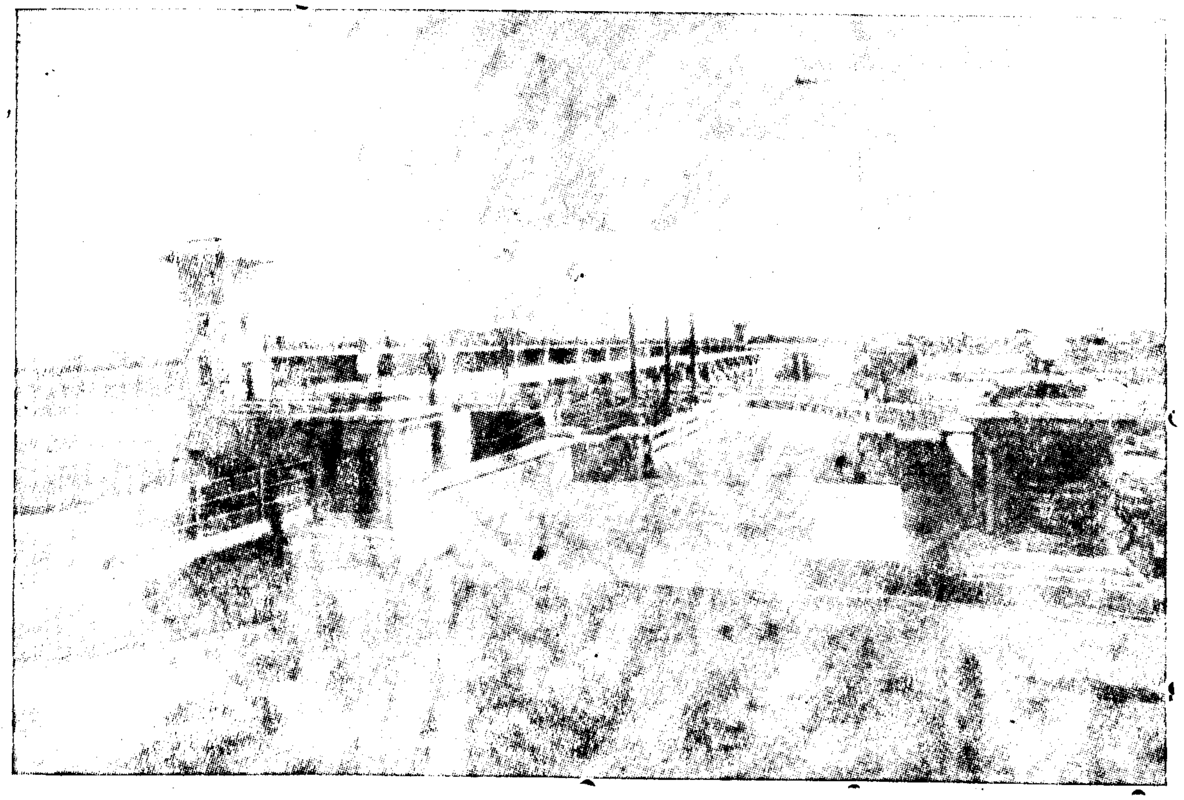
பாசன முறையின் மறுமலர்ச்சி செய்தி

The New Irrigation Era



A view of Virahanur Regulator.

JANUARY 1978
VOL. XVI—ISSUE No 1.
ஜனவரி, 1978
தொகுப்பு 16 — மலர் 1

The New Irrigation Era

1978 - January

Vol. ... No. 1

EDITORIAL.

In the first article, the author Er. K. Ethirajan, Assistant Engineer, P.W.D., Institute of Hydraulics and Hydrology, Poondi analyses the method of recharging the out going hot water from the coolant water system of a power plant into the ground water aquifer and dissipitating the high temperature of the water. This is necessitated on account of surface water scarcity due to the failure of monsoonic rains. New method of cooling and recirculating the coolant water has been enunciated without going in for detailed study of the whole coolant system.

Secondly, Prof. R. K. Sivanappan has explained in his article the need for systematic water management practice to minimise the use of water without the yield potential being affected.

Thirdly, Er. C. S. Ramadoss, and Er. K. V. Padmanabha Rao, Engineers of Valparai division have brought out in the article, the economy in the construction of a bridge adopting method of composite construction in steel and concrete in preference to the conventional T-Beam and slab method for information of readers.

Lastly, but not least, Er. M. A. Varadarajan, has explained regarding as to what is gaugings starting from selection of gauging sites, different methods of observing velocities and discharges and also regarding current meter gaugings, etc.

We again request all Engineers to contribute more articles for the forthcoming issues and serve a noble cause.

EDITOR.

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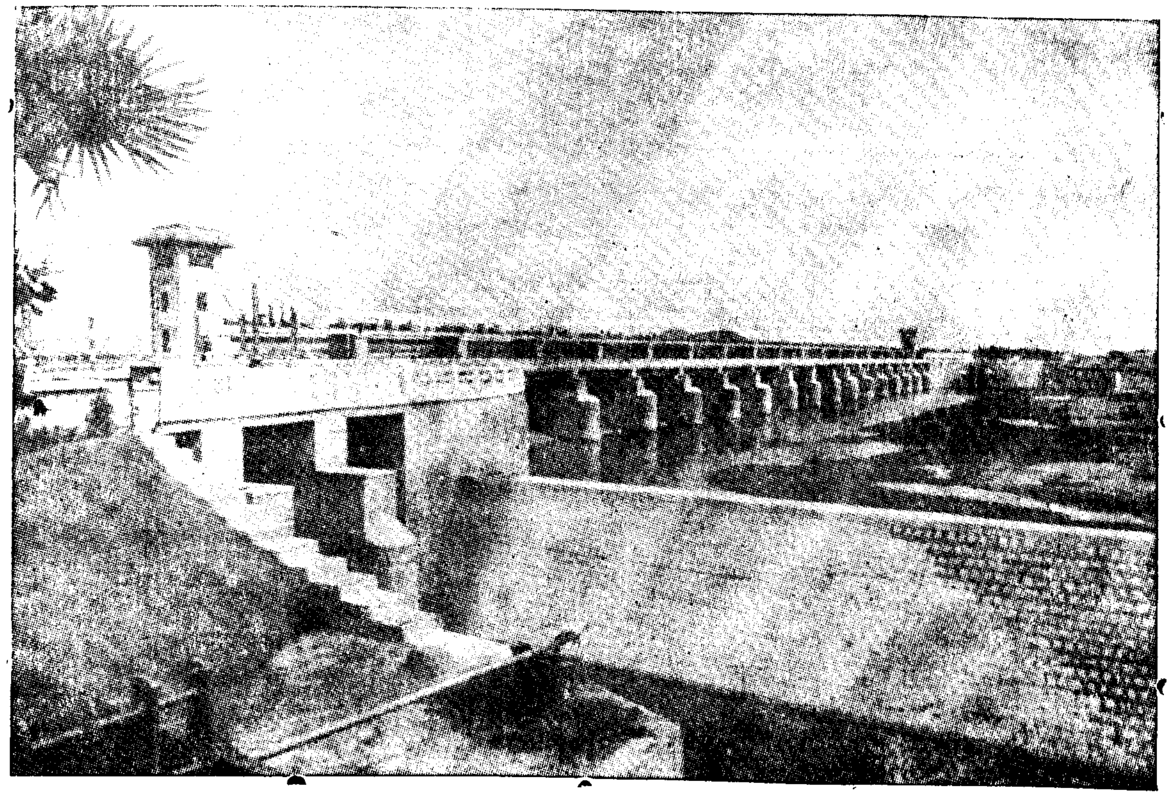

Another view of the Virahanur Regulator

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THE NEW IRRIGATION ERA

Vol. XVI **January 1978** **Issue I.**

CONTENTS

STUDY OF COOLANT WATER CIRCULATION TO THERMAL PLANTS BY GROUND WATER RECHARGING.

By

ER. K. ETHIRAJAN, B.E.*

Scope.—Almost all the thermal plants which are adopted for various purposes like power generation, pumped storage works, etc., need some sort of cooling system to minimise the rise in their working or limiting temperature. Of those various purposes, thermal plants adopted for power generation invite our special attention on their coolant water system in these days of scarcity of both water and power in Tamil Nadu. Though, it is a general principle that those power plants are to be located and situated near or at any surface water storages either natural or artificial for the purpose of obtaining adequate coolant and other water supply, there arise certain practical and unavoidable difficulties in drawing and disposing the coolant water. For example, from a surface water storage with multipurpose, the rate of intake of water for coolant system will vary according to the availability of storage in the reservoir, which will vary season to season. This may lead to a severe shortfall in the coolant water supply to the power plant. The drawal of coolant water for power plants from sea or any other natural storage may be affected by the formation of sandbars of littoral drift at the intake, which gives rise to the difficulty of reduction in the intake rate of coolant water continuously. Present methods that are being handled to avoid such difficulties do not prove to be efficient and economic. On the other hand the surface water scarcity due to the failure of monsoonic rains pricks us to evolve any other alternative to avoid the above problem, by all the advancements in science and technology.

With the above problem in mind, the scope of this paper is vested or devoted to illustrate in principle, the method of recharging the out going hot water from the coolant water system of the power plant into the ground water aquifer and dissipating the high temperature of the water. Fresh and cold water may also be pumped into the coolant water system by direct connection of the discharging limb of the recharging assembly installed nearby for the purpose. The scope of this paper is further limited to understanding of the new method of cooling and recirculating the coolant water without going in for detailed study of the whole coolant water system and its working or operation.

Introduction.—The thermal power plants in our state experience insufficiency of coolant water supply and the enormous cost of the supply of coolant water at present Secondly, the methods of cooling in reusing the effluent hot water from the plants, by reducing the rise in temperature, are not also effective and quick. Hence it becomes necessary that any other alternative method of cooling and supply of the coolant water which will be effective and quick must be introduced atleast to safeguard the power plants from their break-downs due to the insufficient rate of coolant water supply into them to absorb maximum temperature of the plants during their operations for power generation and to keep them in their working or limiting temperature for their best performance with maximum efficiency. Now, it is known that sufficient knowledge of the ground water study and its tapping for various purposes in place of the surface water is available and thus the artificial recharging of the effluent hot water coming out from the power unit into the ground water aquifer in the nearby area, getting it cooled by mixing with the cold water of the aquifer and recirculating the cold water pumped out from the aquifer directly into the coolant system of the power plant unit continously, become effective and quick for the purpose.

Procedure outlined.—The execution of this system or method will mainly comprise the following works:—

1. *Investigation* of sub-surface and ground water potential and the characteristics of the underground aquifer in the near vicinity of land where the power plant is located and

2. *Installation* of the recharging and discharging unit with other accessories required for pumping, measuring and recirculating the ground water directly into the plants.

1. *Investigation of the ground water aquifer* :—This work comprises of conducting a site survey of the land, drilling and boring operations to obtain details of the sub-strata of the land, boring *number* of observation wells to the maximum depth of the aquifer and conducting pumping and recuperation tests on the observation wells, etc. The sub-surface investigation will reveal the configuration of the geologic formation of the various strata of the ground and their physical characteristics in the storage and transmission of ground water. This investigation is done by actual drilling and boring operations into the ground and analysing the soil samples taken for the specific object of determining their water holding, retaining and transmitting capacities. A *number* of observation tube wells in the land area located in a planned manner may

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be board to determine the ground water table level, depth of aquifer and also the extent of the aquifer; pumping and requperation tests may be conducted to determine the rate of the ground water movement (Transmissibility) and the capacity of the aquifer, whether it is a confined or unconfined one (storage). From all the above investigations, we can visualise or forsee the whole ground water flow around the power plant site and the nature of sub-soils which will be of much use in deciding the rate of recharging of hot water from the plants into the ground and the ground water for dispersion and cooling. The observations on the storage and transmissibility of the aquifer underneath are to be made continuously for different seasons to realise the fluctuations and variations in them. The location of the observation wells may be carefully planned in such a way that the observations in those wells reveal the exact nature of the aquifer and those wells can well be adopted as either discharging or recharging ones later, when the artificial recharging unit is executed. The preparation of "Bore-log" details for each observation well and determination of the ground water flow parameters like the "storage coefficient" and "Transmissibility" are the most inevitable parts of the whole investigation. The initial cost of this investigation may be worked out and compared with that of the other methods of coolant water supply, along with other factors to decide the applicability and reliability of the new method. Finally, it is to be noted whether the ground water aquifer studied would supply the requisite rate of coolant water to the plants as required by them and whether the recharging of the hot water from the plants would be efficient with the rate of effluent water or not. All the above investigations are of paramount importance to study and determine the applicability of the method in the particular locality of the thermal power plants even just to decide their location.

2. *Design and Installation of the unit.*—The design and installation of the various components of the recharging unit depend on the results of the investigation made as described earlier, on one hand and the requirements of the power plants in respect of their cooling system, on the other hand. The depth of ground water aquifer, storage co-efficient and transmissibility, etc., can decide the type and horse power of the pump and motor to be used for discharging the coolant water supply from the aquifer. The bore-log details and fluctuations in the ground water table level, etc., can help in deciding the rate of recharging and accessories required. Alternatively the capacity of the discharging pump and motor may, also be determined from the rate of coolant water supply required by the plants for their best performance. The design of the recharging limib of the system may be made with reference to the rate of the effluent hot water from the plant, which has to be dispersed and cooled. The temperature rise in the coolant water has to be simultaneously observed. Necessary fittings like pressure guage, watermeter, theremometer, control valves, manometer and strainer arrangements, etc., are to be carefully selected and installed in the unit with special attention. Necessary outlet arrangements and water tappings, etc., may also be installed in the unit for using the pumped ground water for some other allied, domestic and gardening purpose for the locality.

The sketch enclosed is assumed to illustrate the actual installation of the discharging and recharging limbs with all their fittings, etc. The spacing between wells, dimensions of screens and casing pipes and quality of the screens, etc., are to be properly chosen to avoid improper mixing of hot water, soil particles entering into the impeller of the pump and the cooler unit, etc. The perforations in the casing pipes should provide adequate water way without affecting the natural formation of sub-soil strata to avoid caving and undermining and surging, etc., Sufficient air escape line is also necessary to get rid off the air entraining and entrapping into the recharging limb.

Finally, it is suggested that the present day problems on the coolant water supply and disposal or recirculation may be tried to be overcome by the above method wherever it is feasible; it is presumed that the method will be much better suited and economic for the power plants situated near the natural storages or coastal areas where the ground water table level will be relatively at a shallow depth and the very aquifer underneath will be almost unconfined in that area. This will totally solve the insufficiency of coolant water supply and promote the cooling and dissipating the rise in temperature of the effluent water, with all other allied advantages in any thermal power generation project. This paper is only an outline and pre-thought to the above study.

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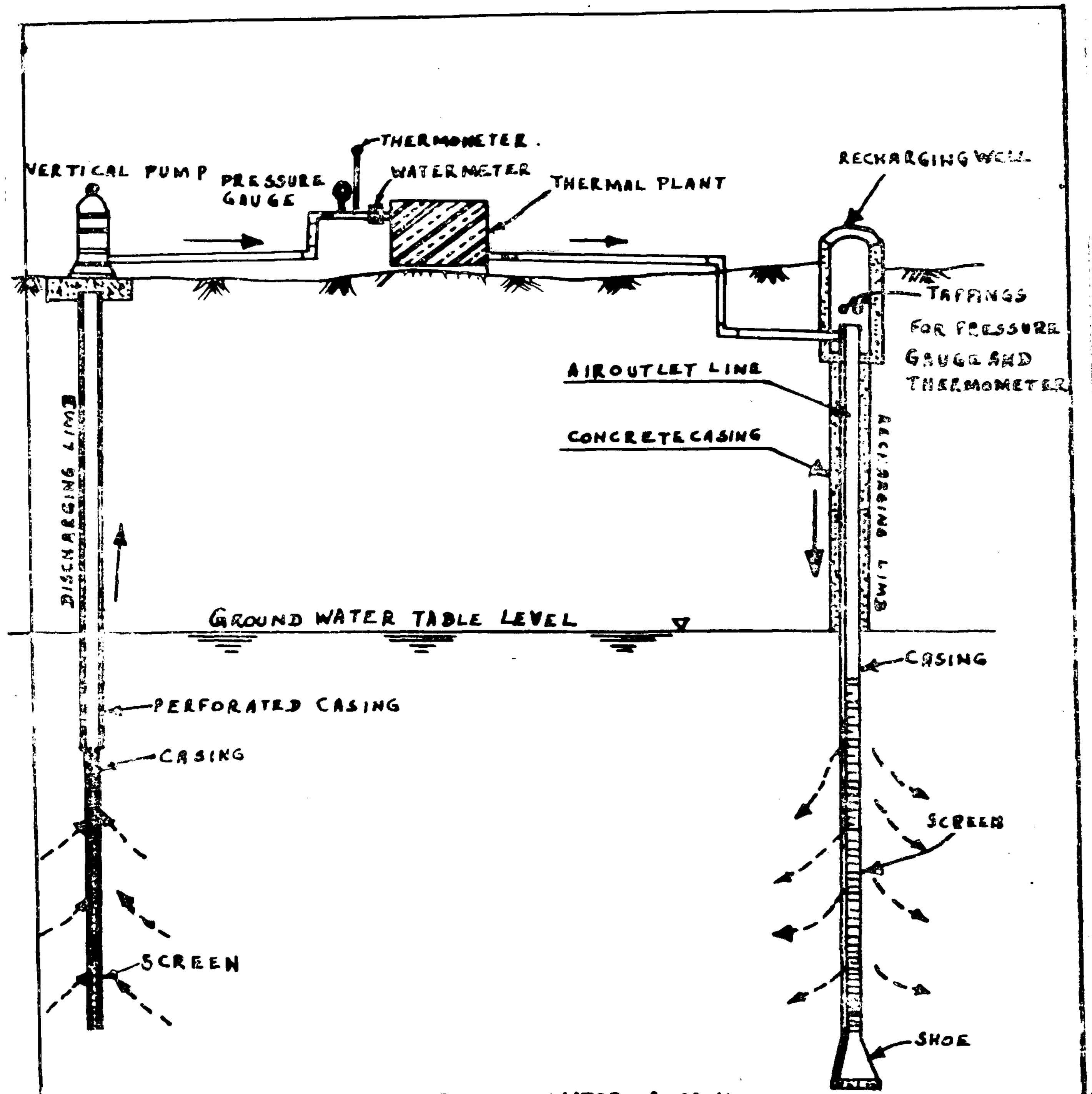

SKETCH SHOWING THE GROUND WATER SUPPLY AND RECHARGING ASSEMBLY FOR COOLANT WATER RECIRCULATION TO THERMAL PLANTS -- (NOT TO SCALE).

WATER MANAGEMENT RESEARCH

Er. R. K. SIVANAPPAN *

Introduction—

Water is a vital natural resource, most often neglected in use. The demand for water is to be doubled in twenty years. Hence conservation and management of water through integral process of diversion, conveyance, regulation, metering, distribution and application at appropriate time becomes vital to conserve this natural resource for the benefit of posterity.

2. **Balance Sheet—**

Total surface water potential in Tamil Nadu is 4.00 m.h.m. Ground water potential is estimated to be 1.50 m.h.m. 95 per cent of surface water and 67 per cent of sub-surface water (about 1.00 m.h.m.) are being utilised in this state. About 3.68 m.h. of land is under irrigation which accounts for 46 per cent of cropped area. About 2.50 m.h. of area is under paddy cultivation which consumes more than 85 per cent of the water available for irrigation. About 30 per cent of the irrigated land utilises the remaining 15 per cent for raising crops other than paddy. The water requirement studies conducted at Tamil Nadu Agricultural University and elsewhere have brought out ranges of water use for some importance crops, as indicated below :

Paddy : 1,000—2,500 mm.

Millets : 350—700 mm.

Banana | 1,500—2,500 mm.
Sugarcane |

The advancement of water management practices have given scope to narrow down this range of water-use without sacrificing the yield.

3. **Results of Water Management studies—**

In Tamil Nadu the need for water management research was started as early as 1920. The water requirement studies conducted at Aduthurai, Tindivanam, etc., brought out varying results owing to the differences in agro-climatic factors, advective and oasis effects, differences in seasons and methodology. Water management aspects of paddy crop was studied systematically after 1960 at Bhavanisagar, Coimbatore and Thanjavur. The results indicated scope for effecting economy in water use. After the establishment of the Water Management Institute at Thanjavur and the Tamil Nadu Agricultural University at Coimbatore, intensified studies were taken up on conveyance, land shaping, management practices, etc., in relation to water application and use for crops and the following findings were obtained:—

(*a*) Conveyance loss through unlined channel is more than 40 to 50 per cent.

(*b*) Land-levelling operation alone resulted in a saving of 30 per cent of water needed.

(*c*) Maintenance of 3 to 5 cm. of submergence during a limited period and a thin layer of film during the remaining season not only resulted in savings of water, but also did not decrease the yield, resulting in higher water use efficiency. With regard to conveyance, use of prefabricated structures can be effectively used since the initial investment can be reaped back within years, The advancement of mechanical gadgets facilitates preparations of ideally level fields. On stock taking of the results so far obtained, it has been noticed that the work done so far is only a fringe of the problem of the State.

4. **Research Needs—**

The emphasis for better management of water needs concentrated research needs as follows :—

(*a*) Utilisation of existing knowledge.— The application of research findings will have to be diffused to the farmers' level. In such endeavour, there is need to confirm the results so far obtained at different physiological, soil and climatological conditions.

(*b*) Adaptive research need.— In order to verify the research findings adaptive research is vital. This can be done by magnifying the area of approach, including more relevant practical parameters, if possible in the farmers' fields.

(*c*) New research.—Sometimes adaptive research will bring back new problems which can be posed to the research scientists, who, in turn, will identify the cause for such problems and work out a reliable solution. The introduction of modern agricultural technology through high yielding varieties, mixed cropping, relay multiple cropping, short duration and hybrid varieties call up different management practices. For istances, the coincidence of flowering stage and monsoon will have certain significant effects in certain crops. The critical water need coincides with certain growth reason. There is a deficiency of knowledge in "on-farm water management aspects".

5. **On-farm water management studies—**

The research needs can be classified as direct or basic research needs which will directly benefit the farmer and indirect or fundamental research needs which will help the scientists to chalk out a solution to the problem. The following problems have been identified which are more relevant with regard to "on-farm water management".

(a) ***Fundamental studies***

(i) Physiological responses of crops in relation to environmental stresses.

(ii) Interaction of water with other inputs with regard to crop productivity.

(iii) Identification of critical technological innovations or systems and their impact on the management practices.

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(b) *Basic studies*

(i) Metering, control and regulation of drainage/irrigation water to improve farm-irrigation system.

(ii) Impact of quality and quantity of on-farm water management with surface and sub-surface water resources.

(c) *Socio-economic studies*

(i) Farmers' involvement in management and distribution of community water resources—Ways and means analysis.

(ii) Prediction of economic consequences of alternative—both short-term and long-term strategy.

(iii) Integrated approach of water management for irrigation, industry, etc., and the prediction of socio-economic hazards such as pollution, water logging, etc.

(d) *Applied Research Needs*

It is not possible to wait for the results of the water management studies spelt up above. Immediate steps are required to concentrate on the following problems simultaneously :—

(i) Conjunctive use of surface and ground water.

(ii) Development of cropping patterns for efficient water utilisation. The fluctuation of price levels of agricultural commodities create varieties of problems to farmers who switch over from one crop to the other. The findings of such studies will benefit the farmers.

(iii) Volumetric water delivery/distribution.

(iv) Maximising crop yield per unit of water through land management, application management and crop selection besides other cultural practices such as skip-furrow/alternate furrow practices.

(v) Resource allocation for maximum return.

(vi) Resource enrichment programme through percolation tank, detention storage structures, water harvesting technique, re-use of drainage water, etc.

Apart from the above research work, the following distinct steps needs to be taken to improve water management in the command areas. Many of these steps can be attempted simultaneously in order to save valuable water.

1. Creation of an informed official opinion.
2. Creation of an informed public opinion.
3. Introducing the subject at the technical level.
4. Creation of an adequate technique infra-structure.
5. Creation of the necessary infra-structure.
6. The phasing of implementation.

6. Formulation of programmes —

Works on water management are carried out by the various organisations in the state, namely (a) Tamil Nadu Agricultural University, (b) Department of Agriculture (c) Department of Irrigation (P.W.D.), (d) Ground Water Directorate (e) Private Agencies. There is no co-ordination among the research workers working in the various organisations at present and they themselves do not know what is happening in other sides. Therefore, it is suggested that there must be a separate organisation to take up research and implementing the research findings at the State level. And there must be a State level committee or board on "water management implementation". The Director of Agriculture, Chief Engineer (Irrigation) Chief Engineer (Ground water), Tamil Nadu Agricultural University, Centre Ground Water Board, Representatives of private bodies who are interested in the water resources and development should be members of the board in order to chalk out the programme, priorities and implementation of the various research findings. There must also be a State level institute for water management which should be located at the central place and this centre can take up research need of that region in addition to co-ordinate the activities of the entire State. There must be five regional stations attached to the Institute for the remaining five agro-climatic regions of the state.

The Water Management Institute at Thanjavur and the Water Resources Institute at Madras may be brought under this set up. The Institutes should have the following separate department to tackle the various problems :—

(*a*) Co-ordination.

(*b*) Data collection.

(*c*) Research.

(*d*) Training.

(*e*) Extension Education and

(*f*) Irrigation Information.

Each regional institute should have a research wing for both irrigated (canal, wells) and dry land training wing and extension wing, so that the research findings can be popularised and implemented soon.

The State institute should organise the work in collabo ration with the National Institutes like Water Technology Centre, New Delhi, International Crops Research Institute for the Semi-Arid Tropics, Hyderabad, All-India Co-ordinated Research and on Water Management and Soil Salinity, etc. In addition, it can also collaborate with International Organisations which are devoting their attention in water resources and management namely, Water Resources Council of Australia, The Council of U.S. University for Soil and Water Development in Arid and Sub Humied Areas, Logan, Utah, U.S.A., Water Resources Council of U.S.A., Washington, U.N.D.P., etc. Since water management and conservation is a global problem, the help of the international agencies, world organisations and other foundations can be obtained to organise and to implement the above proposal who have already sized the problem and implementing in other parts of the world in a big way.

7. Suggestions for future work —

Tamil Nadu has already harnessed the entire water and is being used for irrigation and other purposes. In order to increase production for the growing population and the water needs for industries and for drinking, the requirement of water has to be found out. Substantial quantity can be met by adopting suitable cropping patterns. In order to plan and execute the same, it is necessary that an irrigation (water) commission is appointed by the Government for collecting all the data and suggesting the means and ways for the future needs just like Irrigation Commission of India, 1972. Water has become already a scarce commodity in Tamil Nadu and hence it is very necessary that Extension Officers (Water Management) may be created for each block, in order to give proper guidance and help the farmers in using the water. In U.S.A., extension specialists (water management) are working in counties in order to utilise the available water most efficiently. Further legislation should be brought to check and guide to prevent the over-utilization of water. The State is not having trained personnel in water management. Hence training facilities should be created in the proposed institute for training variuos level of persons, including farmers on water management. It is also necessary to have an irrigation information centre to deal with all aspects of field irrigation. This can be attached to the training centre mentioned above. The centre will publish materials in simple form (popular papers in Tamil) and research reviews on the water management of crops and in the extension methods, suitable for agriculture in all regions will be undertaken. It also will act as a clearing house and regional services in basic and applied aspects of irrigation. Further, this institute can arrange to get top scientists from other countries for short periods for consultation to improve the water management in Tamil Nadu and also try to send the research personnel working in the field to get acquaintance with the latest techniques in other advanced countries. To organise this programme, the help of the various organisations like Ford Foundation, Rock feller Foundation, USAID, World Bank, Food and Agricultural Organisation, UNDP, etc., can be approached. All the above suggestions may be implemented without postponement as the State has already feeling the pinch of the shortage of water and no time should lost further more.

Summary —

The water supply is limited (constant), but demand of water is increasing day by day. Fortunately, some of the problems could be solved by utilising and adapting the existing knowledge on water management. The problems, the crops grown, the source of water are all different in different situations. Though some work has been done in the State in the last 10—15 years, it has not touched the main problems. Many departments or organisations are interested and involved in this subject. All these can be clubbed and a separate organisation can be formed to tackle the very important and essential problem. The problems to be tackled have been identified and the suggestions are given in the previous pages. The work to be done is a complex nature as it involves many agencies including the farmers. Therefore concrete steps should be taken with the co-operation of the farmers and public and also with the help of State level, national and international agencies. The farmers and extension officers should be trained in order to utilise the water economically and to produce more. The suggestions should be implemented by forming a State-level committee which can co-ordinate and guide the problems on water management.

—O—

CONSTRUCTION OF A BRIDGE ADOPTING METHOD OF COMPOSITE CONSTRUCTION IN STEEL AND CONCRETE.

By

*1. THIRU C. S. RAMADOSS

AND

*2. THIRU K. V. PADMANABHA RAO.

This article deals with construction of a bridge in Parambikulam Aliyar Project, adopting a composite section of steel girders and concrete slab. (Please see sketch at last Page.)

Flood waters of the Nirar River are diverted to Sholayar River by means of two tunnels, the Upper Nirar Tunnel and the Lower Nirar Tunnel. The three mile long Upper Nirar Tunnel takes off just above the Nirar Weir in the Chinchona area and is capable of carrying a maximum discharge of the order of 3500 c/s. The diverted waters enter a small stream called Karumalai Ar. which ultimately joins Sholayar River above the Sholayar Reservoir of Parambikulam-Aliyar Project. The catchment yield and catchment flood of Karumalai Ar. are quite small. There are a number of small bridges across this stream, put up by the local estate owners for the purpose of vehicular traffic as well as for the labourers to cross from plot to plot. With the establishment of the Chinchona Plantations and factory, introduction of bus services and due to the construction activities of Parambikulam-Aliyar Project, traffic over some of these bridges have considerably increased. Hence PAP authorities have reconstructed a few of the bridges in this area to accommodate the increased load and traffic over them. The original design of foundation of all the bridges was only for the maximum flood carried by the stream; from the point of view of headroom most of the bridges were comfortably off, as the nature of the rain automatically left sufficient free board in most cases. Where the bridges are founded on rock, the scour was no problem, but in some cases where the bridges were founded in gravelly or loamy soil, the quantity of discharge, extra velocity generated and the maximum scour depth, assumed very great importance. Some of the old bridges which were not rebuilt had their foundation undermined when the diversion of flood waters took place during the monsoon period of 1974. One such bridge was located at a distance of 4.07 Km. from the exit of Upper Nirar Tunnel. During the peak flows, the abutment and piers of this bridge were undermined due to heavy scour and they collapsed.

On an inspection of the site after the flow receded it was obvious that the bridge could not cope up the discharge due to insufficient waterway as well as insufficient depth of foundation. Hence while designing the new bridge, the following points were considered :—

(i) As there was no rock at site or anywhere in the vicinity, the bridge is to be located at the same alignment, with the foundation laid in loamy soil taken down below the maximum scour depth;

(ii) To prevent generation of higher velocities and consequent deeper scour depths, constriction of river width between abutments was not advisable;

(iii) For the same reason as above providing many piers in the river bed to restrict the span was also not possible.

Considering these aspects it was decided to adopt two spans of 12.2 m (40'-0") each, a single pier in the middle thereby leaving a total clear waterway of 24.4 M.

Original intention was to go in for a conventional Tee beam bridge. The main problem in going in for this type of construction was provision of shoring and strutting for the centering for the Tee beams and slab. The stream was almost perennial. The period of lowest flow had to be utilised for excavation and concreting of the foundations as it was a very difficult job to prepare the foundation pit in a flowing type of soil, with water also gushing from all sides; for a small unimportant bridge of this type highly sophisticated methods, of river-bed-tackling was not considered worthwhile or economical. Hence there was hardly any time left to do the shoring and strutting from the river-bed and to maintain it for a minimum period of three weeks, with the possibility of flash-flows at any time. The soil condition in the river-bed was also not suitable to provide massive shoring and strutting arrangements for the centering.

Provision of conventional steel girder bridge with deck slabs for a 12.2.m span was found to be exhorbitantly costly. It was therefore examined whether we could not make the deck slab to take the compression in the longitudinal direction, i.e., in the direction of the main span and use steel girders to take only the tension, employing shear connectors at the interface of steel and concrete to provide composite action. On a rough assessment it was found to work out even cheaper than a conventional Tee beam and slab structure, at the same time facilitating the centering for the decking to be strutted from the lower flanges of the girders. Thus an arrangement for carrying out the execution uninterrupted by the flash floods during summer storms and the sustained floods of the South-West monsoon was made possible by the adoption of this system of construction. There was also saving in use of cement besides achieving an overall economy in construction cost.

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*2. Assistant Executive Engineer, P.W.D., Lower Nirar Tunnel Sub-division, Valparai.

DESIGN OF BRIDGE DECKING ARRANGEMENTS.

Data

Clear span .. .. 12.2 m.

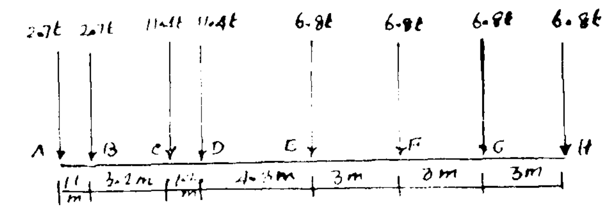
Clear Roadway .. .. 3.60 m.

Live load .. .. .. I RC class A loading.

I. Design of slab :

The Bridge shall be designed as a slab resting on R.S. Joists designed as a composite section with shear connectors.

The load system is as shown in figure.



(i) No other live load will cover any part of the carriage way when a train of vehicle is crossing the bridge.

(ii) In the slab the maximum bending moment occurs when loads (C and D) are in one panel.

In the beams the maximum bending moment occurs under the wheel C or D in the slab for a centrally placed load.

Bending moment along short span; ($m_1 + 0.15m_2$) W

Bending moment along lone span ($m_2/ + 0.15m_1$) W

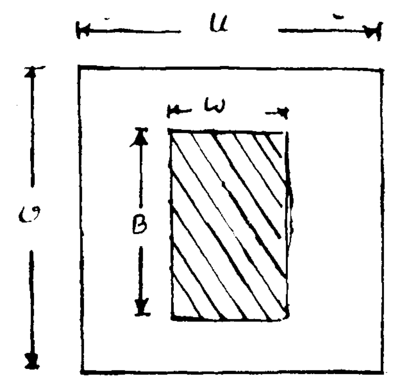
Where W is the Wheel load and m_1 and m_2 have the following values (*see* fig)

where ($u=W+2t.$)

$v=B+2t$ where t is the thickness of wearing coat.

U/L	U/L	m¹	m²
0.07	0.36	0.173	0.110
0.35	0.36	0.128	0.026
0.49	0.36	0.106	0.014

Clear span of the bridge : 12.2 m



Effective span :

In the case of free supports on line bearings the effective span shall be the distance between the centers of bearings.

In the case of a free support not on line bearings the effective span shall be l+d or L whichever is smaller.

Where L =distance from to centre of supports.

l =clear span.

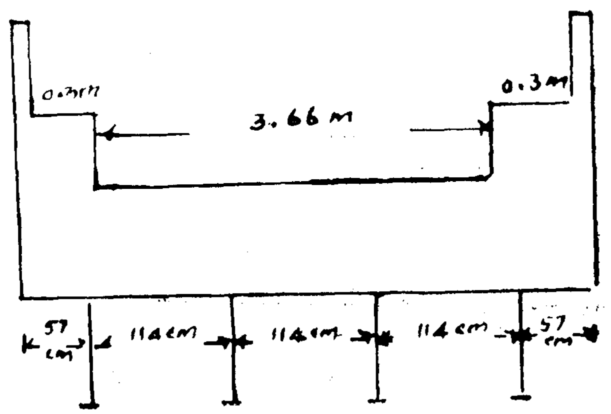
d =Depth of beam.

Using 500 mm depth of girder,
effective span=12.2 = 0.05=12.7m

Therefore dispersion of the wheel load.

$$L= \frac{12.7}{2} =6.35$$

l = 1.14 m.



Thickness of wearing coat = 8 cm.

Ground contract Area: B = 25cm.
W = 50cm.
V = 25+2X8 = 41 cm.
U = 50+2X8 = 66cm.

$$\frac{V}{L} = \frac{41}{6.35x100} = 0.0645$$

$$\frac{U}{L} = \frac{66}{1.14x100} = 0.58$$

But smallest valve for U/l = 0.36.

Hence m' and m corresponding to V/L = 0.07 and u/l = 0.36

m_1 = 0.173

m_2 = 0.11.

1. BM along short span = (m_1 + 0.15 m) W
= (0.173+0.15x0.11) x 5.7 x 1000 kgm.

= 108000 Kg. cm.

(ii) Impact factor $\frac{4.5}{6+1}$ $\frac{4.5}{6+1.14}$ = 0.623.

But the impact factor is not to exceed 0.5

BM due to impact = 108000 x 0.5 = 54000 kg. cm.

(iii) BM due to D.L. :—

Assume 20 cm thick slab :

D.L. of slab/m = 20 x 24 = 480Kg/m

D.L. of wearing coat = 8x 23 = 184 Kg/m.

Total Dead load .. 664kg/m²

Dead load B.M. $\frac{664 \times 1.14}{8}$ = 107.00 Kg. m.

= 10700 kg. cm.

Allowing for continuity, the actual B.M. to be taken for design = 80 % of the max. free moment.

Total B.M. = 108000+54000+10700

= 172700 kg. cm.

0.8 of this = 0.8 x 172700 =

= 138160 kg. cm.

Using M-200 concrete mix the following working stress will be adopted.

C = 70kg./cm.²
t = 1400 kg./cm.²

$$m = \frac{2800}{3c.} = \frac{2800}{3 X 70} = 13$$

nc = 0.394 d

a = 0.087 d

Q = 12

12 X 100 d² = 138160

d² = 138160/12 X 100

$$\therefore d = \sqrt{\frac{138160}{12X100}} = 10.75$$

Providing 25 mm cover and using 12 mm 0 bars.

depth of slab = 20—2.5—0.6 = 16.9 cm

$$At = \frac{138\ 160}{1400 \times 0.87 \times 16.9} = 6.735\ cm^2$$

Spacing of 12mm rods = $\frac{1.13 \times 100}{6.755}$ = 16.78 cm (or) 17.00cm c/c

Distribution steel = 0.2 x live load moment + 0.3 x dead load moment.

= 0.2 (108000+54000) + 0.3x10700
= 32400+3210
= 35610 kg. cm.

Using 8 mm ∅ bars = 20—2.5—1.2—0-4

Effective depth = 15.9 cm.

$$At = \frac{35610}{1400 \times 0.87 \times 15.9} = 1.839$$

Spacing of 8mm bars = $\frac{0.5 \times 100}{1.839.}$ = 27cm or say 25 cm/c/c.

Check for shear :

Dead load shear. $\frac{664 \times 1.14}{2}$ = 378.4 Kg. = 378.4 kg.

Live load shear with impact effect. = $\frac{5.7 \times 1000}{2}$ x 1.5

4275.0 Kg.

Total shear = 4653.4 Kg.

eff. width from S.f. consideration =

$$b = 27.6\sqrt{d+(B+2t)(1-\frac{2.21}{d})}$$

$$27.6\sqrt{20+(25+2x8)(1-\frac{2.21}{\sqrt{20}})}$$

= 176 cm.

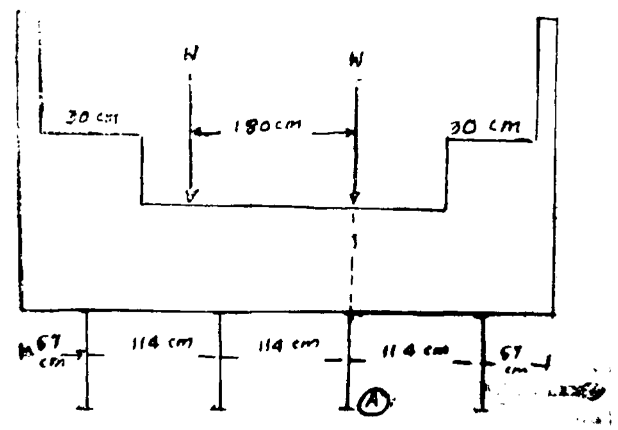
$$\text{Shear stress} = \frac{S}{a \times b} = \frac{4653.4}{0.8 \times 16.9} \times 176 = 1.8\ kg/cm^2$$

Less than 7 kg/cm²

Hence safe.

II. DESIGN OF GIRDER

Effective span = 12.7 m.



15 MB 500 x 180

The load transferred to a girder will be max. when one row of wheels of one train is exactly on the girder and one row of wheels of a passing train is closest to the girder.

max. B.m. occurs under wheels C and D.

Since the span of the girder is 12.7m effective and since the max. B.M. occurs near the centre of the girder, the loads.

Wa Wb Wc Wd and We will be on the span corresponding to max. B.m. condition.

Hence max B.m. will occur under wd. The load system will now be placed on the span such that the load W. and the resultant are equidistant from the mid point of the span.

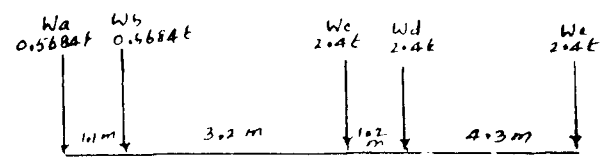
$$\text{Load on the girder A} = W - \frac{0.66}{1.14} W$$

= W(1—0.5789)

= 0.4211 W

But axle load = 2W

Therefore load on the girder A 0.211 W/2 = 0.2105 W



Wa	=	0.2105	x 2.7	—	0.5684t
Wb	=	0.2105	x 2.7	—	0.5684t
Wc	=	0.2105	x 11.4	—	2.40t
Wd	=	0.2105	x 6.8	—	1.4314t
We	=	0.2105	x 6.8	—	1.4314t
Wf	=	0.2105	x 6.8	—	1.4314t
Wg	=	0.2105	x 6.8	—	1.4314t
Wh	=	0.2105	x 6.8	—	1.4314t

Distance between we and resultant or the load system

$$\frac{(2.4 \times 4.3) + (2.4 \times 5.5) + (0.564 \times 8.7) + (0.56 x9.8)}{0.5684 + 0.5684 + 2.40 + 2.40 + 1.4314}$$

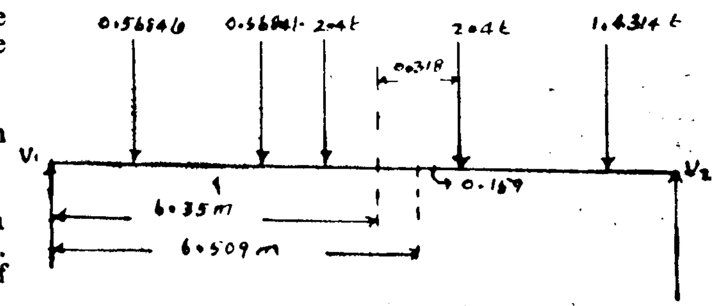
= 4.618m.

Therefore Distance between Wd and resultant

= 4.618—4.30 = 0.318

$$V_1 = \frac{7.3682 \times 6.509}{12 \cdot 7} = 3.7763\ t.$$

V_2 = 3.5919 t



(i) Therefore absolute max. B.M.=(3.5919 x 6.191)—(1.4314 x 4.3)

=22.2374—6.1550

=16·0824 tm.

=16·08240 kg. cm.

(ii) Impact factor $= \dfrac{9}{13.9 + 12.7} = 0.35$

B.M. due to Impact=0.35 x 16088240=562884 kg. cm.

(iii) B.M. due to D.L.

D.L. due to slab =20/100 x 114/100 x2400=247.2 kg/m.

D.L. due to self.
wt. ISMB 500 x 180 at 86.9 kg. m. = 86.9 kg/m.

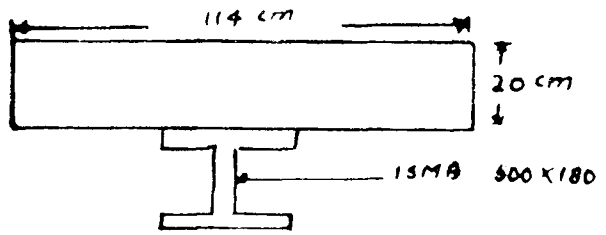
Therefore B.M. due to dead load = $\dfrac{543.9 \times 17.7^2}{8}$

=1701000 kg. cm.

Total b.m. =1608240 + 562884 + 1701000=3872124

Therefore %required=387212/1400=2765.80 cm^3.

Let us try 1 SMB 500 x 180 at 86.9 kg./m. The girder shall be designed as a composite section.



Area of steel (Joists) 110.74 cm^2.

Area of concrete=114 x 20 =2280 cm^2.

M.I. of axis=section about its own neutral
45218.3 cm^4.

M.I. of concrete section about its own neutral axis.

$$IC = \frac{114 \times 20^3}{12} = 75980 \text{ cm}^4.$$

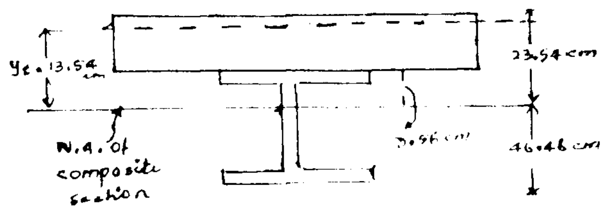
Yes=Distance between C.G. of concrete section and C.G. of steel section=25+ 10= 35 cm.

Area of equivalent steel section.

$$= As + \frac{1}{m} Ac = 110.74 + 2280/13$$

= 110.74 + 175.38=286.12 cm^2.

Ye.=Distance of C.G. of concrete section from n. a. of composite section.



$$= \frac{35 \times 110.74}{286.12}$$

=13.54 cm.

Yes.=Distance between n.a. of composite section and C.G. of steel section.

=Yes—Yc.

=35—13.54=21.46 cm.

Yes.=M.I. of equivalent section about n.a. of composite section.

$$= Is. + \frac{1}{m} Ic + As \times Ys \times Yes.$$

=45218.3 + 75980/13 + 110.74 x 21.46 x 35.00=134239.7 cm^4.

Check for bending stress consideration :

(i) Max. tensile stress in steel = $\dfrac{3872124 \times 46.46}{134239.7}$

= 1340.12 kg/cm^2.

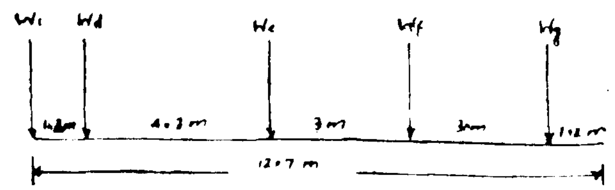
(ii) Max, comp stress in steel = $\dfrac{387214 \times 3.54}{134239.7}$ =102.11 kg/cm^2

(iii) Max. comp. stress in concrete

$$L/13 \times \frac{3872124}{134239.7} \times 23.54$$

=52.23 kg/cm^2.

Hence safe



II *Design of shear connectors* :

(i) Shear due to live load.

$$= 2.40 + (2.40 \times 11.5) + \frac{(1.4314 \times 7.2) + (1.4314 \times 4.2) + (1.4314 \times 1.2}{12.7}$$

=2.40+ 3.66=6.06 t=6060 kg.

(ii) Shear due to impact =0.25 x 6060 =1515 kg.

(iii) Dear load shear $= \dfrac{843.9 \times 12.7}{2}$ =5359 kg.

Total shear= 12934 kg.

hz shear/cm at the junction of concrete slab and steel near the support.

Sh =Vms/I=where V=Max. square feet. at support= 12934 kg.

ms.—Static moment of the transformed area of the slab about the n.a. of the composite section.

$$= \frac{(114 \times 20)}{13} \times 13.54 = 2374.70$$

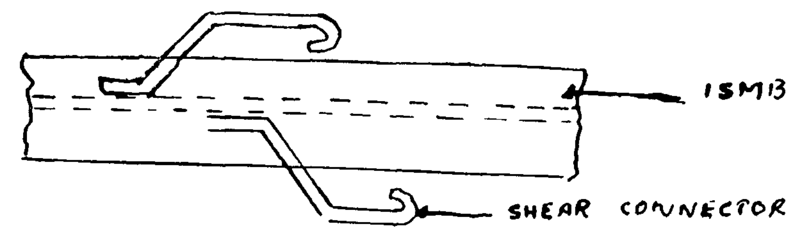
I=Ios=134239.7

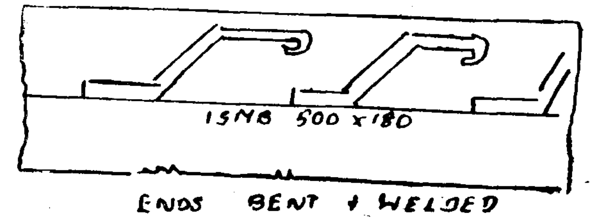
Therefore Sh $= \dfrac{12934 \times 2374{\cdot}7}{134339.7}$

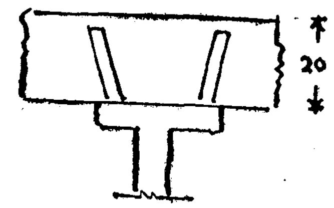
= 228·80 Kg./cm.

In case of connection between in situ concrete and the pre-fabricated steel unit the resistance to the horizontal shear shall be provided by mechanical shear connectors at the junction of the concrete slab and the steel beam or the girder. The connectors shall be capable of resisting the shear force between the slab and the structural steel member and at the same time prevent vertical separation of the steel member at the inner face.

Let us adopt bond or anchorage connectors consisting of mild steel bars welded to the flange of the pre-fabricated unit in the form of inclined bars with one end welded to the flange of the steel unit at each loop and the other end suitably bent.







Shear Resistance of anchor connector

Q=K σst. At

Where K =Coef = 1 =When the anchors with bend length of not less than 40 times dia of bar including rock in compression zone of concrete are used.

σst=Permissible tensiles stress in steel.

At=Cross sectional area of anchor bar.

Using 16 mm bars, Q=1 x 1400 x 2.01=2814 kg.

Therefore spacing=2814/228.8=11.99 cm (or) 12 cm c/c

As per IS 3 935—1966 pacing of anchors shall not be less than 0.7 times depth of slab or shall not be more than twice depth of slab. Hence 18mm rods will be used.

= Q—1 x1400 x 2.54 =3556 kg.

= Spacing $= \dfrac{3556}{228.8}$ 15.54 cm or 15 cm c/c

Design of Web stiffness :

Shear force =12934 kg.

Web area =50 x 1.02

Therefore shear stress $= \dfrac{12934}{5 \times 10.2}$ =13069/51.0

=253.60 kg/cm^2

= 945 kg/cm^2

Hence no stiffness are necessary.

But however, ISA 50 mm x 50 mm+6 mm may be provided as web stiffness at supports for 1.2m from support at 50 cm c/c.

Check for deflection :

W = Live load including impact

= 0.2105 11.4 x 1.5 = 3599.5 kg.

= (5.75 W x 0.6 x 6.05) + 5.75w x 5.75 2 x 5.75

1 3

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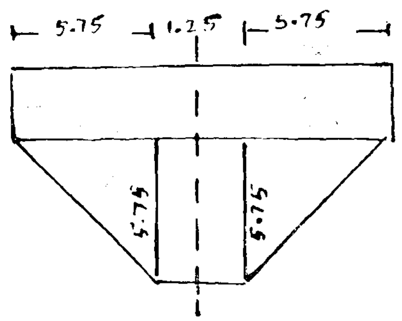

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= ———

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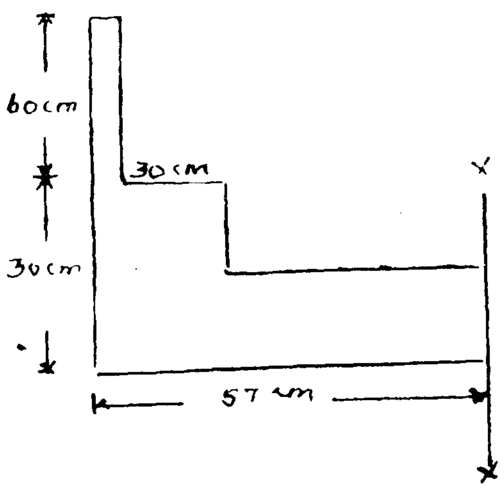
— 210.97 x 3599.5 x 10[6] = 2.69 cm

2· 1 x 10[6] x 134239.7

1/D = 1270/2.69 = 472.11

Hence safe.

CHECK FOR CANTILEVER MOMENT.



1. B.M. due to weight of parapet about x x
= 0.15 x 0.6 x 2400 x 49.5 = 10692 kg. cm.

2. B.M. due to weight of kerb = 0.3. x 0.3 x 2300 x 27.0
= 5589 kg. cm.

3. B.M. due to weight of slab = 0.57 x 10/100 x 2400 x 28.5 kg. cm.
= 3890 kg. cm.

4. BM. due to weight of wearing coat = 8/100 x 12/100 x 6.00
= 132.5 kg cm.

5. B.M. due to live load class A :—As the width of the loading. max load of 11.4 t is 50 cm. the live load will not come in the cantilever portion. As such the wheel load will be over the girder, for which condition, the girder has been designed. Hence live load moment will be nil.

Hence total contilever moment = 20254 kg. cm.
138160 kg. cm.
i.e., the design moment.

Results :

(i) Thickness of slab = 20 cm.
main reinforcement = 12 mm bars at 17cm c/c
distribution steel = 8 mm at 25 cm c/c.
concrete mix — M-200.

II Girder—ISMB500 x 180 at 86.9 kg/m.

III shear connectors —18 mm bars at 15 cm c/c ends bent and welded length of anchor being the bond length (40 x d).

IV Web stiffness — 50 mm x 50 mms. of 6 mm thick may be provided at supports for 1.20 m from supports at 50 cm c/c.

Details of construction—

The foundations of the bridge were tackled during the summer months when the flows in the river was meagre. Ring bunds were formed and with the help of shoring, foundations were excavated upto calculated scour depth. As an additional precaution, old 18 lbs rails were driven to depths of 3 ms. vertically to function as piles over the rails a mat of rods were placed and foundation concrete was laid to function as a R.C. raft. the masonry abutments and pier were raised to the required level and a bed block of reinforced concrete was provided on top.

Launching of Girders—

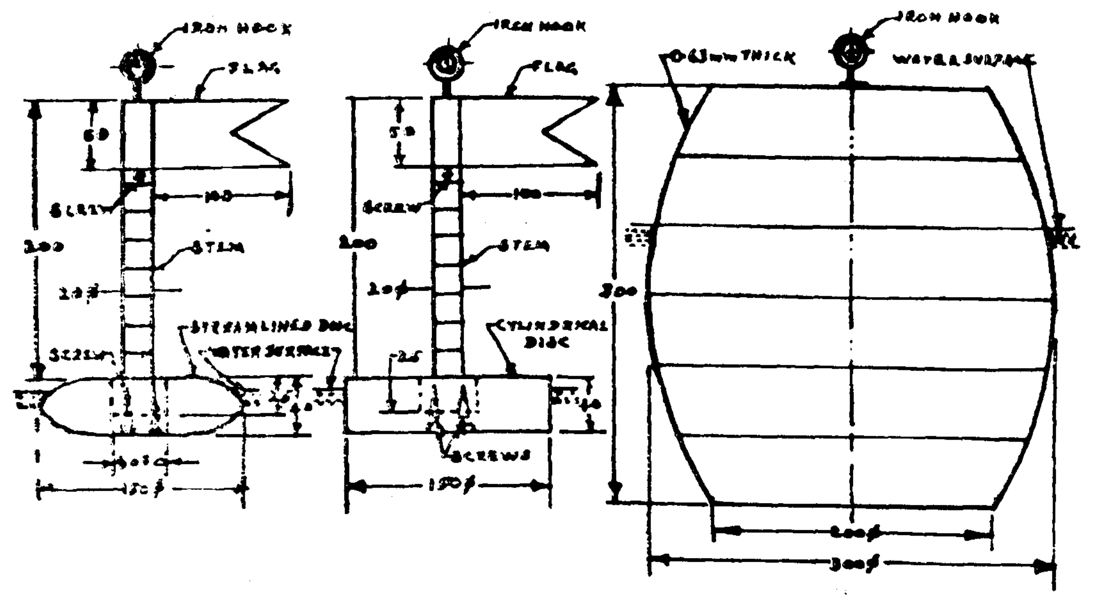
As the span was 12.2 ms. girders of size 500 mm x 180 mm. were fabricated to a length of 13 ms. with web stiffeners and were placed on the bed blocks with the help of tripods and pulleys. When once the girders were positioned the bottom flanges of the girders were used to support cross wooden scantlings which in turn supported the form work for the slab by shutting and wedging. The shear connectors in the shape of bent rods were then welded on to the top of the flanges of the girders at site and R.C. concrete for the road slab laid. Removal of centering was not difficult as the lower most cross wooden scantlings resting on the flanges, could be knocked out in a slanting direction from the flanges. The form work above was then easily removed. Thus the erection and removal of centering was done without reference to flow condition of the stream below.

Economics as observed—

There was a savings of nearly Rs.10,000 by the adoption of composite method of construction, over the conventional 'T. beams and slabmethod. A comparative statement between the cost of conventional type of decking over T beams and the composite method of construction is given below as per the actual field data. 3.660 m. wide Brdige—Span 12.20 ms.

Serial number and description of work.	*Conventional type of decking over T beams.*			*Composite methoa as adopted.*		
	Qty.	*Rate.*	*Amount*	*Qty*	*Rate*	*amount*
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1.R.C. C., 1:2:4: usfng 20 mm metal including centering charges nnd excluding cst of M.S. rods bendin and tying grills.						
a. T beam ..	22.1 m³	628.90/m³	13.899	..	..	..
b. Slab portion	22.3 m³	417.42/m³	9.517	24/m³	417.42 m³	1001
c. Kerb ..	12.2 m³	307.92/m³	3.757	7.1/m³	307.92/m³	2.187
d. Hand posts & rails.	1.4 m³	927.77/m³	1.299	1.4/m³	927.77/m³	1.299
e. Bed blocks	6.2 m³	278.27/m³	1.725	6.2/m³	278.27/m³	1725
II. Supply ofM.S. rods includinng bending & tying grills.	11.5 M.T.	267.80/Q1	30.797	6.5/M.T.	267.80 Q MI	117.40 2100/
III. Supply of R.S. Js. 500 m.m. x 180 mm size.	..	..	..	9.0	MT.	18900
Total cost	..	..	..		60,99,400	51,536.00

Savings got by adopting composite method-Rs.9,458.00.



TYPE A (STREAMLINED)   TYPE B (CYLINDRICAL)   TYPE C (METALLIC)

All dimensions in millimetres

DETAILS OF SURFACE FLOATS.

From experience it could be said that gaugings with float method has some disadvantages. They are (1) Extra men and amount is required for the work (2) inaccuracy in (*a*) the deviation of floats from the line of run(*b*) difficulty in viewing the floats during sun shine or during wavy condition of flow (*c*) affected by wind and eddies and other practical difficulties.

Measurement by velocity rods—

The velocity rods are wooden rods or tin tubes about 1" or 2" in dia. and of length approximately equal to the depth of water on the line of the rod run. The rods are weighed to the extent that they float vertically with about 1" projecting above the water surface. It may be a square section also. The lower end of the rod should reach down to 0·94d. of the depth of submergance cannot be possible to give mean velocity, observed velocities have to be converted into average velocities by using co-efficients or correlations based on current meter readings.

A large number of rods are required to suit the varying depths of cross sections. The advantage of these over surface floats is that they give the approximate mean velocity and are not much affected by wind. They can be used advantageously in artificial channels in which uniform depth of flow could be obtained. The following relation between V mean and velocity (rod) can be followed.

$$V\ mean = V\ rod\ (1.012 - 0.116\sqrt{\frac{D-L}{D}})$$

where D is depth of water in feet and L is submerged length of the rod in feet.

Measurement of velocity by using current meter—

Gaugings with current meters are done by one of the following methods depending on the nature and dimension of river, channel, etc., gauged.

(a) *By Wading—*

Measurements are made by wading wherever the depth and velocity of the river or stream permit the member who is measuring discharge to reach all the measuring points and to hold the current meter in position. That is to say it can be used for shallow streams with low velocity not exceeding about 2' 0"/sec. or 0.6 m/sec. the maximum depth being 2' or 2½ (0.6 to 0.76 m.)

To mark the points of observation, a metallic tape may be stretched between two iron rods that have slits at their ends in case of small streams or channels of shallow depths. For lengths exceeding 200' and when the wind velocity is more it is better to use silk braided wire duly marked, earlier before it is stretched across the stream. The marking can be for 4' 0" length with any paint using special marks every 20' 0". For still greater lengths a galvanized telephone wire or twisted fence wire may be used.

While taking measurement the observer should stand below the guage line and to one side of the meter in order that he may not disturb its action.

Another practical method of avoiding obstruction that is followed is to have the wading rod with the meter vertically tied to an horizontal pole 5 to 6' long which is held by two men one at either end and standing apart in such a way that the meter is not influenced.

(b) *Measurements from Bridges—*

Occasionally bridges may be found located as regards stream gauging. Before starting the gauging work all measuring points are marked with paint on the rail or the string piece of the bridge. The initial point should be carefully referred to any permanent object and its description noted in the record. The meter is suspended by a rigid rod or thin wire cable. Masonry bridges with piers are generally unsuitable for gauging on account of cross currents and eddies which are invariably formed. A light temporary bridge of casurina, bamboos, rails with a few intermediate supports as possible may be constructed at a very small cost in many cases. Both velocity and sounding points are painted as referred above or rails on the longitudinal stringers tied on the decking.

Current meters used for measuring velocity are suspended either by the rod or by a thin wire cable (1/8") In case they are cable suspended a small hand winch mounted on a wooden platform with a pulley at one end is used to enable easy raising or lowering of the meter. If rigid rod is used, suitable clamping arrangement at platform level is to be made. The observations are made invariably on the upstream side of the bridge. An auxiliary cable with a stay line is used in case of high velocities, to ensure verticality of the suspension rod. In case it is found that intermediate piers are causing obstruction, the upstream platform of the bridge is cantilivered and the section fixed so as to free from the influence of the posts.

The observation of velocity by a bridge construction is an ideal arrangement and however has several factors which preclude its adoption.

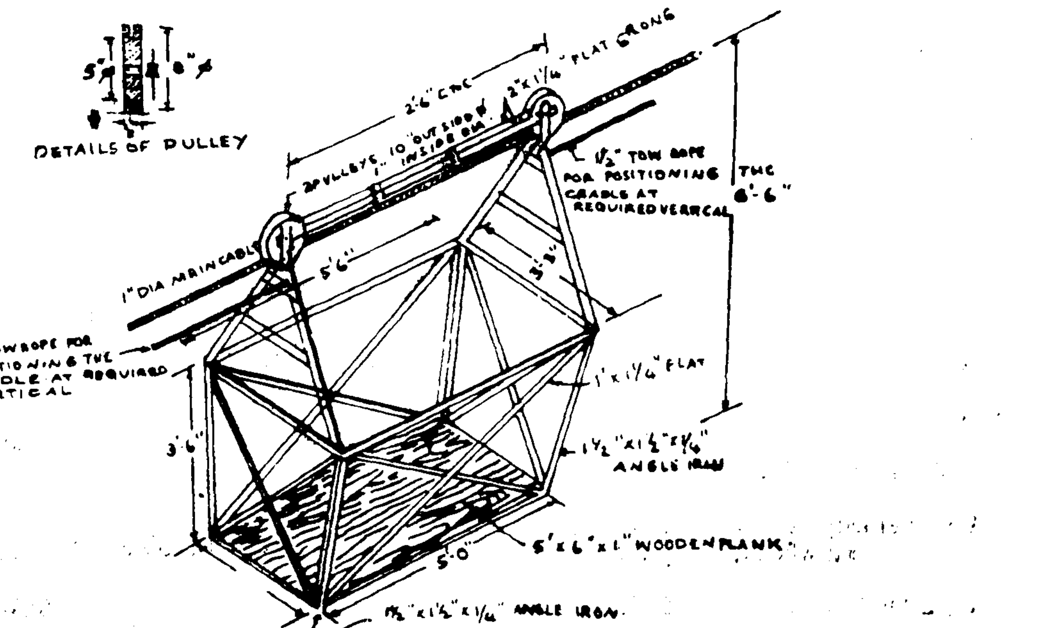
It has the advantages such as savings in initial cost of erecting a cause way or purchase of boat etc., accessibility t all ti.n:s of year and flow being in a confined section.

(c) *From a cradle—*

Where it is not permissible or practicable to have a temporary bridge across the channel, observations are conducted from a cradle or car suspended from an over head cable, stretched across the channel over suitable supporter or trestles.

Cables afford ready means of gauging streams upto 1000' 0" and more in width. The advantage in using this lies in the fact that the station may be placed at the most favourable location independantly of existing structures. Generally one end of the cable is anchored securely while the other end is wound round a drum of 5 or 10 ton winch to enable the cable and with it the cradle being raised or lowered as required. The meter is uespended by means of the meter cable which is thick nsough to afford a comfortable grip and not cut the hands. In swift water a head line is used to hold the meter in a vertical position, the head line being made of No. 10 galvanized iron wire long enough to reach from the top of meter hanger to which one end is attached to a rope above the surface. The rope is carried to pulleys on a stay line some distance upstream and back to the car. The member taking measurement adjusts the stay line as required.

Over the cable is run a cradle which carries the observer, the meter, the Electric cells and the sounding rod, telephones, etc., The cradle can accommodate2 persons. Before the cable is stretched and the equipments are got fixed, a site survey and cross section is taken at the place, and the cable line is ranged out to be normal to the axis of the stream.



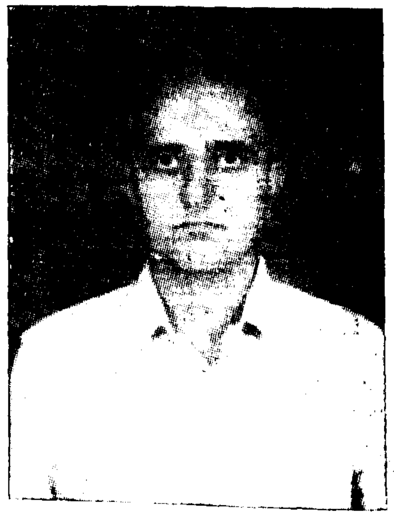
DETAILS OF CRADLE
(ALL JOINTS SHOULD BE WELDED)

GAUGINGS

IN STREAMS,

CHANNELS

AND RIVERS



BY

Er. M. A. VARADARAJAN, B.E.*

1. Introduction—

Assessment of probable discharge in any river during different stages of flow can be done only by gaugings. The availability of water and proper distribution can be properly judged by gaugings only. One of the main function of an Irrigation Engineer is how best to utilise the available water economically and to the maximum advantage. Gauges have been observed daily or often at masonry structures across rivers or channels. By this indirect method discharges are estimated from gauge discharge curves. Such gauge data has enabled irrigation Engineers to have approximate estimation of the maximum river discharges using area velocity method, wherein velocity is arrived by means of surface slope, Mannings formulae, etc.

During cultivation season an Irrigation Engineer will be confronted with the problems of the correct discharges that has to be sent through different branches of channel at different times of a year either through a sluice or regulator etc. This discharge is calculated by the vent area and head of discharge adopting suitable co-efficient of discharge. This system is only approximate and is found that actual realisation is in variation with the calculated, the formulae used being purely empherical. A more accurate method would be necessary for achievement of better results.

For this purpose different methods of gauging the flows are resorted to.

2. Selection of site—

Establishing a gauging station must be done with careful examination. With the help of topographic maps selection may be better made. Sites which may tentatively appear suitable from a study of the map may then be inspected to ascertain whether it satisfies the conditions noted below. It is also essential that the Engineer watches the behaviour of the river during monsoon and then decides on final selection. The correct measurement of discharge (whatever method is adopted) requires a section with an even steady flow, free from eddies and cross currents. To ensure this the river bank and bed should be reasonably straight and stable both in the upstream and downstream of the selected site for atleast 4 times the normal width of a river or 0.8 km. whichever is less. The bed slopes should not be subject to sudden changes and also should not have reverse slope (slope against currents) and should be free from pools, etc.

It can also be 10 to 20 times the width of the channel in stable regime and uniform cross section and ponded sections should be avoided. Proximity of masonry works such as sluices, regulators, bridges and drops, etc., causing disturbance in the flow should be avoided. The gauge stations should be in a position as to have free access at all seasons of the year. 'There is no necessity for measuring the discharge at the same section where the gauge is erected unless the same is also a best site for measuring discharge. The place of measuring discharge can be even be miles away from gauge site but the only limit on the distance apart between the measuring site

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and site of gauge is that the former must be situated such that no inflow or outflow occurs between the places. In such a case a temporary gauge may be fixed near the measuring place and both gauges read at short interval. The character of the stream bed, whether of sand, gravel, boulders or rock and whether it is shifting, regime or changing has to be checked. The relative position of reservoir, inlets considering the effect of these on gauge results may be noted. Available required structures from which measurements are to be made can also be a factor. Selection of a gauging station, should not however be unduly influenced by the existing bridges or other convenient structures for supporting the gauge or from which measurements are made. The introduction of automatic recorders will be successful at isolated places.

When a gauge station is located at the downstream point of a confluence of a tributory or drainage to the main river course the place should be a Kilometre length below or distance equal to 3 times the width of the main river whichever is smaller.

The site selected should not be exposed to wind unduly. It is preferable that the river upstream and downstream be clearly visible from the gauge site selected for a distance of about half a mile or 4 times the width of river whichever is less.

The approach to the site selected should not as far as possible be through any private land or road, etc. Availability of easy and cheap facilities for transport of construction materials, equipment and personnel should not be lost sight of in choosing a gauge site.

3. Measurement of velocity—

Of the various methods used for the correct measurement of discharges, the velocity area method, involving measurement of velocity and depths at various number of points in across section is the one commonly used in measuring the discharge.

4. Velocity Area Method—

In this method, the velocity of water is measured directly by noting the time taken by a float or rod or drum or any object over a measured length or by the current meter or by measuring the slope and calculating discharge by the empherical slope formulae.

Measurement of velocity is done directly by noting the time taken for (1) the passage of a float (2) rod over a measured run (3) indirectly by noting the rate of revolution of the wheel of a current meter or by slope measurement. Measurement of velocity by float even though is easy, the results are not as accurate as with current meter. But occasions may arise when it may be impossible to observe velocity by a current meter when the velocities are too high and beyond the range of current meter.

5. Measurement by float—

A length of about 100'0" (for small canals) and 200 to 500'0" (in rivers) in which section the flow is uniform may be selected and the same area divided into two length by transverse dividing line I, II and III set out at right angles to the direction of flow by poles erected on the margin of the stream. The float method can be adopted when there is not much of wind and current velocity is not less than 1/4 feet/second. In any case the minimum distance between two reaches should be such that the time taken for a float to transverse from I reach to II reach is not less than 20 seconds. About 50' 0" above the upstream line I, a chord "AA" tagged or otherwise marked at suitable known intervals is stretched across the stream. Corresponding points in I, II and III reaches are also marked by means of flags erected on either banks. Floats will be released at successive point "AA" from a boat (The releasing of floats from 50' point is only to see that the floats attain the velocity of flow of water when they cross the I reach, etc. The time taken by these floats to pass in succession line at I, II and III will be recorded by an observer stationed at the margin for the purpose and provided with stop watch. Several floats (generally not less than 3) are let down successively from each point and the time for each run recorded. When the float does not fairly run and truly down the section through out the run as indicated by the float passing with marked deviation from the corresponding points in the sections I, II and III or where the time of any run greatly differs from others the observation is discarded and an additional one is released. To facilitate correct observation, three persons may be stationed along the three cross sections line to give signals by means of flags or whistles when the float crosses the respective cross sectional reaches. Time taken between passing of the float in two reaches is noted by a stop watch and to be read correct to 1/5 of a second. In using the float the time of passage over a measured course consists in the direct observation of the average velocity of that thread or filament of water in which the float has travelled over the course and as such accuracy depends on the uniformity of velocity through out the length of run as well as on the sufficiency of the run. In small channels the floats can be released by men wading in water and for rivers when there is more depth of water the floats are released by boat, or bridge or by a cable way. The spacing of the points from which floats are released depends on the facilities available at site and time available and width of channel

Surface floats can be wooden disc. 3" to 12" dia. corked bottles, partly filled with water, sealed kerosene drum cocoanut, palmyrah, fruits, oranges and lime. It is preferable to use weighted floats greater part of which is submerged in water since it should not be affected by wind. For bigger depths for streams in flood, a kerosene drum properly weighed and sealed and provided with a coloured flag on top may be better. Different coloured flag may be used to avoid confusion. The specifications for surface floats can correspond to the I.S. 3911—1966.

Velocities are observed at various points, and observed surface velocities is reduced to mean velocity in the vertical by a reduction co-efficient from 0.80 to 0.90. The areas used in these float measurements are the effective areas of the sections of the river over which the runs are made and are determined by averaging the areas of the C.S. of the stream measured at the three sections.

The measuring points are generally indicated by paint marks on the second (auxiliary) cable 1/8" dia. stretched right across the stream so as to eliminate due to sag of cable. To ensure verticality of the rod stay line with stay line arrangements is adopted. If the span is more, the observation points are fixed and alignment pegs is made in the form of rays radiating from an apex point. These pegs are fixed permanently on the ground (banks) with reference to the angles formed by the rays to a baseline by means of a Theodolite.

(d) *From Boats*—

This method is adopted only in cases where all the above methods are not possible or convenient. Boat measurements are occasionally used. The boat used for this work should be especially equipped so that all influence of the boat on the current measured is eliminated. The movement of the boat from point to point along the section is ensured by one or a set of two compound pulleys running on a wire cable stretched across the channel upstream of the gauging site. Two Manilla Ropes tied to the beams supporting the platform on the boat are passed through the lower pulley showing the arrangement. During observations special care is taken to keep the boat steady and in proper position with reference to the gauging section by means of stay wires and Bamboo poles, etc. The wire cable suspension of the current meter is the one commonly used being most suitable, meter being suspended from a cantiliver arms at a sufficient distance from the boat to avoid the velocity to be measured, being affected by the afflux and disturbances near the boat.Current meter, gauging section is usually located at about 25 to 50 below the cable line and is defined by fixing posts suitably painted on either bank. The exact positions on the observation points on the current meter gauging lines are fixed either by having them marked on auxiliary cable or by the alignment pegs described in the previous para.

Whenever current meter immersed does not remain steady then velocity observed will not be accurate since the current meter will record the velocity relative to the moving boat. It will be less than the actual velocity. As reported in previous para when the boat drifts the drift velocity have to be found and added to the velocity recorded by the current meter to get correct velocity. As per the numerous datas collected the following formula will have to be used.

$$V_p = 0.21 + 0.98\ V_b + 0.98\ V_d.$$

where V_p = True velocity

V_d = Drift velocity

V_b = Velocity at observation point with the boat when drifting and

$$V_d = \text{Drift velocity} \quad \frac{\text{drift i ft.}}{\text{120 sec. (i.e. period of Observation)}}$$

General application of this equation needs to be tested.

Drift velocity may be observed by (1) Flag method (2) by Box Sextent or Marine Sextent (3) by Theodolite.

Discharge estimation on account of boat obstruction is likely to be incorrect unless precautions are taken to avoid this. For this purpose meter should be from the bow of a flat bottomed boat using an extended boom facing upstream.

Determination of verticals—

The quantity of water flowing in a stream in a section is worked out by totalling the components of the discharges obtained by sub dividing the cross sections into compartments by verticals. In order to determine the areas accurately the verticals are spaced sufficiently closely. Number of observations and their distances apart are dependent on the practical considerations of the configuration of the channel and the time available for gauging. Number of observation points may be so fixed so that the whole observation can be completed in a period of about 4 hours.

The velocity is more in the centre of the stream than near the banks and hence observation points are to be closer at centre area. The following is the guide line for fixing observation points.

Small channels up to 20' bed width— 2, to 4'0"

Small channels of 20' to 50' bed width—4' to 8, 0"

Small channels over 50' 0" bed width —8' to 15' 0".

The observation point should be marked with reference to any permanent guide mark on the bank. When observation points are marked on cable, the distance should be checked with a steel tape occasionally as the cable may stretch or sag and the tags slip.

If observations made by wading or by boats with a flexible cable for marking off the observation points, care should be taken to see that the cable is not pulled off the line.

Method of marking varies with the method of discharge observations. If the width of the stream is upto 500' 0" steel wire rope can be stretched using a winch if necessary. Correction is applied to account for the sag. Formula giving the correct span S in terms fo actual length of the line L and Sag

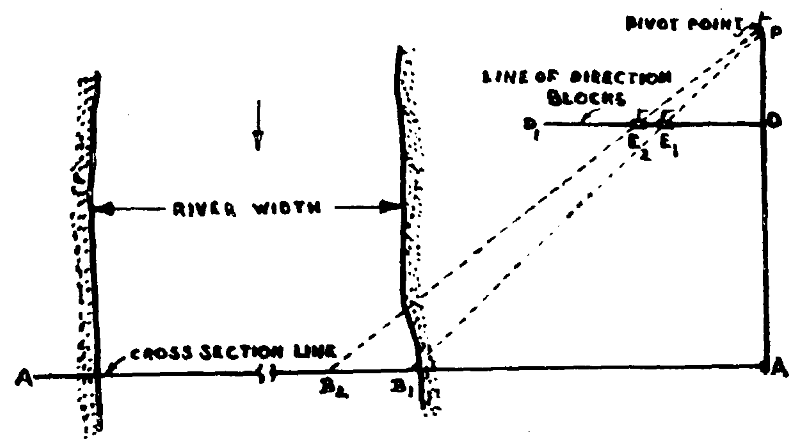
$$\frac{DL - S(1+8\ D2)}{3\ S2} \quad \text{Sag D in}$$

the line can be obtained from the relation

$$D = \frac{SxWS}{8H}$$

where W is the weight of the line in pounds/foot length and H is the Horizontal tension in pounds.

For longer lengths a cable line erected on towers at either end is used on which a cradle is mounted and moved to the desired vertical. The method of marking observation points is detailed below.



Pivot point layout

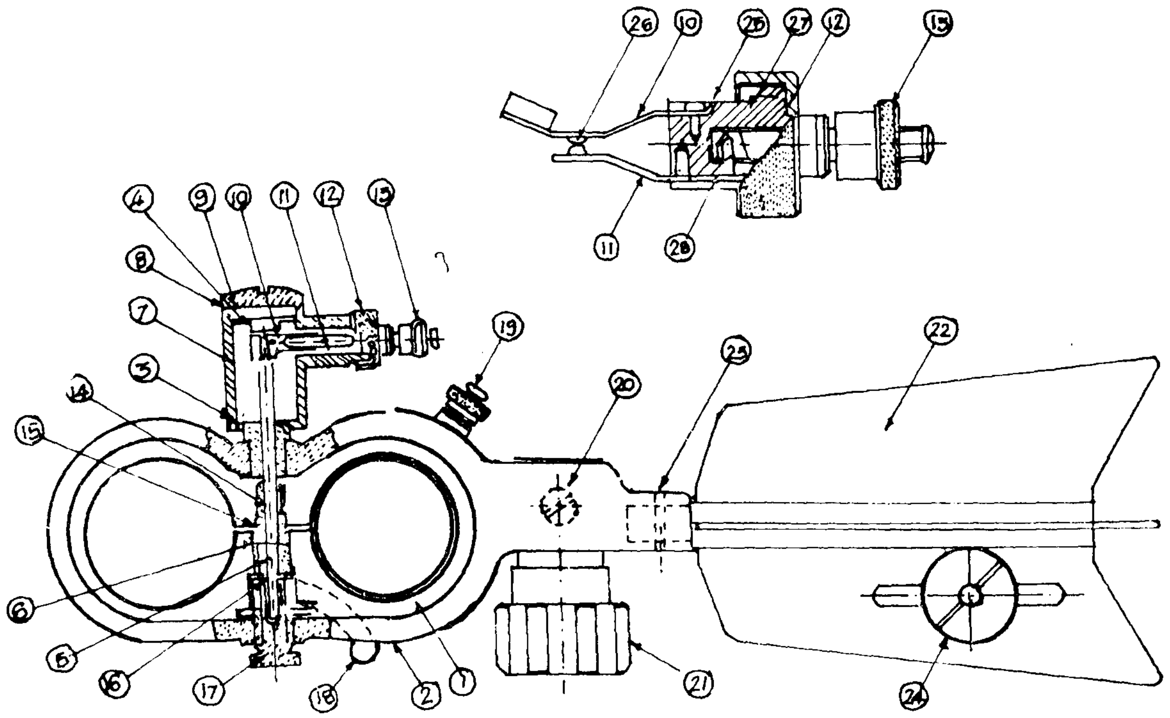
From point A a line AP at right angle is laid. This should generally be of about 1000'0" or up to half of the width of river. From Point D on AP, a line DD1 is drawn parallel to cross section. The ratio PD to PA is generally 1:5 on the line DD1 points are marked at fixed intervals depending on the distance between verticals. For example for observation point 100 ft apart points E1 E2 have to be 'fixed 20'0" apart so that the radial rays from P passing through these points will intersect the cross section at B1, B2, B3, etc., each 100'0" apart. During gauging operation the boat can now come to a point on A A1 and anchored at B1, B2 etc., when it corresponds to point E1, E2, etc., in line with P. Concrete or masonry blocks with pipes fixed at centre so as to insert flag during observation days will be enough, upto half the width of the river the blocks can be conveniently constructed on one bank and for the other half the blocks can be put up on the other bank.

Current meter is used to obtain the velocity at different points on the transverse section of a stream and thus divide up the section of flow into a number of compartments, the discharge of each of which is separately calculated. While measuring velocity the current meter is immersed in water to the calculated depth and allowed to attain full velocity and able to hear undisturbed continuous sounds of the bucket wheels of current meter. If the velocity is of the order of one ft/sec. it may take even a minute to attain the above stage. As a general rule velocity observations with current meter at each vertical are generally made by the point method the meter being successively immersed to 0.2 d and 0.8 d of depth of water at each point. If the depth at the observation point is less than 2¼ feet and exceeds 1¼' the velocity need be measured at one point viz., 0.6 d. If the depth of water is less than 1 1/4' the current meter is immersed only to 0.5 d and at no time the current meter should be kept closer to 6" from bed. In cases where great accuracy is not required and a saving of time for gauging is necessary on canals and distributaries not greater than 6" in depth, the one point method can be adopted. In cases where the flow is not ideal the necessity for the application of a co-efficient to derive the correct mean velocity of the vertical from these observed point velocities have to be examined by vertical observation and correction co-efficient if any determined and applied. The meter reading is made for 150 seconds or the time for 150 revolution is measured whichever is less. The two methods viz., to note timings for specific number of revolutions and to note revolutions during specific time can be adopted. But as counting of fraction point a revolution is not possible the former method is only adopted. Time is generally read correct to 1/5th of a second using reliable stop watch.

Under current meters though there are different types of current meter like Fteley—Stearm, Harkell and Hoft of the propeller type, price meters like Gurley, Watts of the cup type are used and are more convenient to use on account of its simplicity in construction and design. Both types take low friction factor generally negligible for velocities higher than 0.5 ft. per second.

(i) *Vertical Axis cup type meter*—

Cup type meters are known as price meters. The important ones are Watts and Garley. This is versatile instrument and combine accuracy with ruggedness.



1. Bucket wheel cup
2. Yoke
3. Spindle
4 Upper spindle bearing
5 Pivot
6 Bearing cup
7 Contact chamber
8 Sealing washer
9 Cap for contact chamber
10 Contact spring
11 Insulated contact bar
12 Contact breaker clamping ring
13 Terminal knob
14 Spindle collar
15 Bucket hexagonal lock nut
16 Pivot lock nut
17 Pivot holder
18 Bucket-raising lever
19 Earth terminal
20 Central adaptor pivot
21 Adaptor clamp
22 Tailfin
23 Tail fin securing pin
24 Tail fin counterweight
25 Contact to earth
26 Platinum
27 Vulcanite mount
28 Insulated contact bar securing screw

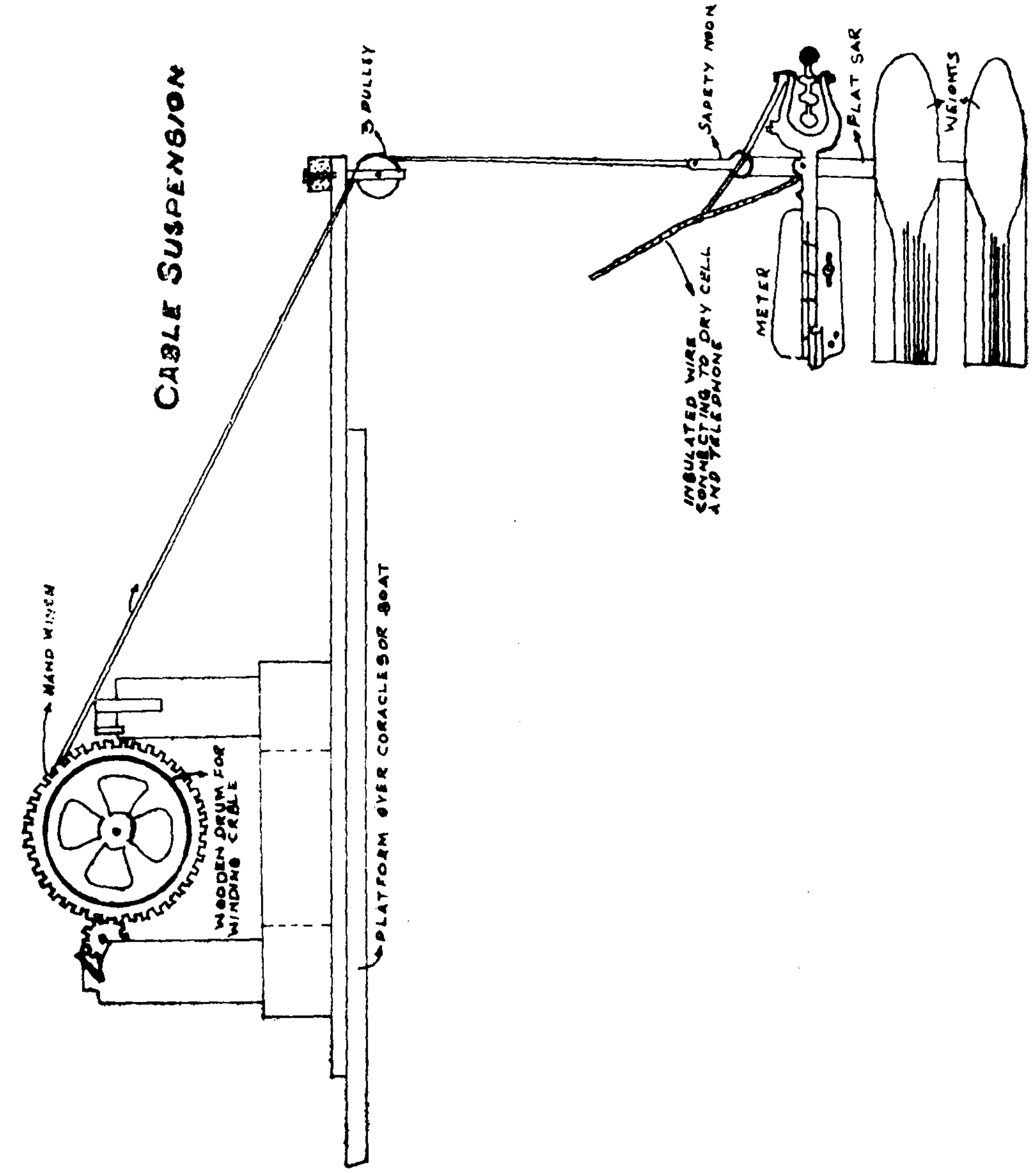
CURRENT METER (CUP TYPE)

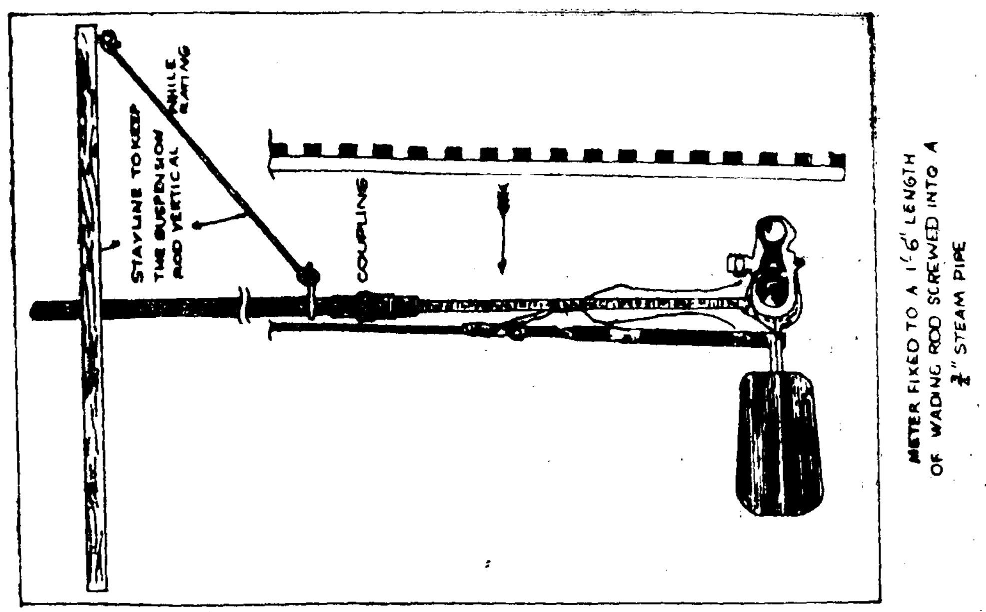
(ii) Horizontal axis screw or Propeller type current meter :—

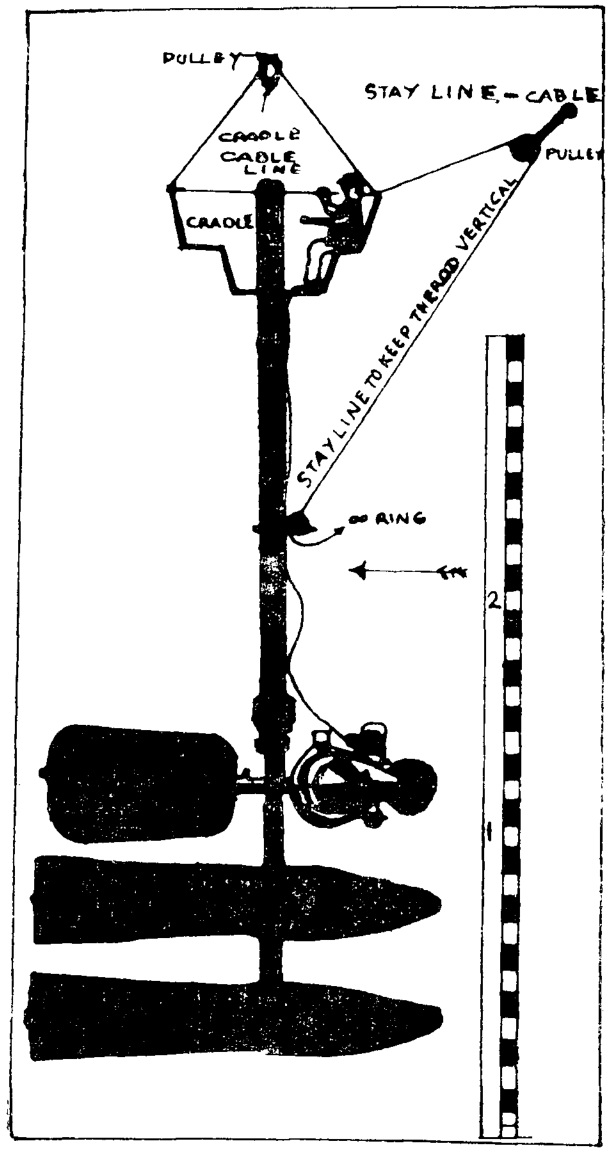
Meters like Fteley, Ellis, Gettner, Hoft, Ott etc., are a few. These current meters are very much suited for measurement in clear water within the range stipulated by the makers. Some of these are very well suited for measuring low velocities but highly sensitive. These are not commonly used. when current meter is used one or the other of the standard suspension (Rigid rod orcable suspension with weight) for which the meter is rated is used. If the rigid rod is , used, care should be taken to see that the meter is held with its axis normal to the gauging section, i.e. with tail vanes exactly in the line of flow. To facilitate this a pointer should be fixed to the top of the rigid rod to indicate the exact direction of the said axis when the current meter is immersed in water. If observations are made from a cradle a stay line will be necessary for velocities over 2'0"/sec. to keep the rod steady and vertical. The stay line (cotton rope of 1/2" dia) should be tied to the rod of about 3'0" above the meter and passed through a running pulley on the stay line cable stretched across the channel 30' or 50' up stream of the gauging site. The rope may be tightened sufficiently and tied to a hook fixed in the side of the cradle.

Corrections necessary while using sounding lines—

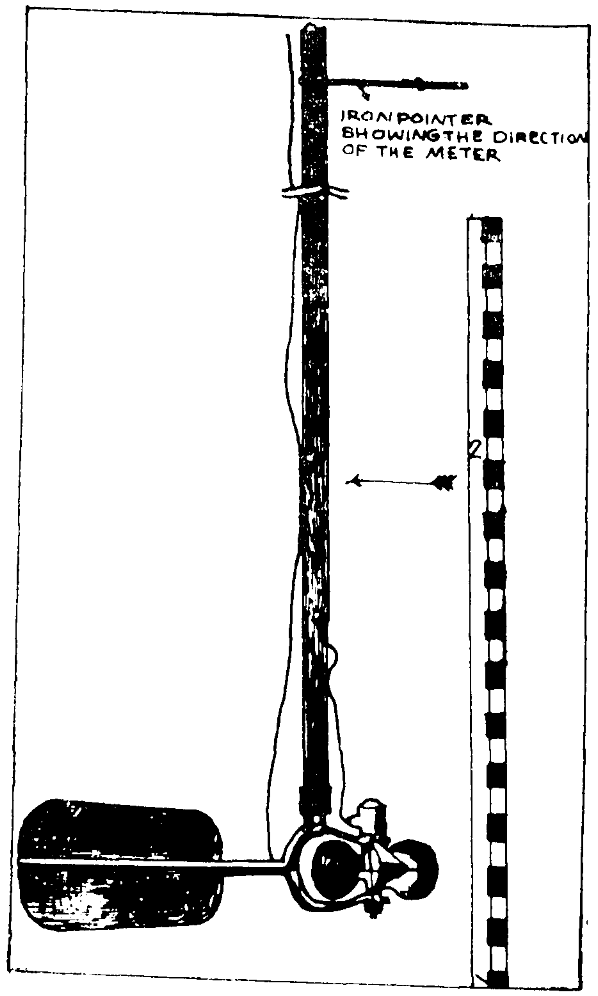
The draft on the sounding cable may cause it to deflec from vertical. Measurement of depth will be incorrect.







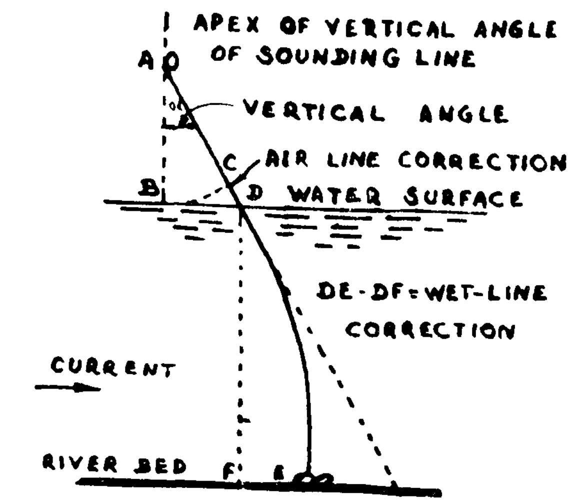
METER SLUNG FROM A $\frac{3}{4}$" STEAM PIPE



METER FIXED TO A $\frac{3}{4}$" STEAM PIPE

When the meter is suspended by a cable 1/8" wire cable with one fish weight of 30 lbs. or 2 fish weights of 17 lbs. each below the meter is generally used. Ordinarily in channels where velocity is $2\frac{1}{2}$' maximum only, the drag of the cable with meter and weight is found to be within very low and permissible limits, and no correction is necessary. If higher velocities are involved additional or heavier weight of 60 lbs. are used. Generally sections are chosen so that the flow is axial or normal to the cross section. But where the flow is found to be oblique, the angle of obliquity of the current (flow) to the normal is measured by using a deflecto meter.

When the direction of flow of water at an observation point is not normal to the cross section but is inclined away from it, a correction has to be applied. This is done by measuring the angle of deflection of the current from the normal and deriving the equivalent normal velocity. When the meter is to be immersed in water in order to ensure that the meter is held in the line of the current, rpointer may be attached to the top of the pipe or rod and this is at the outset so fixed as to be parallel to the axis of the meter. The meter so immersed that this pointer will be normal to the cross section.



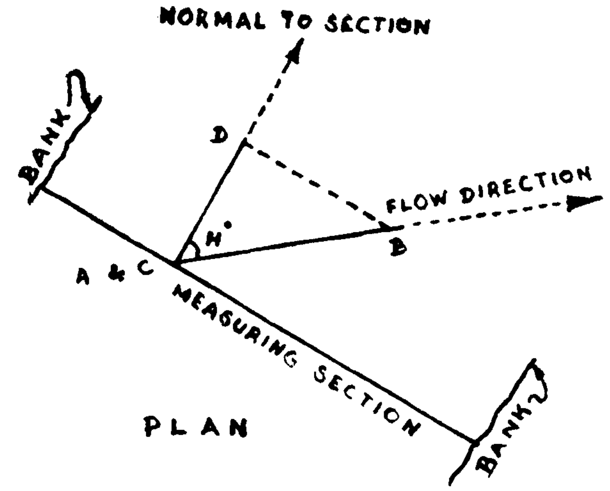
Deflection of the sounding cable from true vertical due to drag of the current.

CD is the air line correction and DE—DF=wet line correction. Air correction CD =(AD—AB)+ (1—Cos.—1)AB.

Wet line correction—

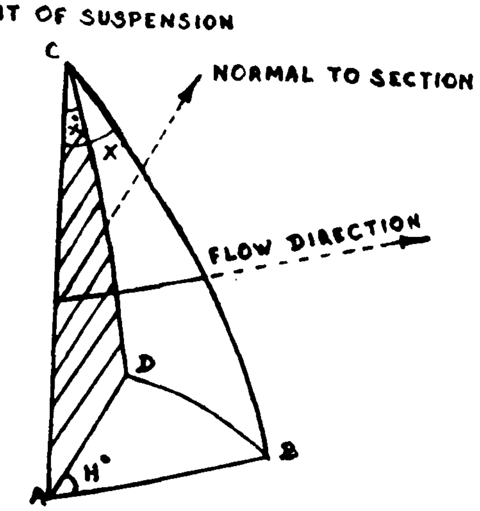
The wet line correction is = DE—DF and is on account of curvature of the cable caused by flow of water. The flow sometimes may not be perpendicular to the section line. Horizontal angle between these two introduces an error in the observed angle. α This is because the measured vertical angle, is the projection of the actual angle X as shown in sketch. If the horizontal angle between the direction normal to the measuring section and the flow direction is H° from tan $X = \frac{\tan \alpha}{\cos H}$

The angle of deflection of the current from the normal is measured with deflect meter. The angle which the current makes with the section is read off directly on the graduated dial with the help of the pointer, which is rigidly attached to the tail vane below the movement of which is regulated by the direction of current. For systematic record of depth observation the following form can be used.



PLAN

POINT OF SUSPENSION



ELEVATION

NOTE :—

$$\frac{\tan \alpha}{\cos H} = \frac{AD}{AC} \times \frac{AB}{AD} = \frac{AB}{AC}$$

$$\tan X = \frac{AB}{AC} = \frac{\tan \alpha}{\cos H}$$

Error in vertical angle due to horizontal angle

FORM

Record of Cross section.

Station.................. River System..........

Name of stream.................Gauge No...........

Zero of gauge................Method of measurement..
........Started............hr..........19 ....
Gauge reading ....*

Completed.............hr..........19 ....
Gauge reading...............*

* Water level..............

Cross section No.

Measuring point.	*Angle or distance.*	*Reduced distance.*	*Depth.*	*Average depth.*	*Area of section.*

Name of observer..........Designation.............

Signature.................. Date..................

Correction factor is to be subtracted from the observed velocity when the current is at an angle to gauge line. With the angle observed either with sextant or any other means.

Readings of the gauge or gauges to be calibrated and of the site gauge, should be observed at the commencement and close of each gauging. When the gauge reading is likely to alter appreciably during a gauging readings the gauge should be observed at intervals of say 15 minutes during the gauging and the gauge reading at the time of each velocity observation then reduced by interpolation The rear gauge reading for the gauging will be the average of gauge readings for the various compartments.

The observation should be systematically entered in the form noted below and the gaugings numbered serially.

All the particulars of condition which may affect the flow in the channel, such as varying conditions of draw off, drainage inflow etc., should be recorded. Any draw off or inflow in the reach between the gauging site and the calibration gauge should be determined. The strength and direction of wind with reference to the direction of flow should be noted in the field book. When the gaugings are being conducted to determine the coefficient of discharge at sluices, weirs or drops all necessary levels, vent openings etc., at such works should be recorded for each gauging.

Current meter gauging No.........dated........at....

Meter No......for rigid rod suspension with flat bar and stay line.

Case......site gauge at start......weather...........

Stop watch........calibration gauge at start. Time at....... close.

Wind strong......attached......mano or penta counters site gauge at close........

Time at start......observers i. ii. Calibration.... Gauge at close ....

Measuring station N.	Distance from initial point.	Distance between adjacent measuring point.	Depth of measuring point.	Point of immersion.	Depth of immersion of meter.	Time taken (seconds).	Revolution	Revolution per second.	Velocity as per chart for meter used.	Reading Gauge. Time.
For example—										
20.	158.0 water edge.									
	161.5	3.5	1.0	—						5.96 6.30.AM.
	166.5	5	1.8	0.6	1.08	150.2	27	0.18	0.46	5.97
	171.5	5	2.7	—						
19................etc.										

Effect of wind—

Current meter observations are affected on account of wind. Considerable variations in velocities may result on account of wind waves especially near the surface. Wind incident on water surface generates oscillatery wave and induces velocity of oscillation and also net forward velocity. This depends on strength of the wind and on the fetch of water surface. If the direction of wind is same as that of flow of water wind waves are flattened, their heights reduced and error in the current meter observation minimised. If the direction of wind is otherwise, wind waves are started and their heights augmented and error is increased. There will be reduction in velocity the tolerance limit is about 3 per cent. The graph below indicates the maximum permissible wind velocity in knots to restrict error to 1 per cent and 3 per cent.

It might be said from investigation carried out that the temperature has practically no effect on current meter rating over the range of velocities tested and presumably even for higher velocities than 2.61 ft.sec.

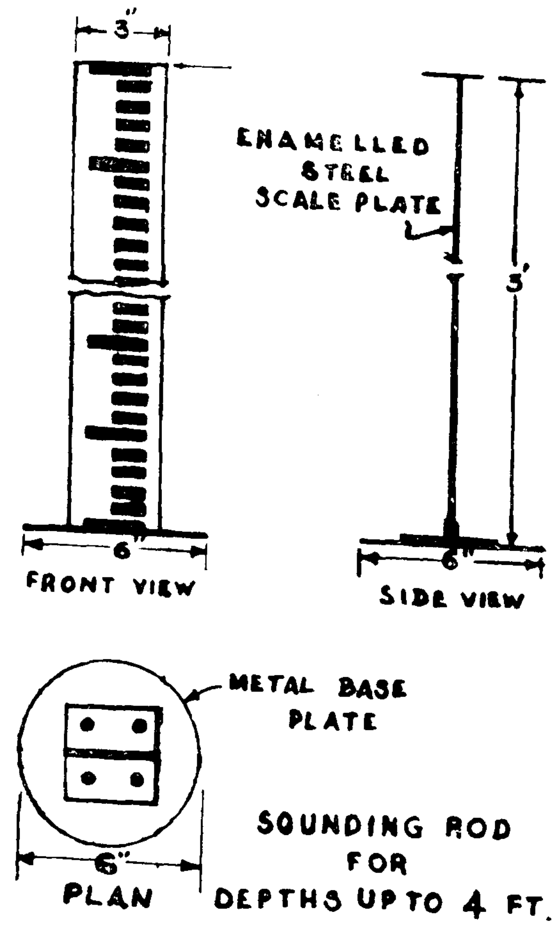
Measurement of Discharge by current meter is the best method at present which can be used for gauging streams. Slight difference in rating of a meter, small fluctuations in discharge turbulance of flow, wind effect, restricting the number of observation points, personal errors, in taking depth and velocity measurements are some of the items witch creates errors in current meter observations. Even under ideal conditions maximum accuracy under field conditions in a lined canal cannot be expected to exceed 98 per cent. For natural stream on account of so many factors the accuracy may be about 95 per cent or even less.

Different methods of computing discharges—

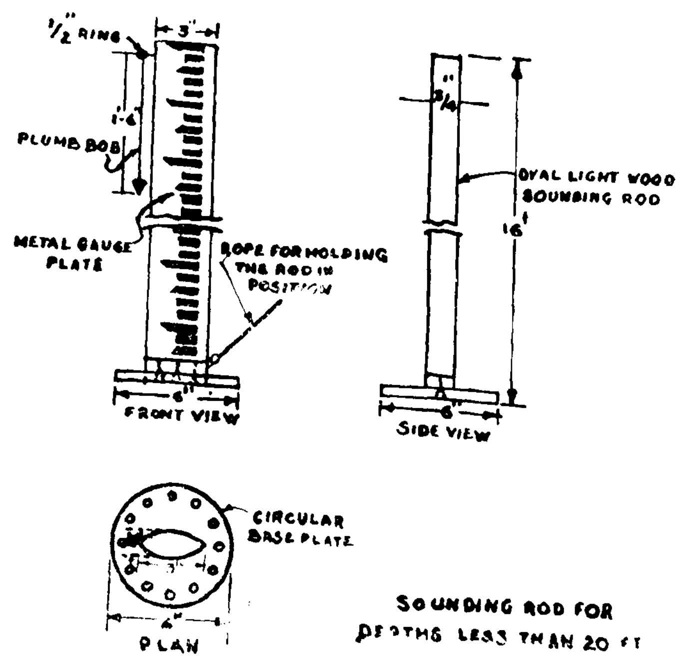
The discharge measurements are computed from the following datas (1) soundings at known intervals across the channel, (2) the velocity is observed by means of current meter at one point or two point method and mean velocity calculated, (3) the distance between observation points.

(i) *Soundings—*

Soundings to determine areas and to compute discharge is taken with a oval shaped rod (wooden) or 1/2 to 3/4" dia. pipe with a base plate attached to have a flat bottom in order to prevent the pipe sinking into bed. The rod is to be held vertical while taking soundings.



SOUNDING ROD FOR DEPTHS UPTO 4 FT.



SOUNDING ROD FOR DEPTHS LESS THAN 20 FT

Soundings are to be taken at close intervals in order ot arrive at the cross sectional area to a more precise extent and edges of berms, etc., should be noted carefully and distance from water edges noted for each gauging.

Soundings are to be taken afresh for each gaugings. In cases where the regime of a channel is not likely to change on account of hard bed or rocky bed, it is enough if standard cross sections are taken with the help of levels and soundings.

Measurement of depth and velocity by current meter are made successively. Two observations for depth measurements are to be taken at each vertical and mean of the two values used for computing area. If the difference in the two values are more than 5 per cent a third readings should be taken and mean of three values taken.

The rod is generally of 10'0" (3.05m) in length with a metal base plate as reported earlier. The observation of depths is likely to be affected if the observer stands in line with the rod as the flow is naturally disturbed. In such cases it is necessary that the observer stands 1' to 3' downstream of gauging line and minimum distance of 1' 6" to one side. Plumb bob can also be attached to the rod to see that when rod is immersed in water it is truly vertical.

An accurate and rapid method of measuring and recording depths is by Echo Sounder. This may obviate the defects such as (1) deflection of the line due to current, (2) tuly verticality, (3) sinking of base plate in the bed without the observer noticing it.

The equipment of an echo sounder can be installed in a boat (small launch). The only point to be borne in mind is that the maximum efficiency in echo sounding depends on having free flow water beneath the equipment that is absence of air, since air bubbles will form a sound block.

The field work portion of conducting gauging operations, across a stream consists of in measurement of depth and velocities at several points transversely across a stream by combining which the total discharge across the section is computed. Number of observation points and their spacing across the river are dependant on the width of river, unevenness of the bed and velocity distribution, so as to correctly allow for the variations in the configuration of bed and the volocities. They are spaced closer near the banks where there is sudden reduction in depth and velocity. The spacing of observation points may be kept from 40' to 60' at centre with a spacing of 20 to 30' near banks. Though velocities are observed at 40' to 60" points,soundings are taken every 10'0". velocities observed also should be at points midway between regular observation points (O.Ps.) on alternative days.

For greater depths than 6 to 8[1] especially at high stages, weight and line consisting of a 1/8" or 1/4" wire cable with a weight ranging from 15 to 100 lbs. tied to one end has to be used. Stay line has necessarily to be used on such occasions to hold the weight of cable in vertical position and prevent it from being dragged down by current. For deeper depths 13' to 15' 0" especially on windy days a ladder rod sounding apparatus (a broad frame intended to withstand the deflecting section of water) is used. The ladder is founded on two pipes and one thin iron flat bar. The flat bar is in front placed edgewise to the flow of water to obviate afflux and joined 1 1/4' at intervals to the two pipes placed in rear. The 18'0" sounding ladder is lowered and raised by a winch with a system of pulleys. These things can be used satisfactorily upto 18' 0" only. For greater depths than this weight and line arrangement will have to be used and if velocities are high and depth great the same will be dragged so that the measured depth will be more than the correct depth and require correction if there is considerable deflection. The deflection is reduced by increasing the weights used.

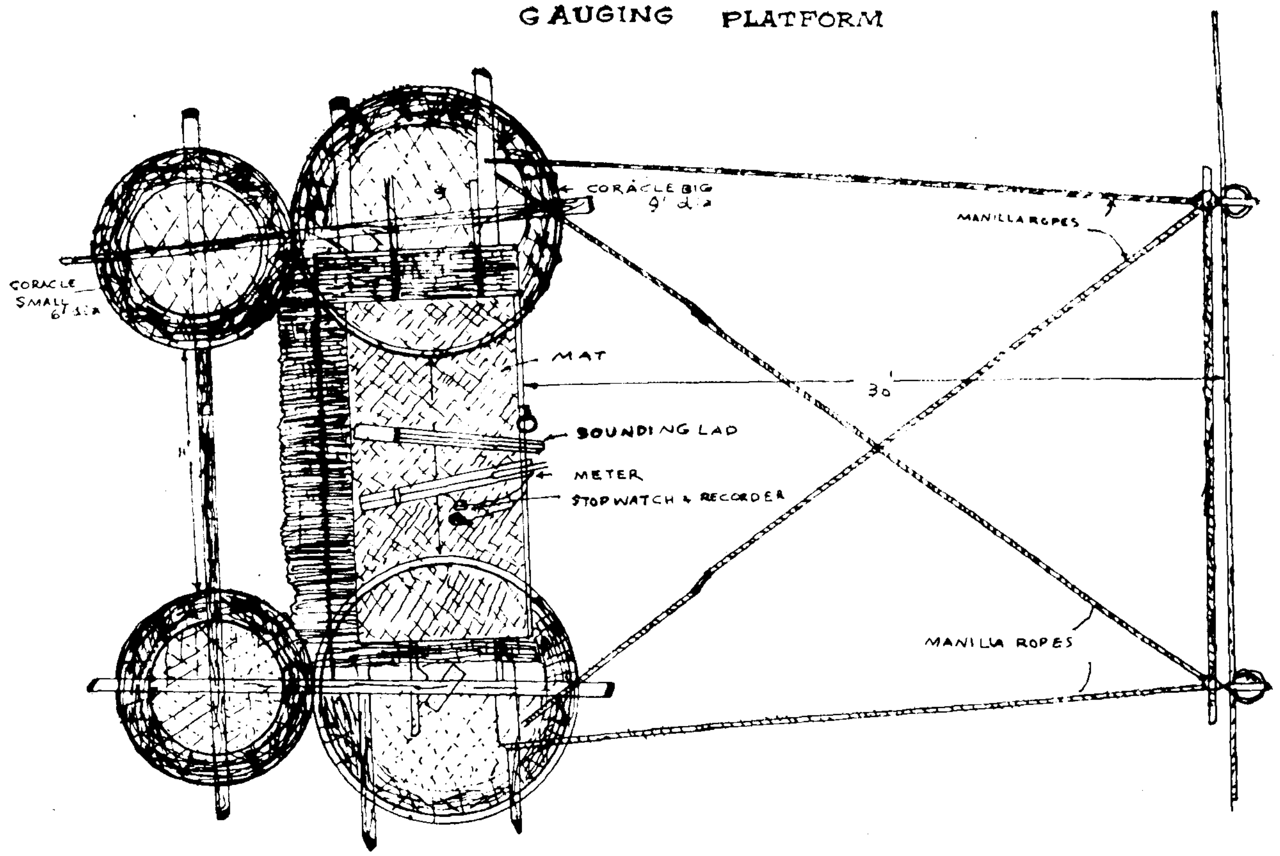
In making a measurement of discharge, the depth is measured first at each observation point to enable depth of immersion of current meter. The following may be taken as a rough guide in fixing the observation points in the bed portion of the channel.—

	Distance between observation points.	*Soundings for areas.*
	1	2
Small channels upto 20' width.	2 to 4' 00"	At all observation points.
Channels 20' to 50'	4 to 8' 0"	
Channels over 50' 0"	8 to 15' 0"	
Large streams	20 to 60' 0"	5'to 20'

The observation point should be located with reference to an initial point permanently marked on bank. An end point on the other bank at a known distance from the initial point will be useful as a check. If the observations are made from temporary bridge, the sounding points and observation points can be marked on it with nails and paint. Otherwise a 1/8" dia. or 1/4" dia. flexible wire cable should be stretched tightly across and observation and sounding points marked on it by paint and tags. The distance marked on the cable should be checked with a steel tape occasionally as the cable may sag or stretch and the tags slip.

The total discharge is worked out by totalling all the partial discharges calculated in each compartment divided by the verticals. The formula adopted for computing the discharge is based on assumption that the bed of the stream and horizontal velocity are made up of straight lines or of Parabola or between two successive verticals, the former called the rectilinear and the other curvilinear method. In the former method partial discharges for any compartment is calculated on the basis that the average depth for the compartment is the average of the depths observed at the two end verticals formin

GAUGING PLATFORM



the boundary and average mean velocity is the average mean velocities observed at the two verticals. Thus if V_1 and V are mean velocities at the two end verticals spaced 1'0" and d_1 and d_2 are the depths, then Q, the partial discharge is worked out as $Q = \frac{(V_1+V_2)}{2} \times \frac{(d_1+d)}{2} \times 1$

In cases where the verticals are evenly spaced the calculations are also some times based on $Q_1 = \frac{(V_1+2V+V_3)}{4} \frac{(d_1+2d+d_3)}{4}$

The prismoidal formula can be used for giving greater weightage to depth and velocities observed at the middle of observation point.

$$Q = L \frac{(V_1+4V_2+V_3)}{6} \times \frac{(d_1+4d_2+d_3)}{6}$$

where V_1, V and V_3 etc., and d1, d_2 and d_3 etc., have the same conversion. It is found from experience that the simple formula based on trapezoidal rules gives result that area as accurate as the data collected and warrant flow and depths, the difference in the results as worked out will not matter whatever method is used and hence the simple formula is used in every day practice in preference to others.

Curviliner formula are justified only when bed or streams and horizontal velocity curves are continually concave to the axis of the reference and is seldem used.

These measured velocities when plotted with depth as ordinate and velocities as abscissa as defined for each vertical, the vertical velocity curve which shows the velocity at every points in vertical and from which the mean velocity can be determined by dividing the area bounded by the curve, the top and bottom ordinates, and the axis of depth by the total depth. The area may be calculated by planimeter.

Studies of vertical velocity curves taken on many stream under various conditions of depths, velocity, and roughness of bed, show that these vertical velocity curves have approximately the form of a parabola in which the axis coinciding with the maximum velocity is parall to the surface and is in general situated between the surface and 1/3 of depth of water. From the maximum, the velocity decreases gradually upward to the surface and downward nearly to the bottom where it charges more rapidly on account of the friction on the bed. As the depth and velocity increase, the curve approaches a vertical line as its limiting position.

A sample form for discharge computation is noted below. (vide separate statement attached.)

Other particulars.

Current meter gauging No........of.............. 19

Dated.....................

Time at start.............at finish.................

Station gauge readings.

Gauge at.	*Site.*	*Well.*
R.L. of zero	..	..
at 7.30 A.M. Example	5.96	5.96
at 9.50 A.M.	6.00	6.00
Average	5.98	5.98
Arithmetic mean	5.99	5.99
Change per hour	0.017	0.017

Meter No .. .. .. Mono or Penta counter.
Method of suspension .. Rigid rod with flat bar.
State of cone .. .. Medium.
Revolutions counted by
Rating equation of meter used—V = 2.18 R + 0.065 up to R = 0.61 + V = 2.27 R + 0.010 above R = 0.61
stop watch No. condition Good.
Wind—strong, etc. .. Weather—Cloudy, etc.

Abstract (Example).

Area—849 s.ft.
Discharge—276 cusecs.
Mean velocity—3.25 '/sec.

Remarks—

A velocity chart showing velocities for every 0.01 increase of revolutions/second prepared from the latest (rated meter) equation will have to be used for calculating discharge.

The discharge statement for each gauging should be worked out as soon as possible after the gauging. All computation and entries in the discharge statement should be checked a second time to avoid errors. The statement should also be checked at division level before the results are recorded.

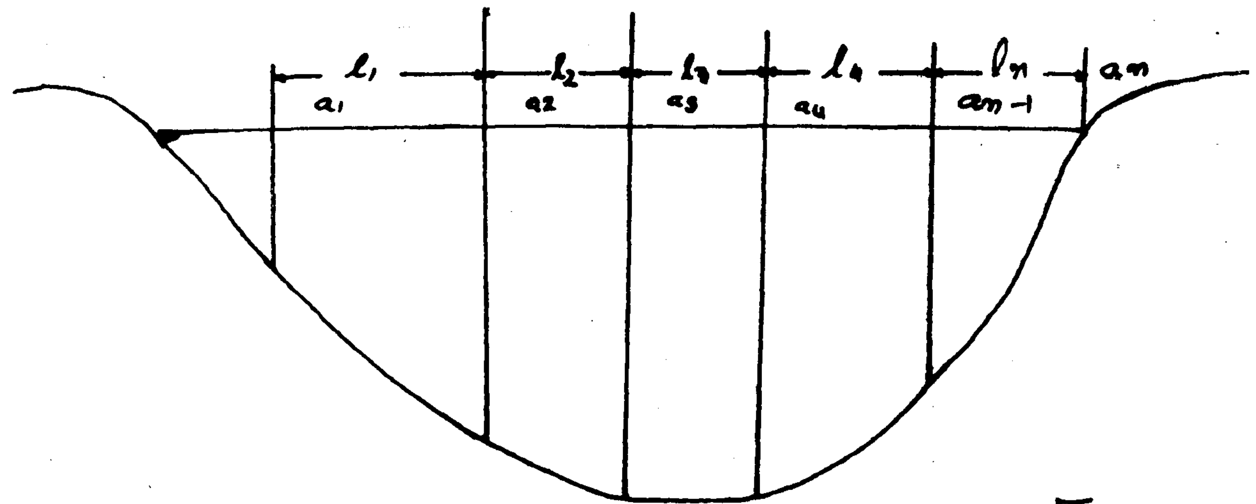
Gaugings should cover as far as possible the full range of the condition of flow in the channel stream gauged. This can only be done satisfactorily by conducting gaugings daily or at frequent intervals through out the season when there is flow in the channel or stream. A single current meter gauging even when carefully done may be as much as+5 per cent in error and hence there should be sufficient number of repetition of observation for approximately the same stages.

In regulated channels it will be possible to arrange for various stages by suitable regulation and this should be done to fill up gaps in the distribution of observations dor various stages.

Gaugings should not be done immediately after each such regulation as sufficient time should be given for the flow to become steady after the regulation. Every year after taking observations over a wide range, the cross sectional area, the mean velocity and the discharge are plotted on different curves with the gauge reading as ordinate. The area and the mean velocity curves help to explain abnormalities. The gauge discharge curve is finally utilised for regulation purposes.

The following points are to be noted. 1. Distance from the initial point of each vertical in which soundings and velocity observations were made. The distance between successive stations gives the width of the partial area. 2. Depth of water in the vertical at which observations are made as noted by taking soundings are described earlier. 3. Duration of the velocity observation in seconds by stop watch. 4. Number of revolutions of the cups during the time noted.

Mean velocity in the vertical at any measuring point is the average of the velocity observations made in that vertical. When a single velocity observation is made in any vertical the same value can be noted in the corresponding column. In order to get mean in the section or partial area, values for consecutive verticals may be taken. Mean depth of each partial area is computed by averaging the depth at each vertical with depth at the following vertical. From this mean velocity and area, discharge are calculated. The following sketch also indicates the method of calculation.



$$DE.\ l_1\left[\frac{d_0+d_1}{2}\right]\left[\frac{V_0+V_1}{2}\right]+l_2\left[\frac{d_1+d_2}{2}\right]\left[\frac{V_1+V_2}{2}\right]+l_n\left[\left(\frac{d_{n-1}+d_n}{2}\right)\left(\frac{V_{n-1}+V_n}{2}\right)\right]$$

where d_0, d_1 etc., and V_0 and V_1 are depth and velocities at the respective area of section a_0, a_1 etc., which are spaced at distances l_1, l_2 etc.

It is of great importance to use correct gauge height when plotting a measurement. The gauge height should be read at frequent intervals during the measurement, and the reading noted together with time. The vertical at which measurement is being made at the time of reading the gauge should also be shown in the note.

Methods of suspension of meter .—

Three methods are adopted.

1. The meter is suspended by an 1/8" wire rope cable and two torpedo weights of either 17 lbs. or 11 lbs. are attached below the meter to the small flat arm to which the meter is secured.

2. A set of rigid rods is attached to the meter at lower end by screwing (as per sketch). This is adopted, when the depth is small about 1'6" modified by attaching 3/4" pipe for larger depths.

3. The meter is suspended as mentioned in item 1 but fish weight is fixed above meter (as per sketch).

First method is adopted generally in all cases where plat forms with coracles are used for gaugings. The third method is used when we have to measure the velocity close to the bed. The second is used for all cradle gaugings.

It may be thought that cradle gaugings are preferable to platform gaugings since the former do not disturb stream lines but the errors on account of impracticability of lowering the meter exactly to a particular depth and consequent distribution of cross sectional velocity not being correct. Coracles introduce very little error as the coracles we use are shallow and depths comparatively large.

The result of gauging can be best examined by plotting.:

(i) A.site gauge area curve for the site gauge:

ii) A site gauge mean velocity curve and

(iii) Gauge discharge curve for the site and calibration gauge.

The scale should be so chosen to cover the full range of flow on a single sheet with the main portions of the curves inclined at 45° to the gauge axis.

The points representing each gauging should be marked neatly by small circles. The plotted points should be checked and inked in, and the serial number of the gaugings also entered for each point. The plotting of these curves should be done immedicately after each gauging (and not postponed until after all the gauging or the season are completed) as they will be of great assistance in detecting then and there, errors on observation changes in areas or mean velocities of the channel, etc.

For any gaugings done, a brief report on the gaugings indicating the object, description of the site, and equipment total number and distribution of the gaugings and any other useful information regarding the changes of "regime" of channel may be prepared along with the results. The other particulars that have to be prepared are 1. Abstract of gaugings, giving serial No. date, gauge reading, area, discharge and mean velocity for each guagings, 2. rite plan, 3. statement of check of zero of gauge with description of permanent bench marks to which the levels have been connected, 4. the cross sections of channel at the site of the calibration gauge and gauging site. 5. Curve and discharge tables. 6. Comparative statement of current years gauging with previous observation, if any at the same sites. 7. A statement of co-efficients at masonry works, if any, worked out from the results.

In canals which are subject to variations in draw off from the normal, due to silting up, or scour or due to weed growth, or due to variations in surface fall caused by the draw off conditions in the lower reaches or other factors tending to cause a decrease or increase in area or velocity, different stage discharge relation curves are drawn corresponding to different conditions that obtain at the time of observation. These curves are also checked up with the corresponding area and mean velocity curves and values tabulated for every stage.

Other methods of measurement of discharge—

1. In the velocity area method of measuring discharges instead of measuring velocity directly by floats or rods or indirectly by current meter, another method that is commonly resorted to is by measurement of slope and using slope formula and the discharge measurements are distinguished in accordance with the method of measurement of velocity adopted. Slope measurement of measuring discharges consist of determination of mean area of cross section, mean hydraulic mean radius and slope of water surface, with character of river bed and banks to assess requisite co-efficient. Mean velocity is determined by the formula.

$V = C\sqrt{RS}$ where C is co-efficient depending on

roughness of stream bed, $R = \dfrac{\text{Mean area of cross section}}{\text{mean wetted perimeter.}}$

and S = slope of water surfacee.

In order to find out the surface fall it is taken for a fairly straight reach of the river of length of about 20 times the width of river where slope and cross section are uniform fairly and condition of bed and bank are permanent. A staright reach for 200 feet to 1000 feet must be selected and measured for the course. Surface fall in a reach must be sufficiently large to be measured without a large percentage or error. This surface fall can be measured by reading simultaneously guage readings (gauge posts to be fixed at relevant fixed places) at the comencement and at the end of the run. If one more. gauge at middle is also noted it will give more accurate results. It is desirable that there should be at least two gauges at each and (one on each bank) and the average of the two bank readings are noted. Gauge plates to 1/100 of a foot are desirably to be fixed. For ensuring correct reading these should be got checked by a responsible officer along with the zero values connected to the common permanent datum. The accuracy of the measurement depends largely on the proper placing of gauges, precision in gauge readings and care in setting the gauges to read from the same datum. The value of "n" (rugosity co-efficient) should be fixed based on the roughness of bed and other obotructions that prevent free flow of water. "N" is larger for berm areas (over

flow portions) than for the confined river section. The value V is arrived by Kutters formula.

$$V = \frac{1.811}{M} \qquad C = \sqrt{RS}$$

$$C = 41.65 + \frac{0.0281}{S} + \frac{1.811}{M}$$

The value of "n" varying from 0.020 to 0.025 is smooth sandy and find gravel beds and 0.030 to 0.035 in rough beds and for overflow banks (berms) with vegetation 0.040 to 0.055.

other formulae are :

$$\text{Mannings } V = \frac{1.486}{N} R^{\frac{2}{3}} \times S^{\frac{1}{2}}$$

$$\text{Lacys } V = \frac{1.346}{na} \times R^{\frac{3}{4}} \times S^{\frac{1}{2}}$$

or

$$16\, R^{\frac{2}{3}} \times S^{\frac{1}{2}}$$

The slope method is generaly used for estimating high flood discharge, often after the highest flood has passed and when the only data available are the slopes as determined by gauge readings or marks along the banks and the area of section and the measurement of discharge with instruments are impracticable.

The results obtained by this method are at best only approximate owing to the difficulty of obtaining accurate measurements of slopes and the difficulty in the selection of proper roughness co-efficients for natural streams. This method of calculation of discharge can be used in the preliminary calculations of discharge to know the quantity of water available and thereby decide whether detailed investigation is to be undertaken or not.

2. *By weirs*—

Standard type of weirs of suitable form furnish an accurate method of measurement of water, as water has been found to flow over them in accordance with known definite laws. However on account of the cost of its construction, this method cannot be used in all places as a measuring device.

In the case of a normal weir, a record of flow of water over crest of weir is to be noted. Co-efficient for the weir is also to be known. Then also the discharge as per the formulae $Q = \frac{2}{3} C \times \sqrt{2g}\, LH^{1.5}$ (i.e. 3/2). The value of C depends on the nature of crest (sharp edged or broad crested) or round crested the length of weir over which discharge has taken place the width and breadth at bed level of the approach channel i.e., upon the end contractions, when velocity of approach is to be allowed for one formulae to be adopted is $Q = \frac{2}{3} CL\sqrt{2g}\,[(H+h)^{3/2} - h^{3/2})]$ where H is head over crest and =head due to velocity of approach. For sharp created weir with end contractions the formulae that can be adopted $Q = 3.33\, LH^{3/2}$ and with velocity of approach $Q = 3.33\,[(L-2H)\ (H-h)^{3/2} - h^{3/2})]$.

In the case of broad cresated weir the formulae to be used = $Q = 2.64\, LH^{3/2}$. Trapezoidal and triangular weirs in the case of small channel are used occasionally with proper co-efficients in the formula.

In practice weirs are not constructed for gauging purposes. But when drops in a channel are to be provided in a canal, advantage is taken for measuring discharges. In the case of clear over fall discharges, discharges are worked out based on the head given by the front gauge readings, and in the case of submerged weir by taking into account the readings on both the front and rear gauges As the accuracy of the methods depend upon the correct measurement of head and the value of co-efficient selected, a value which is again based on experiments whose applicability depends on actual conditions cannot be vouchsafed without being verified by any other method at site.

In the case of sluices provided at anicuts or reservoirs etc., determination of discharge is done by calculation of water flow by the formula $CA\sqrt{2GH}$ or other modified forms. Here also the gauge readings in front and rear of sluice (i.e. in anicuts or reservoirs) are to be read and effective head causing flow is calculated. The area of vent way is worked out by noting width and height of shutter opened. Here also a suitable value of "C" has to be used. The adoptability of the value of "C" is checked by measuring discharge by other means or by gauging with meters. Measurement by sluices is followed in exceptional case where there are no facilities for calibrating a gauge. An error of 0.05 in noting the head itself is likely to give such error in results. Similarly leakage through vents also affect the results. Again co-efficient varies with the front head, rear condition of flow and extent of opening, etc.

Volumetric method of discharge—

Another method to compute discharge is by direct measurement if possible. This is most accurate method except in case of small channels where flow can be regulated and diverted or in other extraordinary cases where suitable facilities are available. Water flowing through a channel is diverted to a measuring cistern or receptacle whose volume is calculated earlier. By noting the time taken to fill up, rate of discharge is calculated. By letting down water through a known height of opening of a sluice or sluices, for definite interval discharge let down into the chamber was determined by the rise in levels and the consequent increase in volume. From this the value of "C" for the sluices for the particular opening and reservoir levels are determined.

Discharges at bridges and regulator sites—

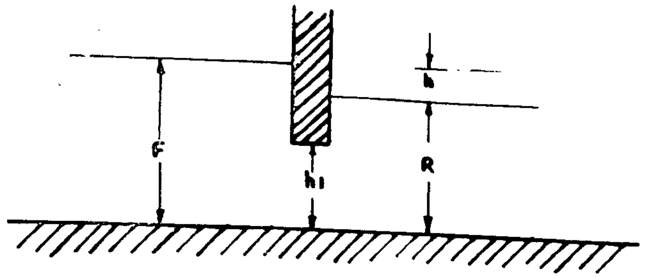
Discharges are calculated for bridges etc., during high flood condition when the standard methods of accurate measurement are impracticable due to difficulties involved. They are used as good check on discharge calculated by slope formula under such flood condition. The discharge formula $Q = CA\sqrt{2g\ (X+ha)}$ (with velocity of approach) where A = Area of C.S. at greatest contraction. point C = 0.9; h = head due to velocity of approach X = actual heading up or afflux. Accurate discharge by this method may not be possible since there will be difficulties in the correct measurement of afflux and to the uncertainity of the value of "C".

CONDITIONS OF FLOW

(1) SUBMERGED CONDITION:-(S.C)

FRONT AND REAR GAUGE READINGS ARE MORE THAN THE HEIGHT OF OPENINGS.

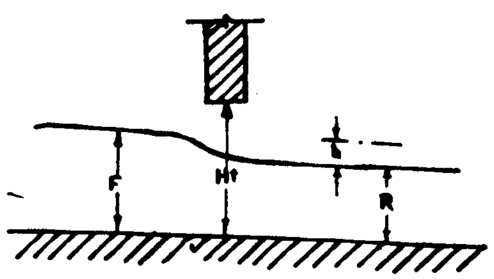
$Q = CA\sqrt{2gh}$



(2) FREE CONDITION (F.C)(FOR VENT OPENINGS).

WHEN THE FRONT AND REAR GAUGE READING; ARE LESS THAN THE HEIGHT OF OPENINGS.

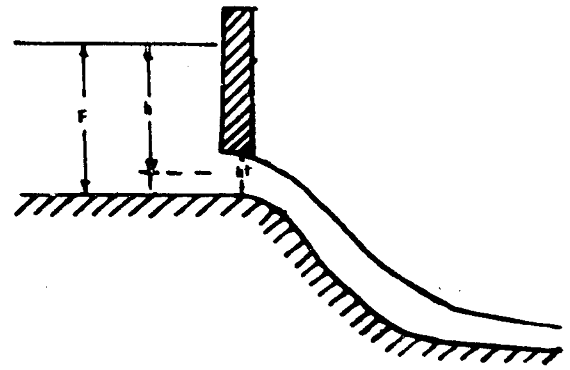
$Q = CL\sqrt{2gh}\,(F - h/3)$.



(3) SMALL ORIFICE. (S.O)

WHEN THE FRONT WATER LEVEL IS MORE THAN 8 TIMES OF VENT OPENINGS AND THE REAR WATER LEVEL IS BELOW THE FRONT SILL

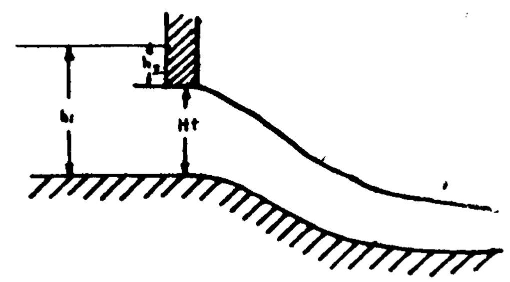
$Q = CA\sqrt{2gh}$



(4) RECTANGULAR ORIFICE (R.O)

WHEN THE FRONT WATER LEVEL IS LESS THAN 8 TIMES OF VENT OPENING AND THE REAR WATER LEVEL IS BELOW THE FRONT SILL.

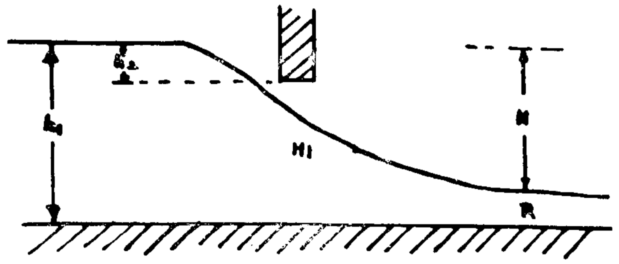
$Q = CL(h_1^{3/2} - h^{1/2})$



(5) PARTIALLY SUBMERGED ORIFICE (P.S.O)

WHEN THE FRONT WATER LEVEL IS MORE THAN THE HEIGHT OF OPENINGS AND THE REAR WATER LEVEL IS LESS THAN THE HEIGHT OF OPENINGS.

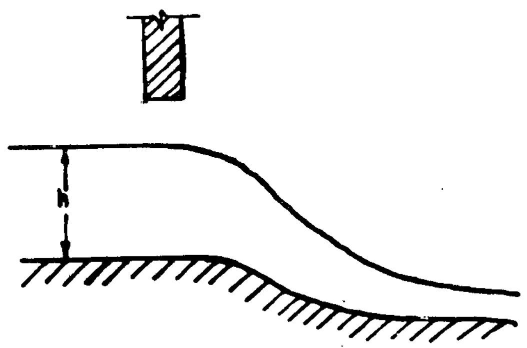
$$Q = CL\sqrt{2g}\left\{\frac{2}{3}\left(h^{3/2} - h_2^{3/2}\right) + (h_1 - h)\sqrt{h}\right\}$$



(6). CLEAR OVERFALL CONDITION. (C.O)

WHEN THE FRONT WATER LEVEL IS LESS THAN THE HEIGHT OF OPENINGS AND REAR WATER LEVEL IS LESS THAN THE CREST (OR SILL LEVEL).

$$Q = CLh^{3/2}$$

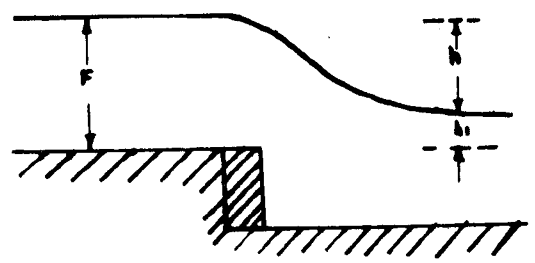


(7) DROWNED ANICUT CONDITION (D A)

WHEN THE FRONT WATER LEVEL AND REAR WATER LEVEL ARE ABOVE THE CREST OF THE ANICUT.

$$Q = L\left[3.1\left\{(h + ha)^{3/2} - ha^{3/2}\right\} + C_2 d\sqrt{2g(h + ha)}\right]$$

$$= L\,(D + d + h)\,Va, \quad \text{WHERE } Va = \sqrt{2gha}$$



NOTE:-

$F = h_1$ = HEAD OF FRONT WATER

$R = d$ = HEAD OF REAR WATER

H_1 = HEIGHT OF VENT OPENINGS

h – DIFFERENCE BETWEEN FRONT AND REAR WATER LEVELS.

h_2 – FRONT HEAD ABOVE THE HEIGHT OF VENT OPENINGS.

L – LENGTH OF EACH

A – AREA OF VENT (WATER WAY) lo. (LENGTH OF VENT X HEIGHT OF VENT OPENINGS.)

h_a – HEIGHT DUE TO VELOCITY OF APPROACH.

$2g$ = 8.00

D – HYDRAULIC MEAN DEAPTH

C – COEFFICIENT OF DISCHARGE

C_2 AS PER M.D.S.S.,

Reference—

1. Stream Gaugings by M.G. Hiranandani and S. V. Chitale.
2. Instructions for conducting current meter gauging (issued by Chief Engineer (Irrigation) in 1934).
3. Manual of Gurley Hydraulic Instruments by Gurley makers.
4. A Brief note on gauging of River by Engineer N.S. Ananthanarayanan.

INVITATION TO READERS.

Kind attention of Engineers of Tamil Nadu Public Works Department and other readers of the journal "NEW IRRIGATION ERA" is drawn to the meagre response to repeated requests for contribution of articles to the journal.

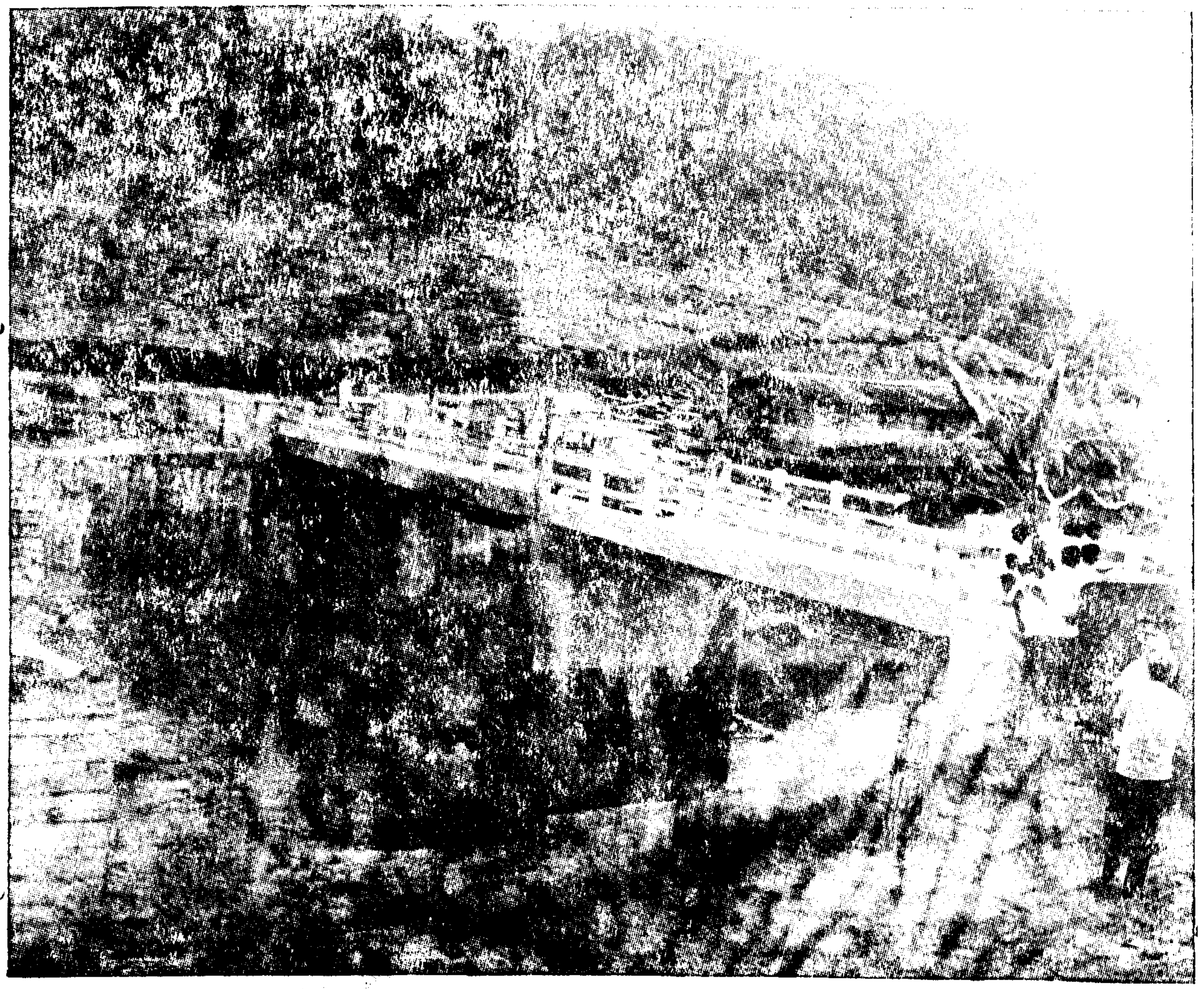
As all are aware the objective of the journal is two-fold, viz., to promote the interest of Engineers and assisting their specialised work by placing before them the different problems of technical importance and ways and means of solving them and secondly to offer to all Engineers to express their own experience in various fields. It is 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 articles to the journal can be mostly on Irrigation, but articles which have direct bearing on subjects as noted 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.
4. Soil erosion problems and conservation and reclamation.
5. Silt problems in rivers and reservoirs.
6. Sanitation and Public Health problems and possbilities of utilisation of sewage.
7. Inland water ways, navigation, fishing schemes and protection of land against sea erosion, etc.
8. Problem of drainage in deltaic tracts and entrance of rivers into the sea.
9. Run-off studies in catchments and evaporation studies.
10. Other subjects.

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

The Engineers of other departments are also requested to contribute liberally articles.



View of Completed Nadumalai Bridge at 4·07 km of VNT Exit Course.
Adopting method of Composite Construction in steel and Concrete.