition. "Depreciation accounting is a system of accounting which aims to distribute the cost or other basic value of tangible capital assets, less salvage (if any) over the estimated useful life of the unit (which may be a group of assets) in a systematic and rational manner. It is process of allocation not of valuation. Depreciation for the year is the true portion of the total charge under such a system that is allocated to the year. Although the allocation may properly take into account occurrences during the year, it is not intended to be a measurement of the effect of all such occurrences".

The decrease in value of property from age zero to any service age results from a reduction, since it was first put into service now, in the present worth of its probable future services. Physical property decreases in value with age and use because it thereby suffers a decrease in the amount and character of the future service it can render before it will be retired. Good maintenance can prolong the useful life of the machine but maintenance operations do not entirely overcome the forces that ultimately cause the retirement of the machine.

3.2. *Classification of depreciation.*—A common classification of types of depreciation sufficient for most purposes is:—

1. Physical depreciation.
2. Functional depreciation.
3. Depletion.
4. Depreciation and its reverse appreciation due to fluctuations in price levels.
5. Accident.

The causes usually considered to result in physical depreciation are (1) deterioration due

to action of the elements; under this classification are such causes as corrosion of pipes in soil, rotting of structural elements, chemical composition and bacterial action all of which are substantially independent of use. (2) Wear and tear from use. The use of assets subjects them to abrasion shock-impact, chemical and bacteriological action and other forces that reduce their life.

Functional depreciation results not from a deterioration in an assets ability to serve its intended purpose but from a change in the demand for the services it can render. Obsolescence and inadequacy are the terms used in this connection. Assets become obsolete because they are no longer needed or because another asset, the superiority of which makes it uneconomical to continue using the original asset, becomes available. An asset may be in excellent condition but unable to meet the volume of the demands placed upon it. Rapid advance in technology and rapid growth and change in industrial output make obsolescence and inadequacy as important causes of depreciation. Depletion differs in theory from depreciation in that the later is the result of use and the passage of time while depletion is the result of the intentional piecemeal removal of an asset, where replacement is not contemplated. Depletion applies to cases like mines, petrol wells, etc. A change of price levels is normal with the passage of time and these fluctuations may influence the value of the assets. Accidents like fire, flood, etc., may result in very rapid depreciation. As already mentioned that even the best of care will not prevent deterioration with age. The best maintenance can only retard the rate of depreciation.

3.3. *Methods of estimating depreciation.*—Depreciation cannot be known in advance with certainty. With experience some forms of physical depreciation can be approximated but functional depreciation can rarely be determined objectively. It is impracticable to defer calculation of depreciation costs until after an asset has been disposed of at the end of its life. In fact, depreciation costs of an asset should be taken into account prior to the

purchase of the asset as one of the factors to be considered in arriving at the desirability of purchasing it. The calculated depreciated value of an asset also known as the "book value", at any time during its life is necessarily no more accurate than the estimates upon which it is based. The usual method is to assume that depreciation of an asset from its first cost to its salvage value takes place year by year in accordance with one of several mathematical formulae or curves. Some methods are arbitrary and others are based on theory.

1. Good-as-new depreciation method.
2. Direct appraisal method.
3. Arbitrary lumpsum method.
4. Depreciation as a percentage of revenue.
5. Sum of the Digits method.
6. Declining balance method.
7. Straight line depreciation method.
8. Sinking fund.
9. Present worth principle.

The four methods items 5, 6, 7, 8 only are most commonly used and it is enough if these methods are explained below.

3.3.2. Sum of the Digits method.—In this method, the sum of the arithmetic series of numbers from 1 to n is used where n represents the probable life of the unit, as the denominator of a series of fractions, n in number. The numerator of the fraction for any specific year is the expectancy of life as of that year. If 6 years is the estimated probable service life of a machine the denominator is $1+2+3+4+5+6=21$. The depreciation allocations for the first through the sixth year are found by multiplying the depreciable cost by a series of fraction, $6/21, 5/21, 4/21, 3/21, 2/21$ and $1/21$. This method an arbitrary method of allocation in which the depreciation base is held constant and the yearly rate decreased. As with the declining balance method, the result is to allocate the larger amount of depreciation to the first year and to decrease

the amounts each succeeding year. This method is not used to any great extent partially because it is difficult to apply to groups of units and because the declining balance method achieves similar results with greater ease and flexibility.

3.3.3. Declining balance method.—The declining balance method or fixed percentage diminishing balance method, is a variation of the fixed percentage depreciation idea in which the fixed percentage is applied to the unallocated balance of the base instead of to the cost new base. When the declining balance method is applied to a single unit, the annual depreciation is always a fixed percentage of the units' unallocated base at the beginning of the year. The formulae for the unallocated base of the unit at any age by the declining balance method is $B_x = B(1-f)^x$ where B = depreciation base including the salvage value V_s .

B_x : Unallocated base at age x .

f : the annual rate of depreciation.

Although the expression $(1-f)^x$ is always less than unity and decreases as the age x increases, it does not equal zero until x equals infinity. So the accrued book depreciation theoretically would never reach 100 per cent of the base. A finite ending of B_x can be obtained by the use of a salvage value. The above equation may be written in terms of the salvage value and the service life, n , $V_s = B(1-f)^n$ when B_x becomes equal to V_s the depreciable portion of the base B would have become fully allocated. It is possible to determine f

$$f = 1 - \sqrt[n]{\frac{V_s}{B}}$$

From this it may be seen that the fixed rate of depreciation to be applied to the declining balance is dependent upon the service life period and the ratio of salvage value to the depreciation base. This method results in a relatively high depreciation during the first year and progressively diminishing depreciation during subsequent years. Therefore it is more conservative than the straight line method in the event,

retirement takes place prior to the attainment of the contemplated service life. This method of depreciation is little used in the United States and has little application in economy studies because of the difficult calculations it requires.

3.3.4. Straight line depreciation method.—In this method it is assumed that depreciation is equal for each year of the life of the asset. The life and salvage value are but estimates and consequently the annual depreciation arrived at is also an estimate. Money is usually invested in an asset for the purpose of gain or return. The first step in the attainment of this purpose is the recovery of the capital that has been lost through the depreciation of the asset. The benefits received from the use of the asset are a charge against the products it produced. The benefits received are recovered capital. The second step is to secure a gain or return (interest) on the unrecovered capital remaining in the asset each year.

A general expression for the straight line depreciation and average interest is derived in the following manner; Let

P = first cost of the asset.

L = estimated salvage value at end of estimated life.

n = estimated life of the asset in year.

i = rate of interest.

(1) Depreciation in a year $\frac{P-L}{n}$

(2) Depreciation per year $\frac{P-L}{n}$

(3) Interest on unrecovered balance first year : $(P-L)i + Li$

(4) Interest on unrecovered balance n th year $\frac{(P-L)i + Li}{(n)}$

(5) Average of (3) and (4) $\left\{ (P-L) + \frac{P-L}{n} \right\} \frac{i}{2} + Li$
 $\left\{ (P-L) \frac{n+1}{n} \right\} \frac{i}{2} + Li$

Consolidating (2) and (5) results in a straight line depreciation and average interest equal to

$$\frac{P-L}{n} + \left\{ (P-L) \frac{n+1}{n} \right\} \frac{i}{2} + Li$$

$$\text{or } (P-L) \left\{ \frac{1}{n} + \frac{(n+1)}{2} \frac{i}{2} \right\} + Li$$

It should be borne in mind that this is only an approximate solution. It is fairly accurate for relatively low interest rates and for a relatively short life.

This method is more widely used in depreciation calculations than any other. A variation of the straight line method is the multiple straight line and this results when there is a change in the estimated probable life or salvage value from that used in the first year. The straight line method is equally applicable to single units, a group of units all of the same age or groups of units of mixed ages. This general applicability of the straight line method is another of its characteristics that has caused it to be widely used and accepted.

3.3.5. Sinking fund method.—In this method, the total allocation of the base to any date is equal to the corresponding accumulation (of annual payment to the sinking fund and compound interest thereon) in a hypothetical equal-annual-year-end-payments at sinking fund in which the total accumulation at the end of the service life of the unit would be just equal to the depreciable base of the unit. That is, in other words, one of a series of equal amount is deposited into a sinking fund at the end of each year of the asset's life. The sinking fund is ordinarily compounded annually. At the end of the estimated life of the asset, the amount accumulated in the sinking fund equals the amount of the estimated depreciation during the life of the asset. The amount deposited into the sinking fund annually is equal to the estimated total depreciation times the sinking fund factor corresponding to the estimated life and the interest rate thereon. It will be noticed that in the sinking fund method, the sum of depreciation and the interest on the undepreciated balance is the same for each year. While

the calculation of depreciation by this method is somewhat laborious, the sum of the sinking fund depreciation plus interest on the undepreciated balance is easily found. This sum is determined with mathematical accuracy by means of the capital recovery factor. This fact and the convenience of this method for calculating capital recovery with a return make it a preferred method with many, particularly for theoretical analysis of business problems. It will be noticed that depreciation becomes progressively greater as the age of the asset increases.

It is therefore less conservative than the straight line method in the event that an asset is retired prior to the end of the estimated period of depreciation.

3.4. *Evaluation of methods as related to cost accounting.*—The main objective of the accounting procedures for depreciable property is to allocate the cost of the property to production expense. An allocation equal to the full cost of the property is not made at the time of purchase for 3 reasons (1) The practice could result in extreme fluctuations in the proforma (2) all depreciable property would be immediately credited off the investment account, thus leaving no such assets on the balance sheet and (3) the usefulness of the property would not have been consumed by the date of its charge to expense. Charging the entire cost of the depreciable property to expense at the time of its retirement is not the preferred practice either because (1) the result could be extreme fluctuation in the statement of profit and loss or balance (2) the series of balance sheet statements of physical assets prior to retirement of the property would not reflect any reduction from cost to accounts for the partial consumption of usefulness of the property yet in service. The desirable accounting procedure then, is to follow an allocation method that tends to equalise the charge per unit of service for depreciation year to year but at the same time makes the allocation in conformity with the consumption of the useful service of the property. If these two objects are to be achieved the direct appraisal, good-as-new and arbitrary lumpsum methods are not usable as devices for

allocating depreciation expense. The arbitrary lumpsum method is ruled out because it follows no certain system which could be followed by another person or which could be depended upon for consistency from year to year.

The sum of digits methods when applied to a single unit has the merit of a positive scheme of allocation, subject only to the judgment of its probable service life. For group property the sum of the digits method is not usable because there is no single logical basis of arriving at the fractions which determine each year's allocation.

The declining method has considerable merit when applied to single units but is not practical for groups of units. For purposes of financial protection, a heavier charge for depreciation of property in early life than in latter life is sound, further from a production viewpoint, a property probably gives us its usefulness more rapidly in its early life than it does during its latter years.

The straight line method has found universal acceptance for depreciation cost accounting. No other method results in a distribution of the cost attributable to the consumption of useful service of the property by a procedure so easily applied and understood. Further the method is related to a reasonable assumption as to the rate of usage of the useful service of the property. Also the method is equally applicable to unit or group accounting. The method is objected to only because it does not allocate a greater portion of the base to early years than it does to latter years.

The sinking fund and present worth methods are somewhat troublesome in their application to accounting and are further objected to because they allocate more heavily to the latter years of service than to early years.

The foregoing discussions lead to the conclusion that for day-to-day cost accounting, the straight line depreciation method is superior to all other methods yet proposed. Any suitable service unit may also be used.

(to be continued.)



Radhapuram canal excavation in progress in Reach 5/3-0, Chittar-Pattanamkal Scheme.



Canal syphon at 0/5-55 of Pattanamkal—Chittar-Pattanamkal Scheme.

INVITATION TO READERS

Kind attention of the Engineers of the Madras Public Works Department and readers of the journal elsewhere is drawn to the meagre response to repeated requests for contribution of articles to the journal.

May we once again emphasise the two-fold objective of the journal, viz., to promote the interest of engineers and aiding their specialised work by placing before them the many problems of technical importance and ways and means of solving them, and secondly, opportunities being scarce for engineers to express their own experience in solving practical problems of design and execution, we wish to add that this journal has been the only medium of expression for those in the field. It is earnestly desired that the Engineering fraternity will come forward in large numbers to utilise this medium to its best advantage by offering their valuable contribution.

The scope of the articles to the journal need not be rigorously confined to the subjects on irrigation alone. Articles which have a direct bearing on subjects given below are also welcome:—

(1) Technical aspects of foundation soils and foundation Engineering.

(2) Auxiliary works in the field of irrigation and other water-retaining structures and works.

(3) Practical solution to hydraulic problems in canals and distribution systems with reference to their maintenance.

(4) Problems of soil erosion and conservation and reclamation of land.

(5) Practical silt problems in rivers and reservoirs.

(6) Problems of sanitation and public health and possibilities of utilisation of sewage.

(7) Inland waterways and navigation, fishing schemes and protection of land against sea erosion, etc.

(8) Problems of drainage in deltaic tracts and entrance of rivers into the sea.

(9) Run-off studies in catchments and evaporation studies in reservoirs.

(10) All allied subjects.

The Executive Engineers of the Madras Public Works Department are specially requested to encourage their Subordinate Officers to come forward with articles and enable the journal to flourish and serve the common cause of the Engineers of the State.

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22-11-1957
10-11-1957

