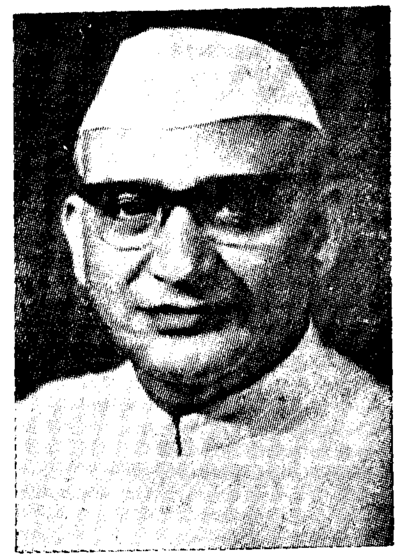
INAUGURATION OF PARAPPALAR HOUSE BY HON. MINISTER FOR WORKS.

DISCUSSION OF HON. MINISTER FOR WORKS WITH CHIEF ENGINEER PARAMBIKULAM ALIYAR PROJECT AT PARAPPALAR HOUSE.



RAJ BHAVAN
MADRAS-600022
December 16, 1975

MESSAGE

I am glad to know that the journal "THE NEW IRRIGATION ERA", the Quarterly Publication of the Irrigation Branch of Public Works Department, has reached the 25th Year of Publication and is shortly publishing its 50th Issue. I offer my best wishes for its continued success.

(Sd.) K. K. SHAH
Governor of Tamil Nadu

The New Irrigation Era

[illegible] December

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MADRAS-9
6th December 1975

MESSAGE

Engineering is a dynamic field. Engineers have to keep themselves abreast of current developments in technology. They should have an alert and inquiring mind, a sense of urgency to solve the problems before them and the determination and the expertise to solve them.

I am confident that the New Irrigation Era will be an adequate vehicle to promote the required expertise among the members of the Public Works Department and to enthuse them to face ever challenge.

I wish the Journal all success.

B. VIJAYARAGHAVAN, I.A.S.,
Secretary to Government, Public Works Department.

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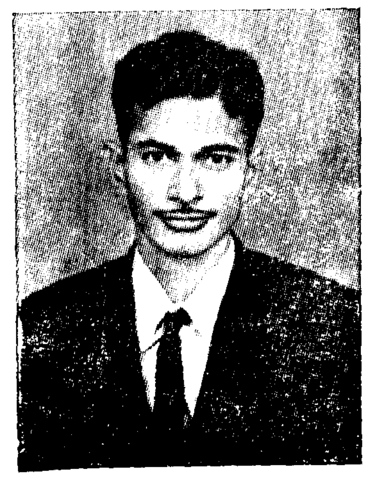

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The Editorial Board of New Irrigation Era of Tamil Nadu Public Works Department accepts no responsibility for the views expressed by the authors.

THE NEW IRRIGATION ERA

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CONTENTS.

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GENERAL AND IRRIGATION CHIEF RETIRES.

Born in the year 1917, Thiru S. P. Namasivayam was a distinguished student throughout his studies in the Engineering College, Guindy. After devoting his duties for the maintenance and operation of parts of Periyar Irrigation System in Madurai District, he began his services as one of the directly recruited Assistant Engineers in 1948 in Composite Madras Province. He was sent on deputation in 1954 to Bihar after selection by them as Executive Engineer in-charge of Investigation and Construction of a number of Irrigation Schemes in Gaya District in which part he had served till 1956. Afterwards he had served as Executive Engineer, P.W.D., in-charge of major Building Construction and Maintenance in the City. He had served as Deputy Chief Engineer (General) also for a short spell.

As Superintending Engineer he was in charge of Construction of major Buildings of Colleges, Hospitals, etc. He went on deputation to Department of Atomic Energy, Trombay, as Superintending Engineer in-charge of Construction of Laboratories, Workshops, etc., in Atomic Energy Establishment. Later he worked as Project Engineer for the Township Construction for the Department at Trombay. He was with the Department of Atomic Energy from 1962 to 1969. Then he served as Chief Construction Engineer for Tamil Nadu State Electricity Board from April 1969 to January 1971 for the investigation formulation of and detailing of designs and construction of Hydro-Electric Projects like Kodayar, Aliyar, Sholayar Power House, etc.

From 11-1-1971 onwards he was selected by the Tamil Nadu Government to work as Chief Engineer (Irrigation) in Public Works Department. During the period he was in-charge of all works including investigation, formulation, designs, research and construction of major, medium and minor Irrigation Projects and also he was in-charge of remodelling the old Cauvery System for minimising the use of water for Irrigation purpose and also he was handling the Inter-State Cauvery waters question from Technical side.

From January 1972 onwards he was given the General Portfolio also in Tamil Nadu Public Works Department in addition to the Irrigation wing and retired from State Service on 28th August 1972 as Chief Engineer (General and Irrigation). He was reappointed in the same post and retired from State Service on 1st March 1974.

With such a distinguished record of service for the last 32 years he retired in 1974 at the age of 57. We wish him long life and happiness in his retirement.

All correspondences should be addressed to the Executive Engineer, P.W.D., Gauging Division, Chepauk, Madras-600005.

EDITORIAL

In the event of water shortage productive use of all the available water let off for irrigation is very much on discussion. An *Irrigation network* is a group of canals and structures for transporting water from its source, be it a river, drain, lake or reservoir, to places where it is used to agricultural ends. The requirements of agriculture dictate the laws according to which the water should enter and be transported by the network. With this aim in view, the operation of the network should include the establishment of a programme for the allocation of water for each canal based on the crop cultivated and should ensure that such a programme be carried out working facilities are closely related to the efficiency of distribution the arrival of the water on plot of land is dependent on the nature of the soils in which water flows, structures, canals and ditches which connect this land to the place where the water is stood and also on the operation of flow adjustment between these two points. In order that the water may reach a given destination at a given time it is necessary that these adjustments be made a certain length of time before hand. Further minimising the losses of flowing water is an absolute necessity for greater efficiency. This can be achieved only through closer co-ordination of the staff engaged in distribution network. The operation will have to work like an automatic signal around the junction of highway roads. Such precision could be obtained by computerising the entire process of distribution before hand and prepare a schedule of operation to be supplied to the distribution staff. We hope that such a day is not far off in as much as the Government is bent upon modernisation of systems of distribution.

The Editorial Board is glad to inform that this issue, crossing the Fiftieth one, has completed Twentyfive years of publication.

INSTRUMENTATION IN EARTH DAMS.

By Thiru R. Rajaraman*

1. *Introduction*—

1·1· Dams and related works are built to last many years and they form important structures useful in the development of the Irrigation and Power potentials of river basins. The water stored upstream of the dam possesses tremendous forces which the structure is expected to withstand throughout its life. The failure of a dam will have serious and disastrous effects in terms of loss of life and damage to property very much larger than the failure of any other Civil Engineering work. The failure of the Panchet dam in Maharashtra which initially started with the settlement and over topping of a small reach of the earth dam finally led to devastating consequences including the collapse of another dam lower down in the course of the river and untold suffering and damage in the city downstream. It is therefore most essential to assure ourselves about the continued safety of such important structures. One of the methods to ensure this is to embed various instruments in the dam structure which will enable us to keep a continuous watch over the performance of the structure and locate trouble spots which call for immediate remedial measures. In addition, instrumentation in earth and rockfill dams could be designed to serve additional purposes.

2. *Purpose of Instrumentation*.—

2·1· Most of the instrumentation is done during construction of the dam itself and it is intended to help in studying the behaviour of the dam both during construction as well as during subsequent reservoir operations. The specific uses of instrumentation are threefold.

(i) *During construction*.—To provide information necessary for a continuous check as to whether the design assumptions adopted are realised or whether modifications in the design and/or of further construction are required to ensure that during no stage, the actual field conditions are worse than that taken in the design. For example—Observations on the pore water pressure in the earth dam and foundations provide a guide to the permissible rate of fill placement.

(ii) *During operation*.—To furnish data to determine whether the structure functions as intended and to provide a continuous surveillance of the structure to warn of any developments which may endanger its safety. For example—A record of the progressive settlement and internal movements obtained from gauges installed in Earth dam provides information with which we could ensure that adequate free board is available and if not remedial measures could be initiated. Also any sudden increase in the pore pressures or the rate of seepage may indicate cracking in the core zone.

(iii) *To assist in the design*.—The solution of many important practical problems faced during design of dams such as prediction of pore pressures during construction in the core materials, deformation of rock fills under gravity loadings and high pressure effects, seismic effects on overall stability, etc., now seem to be possible only by field observations on completed dams. As on date many theoretical analysis of great refinement are in progress throughout the world. The value of such theoretical analysis could be ascertained only by comparison with detailed and reliable information on the behaviour of completed earthern dams. Example—The records of the Seismographs installed at the Koyna dam are perhaps one of the few actual records of the vibration amplitudes and intensities near an epicentre of an earthquake and thus forms the basic data of immense value in the analysis of earthquake stability of dam.

2·2. The Government of India recognising the importance of the study of such instrumentation installed in dams, retaining walls, etc., constituted a committee in 1964 to review the instrumentation done so far in the country and also to make recommendations for adoption in the future. The committee collected data on the instrumentation so far executed in various dams of the country and the report of the committee contains information on the state of art on this subject in the country. The report of the committee also contains information on various types of instruments embedded in concrete, masonry and earth dams as well as instructions for their installation, observation and record of the data[1]

2·3. The committee, after detailed study of the practices prevalent in other countries vis-a-vis the stage of development and conditions prevailing in this country, has recommended that for all Earth dams over 50 ft. in height, the following measurements may be taken as an obligatory measure of safety :—

(i) Internal vertical settlements and strains during and after construction.

(ii) Surface settlements and horizontal displacements during and after construction.

(iii) Stand-pipes and porous tube piezometers for appraisal of seepage control measures both in the body of the dam as well as in the foundations. These must be combined with measurement of seepage.

(iv) Hydrostatic piezometers in regions liable to development of high positive pore pressures during construction and in the upstream casing and semi-pervious zones of dams with large range of draw-downs.

Additional instrumentation would be necessary in earth dams where special site conditions exist or to obtain information for advancement of knowledge regarding strength and stability of earth masses and other special problems, as for example—

(i) Measurements of horizontal strains would be necessary where significant movement is expected due to plastic nature of the soil in the embankment or foundation

*Research Officer, Concrete and Soil research Laboratory, Madras-600005.

(ii) In the case of high dams with narrow cores or abutments with steep and irregular profile, inclinometer installations can provide very useful indication of tendency for separation between core and abutment.

(iii) In cases involving problems of long-term stability various methods of detecting movements such as surface markers, horizontal cross arms, inclinometres, settlement plates, etc., could be used and a continuous review of observations would reveal movements, etc.

2·4. The Committee on Observations of dams and models of the International commission on Large dams, has also brought out a report on the international practice in this field. The various types of instruments embedded in earth and rock fill dams in various countries of the International commission have been described in the report[2]. A study of this report indicates that we in India and especially Tamil Nadu have to advance very much as compared to international practices in this subject.

2·5. In view of the importance of this study, the Central Board of Irrigation and Power also held a Seminar on this subject at which papers on techniques of installation of instrument, precautions to be taken, methods of analysing the data were presented and discussed[3].

2·6. In order to obtain the full benefits of the instrumentation, it is of extreme importance that a systematic and complete programme of the various observations and measurements is prepared in advance so as to obtain as complete an information as possible. Measurements should be planned for the most important zones of the dam. The planned instruments must provide for mutually checking information for establishing the reliability of the results obtained by different methods of measurements. As for example observations on internal settlements should be planned at or near the section in which pore pressure observations are made so that both these data could be correlated. When working out the plan of observations, the cost of the equipment, of the observing personnel and of processing of the observed data and interpretation of the results should commensurate with the cost of the dam.

2·7. The goal of the instrumentation provided as indicated in the earlier paragraphs cannot be reached unless the staff employed for taking the observations, processing the observed data and interpreting the results is of adequate calibre.

3. *Types of measurements in earth dams* :—

3·1. The following types of measurements are generally made in the field —

(1) Horizontal and vertical movements.

(2) Piezometer observations and Neutral stresses (Pore water pressure).

(3) Stress and stain in various directions.

(4) Seepage measurements

3·2. *Horizontal and vertical movements*:

3·2·1. *Surface Movements :*

This is the simplest of instrumentation usually adopted and consists of fixing 2·5 cm. diameter × 2 m long reinforcement rods which are driven vertically into the completed eath dam. These points are established in 3 or 4 rows parallel to the central line of the dam at horizontal spacing of 100 to 200 m centres. After installation all surface settlement points are connected to permanent bench marks by precision theodalite levelling.

It is also preferable to establish a row of points along the toes of the eath dam to observe possible movements. Also settlement points installed along the centre line of the invert or at other locations inside the conduit type outlets would provide information on the differential settlements of the various zones under different loading and foundation conditions.

In the case of soft ground, surface settlement points have also been constructed by welding a short length of 25 mm M.S. rod to M.S. plates 25 cms × 25 cms. These are embedded at least 30 cms. below the surface with the M.S. rod projecting outside the ground and levels are taken on the projecting rods.

3·2·2. *Internal vertical movements* :

The following types of instruments are generally used for mesurement of vertical movements :

(i) U. S. Bureau of reclamation cross arm device.

(ii) Telemac water level indicator system.

(iii) Other devices.

3·2·2·1. U.S.B.R. cross arm installation.—The U. S. Bureau of reclamation cross arm device is the simplest and most widely used in this country. A typical installation of the device in one of the dams of this State is shown in figure 1.

This system consists of a series of 37 mm(1½") and 50 mm. (2") diameter sections of pipes telescoping into one another. The 37 mm pipe section is anchored to the embankment by means of horizontal cross arms at intervals of 3m. For this purpose the 37 mm pipe is attached to about a metre length of 75 mm channel by means of U bolts. The channel sections are embedded in the embankment as the embankment rises during construction. The movements of these cross arms are measured by recording the changes in the elevation of the bottom tips of the attached 37 mm pipe section (This point is designated as measuring point in figure 1).

A specially designed settlement torpedo is made use of to determine the elevations of the measuring point. The significant part of the torpedo is a set of two pawls which open and close in the slots provided, by spring action. The maximum dimension of the torpedo is at the pawls which when fully opened is less than 50 mm but greater than 37 mm. The torpedo is attached to a tape and is lowered down the cross arm installation with the wings in open position. The torpedo will slide through the 50 mm pipe without difficulty and will also pass through the 37 mm pipe in the downward direction. Once it clears the 37 mm pipe the spring loaded pawls will open out and in this position if the tape is pulled up, the bottom surface of the 37 mm diameter pipe will obstruct the upward movement of the torpedo. If just at this position, the reading of the

tape at the top of the pipe is noted, and substacted from the elevation of the top of the pipe the elevation of this particular measuring point is obtained. By further lowering the torpedo and repeating the above operations the elevations of all the cross-arms are obtained. After passing the bottom cross, arms the torpedo hits the latching plate provided in the base extension pipe. This impact closes the wings and the sleeve provided in the torpedo prevents it from opening again. With the wings closed the instrument can easily be pulled up to the top of the embankment.

Full details for the installation and taking observations with the earth dam settlement apparatus are contained in the USBR Earth Manual[4].

Typical observations obtained in the Vaigai Dam are indicated in Table I. Similar installations have been done in Lower Bhavani, Manimuthar, Amaravathy dams. etc., Recent dams such as Ponniar, Chinnar are also provided with similar installation. Data for the period 1957 to date has been collected for Vaigai dam and for the period 1951 to 1955 for the Lower Bhavani Dam. A study of the results provides useful indications regarding the behaviour of the structures. Data relating to the settlement observations in other dams of the country have also been published[6, 7].

TABLE 1.

SETTLEMENT INSTALLATION READINGS.

Date of observation : 21st November 1962.
Water level in installations : + 394·50 ft.
Water level in Reservoir : + 426·25 ft.
Foundation elevation : + 382·26 ft.

Cross arm number.	*Original elevation of cross arm.*	*Original distance between cross arm in ft.*	*Present elevation of cross arm.*	*Settlement of cross arm in ft.*	*Present distance between cross arms in ft.*	*Consolidation between cross arms to date in ft.*
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	387·77	..	387·59	0·18	..	..
2	392·77	5·00	392·42	0·35	4·83	0·17
3	397·79	5·02	397·25	0·54	4·83	0·19
4	402·82	5·05	402·05	0·77	4·80	0·25
5	407·77	5·09	406·86	0·91	4·81	0·28
6	412·77	5·01	411·73	1·04	4·87	0·14
7	417·63	4·98	416·58	1·05	4·85	0·13
8	422·63	5·00	421·46	1·17	4·88	0·12
9	427·58	5·00	426·35	1·23	4·89	0·11
10	432·51	5·03	431·29	1·22	4·94	0·09
11	437·43	4·99	436·21	1·22	4·92	0·07
12	442·27	4·95	441·11	1·16	4·90	0·05
13	447·07	5·05	446·16	0·91	5·05	0·00
14	452·00	4·97	452·18	0·82	5·02	(—) 0·05
15	457·03	5·08	456·27	0·76	5·09	(—) 0·01
16	461·80	4·96	461·23	0·57	4·94	0·02
17	466·58	4·95	466·25	0·33	5·02	(—) 0·07
18	470·50	4·17	470·44	0·06	4·19	(—) 0·02
	7,736·72	84·30	7,722·43	14·29	82·83	1·47
	7,722·43	82·83				
	14·29	1·47				

3·3·4·2. *Vibrating Wire Gauge* :

The action of this type of gauge depends on the principle that the frequency of a vibrating wire depends on the tension on the wire and its length.

$$f = \frac{n}{2L}\sqrt{\frac{T}{S}}$$

Where f = frequency.

n = Harmonic level.

L = Free length of the vibrating wire.

T = Tension in the wire ;

S = Mass per unit length of the wire.

The tension in the wire is proportional to the elongation.

$$\frac{\triangle L}{L} = \frac{T}{a E}$$

where $\triangle$ L = strain.

a = area of section of wire.

E = Youngs modulus of the material of the wire.

In the pore pressure measuring instrument the vibrating wire is so fixed that one end is attached to the body of the cell while the other is attached to the diaphragm. The cell also contains a magnetic coil which is used for plucking the wire and setting it in vibration and also to pick up the vibrations and convert it into electrical signals. Changes in the pore water pressure causes changes in the deflection of the diaphragm and these induce proportional changes in the gauge wire tension and so of its vibrating frequency. This frequency as picked up by the magnetic pick up is compared with the frequency of the standard wire which is maintained in the measuring instrument. Tension of the standrad wire is adjusted by a calibrated screw till the two tones are matched.

The main advantage of this unit as compared to the resistance type is that the frequency of the wire is independent of variations in the battery voltage, contact resistance, lead resistance, etc., which restrict the utility of the resistance wire type of gauge.

3·4. *Stress and strains in various directions* :

Earth pressures are required to be measured only in rare cases and so such instrumentation is not provided as a general rule in earth dams. However when high retaining walls are used in various situations it may be instructive to install earth pressure cells and obtain data on the development of earth pressures. A Gold-Beck Cell suitable for measurement of earth pressure is available. The carlson type stress meter is also adaptable for earth pressure measurements. The Building Research Station, England has developed a cell incorporating the vibrating wire principle [13]. The cell has two parallel steel diaphragms whose deformation alters the tension of the vibrating wire (figure 6). The wire supports are fixed at the point of contrafluxure of the diaphragms and measurements are made as already described.

3·5. *Seepage measurements* :

The arrangements required for this are quite simple. The embankment is divided into different reaches based on the topography of the area. The seepage flow which passes in the toe drain in the different reaches is measured by installing Vnotches or other gauging arrangments suitably.

4·0. *Data and its analysis* :

4·1. Settlement observations—Observations of the earth dam settlement apparatus are taken once in 15 days during the construction period and thereafter in monthly intervals. A typical set of readings taken on a particular date is shown in Table 1. From this table, it is possible to determine the present thickness of the various soil layers and a comparison of such data with that at the time of initial construction furnishes information on the settlement of each layer as well as the overall settlement.

Fig. 7 is a record of the observations taken on a cross arm installation during the years 1957 to 1966 [5]. It can be seen from the figure that between 1957 to 1961, the settlement was at a fairly rapid rate while afterwards it has been almost flat. In December 1965 there has been a sudden increase in the rate of settlement. The construction period of the dam was from 1954 to 1959 and this generally coincides with the period of large increase in settlement which has levelled off after 1961 beginning.

The overall settlement recorded has been 2·65 feet. The total length of the installation was 86 feet and thus the percentage settlement works out to about 3 per cent. An interesting observation is that cross arm No. 1 viz, the bottom most cross arm which measures the settlement of the foundation has recorded a settlement of about 1 ft. i.e., near about 40 per cent of the settlement is contributed by the foundations and the rest by the embankment fill. The settlement of the bottom 5 feet layer of the fill i.e., between cross arm 1 and 2 has shown the maximum consolidation of the order of 0·5 feet or 10 per cent of its height.

Occasionally cross arm installations have recorded expansion of the soil layers instead of settlement and this is probably due to swelling of the layers due to absorption of water.

4·2. Pore pressures—Table II furnishes a typical set of readings taken of the pore pressures developed in Vaigai Dam. It will be noted from the table that the pore pressures are computed from the hydraulic pressures measured by the gauges and the tip constant. The tip constant defines the relative elevation of the location where the tip is embedded as compared to the elevation of the pressure gauges. Pore pressure observations are taken once in a week during construction period and thereafter once in a fortnight. Fig. 8 shows a plot of pore pressure contours as on a particular date. Such plots are useful in the study of the pattern of pore pressure build up or dissipation inside the stricture. Fig. 9 shows the variation in pore pressures recorded by individual piezometers with change of reservoir elevation. A study of this chart in conjunction with the location drawing of the piezometer tips indicates that those piezometers which are installed near the upstream pervious zone follow closely the variations in the reservoir elevation with little or no time lag while piezometer tips installed in the downstream area show relatively no variations in pore pressure due to changes in reservoir elevation.

TABLE II.

VAIGAI DIVISION. VAIGAI DAM SUB-DIVISION.

Piezometer reading taken on 10th October 1972.

River water level + 894·4 ft.

Tip number.	*Tip level.*	*Tip constant.*	*Gauge constant.*	*Readings of gauges in lbs. per square inch.*		*Average readings in lbs. per squre inch.*	*Average readings in feet of water.*	*Pore-pressure in feet of water.*	*Remarks.*
				Inlet gauge.	*Outlet gauge.*				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	818·0	857—818=39	0	12·0	11·0	11·5	26·5	65·5	..
2	818·0	857—818=39	0	11·0	10·0	10·5	24·2	63·2	..
3	818·0	857—818=39	0	10·0	9·0	9·5	21·9	60·9	..
4	818·0	857—818=39	0	8·0	7·0	7·5	17·3	56·3	..
5	818·0	857—818=39	0	7·0	6·0	6·5	15·0	54·0	..
6	840·0	857—840=17	0	6·0	6·0	6·5	13·8	30·8	..
7	840·0	857—840=17	0	6·0	4·0	5·0	11·5	23·5	..
8	840·0	857—840=17	0	5·0	4·0	4·5	10·4	27·4	..
9	840·0	857—840=17	0	4·0	4·0	4·0	9·2	26·2	..
10	840·0	857—840=17	0	4·0	3·0	3·5	8·1	25·1	..
11	855·0	857—855= 2	0	7·0	5·0	6·0	13·8	15·8	..
12	855·0	857—855= 2	0	6·0	5·0	5·5	12·7	14·7	..
13	855·0	857—855= 2	0	6·0	4·0	5·0	11·5	13·5	..
14	855·0	857—855= 2	0	4·0	3·0	3·5	8·1	10·1	..
15	855·0	857—855= 2	0	3·0	2·0	2·5	5·8	7·8	..
16	855·0	857—855= 2	0	2·0	1·0	1·5	3·5	5·5	..
17	880·0	857—880= (—)23	0	12·0	8·0	10·0	23·0	0·0	..
18	880·0	857—880= (—)23	0	12·0	8·0	10·0	23·0	0·0	..
19	880·0	857—880= (—)23	0	12·0	8·0	10·0	23·0	0·0	..
20	880·0	857—880= (—)23	0	11·0	9·0	10·0	23·0	0·0	..
21	900·0	857—900= (—)43	0	19·0	18·5	18·75	43·0	0·0	..
22	900·0	857—900= (—)43	0	19·0	18·5	18·75	43·0	0·0	..

4·3. *Construction pore pressures* :

In a rolled earth embankment where moisture content of the fill is controlled so as to obtain a dense fill, the resulting fill contains considerable volume of air in the moist soil. When this compacted layer is subjected to loads due to raising of the fill, *Hilf* has shown that pore pressures are developed inside the fill. A method for calculating the construction pore pressures has also been proposed by Hilf (14). This pore pressure development reduces stability and it is necessary to allow for this in the design computations. Actual measurements of construction pore pressures during the raising of an embankment will enable a watch to be maintained against any dangerous excess pore pressures developing. Such a study can also help in improving the Hilf method of computing construction pore pressures. Fig. 10 illustrates the development of pore pressures as the height of fill rises. Pore pressures were measured using both a hydraulic piezometer as well as an electrical piezometer. The difference in behaviour between the two has been attributed to the fact that the electrical piezometer is insensitive to negative pore pressures as no water flow is allowed in this device.

3·2·2·2. *Telemac Water level indicator system.*—This device has been developed by the Building Research Station of U. K.[8]. These are very simple gauges and use a plastic tube full of water to connect a container burried at the position for settlement measurements with a transparent stand pipe in an instrument house. Water in the stand pipe falls as water overflows from the end of the tube into the burried container until the level of the water in the stand pipe is the same as the tube and in the container. Settlement of the container is measured by the changes in the position of the water surface in the stand pipe. A second pipe is connected to the top of the container to equalise the air pressure in the container and in the air above the water in the stand pipe. A third tube is connected to the bottom of the container to remove overflowed water. The diameter of the level tube is between 3 mm. to 6 mm. The air tube is 15 mm. in diameter in some of the installations but this diameter is found to be inconvenient and a 6 mm. diameter is sufficient. Before measurements are made the level tube is flushed with air free water. Figure 2 is a sketch of the set-up.

The water overflow gauge thus simply transfers the level of an unaccessible burried container to an accessible stand pipe. It is not usually possible to have the stand pipe at the same elevation as the burried container. In such cases the stand pipe installation can be situated at a lower elevation and can be operated by using a column of water between the air tube from the container and the air above the water in the stand pipe.

The elevation of the stand pipe has to be connected to a stable reference datum by accurate levelling.

The water gauge has an accuracy of $\pm$ 2 mm. The overall accuracy including the Geodetic levelling is $\pm$ 3 mm. which is usually adequate for use in earth embankments.

3·2·2·3. *Other Devices.*

(*a*) *Swedish well.*

This device has been used in several dams in Europe. It consists of a vertical column of large diameter reinforced concrete pipes. The pipes are installed in pits during construction of the embankment and are separated about 3 to 5 cm. by wooden wedges. As the fill progresses the wedges are removed so that each pipe section becomes independent and has no connection with the adjacent ones and is held in place by the surrounding embankment material. With this arrangement the pipes are free to move and the three components of the movement could be measured separately. In a typical installation, the pipe section was 150 cm. diameter and 95 cms. long.

On account of its open structure, the Swedish well can be located only in the core or in the downstream of the core to ensure prevention of seepage water into the well[9].

(*b*) *S.M.R.D. Probe.*

A simple and elegant method for measurement of settlement of embankments has been recently developed in the Soil Mechanics and Research Division, Chepauk, Madras-5. The system consists of placing a set of steel plates, 60 cm. $\times$ 60 cm. $\times$ 6 mm., at 3 m. intervals. These plates are provided with a central circular hole of 75 mm. diameter. 63 mm. diameter PVC. rigid pipes or A.C., pipes are introduced through these holes such that the pipe is in a vertical position with the metal plates embedded in the embankment at 3 m. intervals. A transistorised probe has been developed in the Soil Mechanics and Research Division by which it is possible to locate the position of these embedded plates very accurately within $\pm$ 1 mm. The probe is lowered into the PVC pipe attached to a steel tape and when the probe reaches each of the embedded plates the attached meter shows an indication. At this location the steel tape is read. Such observations could be repeated at suitable intervals of time and therefore the settlement could be deduced. The instrument has been fully developed and is under field trials [10].

3·2·3. *Horizontal movements.*

3·2·3·1. Horizontal cross arm device—The method consists in emdedding a suitable cross arm system for horizontal movements which is similar in principle of operation to the vertical cross-arm device. In the U.S.B.R. device a system of mechanical linkage and counterweight arrangement is used to transfer the horizontal movement into vertical movements and they are measured by the usual torpedo and scale arrangement.

3·2·3·2. However a more elegant method is to use an electrical sensing device. The system consists of a plastic pipe 75 mm. diameter threaded through metal rings or plates and these are placed in a trench during construction to extend from the downstram slope of the dam into the clay core. After installation the possition of the metal plates are meaured with reference to a reference plate near the mouth of the pipe, located in the instrument house. An electrical probe using the induction coil principle is passed into the pipe and this coil detects the embedded plates. Distances of these plates from the reference plate are measured by a steel tape attached to the probe. The probes are pulled into the plastic pipe with a duct motor driven by compressed air and they drag behind them a tube for compressed air, a steel measuring tape and a co-axial cable for the electrical sensing device. These are paid out from drums mounted on a frame and are guided suitably by pulleys to pass into the mouth of the plastic pipe [11].

An alternative arrangement utilises a fixed pulley at the end of the installation and an endless wire system is used to pull the probe forward and backward in the plastic pipe.

Distance measurements are made by first driving the motor to the extreme end and then pulling back the tape and tubes continuously so that the electrical sensor indicated the position of all the plates with reference to the fixed plate at the entry of the plastic pipe in the Instrument house. This reference plate is connected by accurate levelling to fixed bench marks and monuments. Accuracies of the order of $\pm$ 2 mm. are obtainable with this system.

This type of installation has not been extensively used in our country. Typical Installation of Scammondan Dam is illustrated in the figure 3.

3·2·3·3. *Wilson slope indicator.*

This is a precision instrument for measuring the inclination of nearly vertical surfaces[12]. It consists of a precision pendulum which is a moulded circular

resistance element pivoted on instrument ball bearings with an eccentric weight on the rim. This is enclosed in a water-tight alluminium cylinder 60 mm. outside diameter and 375 mm. long Attached to the same frame as the pendulum is a relay which when activated by an electric current, makes contact with the resistance element and thus forms two arms of a Wheatstones bridge. The other two arms and necessary controls are provided in a control box which is connected to the indicator through a heavy duty cable. The instrument has two sets of guide wheels. Fixed guide wheels aligned in the vertical plane parallel to the axis of the instrument are mounted on one side while on the other side are provided spring loaded guide wheels. The inclinometer in effect measures the angle to vertical of a line joining the upper and lower wheels on the instrument.

In practice an aluminium tube is installed in a bore hole. Four equally spaced longitudinal grooves in the tube, guide the wheels on the inclinometer and ensures that the instrument keeps the same orientation throughout. By lowering the instrument into the tube it is possible to measure the inclination of the tube. Changes in inclination indicate horizontal ground movements. Readings of inclination are taken every metre and the inclinations could be converted into displacements, which may be added up algebraically to give the lateral displacements.

3.3. *Measurement of pore pressures* :

3·3·1. The following types of instruments are generally made use of for measurement of pore water pressures:—

(*a*) Open stand pipes.

(*b*) Hydraulic piezometers.

(*c*) Diaphram type piezometers.

3·3·2. *Open stand pipes* :

The installation consists of a length of porous carborandum or alundum tube set in a hole which is drilled into the foundation to the pre-determined elevation. The porous tube is plugged at the bottom and is surrounded by clean sand. A small diameter plastic riser pipe leads to the ground level (Figure 4). The pressure of pore water surrounding the porous tube causes flow into the piezometer until pressures are equalised by the head of water in the stand pipe. The elevation of water in the plastic tube is determined by an electrical sounding device lowered from the ground surface.

Each porous tube piezometer is an independent installation and may therefore be utilised to provide pore pressure data or ground water data along canal banks, etc.

The porous tube is 37 mm. outer dia. with 6 mm. wall thickness and 30 to 60 cm. long. Because of its relatively large size and the requirement of rise of water in the stand pipe to equalise the pressures, the installation suffers from a large time lag. The other disadvantages are that fines from the fill penetrates into the backfill altering the effective dimensions of the porous space. In soils which are partially saturated air bubbles affect the flow of water and slows down the piezometer response.

3·3·3. *Hydraulic piezome e* :

Basically the installation consists of a piezometer tip from which two tubes are taken out to a gauge well where pressure gauges and flushing and clearing system is installed. The porous tip is buried in the fill in the position at which pore pressure measurements are required. This tip separates the pore pressure from the total pressure in the fill; it must be strong enough to withstand the total pressure without crushing and the porous area should be sufficiently large enough to allow the required flow of pore liquid required to operate the pressure measuring apparatus to occur within the required response time. In unsaturated fills it becomes necessary to prevent the entry of gas into the measuring system while allowing the free passage of pore water. This is ensured by the use of porous plates with high air-entry value.

The connecting tubes must be completely impervious to both air and water and strong enough to prevent collapse under the most adverse conditions of total stress. It has been found that the usual polythene is slightly permeable to air but not to water and that nylon is impermeable to air but would absorb water. Therefore a combination of Nylon tube coated with polythene is now made use of. Another suitable material for the connecting tubes is polyvinylide Floride (PVNC). The purpose of providing two tubes from the tip to the instrument house is to enable circulation of air-free water to remove any gas which had got into the system. Gas in the system adversely affects the response time and can also cause erroneous readings.

Figure 5 gives details of a typical installation showing piezometers installed in a chosen cross-section. Tips are located mostly in the central core section in a number of tiers. Leads from all the piezometers tips are taken to a terminal well, the individual tubes are connected to a manifold system and pressure gauges. The manifold system is common to all the tips and it is so designed as to permit the independent operation of any one piezometer tip without disturbing the others. The system enables the circulation of air free water and also to check the accuracy of the individual gauges against a master gauge. Details regarding installation procedure are available in Ref. [4].

3·3·4. *Diaphragm type piezometers* :

3·3·4·1. *Electrical piezometers* :

The reliability of the hydraulic piezometers has been well established. Many of its short comings have been over-come by the use of ceramic porous stones of high air-entry value. However it cannot be used in locations where the tubing passes through an area of negative pressures sufficient to cause cavitation. Electrical piezometers were developed to over-come this defect.

The electrical piezometers basically fall into two categories. In the first the pore water pressure causes the deflection of a thin diaphragm which is compensated by equalising the air pressure on the other side, equality being detected by an electrical contact system. The Gold Beck Pressure Cell belongs to this category.

In the second category a strain gauge is attached to the diaphragm and this measures the deflection of the diaphragm. The strain gauge readings are calibrated against pressures. The strain gauge may be either an electrical resistance type gauge which changes in its resistance due to deformation of the diaphragm or may be a vibrating wire type of gauge.

5. *Conclusions* :

The degree of accuracy obtained from measurements depends not only on the type of the instrumentation but also on the personnel making the measurements. Electrical gauges have the further disadvantage of introducing errors due to unknown changes in the cables and their deterioration with time. The reliablity of measurements could be improved by increasing the number of measurements of the same type in similar locations in the embankment. If measurements are available for several instruments taken under similar conditions, it would be possible to obtain average and omit erratic readings. In addition to providing enough instrumentation of a particular type to obtain reliable data, it is often possible to obtain confirmation of measurements from a different type of instrument.

In conclusion, it would be stated that greater reliability over longer periods of time can be obtained from those instruments for which measurements can be made from the surface by means of sounding devices or probes than from instruments which are completely buried in the embankment.

As compared to the practice in advanced countries such as U.S.A., U.K. etc., we in India have not devoted as much attention to instrumentation of dams and structures as is desirable. Both in the number and types of instruments embedded as well as in the study and analysis of data we are somewhat lagging behind. We should take active step to ensure that the years to come witnesses advancement in this field also in our country .

6. *References* :

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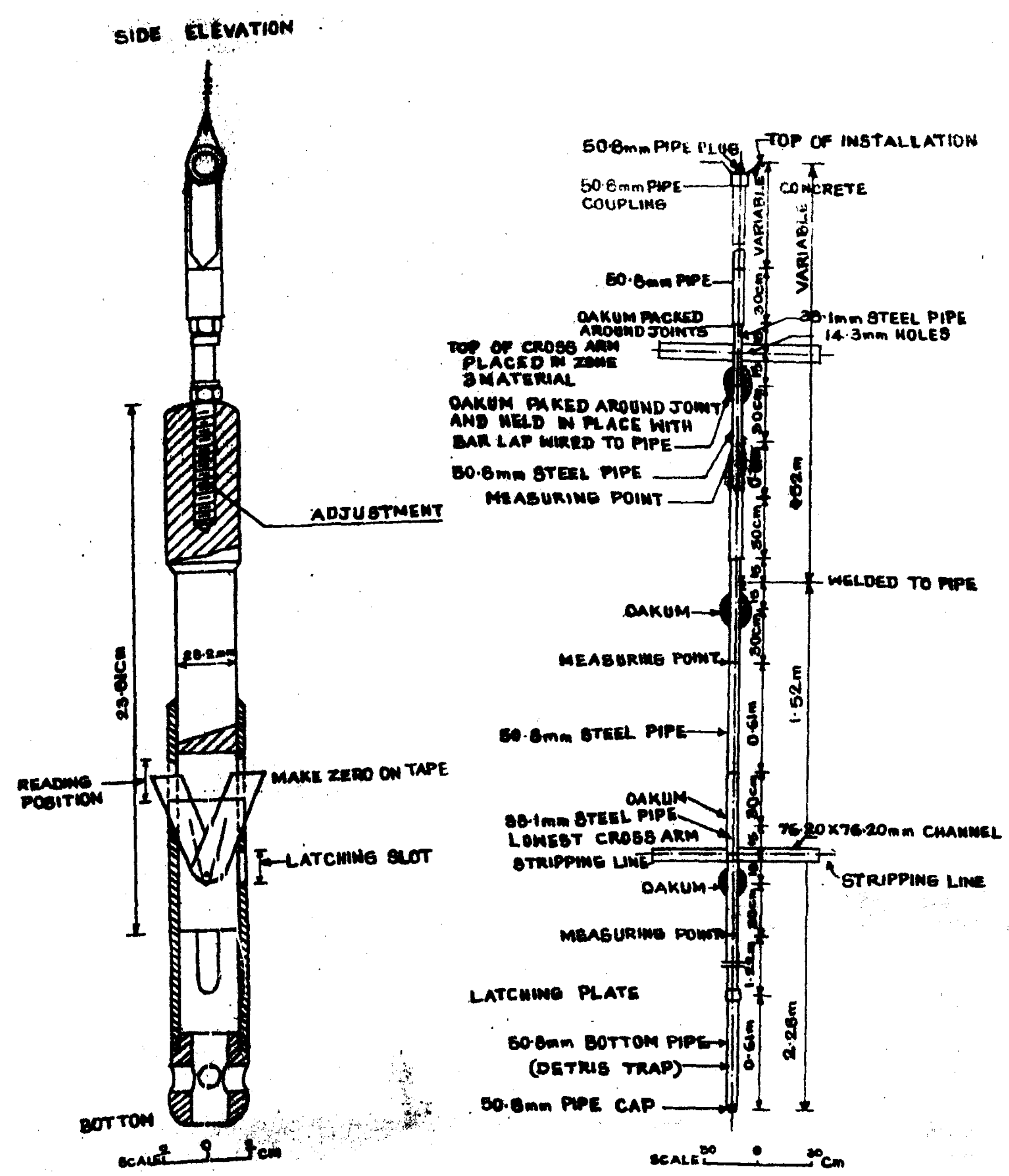

FIG. I

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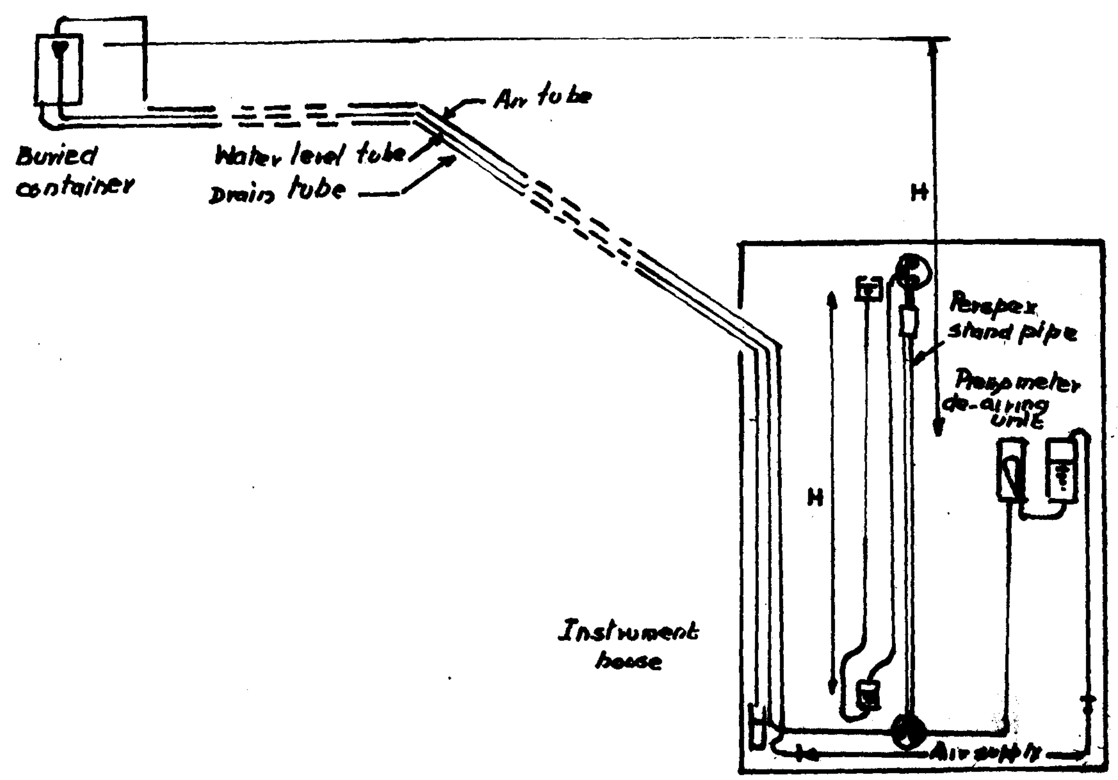

FIG.2 WATER OVERFLOW SETTLEMENT GAUGE.

G.DN/NIE/3/1-75

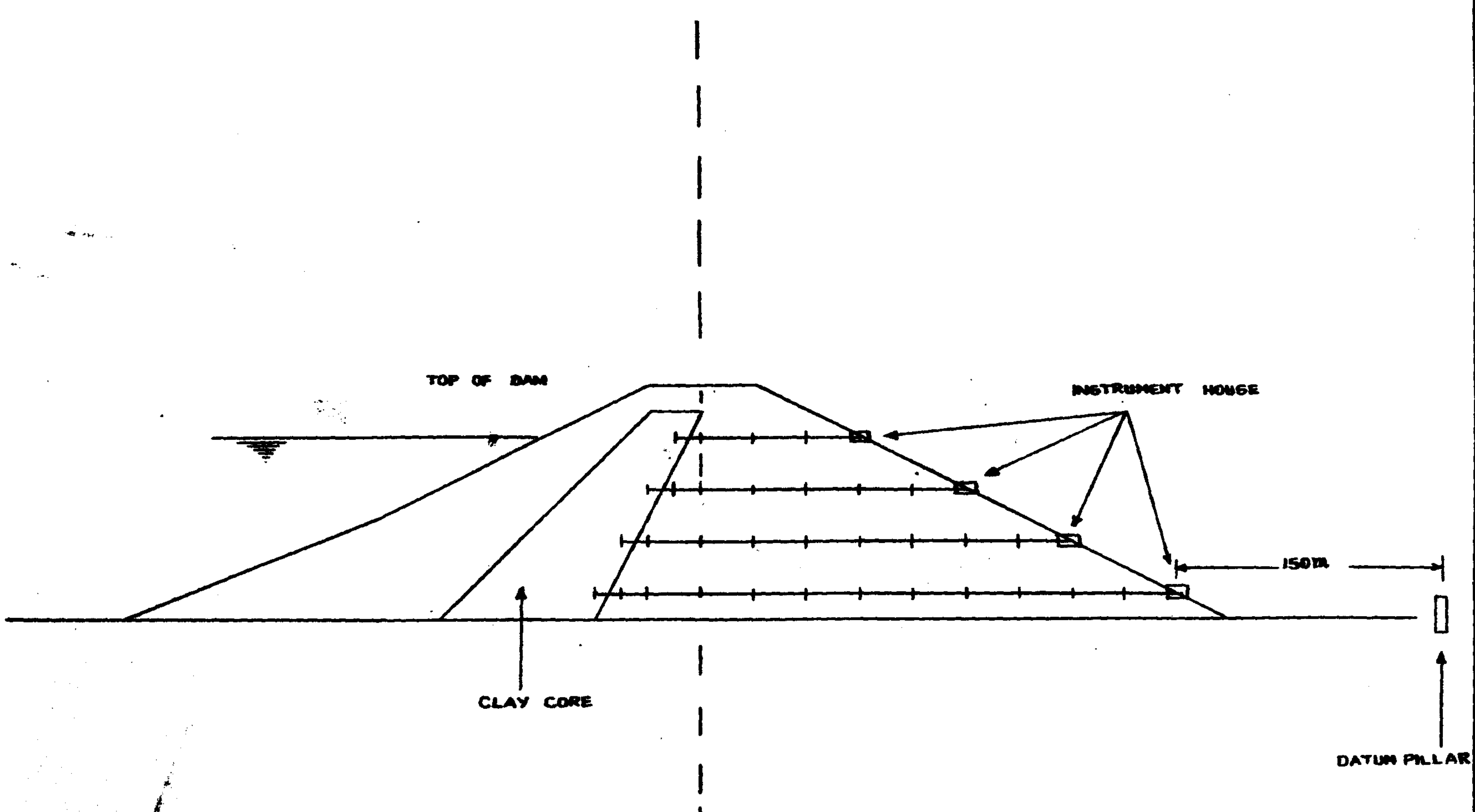

FIG. 3 TYPICAL HORIZONTAL PLATE GAUGE INSTALLATION

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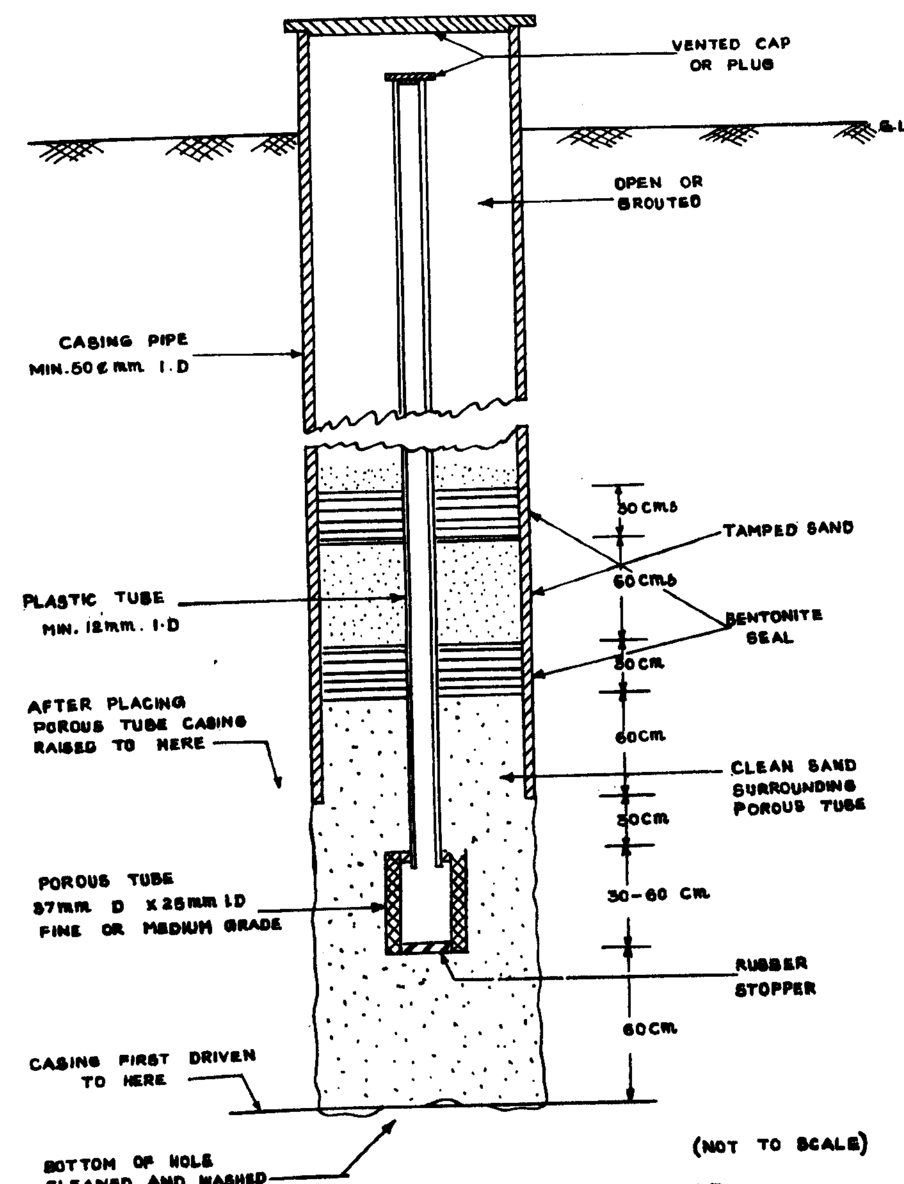

(NOT TO SCALE)

FIG. 4 OPEN STAND PIPE

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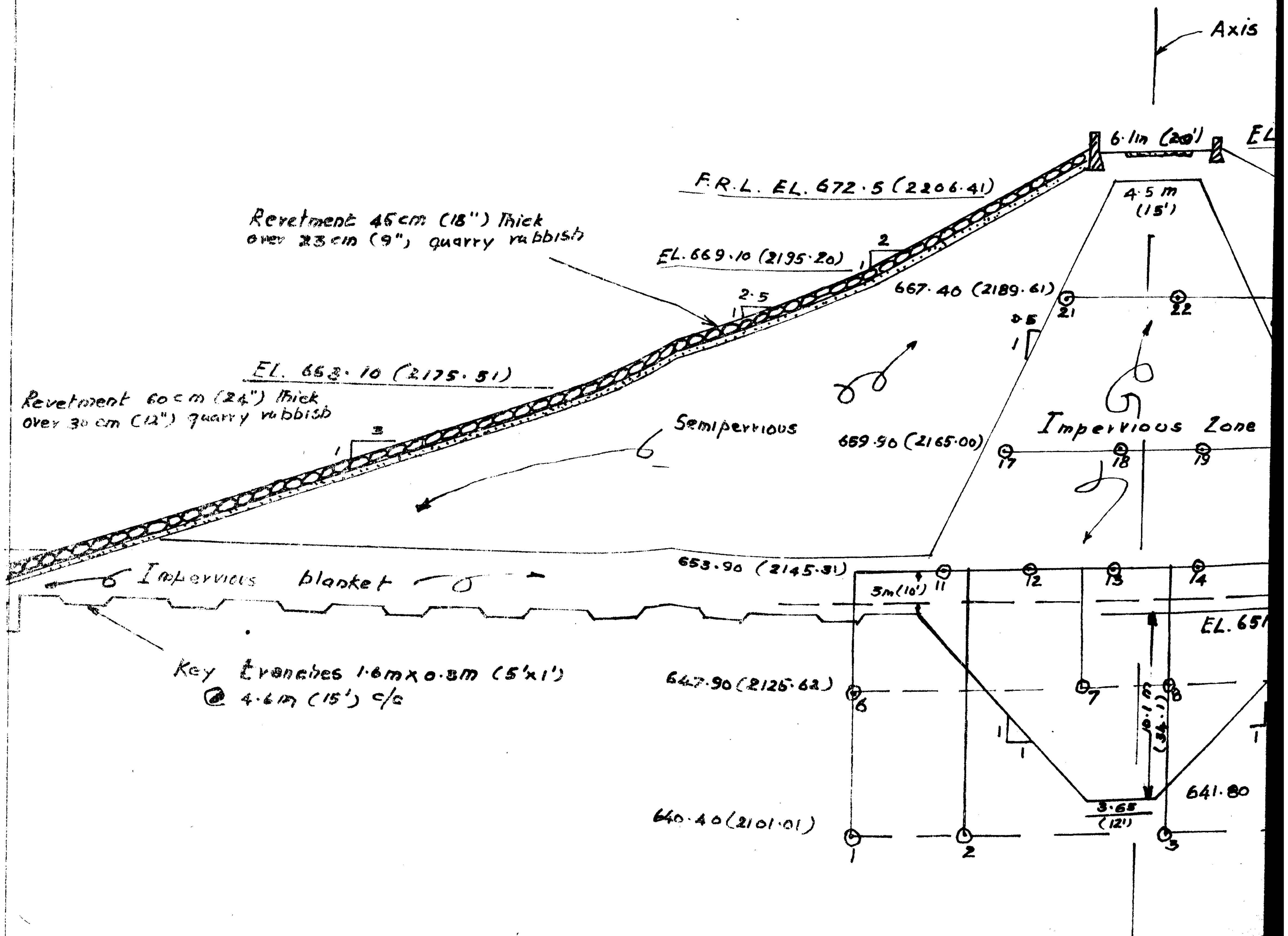
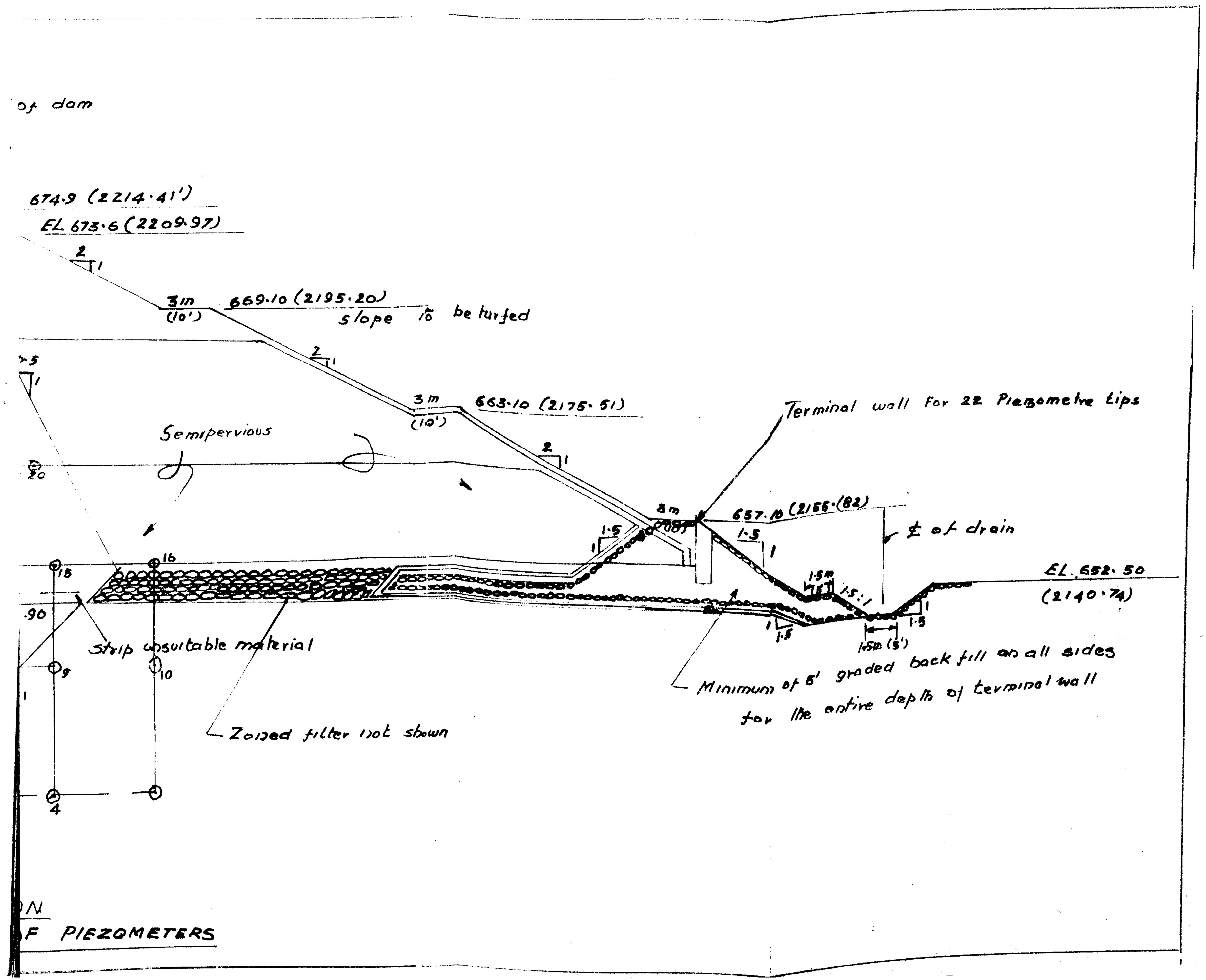


FIG. 5 EARTH DAM SECTI

SHOWING INSTALATION O



of dam
674.9 (2214.41')
EL 673.6 (2209.97)
2
1
3m
(10')
669.10 (2195.20)
slope to be turfed
2
1
3m
(10')
663.10 (2175.51)
Terminal wall for 22 Piezometre tips
Semipervious
2
1
3m
(10')
657.10 (2156.82)
1.5
1
1.5
1
℄ of drain
1.5m
1.5:1
EL 652.50
(2140.74)
16
13
.90
Strip unsuitable material
1.5
1.5m (5')
Minimum of 5' graded back fill on all sides
for the entire depth of terminal wall
9
10
Zoned filter not shown
4
ON
OF PIEZOMETERS

GDN/NIE/9/1-75

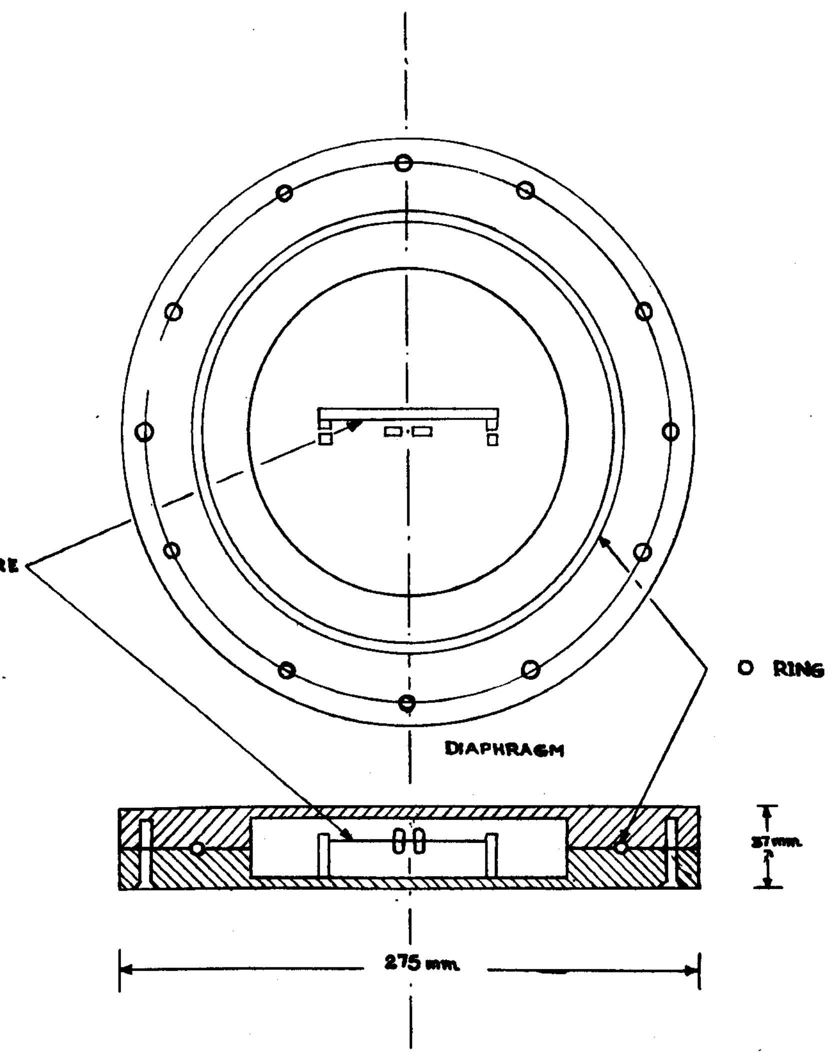


FIG.6 BRS EARTH PRESSURE CELL

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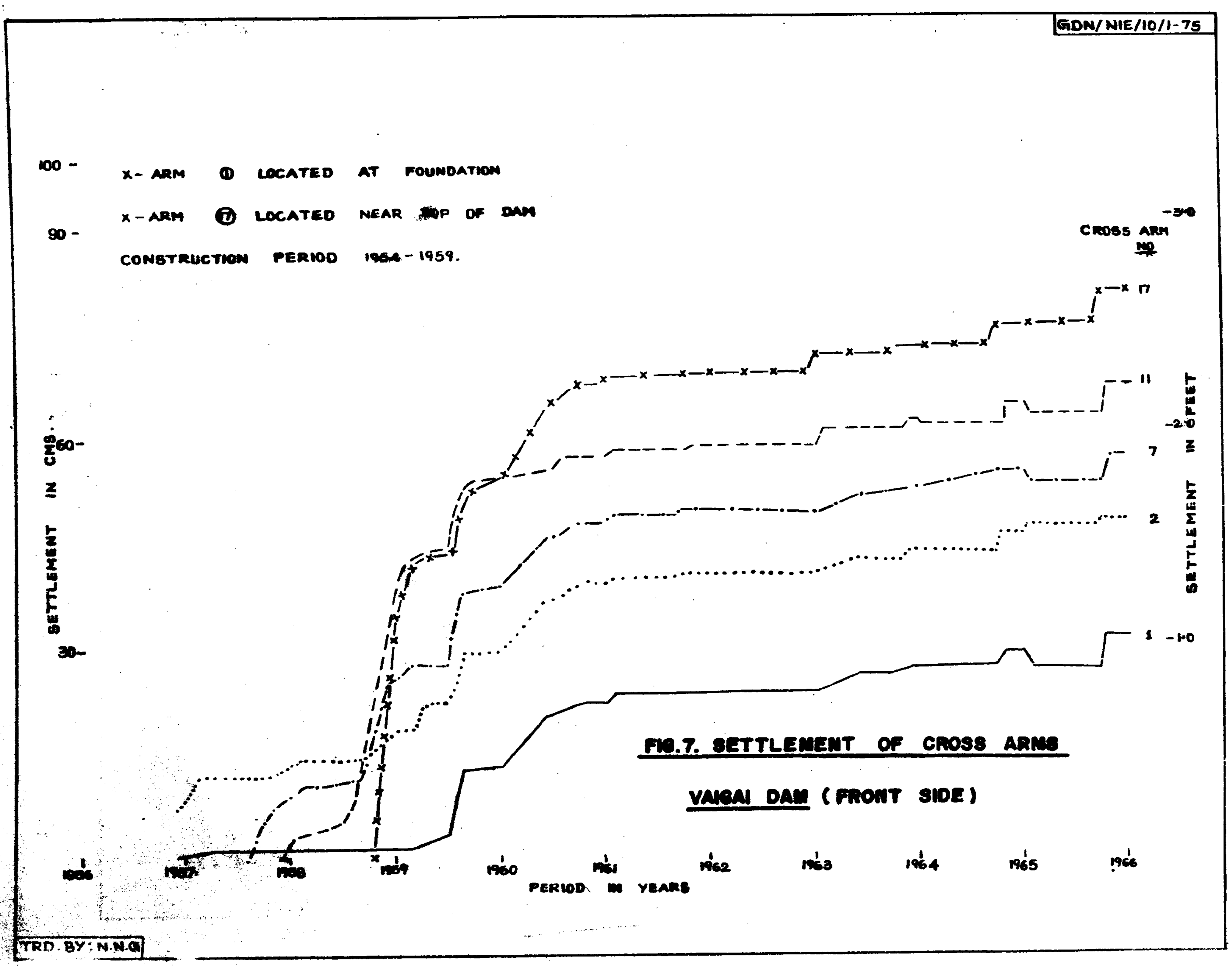

FIG.7. SETTLEMENT OF CROSS ARMS

VAIGAI DAM (FRONT SIDE)

CONSTRUCTION OF HYDRAULIC MODELS.

By Thiru T. INDRAN*

1·0. *Synopsis.*

Now-a-days the hydraulic models have been very familiar for their own advantages of solving number of problems of Civil Engineer in the Projects. The common problems which are facing the Hydraulic Engineers are sand accumulation in Irrigation Canals, Vibration of shutters in the sluices, cavitation in Hydraulic Structures, and energy dissipation in rear of spillways, etc. All these challenging problems are being effectively solved by Hydraulic Engineers with the help of models at economical cost. Nowhere in the world to day is a major hydraulic structure designed and built without prior analysis through model experiments. But, for the models they should be as small as possible, representing all the parameters of the prototype, without any deviation from the hydraulic characteristics of the simulitude. Before constructing the model, scale selection plays an important role. It depends upon the extent of area available for the model, discharge available for each model and the accuracy of measurements required. If the studies are not of calibration nature the degree of accuracy may not be given much importance. In this paper the method of construction of a comprehensive Model for a reservoir project is described.

2·0. *Introduction—*

2·1. A Reservoir Project is generally formed across a river by constructing a dam. Normally a reservoir project will consists of an earth dam, masonry dam for locating spillways, river and canal sluices. River sluices are provided to maintain the Riperian rights of the ryots in downstream of the dam and also to stabilise the existing ayacut. Canal sluices are provided to supply water for the new ayacut.

2·2. The best location for the spillway and river sluice is at the river course itself so that the still level of the sluice may be kept at the lower bed level without any loss of command. During floods the surplus discharge maybe easily allowed through the spillwy to the river without any difficulty. When the soil condition does not permit the construction of any structure across the river course, the spillway will be located at the flanks and suitable surplus arrangement will be provided for the flood discharge. This surplus arrangements may be, either a conventional type of series of drops up to the confluency point, or a modern type of chute.

2·3. In this paper the method of constructing a three dimentional or a comprehensive model of a reservoir project representing earth dam, masonry dam, spillway river sluice, canal sluice and surplus course, if any, is described in detail with the assumed data. Normally these data will be furnished by the project authorities. Preliminary designs are usually submitted to the laboratory with the request that specified features be studied from the stand point of hydraulic performance. In some instances the request is in general rather than specific terms. In either case, the studies, especially when a model is involved are conducted in such a way that the maximum amount of pertinent information is obtained.

3·0. *Data—*

3·1. Data assumed to be furnished by the project authorities are as follows :—

Spillway—

M.W.L. & F.R.L.—El. 1672·50 m. (5486′·49)

Maximum flood discharge—849·5cumecs 30,000 cusecs

Crest of spillway—El.1667·93 m. (5,471·49)

Number of size of vents—3 of 12·2 192m × 4·572m (3 Nos. of 40′ × 15′)

Pier thickness—2·743 m. (9′—0″)

Canal Sluice—

Sill of sluice El. 1657·250m. (5,436′·45)

Number and size of vents—1 No. 1·520 × 1·830m. (5′—0″ × 6′—0″)

Length of sluice—72·00m. (236′·99)

Earth dam—

Top of bund—El. 1,674·90 m. (5,495′·032)

Road width—6·0 m (20′—0″)

In addition to the above contour plans, C.S. of spillways, canal sluice, earth dam and general plan are also furnished.

4·0. *Model set up—*

4·1. The total area of the project to be represented in the model is worked out first from the drawings furnished and for various scales the model area is calculated. Subject to the availability of area, and discharge and nature of study, suitable scale will be decided. For example if the area of the project to be represented in the model is 1,000 sq. metres and the available discharge in the laboratory is 2·5 cfts. and area for the model is 20 sq. metre for different scales, the area and discharges are worked out as given below. Since the model is a geometrically similar one Lr=Vr. Where Lr=Length scale ratio and Vr=Vertical scale ratio. Qr=The discharge scale ratio=$Lr.^{2.5}$.

Scale. (1)	*Qr.* (2)	*Prototype area.* (3) Sq. metres.	*Model area.* (4) Sq. metres.	*Prototype discharge.* (5) Cubics.	*Model Discharge.* (6) Cubics.
1/30	4,930	1000	33·3	30,000 8496 cumecs	6·1 1.727 cumecs
1/40	10,110	1000	25	30,000 8496 cumecs	3 0·850 cumecs
1/50	17,700	1000	20	30,000 8496 cumecs	1·7 0·481 cumecs

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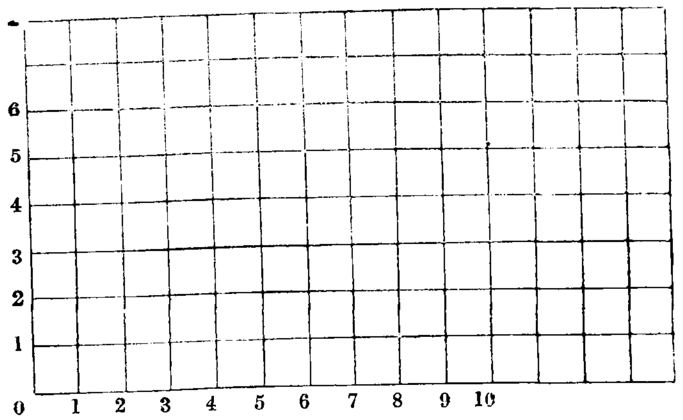
4·2. From the above table it is seen that for a scale of 150/ the discharge is only 1·7 cfts which is easily available it the laboratory, and also the area required is within the available limits. Hence a scale 1/50 is selected for constructing the model.

4·3. For the area of 20 square metre a tray with level bed should be formed. The bench mark may be fixed in such a way that the lowest contour in the general plan is just 5 cm. above the bed. Hence in this case it is fixed at El. 1645·00 m. This reference elevation has to be painted on the top of the bench mark and kept in safe condition. A suitable shed may be provided for protecting the model from sun and rain.

4·4. For transferring the contours from the plan to the tray prepared, 1·00m × 1·00 m Grid lines may be drawn and painted on the tray. The Grid lines may be numbered as shown in the figures given below:—



4·5. Now, equivalent grids of 1·00 m × 1·00 m size has to be drawn in the general plan which represents 50m × 50m in the prototype. After the completion of the grid lines in the general plan all the contours may be transferred to the tray and painted with different colours with their corresponding levels. Then the axis of the earth dam, centre line of surplus course and river course may be fixed and drawn in the tray.

4·6 M.S. rods of 6 m.m. diametre has to be fixed along the contour lines to the respective heights of the contour levels. For example if we consider El. 1660·00 m. contour the total height from the bench mark 1645·00 m. is 15 metres. This is reduced to 30 cm. of height for the model. Hence 30 cm. length of 6 mm. M.S. rods may be cut and fixed along the El. 1660·00,m. contour, with an interval of 30 cm. to 35 cm. When the sharp curves of contour is to be represented, the distance between the M.S. rods can be still reduced and fixed on the tray by cement mortor. (*vide* Photo No. 1)

4·7. To start with, a single brick wall may be constructed along the axis of earth dam upto the road level leaving the space for spillway portion and canal sluice. The canal sluice portion may be made in teakwood from entry to the exit hood and the shutter with perspex sheet. As the teakwoods are all well seasoned there will be no bulking effect during the submersion in water for any amount of time. Countrywood can never be used for all these purposes.

4·8. Regarding the spillway block the profile of the spillway has to be drawn to the scale 1 in 50 in the graph sheet using the given equation of the spillway profile. Based on this curve the female template may be prepared in teakwood and used for constructing the spillways. Similarly a drawing for pier has to be drawn to the scale 1 in 50 and after making the male template the piers may be constructed using cement mortor. The radial shutters may be made in G.I. Sheet and fixed in all the vents with the help of grooves provided on the sides of piers. The opening may also be marked at 30 cm. interval along the curvature of the shutters. Next the proposed chute type surplus course may be started for construction (*vide* figure 2).

4·9. After fixing all the rods along the contours, the entire portion may be moulded by filling with brick jelly and sand. The upstream and downstream of the earth dam, Water spread area, Stilling basin arrangements, river course and surplus course, etc., have to be made leak proof with cement plaster finished smooth (*vide* figure 3).

4·10. For allowing water to the model from the main flume suitable arrangements have to be made such as leading flume, stilling tray, and honey combs, etc. (*vide* figure 4). For measuring the quantity of the water allowed into the model a 90° ' V ' notch has to be fixed across the leading flume. It has to be constructed so that the water from the ' V ' notch freely falls into the stilling bay and to avoid the turbulance and uneven flow on the basin honey combs may be provided (*vide* figure 4). The honey combs may be circular shaped immediately after ' V ' notch and a straight honey comb across the stilling basin may also be constructed next to the circular honey comb.

4·11. In the downstream portion tail gate arrangements may be provided to build up the rear M.F.L. Water discharged through the surplus arrangement may be measured in downstream portion with 90° ' V ' notch. This is to verify whether the actual quantity of water allowed in upstream flows without any leakage in the model and also to maintain a stable and steady condition of discharge for making observations, in the model. In the downstream portion also a small basin has to be constructed and suitable honey comb arrangements may be provided to get an even flow (*vide* figure 5).

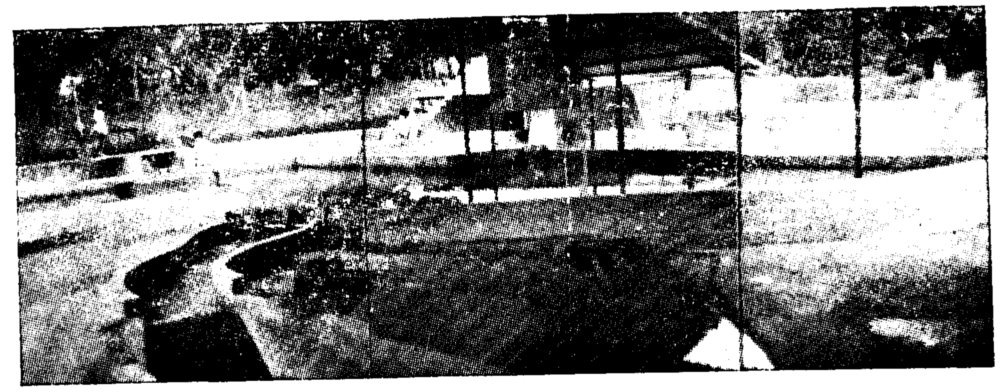
4·12. For measuring the discharges in upstream and downstream side (*vide* figure 7) two gauges have to be used. For this gauge wells should be constructed at a distance equal to four times the maximum head over ' V ' notch by the side of the leading flume and 2·5 cm. dia G.I. pipemay be used to allow water into the gauge well (*vide* figure 6). Similarly gauge wells have to be provided to measure the Head over crest of the spillway, and rear water levels for the surplus course.

5·0 *Conclusion*—

5·1. The same procedure may be adopted for the Construction of all the Comprehensive or three dimensional models.

Reference :—

" Note on Techniques of Hydraulic Models " by Er. P. KUMARASWAMI, Director, Institute of Hydraulics and Hydrology, Poondi.



PANORAMIC VIEW OF COMPREHENSIVE MODEL

Fig. 1 MIDDLE STAGE OF CONSTRUCTION

Photo No. 3

Photo No. 4

Fig. 5 D/S 'V' NOTCH

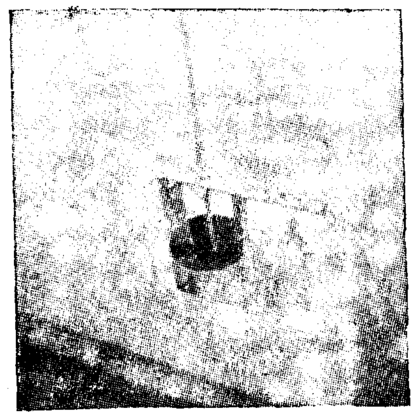


Fig 6.—U/S. GAUGE WELL WITH GAUGE

Fig. 7 D/S. GAUGE WELL WITH GAUGE

FLOW FREQUENCY ANALYSIS—CURRENT METHODS.

By

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1. *Synopsis* :

Prediction of frequency of a given magnitude of flow is important for the design of water use or control projects. Many methods are available for flow frequency analysis. Each one assumes the flow data to follow a particular statistical distribution. Statistical distributions generally used are reviewed.

The flow data of Vaigai River at Peranai has been used to illustrate the application of Gumbel's method of flow frequency analysis. The fitted Gumbel's frequency curve along with the plotting positions are shown in the figure indicate the fit of the computed line.

2. *Introduction* :

Flow frequency analysis is used by engineers as an aid in the evaluation of water use and design of water use or control projects. In some cases such an analysis gives the final solution for a flow problems but in most cases the analysis is only one of the steps in a logical engineering study in which the projects evaluation or design may advance far beyond the scope of flow frequency analysis.

A flow frequency analysis consists of a study of past records of flow to estimate frequencies of future flows. It is used for the determination of flood storage in flood control projects, estimation of runoff from airports and urban areas, design of water ways, cross drainage structures, spillways, diversion dams, bridge openings, culverts and in the study of floods flows and damages.

Frequency analysis can be directly applied to the data of flood peaks, flood volumes, mean flows, minimum flows and river stages.

3. *Commonly used methods of flow frequency analysis* :

The purpose of frequency analysis is generally the prediction of what will happen in the future. In using past records for predicting the future, it must be assumed that there is no change in the nature of factors causing floods. The past record is then considered a sample of the total statistical population consisting of past and future floods. However when predicting the future, the sample must be considered as only an approximation.

There are quite a few methods of frequency analysis differing only in the chosen statistical distribution that the recorded magnitudes of a hydrologic event are assumed to follow.

Most of the theoretical distribution used for the frequency analysis are based on three main assumption :

(i) Number of observations used in the analysis are indefinitely large.

(ii) Observations are pure random.

(iii) Factors producing hydrologic information are time independent or probabilistic rather than stochastic.

The first assumption is seldom satisfied in the real world since the record is mostly of small size. Hydrologic information are likely to be inter dependant to some extent and the factors producing hydrologic observations seldom remain unchanged. In reality, all hydrologic processes are more or less stochastic but assumed probabilistic for frequency studies to simplify the analysis (7).

Hydrologists have long sought one theoretical distribution that would describe flood events. If there were such a universal distribution, the observed distributions of flood events at various sites would differ only in parameters of that universal distribution and in sampling error. Basin characteristics do influence the flood events so that it is very unlikely that any one theoretical distribution would be generally applicable. It is also known that a frequency curve obtained from a small sample differs greatly from the population frequency curve. As a consequence of the variability of characteristics of one basin to another basin and also the sampling variability in time, several theoretical distributions are used in hydrology. In most cases, the selection of probability distribution functions is based on experience. Often two or more probability distribution functions may fit the data very well.

In general, the functions can be one, two, three and even four parameter ones, depending on the number of parameters with which they are described. The more parameters in a function, the more flexible it is in fitting any set of data. But the number of moments needed for estimating parameter increases as the number of parameter increase. The reliability of moments estimated decreases with an increase in their order. Hence a via media is to be chosen between these two conflicting requirements.

The parameters of the distribution define properties of location (central tendency, boundary values), scale (dispersion or concentration), shape, asymmetry, peakedness, asymptotes and others. The goodness of fit depends on the sample size, proper selection of the function and the use of best methods of estimating parameters (4).

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Some of the more commonly used distributions in hydrology are given below :—

Normal distribution.—The equation for the normal probability density curve is

$$f(x) = \left(\frac{1}{\sigma}\sqrt{2\pi}\right) e^{-(x-\bar{x})^2/2\sigma^2}$$

where f (x) = probability density function

x = event described

$\bar{x}$ = Mean

σ^2 = Variance

σ = Standard deviation

Mean and the variance are the two parameters of this distribution.

Mean occurs at 2 years recurrence interval. Mean and mode for this distribution are the same. Values of x corresponding to various cumulative probabilities are tabulated in most of the text books on statistics. Normal probability plotting paper is one on which any cumulative normal distribution will plot as a straight line. The range of this distribution is from minus to plus infinity.

Lognormal distribution.—This is a normal distribution of In X (Naperian log). Interms of 'X', it is a three parameter distribution having a range from zero to plus infinity. It is plotted on a common logarithmic probability paper. It is a normal distribution of logarithms or skewed distribution of untransformed data.

Type I extreme value distribution (*Gumbel's*).

The return period is a statistical function as is the distribution or cumulative frequency. Based on the theory of extreme values, the calculations of return periods are related to the distribution of flows. Many changes have been made in this method since Gumbel proposed it in 1941. Though Gumbel originally specified the use of the method only to maximum daily discharges, it is being currently used with flood peaks (1). Powell in 1943 developed a probability paper for the method. U.S. Geological Survey form 9-179a, is a slight modification of Powells Plot, Whereas Powells paper has a reduced variate scale, form 9-179a does not have a reduced variate scale. He also suggested the use of Weibull plotting position formula. Use of this formula is now accepted way of computing plotting positions. Weibull formula is as follows :—

$$Re = \frac{n+1}{m}$$

n = no. of events

m=Rank of the event

Re=Recurrence interval

This is the first asymptotic distribution having two parameters. It has a fixed skew of 1·139 and therefore is not symmetrical about the mean. The mean of the distribution occurs at 2·33 year recurrence interval when the distribution is cumulated from the upper end i.e., when it is used to represent frequency of floods greater than, whereas the mean will plot at 1·59 years when it is used to represent the frequency of floods less than the given magnitude of flood. Chow has shown that for practical purposes, this is but a special case of log-probability law.

$$\text{Probability density function } f(x) = e^{-e^{-(x-u)}}$$

This can be reduced to a function involving no parameters by means of a transformation $y = \alpha(x-u)$ i.e., $x = u + y/\alpha$

The parameters are—

$u = \bar{x} - \bar{Y}_n/\alpha$

$1/\alpha = S/\sigma_N$

Where u is the mode, shape parameter

$1/\alpha$ is a scale parameter

$\bar{x}$ is the sample mean

S is the sample standard deviation

$\bar{Y}_n$ and σ_n are functions of N

N is the Number of events.

Values of y_N and σ_N for from 8 to 1,000 are tabulated by Gumbel. The magnitude of the event is given by

$x = u + y/x$

x — Magnitude of event in the sample

y — a reduced variate = ln T for T> 10 yrs.

T — Recurrence interval.

The theoretical return period T as a function of $\times$ is given by Gumbel. For T = 10, the asymptote of this function, y = ln T is almost reached.

Pearson Type III distribution.—This is a flexible distribution in three parameters with a limited range in the left direction and unlimited range to the right. When the distribution is fitted to the logarithms of the data, it is referred to as log Pearson Type III distribution. This results in a smaller skew. Pearson Type III with zero skewness corresponds to the normal distribution.

Foster introduced Pearson system of frequency distribution in 1924. The transformation of data to their logarithms results in smaller range of skew co-efficients, covering a wider range of skewness than in Foster's original approach. The results of an analysis are plotted on a probability paper—arithmetic normal paper for Pearson III and log-normal paper for Log Pearson Type III distribution.

Its probability density function with the origin at the mode is

$$f(x) = P_o\,(1 + x/a)\, e^{c - cx/2} \quad \text{where } c = 4/\beta_1 - 1$$

$$a = \frac{c\,\mu}{2\,\mu}\ ;\quad P_o = \frac{NC^{c+1}}{a\,e^{c}\,\Gamma(c+1)}$$

and Mean = Mode — $\mu / 2\mu$

Standard deviation = $\sqrt{\mu}$ and

$$\text{Pearson Skewners} = \sqrt{\frac{\beta 1}{2}}\ ;\quad \beta = \frac{\mu^3}{\mu^2}$$

μ and μ are second and third moments about the mean.

The three parameters of Log Pearson Type III distribution are

$$\text{Mean : } M = \frac{\Sigma X}{N}$$

$$\text{Standard Deviation } S = \sqrt{\frac{\Sigma(X-M^2)}{N-1}}$$

$$\text{Co-efficient of Skew, } g = \frac{N\Sigma\,(X-M)^3}{(N-1)\,(N-2)\,S^3}$$

Frequency Curve Co-ordinates are given by :

log Q = M + KS

in which

M = Means of the logarithms

X = logarithmic magnitude of an event

N = Number of events in the record

S = Standard deviation of the events expressed in logarithms

g = Co-efficient of skewness

K = Pearson Type III Co-ordinates expressed in magnitudes of standard deviations from the mean for various exceedance percentages.

Q=Computed flood flow.

Gamma distribution.—Though this is considered as equivalent to Pearson type III distribution, in flow frequency analysis these two are considered separate, possibly because of the different methods used in estimating the parameters. Pearson's Tables of the incomplete gamma function is used to obtain the cumulative probabilities. Log normal paper is used for plotting. This is particularly useful where the magnitude of the variable is likely to be zero or very near zero many times.

The probability density of this distribution is

$$p(x) = \frac{X^{a}\, e^{-x/b}}{b^{a+1}\,\Gamma(a+1)} \qquad \begin{array}{l} \text{with } o \leq x < \infty \\ \quad b > o,\ a > -1 \\ \text{for } x = 0 \\ \text{and } p(\times) = 0 \\ \text{for } x \leq 0 \end{array}$$

where a and b ⇔ are constants and $\Gamma(a+1)$ is a gamma function.

The cumulative gamma distribution is given by

$$F(x;a,b) = \begin{cases} \dfrac{1}{b^{a+1}\,F(a+1)} \displaystyle\int_0^x x^a\, e^{-x/b}\, dx & \text{for } x \leq 0 \\ 0 & \text{for } x < 0 \end{cases}$$

This is known as the incomplete gamma function and the values are tabulated for different values of a and b.

In using these tables to determine the probability that a random value from a gamma distribution with parameters b and a ; 1 takes a value less than x, we first determine,

$$u = \frac{u\ x/b}{\Gamma\, a+1}$$

Then find I(u, p) from the tables. This value is the desired cumulative probability, F(x, a+1, 1/b), (the probability that a random value from a gamma distribution takes a value less than x)

The statistical parameters are

1. Mean = b (a + 1)
2. Variance $\sigma^2 = b^2\,(a+1)$

The use of two parameter gamma distribution is as common as the lognormal distribution. It is not easy to transform the co-ordinate scales in such a way that all cumulative gamma distributions plot as straight lines.

(a) and (b) are determined from the sample data using the relations.

$$b = \frac{\sigma^2}{\bar{x}}\ ;\quad a+1 = \frac{x^2}{\sigma^2}$$

Frequency Analysis Using Frequency Factors.

In general, if a three parameter distribution is used, the sample parameters are computed and they are treated as though they are population parameters. These parameters are used in the corresponding formula for the distribution. Since the distribution cannot be integrated directly, the relation between magnitude and probability of exceedance is determined from a table of frequeny factors for the chosen distribution and from the general formula (3).

$x = \bar{x} + KS$

where x is the variable

$\bar{x}$ is the mean of the sample

S is the Standard Deviation of the sample

K is the frequency factor for the particular distribution.

Frequency factors for Normal distribution are available in any standard book on statistical analysis. For Lognormal distribution and Type I extreme value distribution, Chow has given frequency factors and for Pearson Type III distribution, Beard has given the factors (3). Also available are graphs connecting K and recurrence interval, with co-efficient of skew as parameter for different distributions.

Graphical fittings.—This does not require the assumption of any theoretical distribution. Each individual in the sample is assigned a probability using any of the plotting position formula. Then magnitudes of the individuals are plotted against probabilities of recurrence intervals and a line is drawn to properly interpret the points. Any probability paper may be used. Only if they plot as a straight line can we say that the magnitudes follow the particular distribution of the probability paper being used.

Hazen Method.—This is one of the more improved graphical fittings. Hazen's plotting position formula is used to compute the frequency of floods greater than a given flood.

$$P = \frac{2m-1}{2n}$$

where m is the rank of event, n is number of events and Pis Probability of equalling or exceeding. They are plotted on a Hazen (Lognormal) probability paper. The subjectively involved in drawing a line to represent the data points is very much reduced by adopting a method suggested by Hazen.

Co-efficient of skew and co-efficient of variation are obtained for the sample. To bring all estimates of skew to the same basis, an adjustment, for values obtained by calculation from short term records is carried out, the adjustment factor being given as

$$F = \frac{1 + 8.5}{m}$$

This means that the computed Co-efficient of skew must be multiplied by F to obtain the adjusted value. Certain factors depending on the Co-efficient of skew are multiplied by the Co-efficient of variation and the products are added to or subtracted from the mean to obtain plotting points for a smooth line. The factors are graded according to the skew and are given by Hazen. Some users omit the application of Hazens' skew factors, but otherwise follow the Hazen Method.

Application of Gumbels, distribution to the flow data of Vaigai River at Peranai—As an illustration, the technique is applied to the annual peak flows of Vaigai River at Peranai. Plotting positions were obtained using Weibull formula.

$$Re. = \frac{n + 1}{m}$$

Where n is the total Number of events

m is the rank of the event

The results are presented in a U. S. Geological Survey from 9-179a probability paper. Computations are given in Appendix 1.

1. X, Mean of the sample— $= \frac{\Sigma X}{N}$

2. $S = \frac{(x - \bar{x})^2}{N-1}$

$$\Sigma X = 825216$$

$$\bar{X} = \frac{\Sigma X}{N} = \frac{825216}{26} = 31740 \text{ C. ft./s}$$

$$S = \sqrt{\frac{12831.359 \times 10^6}{26 - 1}} = \sqrt{513254360}$$

$$= 22660$$

From the tables 'Means and Standard deviations of reduced extremes'

$\bar{Y}_N = 0{\cdot}5309$ and $\sigma_N = 1{\cdot}091$ corresponding to $N = 25$ and

$\bar{Y}_N + 0{\cdot}5362$ and $\sigma_N = 1{\cdot}112$ corresponding to $N = 30$

Since the data available is for 26 years $\bar{Y}_N$ and σ_N value are to be interpolated

For $N = 26$

$$\bar{Y}_N = 0{\cdot}5309 + \frac{0{\cdot}0053}{5} + 0{\cdot}5309 + 0{\cdot}00106 = 0{\cdot}53196$$

$$\sigma_N = 1{\cdot}091 + \frac{0{\cdot}021}{5} = 1{\cdot}091 + 0{\cdot}0042 = 1{\cdot}0952$$

$$\frac{1}{\alpha} = \frac{S}{\sigma_N} = \frac{22660}{1{\cdot}0952} = 20690$$

$$U = \bar{X} - \bar{Y}_N/\alpha$$

$$= 31740 - 20690 \times 0{\cdot}53196$$

$$= 31740 - 11010$$

$$= 20730.$$

the equation of the straight line.

$$x = u + y/\alpha$$

$$= 20730 + 20690\, y.$$

The relation y = In T may be used to define plotting position for large recurrence intervals

$$y = \text{In } T = 2{\cdot}303 \log_{10} T$$

for T=50 yrs. $y = 3{\cdot}91$, and

$$X = 20730 + 3{\cdot}91 \times 20690$$

$$= 20730 + 80910$$

$$= 101640$$

The fitted line is defined on the graph by the points

$\bar{X}$=31740 cfs at 2·33 years and

101640 cfs at 50 years.

The plotting positions are also shown to indicate the fit of the computed line.

Conclusion.—Various methods of flow frequency analysis have been briefly described. Many of them differ only in the theoretical distribution that the data is assumed to follow. However, the estimation of parameters of each distribution may be determined either by the method of moments or by the method of maximum likelihood. While the computations are tedious in the latter method, they are relatively simple in the former method. In the methods indicated, parameters are determined by the method of moments.

As an illustration of Gumbel's method, the technique is applied to the annual peak flows of Vaigai River at Peranai Plotting positions along with the fitted line are presented in the figure. Other distributions can also be applied and the most suitable distribution can be recommended for the basin as well as any other hydro-meteorologically similar basin.

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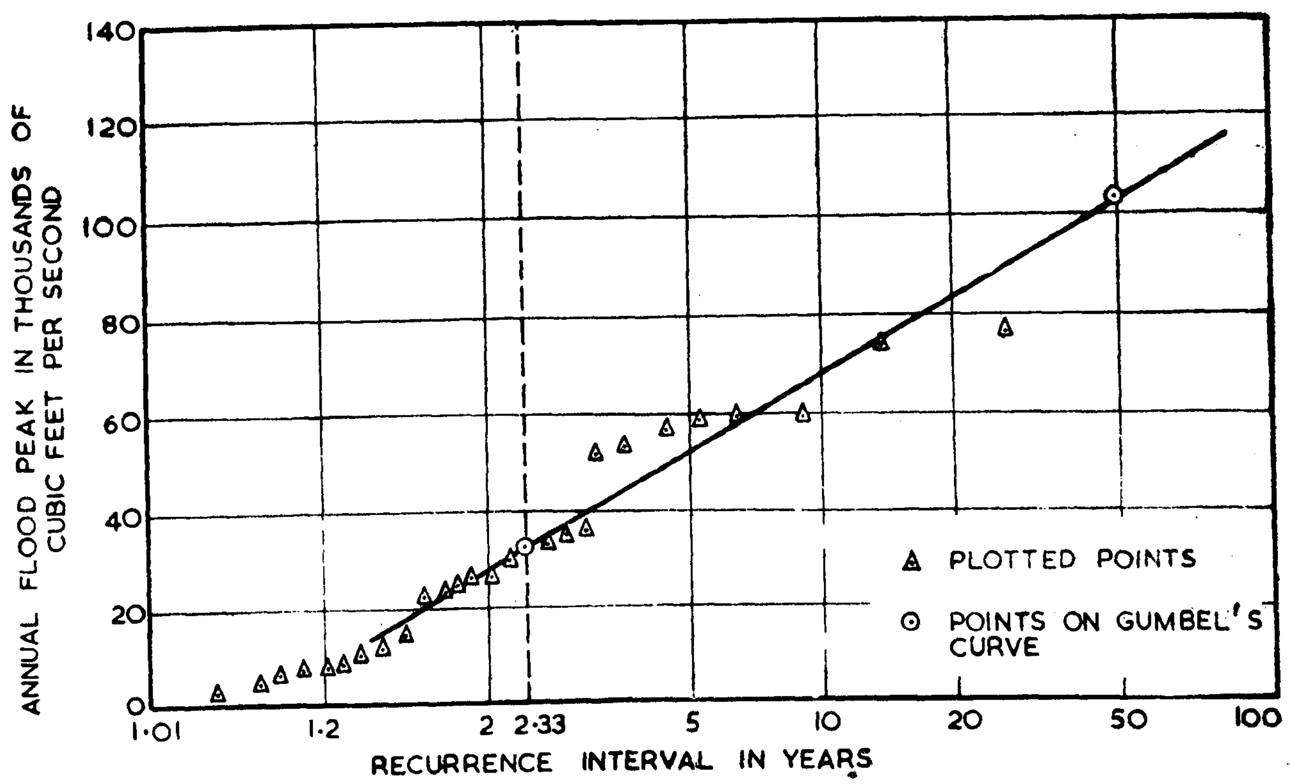

GUMBEL FREQUENCY CURVE SHOWING AGREEMENT WITH PLOTTED POINTS.

APPENDIX 1.

COMPUTATIONS FOR THE APPLICATION OF GUMBEL'S METHOD

Rank.	x	$x-\bar{x}$	$(x-\bar{x})^2$	$R=\frac{n+1}{m}$
1.	76378	44638	1992×10^{6}	27
2.	74385	42645	1819×10^{6}	13·5
3.	59990	28250	7980×10^{5}	9
4.	59389	27649	7645×10^{5}	6·75
5.	58773	27033	7308×10^{5}	5·40
6.	56951	25211	6356×10^{5}	4·50
7.	52583	20843	4343×10^{5}	3·857
8.	51692	19952	3981×10^{5}	3·375
9.	36650	5090	2590×10^{4}	3
10.	34095	2355	5546×10^{3}	2·7
11.	33384	1644	2703×10^{3}	2·455
12.	31737	(—)3	9	2·250
13.	26000	(—)5740	3295×10^{4}	2·077
14.	25443	(—)6297	3967×10^{4}	1·928
15.	24206	(—)7534	5675×10^{4}	1·800
16.	23769	(—)7971	6356×10^{4}	1·688
17.	22000	(—)9740	9488×10^{4}	1·588
18.	14720	(—)17020	2896×10^{5}	1·500
19.	12730	(—)19010	3614×10^{5}	1·421
20.	12205	(—)19535	3818×10^{5}	1·350
21.	8900	(—)22840	5217×10^{5}	1·285
22.	8051	(—)23689	5613×10^{5}	1·227
23	7688	(—)24052	5784×10^{5}	1·174
24.	7200	(—)24540	6023×10^{5}	1·125
25.	4800	(—)26940	7261×10^{5}	1·080
26.	1500	(—)30240	9145×10^{5}	1·038

$\Sigma x = 825216$; $\Sigma(x-\bar{x})^2 = 12831{\cdot}359\times10^{6}$

CHUTE SPILLWAYS FOR PROJECTS AND ENERGY DISSIPATING ARRANGEMENTS AT THEIR EXIT.

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Synopsis.

Firstly, in this paper, the functionary aspects of the elements forming a *Chute Spillway* are furnished. The hydraulic adaptability of a chute spillway in comparison to a conventional drop type in *Curvilinear* alignment of the surplus course is then presented based on model studies conducted on a typical surplus course of an Irrigation Project. The *Subsoil Strata of the surplus course* if purposely assumed to be *permeable* and the problem of *dissipating the energy of high velocity jet at the exit of Chute* under such condition is also discussed.

1. *Introduction:*

The resultant effect of constructing a Reservoir across a river for any purpose is to lift the plane of flow from the river bed level to maximum water lelvel of the Reservoir. Due to this lift the sheet of water that leaves the spillway possesses a datum head equal to the difference between maximum water level of the Reservoir and the river bed level reckoned with respect to river bed as reference. This datum head is responsible for the creep length considerations and uplift pressures in and around the spillway and for the kinetic energy possessed by the flowing water in the spillway channel. It is of an important concern for an Irrigation Engineer to negotiate this potential head before the flow in the surplus channel joins the river below and to make adequate arrangements to have a safe velocity of exit ensuring safety to the structure above. These actions are accomplished in a spillway.

We have different types of spillways used in varying site conditions (1) They are

(a) *Over-fal Spillway.*—It is a common type of spillway suitable for Masonry Dams where hard strata is available to withstand the heavy scour caused due to sudden fall of water.

(b) *Chute Spillway.*—This is a spillway isolated from the Dam and having a discharge channel to the river in an excavated trench.

(c) *Side Channel Spillway.*—A spillway in which the flow after falling from a weir flows in a direction essentially parallel to the axis of the weir.

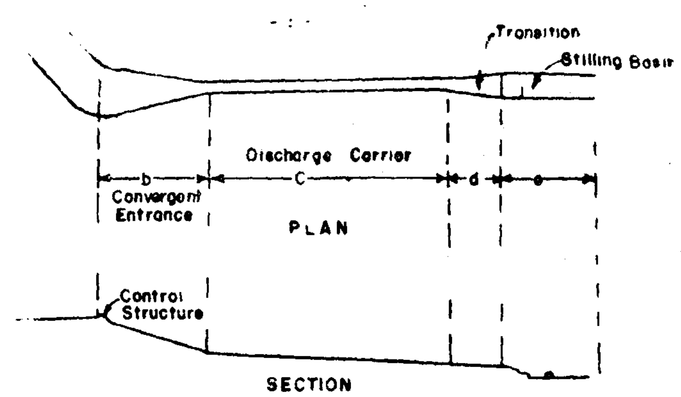
(d) *Shaft Spillway.*—A spillway with a funnel shaped entrance discharging into a shaft or tunnel which leads the water to the river. Suitable where ample space is not available.

(e) *Syphon Spillway.*—Spillways functioning on the principle of syphon.

Out of the above mentioned spillways the main aspects of the chute spillway in general and the problem of energy dissipation at its exit with particular reference to the chute type surplus course for a typical Irrigation Project are given.—

2. *Chute Spillway :*

A chute type spillway in contrast to a drop type spillway negotiates the fall in a continuous manner. The main elements of a chute type spillway are shown in the sketch below :



SKETCH SHOWING ELEMENTS OF CHUTE

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2·1. *The control structure :*

This structure controls the flow from the Reservoir into the chute.

2·2. *The convergent entrance :*

This portion marked ' b ' in the sketch leads the water from the control structure to the discharge carrier marked ' c ' in the sketch. The flow in the convergent entrance is made subcritical so as to turn the flow in the desired direction of the discharge carrier.

Another function of this convergent entrance is to minimise, if not to completely eliminate, the wave propagation and cross currents in the discharge carrier. Due to turning of flow and unsymmetrical gate openings wave fronts are moving inside the dischrage carrier and this causes uneven depth of flow as revealed by the flow contours in Fig. No. 2. This phenomenon was referred by Er. Doddiah and Er. Shammanna in their study on chute spillway for Ambligola Project in Karnataka (3).

The shape of the convergent entrance will have to be decided by model studies keeping the above points in mind for each problem.

2·3. *The discharge carrier :*

This conveys the flow from the convergent entrance to the stilling pool. The flow in this region is supercritical.

This portion is taken in straight or curved alingment ; the bed width may be uniform or varying ; the section may be retangular or tropezoidal all depending on the site conditions. Vertical or nearly vertical walls with necessary superelevation is preferable for curvilinear aligment.

If the discharge carrier is given a flare it must be done according to USBR specification to confirm to the rule that tan ϕ = 1/3 F where ' ϕ ' is the angle of flare and ' F ' is the Froude Number.

The non-compliance with the above rule leads to non-uniform depth of flow and swirl formation inside the stilling pool. During the study on Beas river Earth Dam chute spillway Er. Gazinder Singh and B.D. Sharma (4) on the effect of sharp flare report that the depth of flow varied from 3·5′ to 9′ between sides of Channels to the centre of Channel for discharges varying from 1,00,000 cusecs to 3,41,000 cusecs. The heavy concentration of flow at the centre of course was overcome by them by suitably extending the central and outer piers by more than 140′

2·4. *Transition.*

The transition marked ' d ' is provided to attain an even depth of flow and to reduce the energy perlinear length of flow entering the stilling basin. If necessary flare is maintained in the dischareg carrier and the uniform flow achieved before the water enters the stilling basin, the transition need not be provided.

2·5. *The stilling basin :*

This does its usual function accomodating the Hydraulic jump and dissipation of energy of high velocity jet takes plaec here.

2·6. *Air entrainment :*

The high velocity flow in a chute absorbs air during its motion and this fact has to be taken into account while making provision for free board for various components (elements) of the chute (5).

3·1. *Model studies for chute type spillway for a typical Irrigation project :*

Model studies were conducted at our Institute of Hydraulics and Hydrology, Poondi for a typical Irrigation Project. Both a drop type and a chute type surplus course were tried for the assumed scheme and their relative merits studied. These studies with those conducted for dissipation of high energy jet at the exit of the chute are detailed below.

A typical Irrigation scheme as shown in Fig. 1-B was taken up for studying a chute type spillway having the hydraulic details as given in 3·2. It consists of an Earth Dam across a river with its Regulator at right flank and Head Sluice at centre. It is assumed that the spillway could not be located on the Deep river bed due to the non-availability of hard strata. It is further assumed that the other considerations pertaining to the Project fixes the component parts of the scheme as shown in the Fig. 1-B.

3·2. *Hydraulic details :*

M.W.L. and F.R.L.————El. 1672·50 m.

Crest level of spillway————El. 1667·93 m.

Number and size of vents——————3 vents of 12·22 m. × 4·57 m.

Design discharge 764·6 cumecs (27,000 cusecs).

Normal flood discharge 339·8 cumecs (12,000 cusecs).

River bed level at confluence point————El. 1652·30.

Nature of soil in the Surplus Course—Clayey in the top layers and sand at lower depth.

Length of surplus course from Spillway downstream apron and to confluence point—524 meters.

Level of downstream apron of regulator at the commencement of the chute Pt(A) Fig. No. 1 (L.S.)—El.X1663·83 metres.

Rear water level for 764·60 cumecs—E. 1,655·60 metres.

3·3. *The problem :*

The net head of 11·53 metres the diffeence between top level of Regulator Down Stream apron and confluence point is to be negotiated over a distance of500 meters for the designed discharge of 764·6 cumecs on cayey soil.

For this purpose the proposal is to have he conventional drop type surplus course. A series of drops marked I, II, III, with an increasing width of surplus Channel as shown in Fig. No. 1-B are provided. The major disadvantage in having this usual type in this cae and in any other curvilinear alignment of course is that the high velocity

stream leaving the spillway will not hug to the provided enlarged course fixed from considerations of topography and velocity. The jet will tend to impinge on the Right Flank of course and there will be no flow to certain distance in left side of the course as shown by Photo No. 1. This state of affairs leading to an uneven flow pattern, back flow, and eddies is undesirable and will damage the structure at particular localities. The expenditure on such damages will also be recurring in nature due to the fact such a design does not provide an even flow pattern

3·4. As an alternative a chute type surplus course was studied for the problem through model technique Fig. No. 1-B shows the various elements of the finalised proposed chute spillway for the scheme.

Portion CC—The control structure.

Portion CC KMNL—Convergent enterance.

Portion MOQRPN—Discharge carrier.

Portion QSTR—Transition.

Portion SUVT—Stilling Basin.

The main problem in proposing a chute spillway over a permeable strata like clay is the provision of proper energy dissipating arrangements at the exit of a chute. The stream of water emerging at exit of chute possesses a very high Kinetic energy due to its continued experience of the fall and also due to its flow in a comparatively smaller through along its course. This high energy jet has to be properly dealt with before it is allowed to join the river course at confluence point.

3·5. To achieve this, attempts were made to create a hydraulic jump by introducting baffles at the end of the concrete channel without increasing the channel width at L.S. 408·5 m. The introduction of baffle or flip at the end of the concrete channel (at L.S. 408·5 m) followed by a big basin does bring the velocity at the confluence point to 20′ per second, from 41·5′ per second in the chute. But the arrangements produced heading up of water and a consequent stagnation of water in the concrete channel. No jump is formed. The attempt was hence given up.

3·6. The reasons for the non-formation of a hydraulic jump at the end of concrete channel of 18 m. width was then studied. One of the important conditions to be satisfied for the formation of a good hydraulic jump is that the issuing jet should possess its kinetic and static energies in such a proportion that the jet cannot oscillate. More Scientifically the kinmatic flow factor F (the Froude Number) of the issuing jet must not be less than 4·5.

3·7. *Transition :*
The value of F at the exit point during the trial referred in 3·5 was 2·6 which falls short of the minimum value of 4·5 for a stable jump to form. The increase in Froude number in this case was accomplished by introducing a transition portion marked QSTR in the Fig. No. 1-B.

The high velocity jet from the concrete channel was allowed to expand at level 1,652·30 m. (river bed level at confluence point) and the results are produced in Fig. No. 2 and photo No. 2. Fig. No. 2 shows both the plan and longituditional section of the flow profile. The theoritical boundary of expansion for a supercritical flow given by (6) Rouse, Bhoota, Hsu is also marked in the Figure. The study reveals the following:—

(1) Supercritical flow will not follow any transition that is provided for its expansion but will have the pattern as shown in figure No. 2. The practical splay for the transition is to be decided based upon only on the flow pattern for the given conditions, which is a function of the approaching Froude Number. A splay of 1 in 5 is taken for this particular case. Which also almost coincides with the theoritical one.

(2) The expanding sheet of water in the flumes has got a curved hump to a certain distance depending upon the approaching Froude Number as shown in Fig. No. 2 both in plan and longituditional section of the flow.

(3) Any attempt to create a hydraulic jump to dissipate energy is to be started *only after* a uniform depth is reached in the transition after the central hump.

3·8 The expanded flow thus achieved by introducing the transition and after the central hump, was then tried for creation of the hydraulic jump by the following three methods:—

(1) *Introduction of baffle.*—A hydraulic jump is created by the mere introduction of a baffle at the same level of transition. The height of the baffle varies from 1·5 to 2·5 metres depending upon the distance of the baffle from starting of transition. The following are the disadvantages in this method.

(*a*) The jump is not created for the full length of the baffle.

(*b*) A secondary jump is created.

(*c*) The jump formation is highly sensitive to the location, height of baffle, intensity of discharge and is not at all reliable at all times. Position and height of baffle for varying discharges is not the same which is impossible to maintain.

(2) *Control of jump by blocks and baffle.*—Secondly blocks in a single row were introduced in front of the baffle. The hydraulic jump created is stable and is to the full (length) of the baffle. But the blocks introduced create very high shooting at their location. (10m to 12m height). A double row of staggered blocks in front of the baffle (all kept at same transition level) reduces the shooting (5 m to 6 m) improves the jump formation for varying discharges. The jump created is quite reliable for all discharges. The arrangements and formation of jump with a single row of the blocks and double row of blocks in front of the baffle is shown in Photo No. 3 and 4. The disadvantages of this method are.—

(1) The shooting created looks curious.

(2) There is unsightly back flow in the region up to baffle both during the time of raising and receding of floods.

(3) *Control of jump by an abrupt drop followed by a basin.*—Finally an abrupt drop as suggested in "Open channel Hydraulics" by Vente Chow (7) was tried. A mere drop without a basin does not create a hydraulic jump. A basin of 2 m depth and 48m width provided the necessary stagnation pool of water enabling the water falling from the arbupt 2 m drop to form a hdyraulic jump. A row of blocks of 1·25 m × 1·25 × 1·25; at 2·50 metres/cc. were introduced. This arrangement ensured the formation of hydraulic jump

for all flows. The OGEE profile when given to the 2m abrupt drop further improved the performance of jump. The lower nappe of flow that entres the stilling pool does not follow an OGEE profile calculated from consideration of head of water above it. Trials with a trajectory for the jet velocity were run without chute blocks. It seems that a trajectory with chute blocks at its end shall do better for the purpose. No unsighty hump or back flow are created in this arrangement. The velocity of flow after the formation of the jump in the Down stream is 3·05m/Sec. at point D Fig. No. 1-A. The length of the basin is dicided to give a stable jump. Photos 5 and 6 show the arrangements and performance of hydraulic jump in this manner.

3·9 *Economic consideration.*—Studies were further conducted to reduce the length of concrete channel. Attempts were made to reduce the length of transition, basin, etc. The dimensions given in the final drawing (Fig No. 1 for chute) are the minimum requirements based on model performance.

3·10 Having decided the minimum requirements of the length and width for Energy dissipation arrangements the possibility of shifting the abrupt end of transition itself towards upstream was also studied. This point B in Fig. No. 1-A could be shifted up to L.S. 280m from L. 408m. without any loss in the hydraulic performance. The performance of the Energy Dissipation arrangements for this condition is finalised. The flow after dissipation of energy and with the reduced velocity could then be taken through an earthern channel and led into the river.

3·11 *Effect of "R" jump and "S" jump.*—The tail water level after the formation of jump was 656·00 metres. This is more than the assumed rear maximum water level at confluence point. (656·60 metres) for 764·6 cumecs. Hence there is no possiblity of formation of Repelled jump or the submerged jump referred by (8) Thiru Rajaratnam and Kannkatti Subramaniya in ASCE Journal and no effect will be felt, on the jump due to tail water conditions.

3·12 *Advantages.*—The construction of a chute type surplus course provides an even flow pattern for course of curvilinear nature. This fundamental requisite of any hydraulically sound design is not possible in the conventional drop design.

2. The proposal could also be economically feasible by a study of a particular problem in question. In this case a reduction of length of concreate channel by 128 m has been shown to be of such a measure.

3. The proposal of chute type might be an advantage with particular refenence to the problem in question. For example, the syphoning of main canal from Canal Sluice crossing at line LL in Fig. No. 1-B is to be done for a length of 120 metres in the drop type arrangements. Whereas if a chute type is proposed the length of syphoning will be only for 20 metres. This is not only economically cheaper but has the advantage of dispensing of syphoning for a longer distance at the very beginning of the main canal.

4. *Conclusion.*—4·1 The study reveals that a chute surplus course is not only suitable for rocky region of the course but on permeable soils also. If a rocky strata is met with this scheme will work out cheaper. If permeable strata as in this case is available the expenditure on concentrated drops of bigger dimensions will be spreadover the chute components. Localist drops are accompanied by concentrated flow at particular localities whereas the chute type accomplishes an even flow pattern. As regards the ecnomic feasibility each problem has to be studied with particular reference to site conditions.

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(4) Effect of sharpflare in training walls on stilling basin chute spillway by Er. Gajinder Singh and B.D. Sharma, C.B.I.P., Proceedings 38th Annual Research Session June 21st 30-1968.

(5) 'Irrigation and Hydraulic' Designs by Serge Leliavsky.

(6) 'Engineering Hydraulics' by Hunter Rouse.

(7) Open Channel Hydraulics by Chow. V. T.

(8) Hydraulic Jump below Abrupt symmetrical Expansion and by Nallamuthu Rajaratanam and Kannkatti Submanya Page 481 of ASCE., Journal of Hydraulic Division Vol. 94 HY2 March 1968.

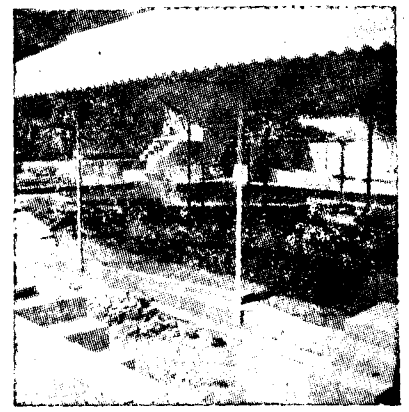


Photo No. 1— FLOW PATTERN IN DROP TYPE SURPLUS COURSE FOR 764·6 CASES.

Photo No. 2—EXPANSION OF SUPER CRITICAL FLOW.

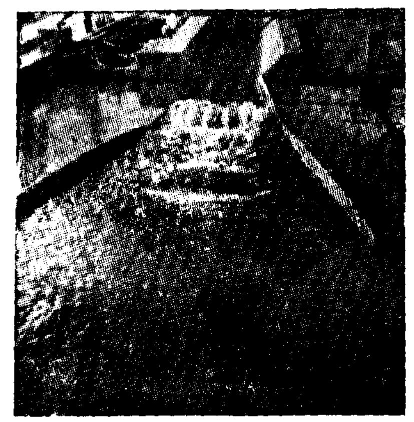
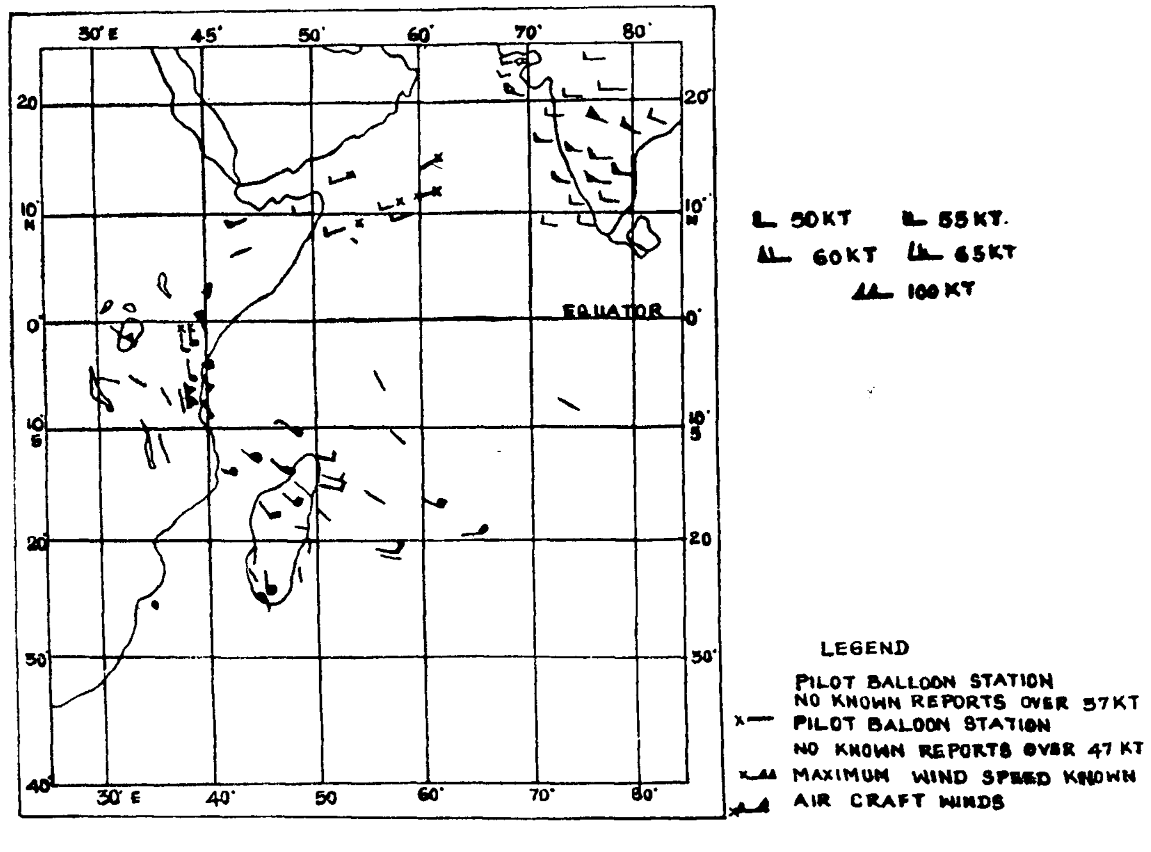


Photo No. 3.

Photo No. 4.

Photo No. 5—ABRUPT 2.1 M DROP WITH STILLING BASIN ARRANGEMENT.

Photo No. 6—HYD. JUMP AT EXIT OF CHUTE FOR 764.6 CUMECS FOR ARRANGEMENTS AS IN PHOTO NO. 5



MAXIMUM WIND SPEEDS KNOWN TO HAVE BEEN RECORDED IN THE LAYER 2000 - 8000 FT. (600 - 2400) ABOVE MSL DURING THE SOUTH MONSOON SEASONS (NORTHERN SUMMER)

EVALUATION OF STRESS CONCENTRATION FACTORS FOR STANDARD 152.4 cm. X 228.6 cm. (5'X7.5') GALLERIES WITH A DRAIN AT THE BOTTOM.

By

[1]Thiru A. N. Raju, B.E. [2]Thiru A. Somasundaram, B.E. [3]Thiru A. K. Sankaran, B.Sc.

1.0. *Introduction*:

1.1. The provision of openings in a dam always cause disturbance in the path and magnitude of stresses in the structure resulting in re-distribution of stresses and concentration of unit stresses at the sides of the opening. The reinforcements are to be provided sufficiently to take up the induced stresses at the immediate faces of the openings. The quantum and location of the reinforcements depend upon the magnitude and direction of the stresses. Analytical solutions are available for giving the stress distribution around various shaped openings in a infinite elastic medium but none could be truly representative of the gallery shape used in concrete dams. The following two methods are being employed to compute the stresses around a gallery:—

(i) Mathematical treatment assuming an equivalnet circular opening.

(ii) From experimental co-efficients deduced from photo-elastic interferometer study for a standard gallery 152.4 cms. × 213·36 cms (5' × 7') size conducted by the U S B.R.

Both the methods are being followed and only the average values of the stress values are taken into account for adoption. Such a comparison has obviously its own demerits and hence an attempt has been made here to evolve stress co-efficient factors for a standard 152·4 cm. × 228·6 cm. (5' × 7·5') gallery with a 30·48 cm. × 30·48 cm. (1' × 1') drain by the photo-elastic technique.

2.0. *Objective of the study*:

2.1. The objective is to evolve stress concentration factors for standard 152·4 cm. × 228·6 cm.(5' × 7·5') with a 30·4 cm. × 30·4 cm. (1' × 1') drain at the bottom, as adopted in Tamil Nadu.

3.0. *Scope of the study and assumptions*:

3.1. As already mentioned in the introduction, both the mathematical method of equivalent circle and the photo-elastic co-efficients evolved by the U.S.B.R. for standard 152·4 cm. × 213·36 cm. (5' × 7') gallery have been used in computing the stresses around the standard gallary of 152·4 cm. × 228·6 cm. (5' × 7·5'). Because of the differences in the dimension of a U.S.B.R. gallery and the standard galleries as adopted in Tamil Nadu such a procedure cannot be expected to give the correct picture of stress concentrations around the proposed gallery opening. Hence, the present model tests, the results of which are expected to be of great use in determining the stress concentration around standard openings.

3.2. In all photo-elastic studies of stresses around openings, it is assumed that the opening is situated in an infinite elastic medium of uniform stress field far removed from external boundaries. It is further assumed that the dimensions of the gallery are small enough to make the changes from top to bottom in forces due to gravity negligible. The U.S.B.R. have conducted studies with similar assumptions.

4.0. *Review*.—

4.1. The U.S.B.R. has conducted photo-elastic interferometer studies to determine the distribution of stresses around a standard gallery of size 152·4 cm. × 213·36 cm. (5' × 7') and have evolved stress co-efficients to compute the induced stresses in the material and the gallery opening. The stress co-efficients are available at severable points on various lines radiating from the centre of the gallery. These are expressed in terms of unit load acting on the external boundaries of the model. These co-efficients have been used to compare the designed values of stresses around standard 152·4 cm. × 228·6 cm. (5' × 7·5') galleries. The persent study was programmed with a view to evolve stress co-efficient factors for 152·4 cm. × 228.6 cm. (5' × 7·5') galleries with the drain at the bottom so as to get a more accurate picture of the true nature of stress distribution for such galleries.

5.0. *Experimental procedure*.

5.1. An important factor to be considered before conducting the model tests is the loading to be imposed on the gallery model. In photo-elastic tests, the practice is to first calculate the principal stresses at the centre of the gallery, assuming that the gallery or opening did not exist. These principal stresses are then applied as the normal loadings far removed from external boundaries.

5.2. Araldite CY-230 sheets cast and annealed in the laboratory have been exclusively used in the preparation of the photoelastic models. The annealing is done by a process of repeated heating and cooling in a suitable bath. The sheet is placed in a rectangular enamel tray with a perfectly horizontal base, a glass plate is used if necessary, and the annealing medium is added so as to submerge the sheet completely. Liquid paraffin or glycerine is used as the annealing medium. It is then kept in an oven whose temperature is then slowly raised to 100° to 105° C. It is maintained at this higher temperature for several hours and then allowed to cool slowly down to the room temperature. The sheet is then removed and examined for residual stresses. If stresses are still present, the process is repeated until clear transparent stress free sheets are obtained

5.3. The gallery was cut to a scale of 1:120 on 10·16 cm. × 10·16 cm. (4" × 4") plate. A milling cutter was employed for shaping the model except for the sharp corners which had to be hand-filed carefully.

5.4. A loading frame with proving ring attachment for measuring the applied load was used for straining the models. The models were clamped between two chucks lined with rubber cushion for uniform loading. The loading frame with the model was mounted in the field a circular ploariscope with mono-chromatic illumination. One 125 watts, 250 volts mercury vapour lamp with a green filter was used in these experiments.

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5.5. Models have been tested for the following four basic cases of loading.

Case I : Uniform vertical load.

Case 2 : Uniform horizontal load.

Case 3 : Uniform load at 45° counter-clockwise from the vertical.

Case 4 : Uniform load at 45° clockwise to the vertical.

In all the above cases, a uniform load of 35·15 Kg./sq. cm. (500 psi.) had been applied. The resulting iso-chromatics were photographed for both parallel and crossed field set up of the polariscope. The tests were conducted within three to four hours of the final finishing of the model to minimse the effect due to time-edge effect. Figure 2 to 5 illustrates the four cases.

6.0. *Collection of data* :

6·1. Photographs of the stress patterns were taken after visual observation of the development of the fringes with increasing load. The fringe orders for the applied load are read off from these stress patterns. The material fringe value of the material of the model was determined by calibration tests on specimens cut from the same sheets from which the models were prepared. The material fringe value was found to be 4·57 kg/sq.cm. (65 psi.) in compression.

7.0. *Analysis of the data* :

7.1. The iso-chromatic pattern given in plates 1 to 4 give the LOC 11 of the maximum shear stress directly. At the free boundaries, since the stress normal to the boundary vanishes, the tangential boundary stresses may be obtained directly from the fringe count. Using the value of the fringe constant determined by calibration, the tangential boundary stressss for the applied load P is determined. The stress co-efficients for unit load is then computed for the different cases vide table 1 and 2.

7.2. The stresses around the gallery due to pure shear, initially acting at the centre of the gallery were determined by considering the load acting at 45° on each side of the vertical axis, with the applied load considered compressive on one side and tensile on the other. The sum of these two effects give the pure shear situation. These results are shown in Fig. 6.

7·3. If P and Q be principal stresses and P inclined an angle θ to the Y—Y axis, then—

$$\sigma_x = \frac{P+Q}{2} + \frac{P-Q}{2}\cos 2\theta \quad \cdots \text{(i)}$$

$$\sigma_y = \frac{P+Q}{2} - \frac{P-Q}{2}\cos 2\theta \quad \cdots \text{(ii)}$$

$$\tau_{xy} = \frac{P-Q}{2}\sin 2\theta \quad \cdots \text{(iii)}$$

For any given value of the principal stresses P and Q and of inclination of P to the Y—Y axis, the values σ_x σ_y and τ_{xy} could be calculated from the above equations. The boundary stresses for each of these compenents may then be found from the tables, by multiplication by the stress co-efficient factors. These separate values may then be summed up to obtain the tangential boundary stresses for the given value of the principal stresses.

8.0. *Conclusion* :

8.1. The tangential boundary stresses around a standard 152·4 cm. × 228·6 cm. (5′×7·5′) gallery have been determined by the photo-elastic technique. The values have been found only for points along the free boundary only. No attempt has been made to work out the stresses at the interial points as has been done by the U.S.B.R. as no interferometer was available for extending the studies to the interior points. The co-efficient factors have been determined for four fundamental cases of loading and are expresssed in terms of unit load. The U.S.B.R. co-efficients for a standard 152·4 cm.×213·36 cm. (5′×7′) gallery has also been furnished side by side with the experimental values. The variations seen may largely be due to the difference between size and shape of the gallery as adopted by Tamil Nadu and the U.S.B.R.

8·2. These coefficient factors may thus be used directly for finding out the stress distribution for a standard 152·4 cm. ×228·6 cm. gallery if the principal stresses P and Q and their direction θ and are known.

9·0. *Application of results*:

The results of these studies are intended to be of use in the designing of reinforcement required to be provided around standard gallery openings of 152·4 cm. × 228·6cm. The coefficient factors furnished in the report are indicative of the true nature of the distribution of stresses and since they have been expressed in terms of unit load, may be applied straightaway to geometrically similar cases, if the values of the pricnipal stresses and of the direction of P at the proposed gallery centre are known. The adoption of these values for comparison of the stress distribution as obtained by design calculation may result in considerable economy in terms of the quantum of reinforcement to be provided.

10. *Project for future research*:

10·1. The immediate object of the present studies have been to obtain stress co-efficient factors for 152·4 cm× 228·6 cm galleries at points around the free opening. Hence no attempt has been made to evaluate stresses at the interior points on any reference lines as has been done by the U.S.B.R. This was partly because of lack of equipments needed for such studies. The U.S.B.R. studies have been made using a photoelastic interferometer. Hence it is considered worthwhile to pursue the studies further to cover the interior points also. This may be done either as an extension of the present studies or as a separate project taking the need for additional equipment required for conducting the same.

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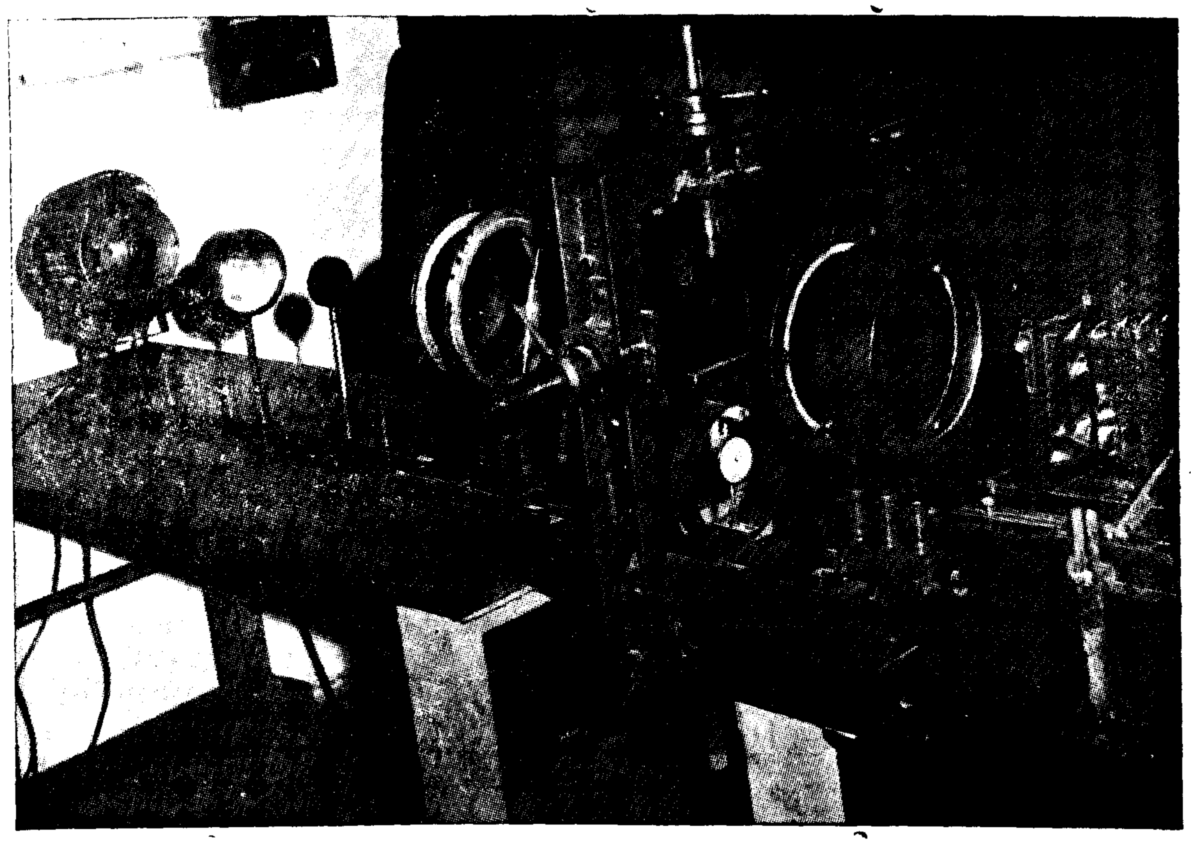
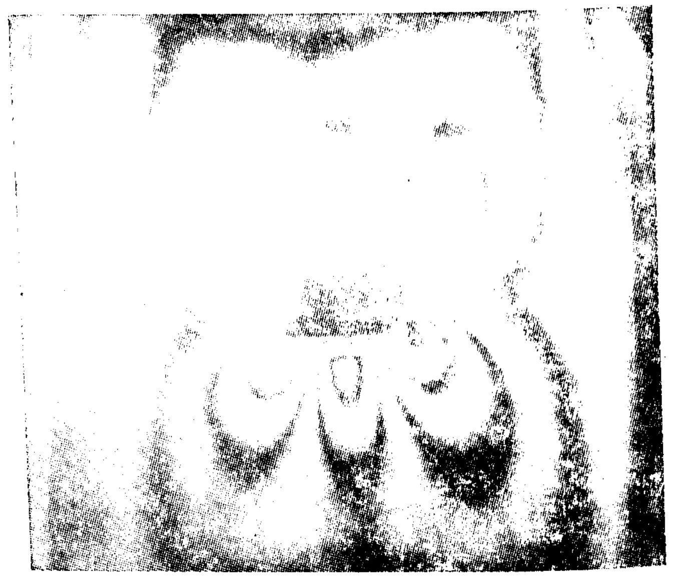
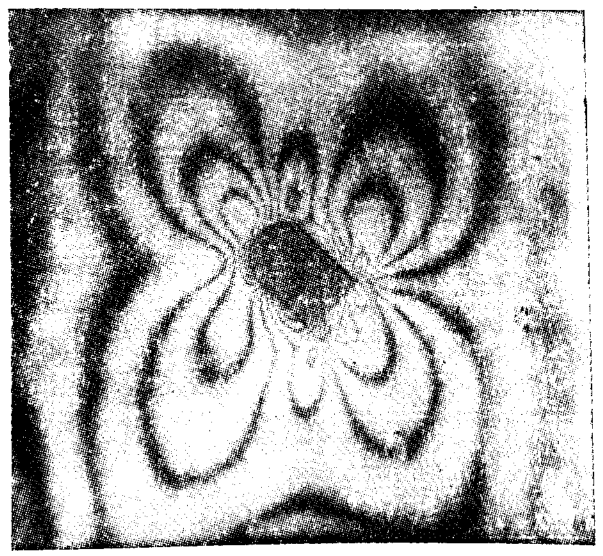


PHOTO-ELASTIC UNIT.

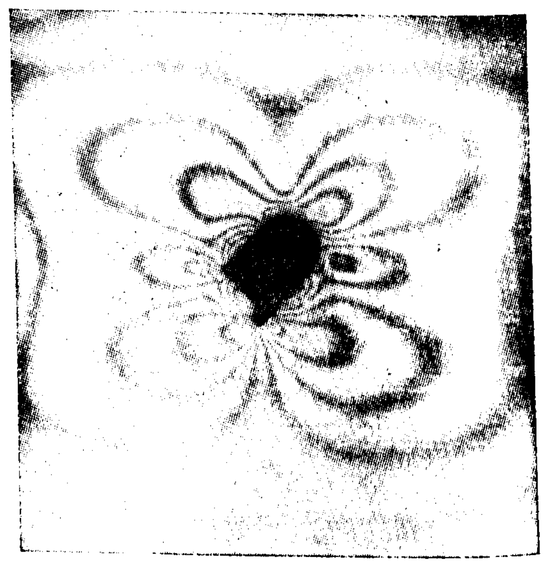
CASE 1—VERTICAL LOADING



CASE 2—HORIZONTAL LOADING



CASE 3—LOAD APPLIED AT 45° COUNTER CLOCKWISE



CASE 4—LOAD APPLIED AT 45° CLOCKWISE

(8)

TABLE I.

STRESS CONCENTRATION FACTORS FOR UNIFORM LOAD APPLIED PARALLEL TO THE VERTICAL AXIS.

Model 1—Load 35·15 kg/sq.cm. (500 psi) applied parallel to the vertical axis.

Reference points on the model.	*Model fringe value F.*	*Fringes.*	*Stress in kg./sq.cm.*	*B.S. for unit load.*
(1)	(2)	(3)	(4)	(5)
0	11·74 kg/sq. cm.	− 3·0	−35·22	−1·00
1	.. ..	0·0	0·00	0·00
2	.. ..	2·0	23·48	0·67
3	.. ..	4·0	46·97	1·34
4	.. ..	6·0	70·45	2·00
5	.. ..	6·0	70·45	2·00
6	.. ..	8·0	93·99	2·67
7	.. ..	0·0	0·00	0·00
8	.. ..	−2·0	−23·48	− 0·67
9	.. ..	−1·0	−11·74	−0·33
10	.. ..	0·0	0·00	0·00
11	.. ..	2·0	23·48	0·67
12	.. ..	7·0	82·19	2·38
13	.. ..	4·0	46·97	1·34
14	.. ..	8·0	93·93	2·67
15	.. ..	5·5	64·54	1·84
16	.. ..	6·0	70·45	2·00
17	.. ..	4·5	52·38	1·49
18	.. ..	2·0	23·48	0·67
19	.. ..	0·00	0·00	0·00

TABLE II.

Stress concentration Factors for uniform load applied normal to the vertical axis.

Refernce points on the model.	*Model 2—Load 35·15 kg/sq. cm. (500 psi) applied perpendicular to the vertical axis.* Model fringe value F.	*Fringes.*	*Stress in kg./sq.cm.*	*B.S. for unit load.*
(1)	(2)	(3)	(4)	(5)
0	11·74 kg./sq. cm.	11·0	129·15	3·66
1	.. ..	7·0	82·19	2·34
2	.. ..	4·5	52·82	1·50
3	.. ..	2·0	23·48	0·67
4	.. ..	−1·0	−11·74	−0·33
5	.. ..	−2·5	−24·34	−0·84
6	.. ..	7·0	82·19	2·34
7	.. ..	4·0	46·97	1·34
8	.. ..	3·0	35·22	1·00
9	.. ..	2·0	23·45	0·67
10	.. ..	0·0	0·00	0·00
11	.. ..	5·0	28·70	1·67
12	.. ..	9·0	105·67	3·00
13	.. ..	6·0	70·45	2·00
14	.. ..	9·0	105·67	3·00
15	.. ..	−3·0	−35·22	−1·00
16	.. ..	−2·0	−23·48	−0·66
17	.. ..	3·0	35·22	1·00
18	.. ..	5·0	35·22	1·00
19	.. ..	7·0	82·19	2·34

TABLE No. III.

Stress concentration factors for 45° angle loadings and due to pure shear.

		Model III load 35·15kg/sq.cm. applied at 45° to the left of the vertical axis			*Model IV- load 35·15kg/sq. cm. applied at 45° to the right of the vertical axis.*					
Reference Points on the model.	*Model fringe value F.*	*Fringes.*	*Stress kg./sq. cm.*	*B.S. for unit load.*	*Model fringe valve F.*	*Fringes.*	*Stress kg/sq.cm.*	*B. S. for unit load.*	*B. S. for unit load with shear diagonal in the 1st and 3rd quadrant.*	*B. S. for unit load with shear diagonal in the 2nd and 4th quadrant.*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
0	11·95 kg/ sq. cm.	5·5	65·74	1·87	11·95 kg./ sq. cm.	5·5	65·74	1·87	0·00	0·00
1	..	10·0	119.52	3·40		5·0	59·76	1·70	2·70	−2·70
2	..	12·0	143·43	4·04	..	−3·0	−35·86	−1·02	5·06	−5·06
3	..	8·0	95·62	2·72	..	−4·0	−47·81	−1·36	4·08	−4·08
4	..	5·0	59·76	1·70	..	−2·5	−29·88	−0·85	2·55	−2·55
5	..	1·0	11·55	0·34	..	1·0	11·55	0·34	0·00	0·00
6	..	−5·0	−59·76	−1·70	..	11·0	131·47	3·78	−5·48	5·48
7	..	−2·5	−29·88	−0·95	..	5·0	59·76	1·70	−2·55	2·55
8	..	−1·5	−17·93	−0·51	..	8·0	35·86	1·82	−1·53	1·53
9	..	−0·5	−5·98	−0·17	..	1·0	11·95	0·34	−0·51	0·51
10	..	0·0	0·0	0·0	..	0·0	0·0	0·00	0·00	0·00
11	..	1·0	11·95	0·34	..	−2·0	−23·90	0·68	1·02	−1·02
12	..	4·0	4·78	1·36	..	−8·0	−95·62	−2·72	4·08	−4·08
13	..	7·0	83·67	2·38	..	−6·0	−71·71	−2·04	4·42	−4·42
14	..	11·0	131·47	3·78	..	−3·0	−35·86	−1·02	4·80	−4·80
15	..	1·0	11·95	0·34	..	1·0	11·95	0·34	0·00	0·00
16	..	−2·0	23·90	0·68	..	5·0	59·76	1·70	−2·38	2·30
17	..	−3·5	−41·83	−1·19	..	8·0	95·62	2·72	−3·91	3·91
18	..	−4·0	−47·81	−1·36	..	12·0	143·43	4·04	−4·11	4·11
19	..	4·0	47·81	1·36	..	10·0	119·52	3·40	−2·04	2·04

TABLE No. IV.

COMPARATIVE STATEMENT OF STRESS COEFFICIENTS FOR U.S.B.R., 152·4 cm. × 213·36 cm. (5′ × 7′) GALLERIES AND EXPERIMENTAL VALUES FOR 152·4 × 228·6cm. (5′ × 7·5′) GALLERIES.

Reference points on the model.	Load applied parallel to the vertical axis.		Load applied pependicular to the vertical axis.		Pure shear load.	
	U. S. B. R.	Experimental.	U. S. B. R.	Experimental.	U.S.B.R.	Experimental.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0	−1·00	−1·00	4·00	3·66	0·00	0·00
2	0·87	0·67	1·21	1·50	−6·12	−5·06
4	2·50	2·00	−1·10	−0·33	−2·43	−2·55
5	2·22	2·00	−1·03	−0·84	0·00	0·00
6	3·20	2·67	3·40	2·34	7·00	5·48
8	−1·00	−0·67	2·72	1·00	0·00	1·53
14	3·20	2·67	3·40	3·00	−7·00	−4·80
15	2·22	1·84	−1·03	−1·00	0·00	0·00
16	2·50	2·00	−1·10	0·66	2·43	2·38
18	0·87	0·67	1·21	1·00	6·12	4·11

GDN/NIE/1/1-75

DIMENSIONS OF DRAINAGE GALLERY AND LOCATION OF REFERENCE POINTS

SCALE:- 2·54 cm = 76·2 cm

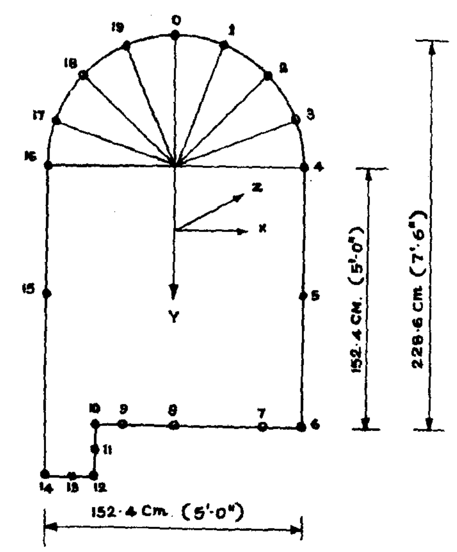


FIG. 1

TRD. BY:- N.N.G.

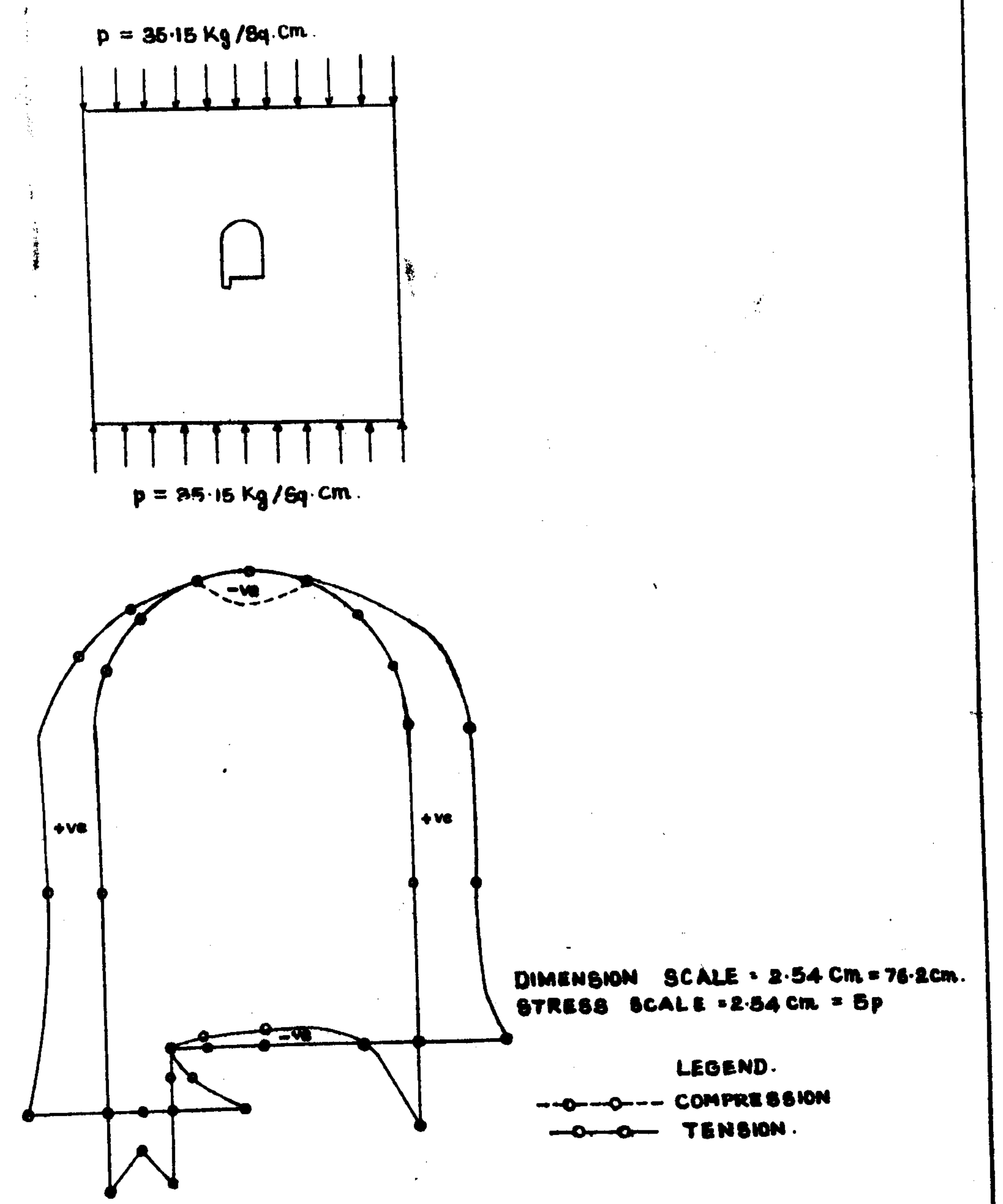


FIG. 2 152·4 Cm X 228·6 cm.

PHOTO ELASTIC STUDIES DRAINAGE GALLERY

TRD. BY: N.N.G

GDN/NIE/4/1-75

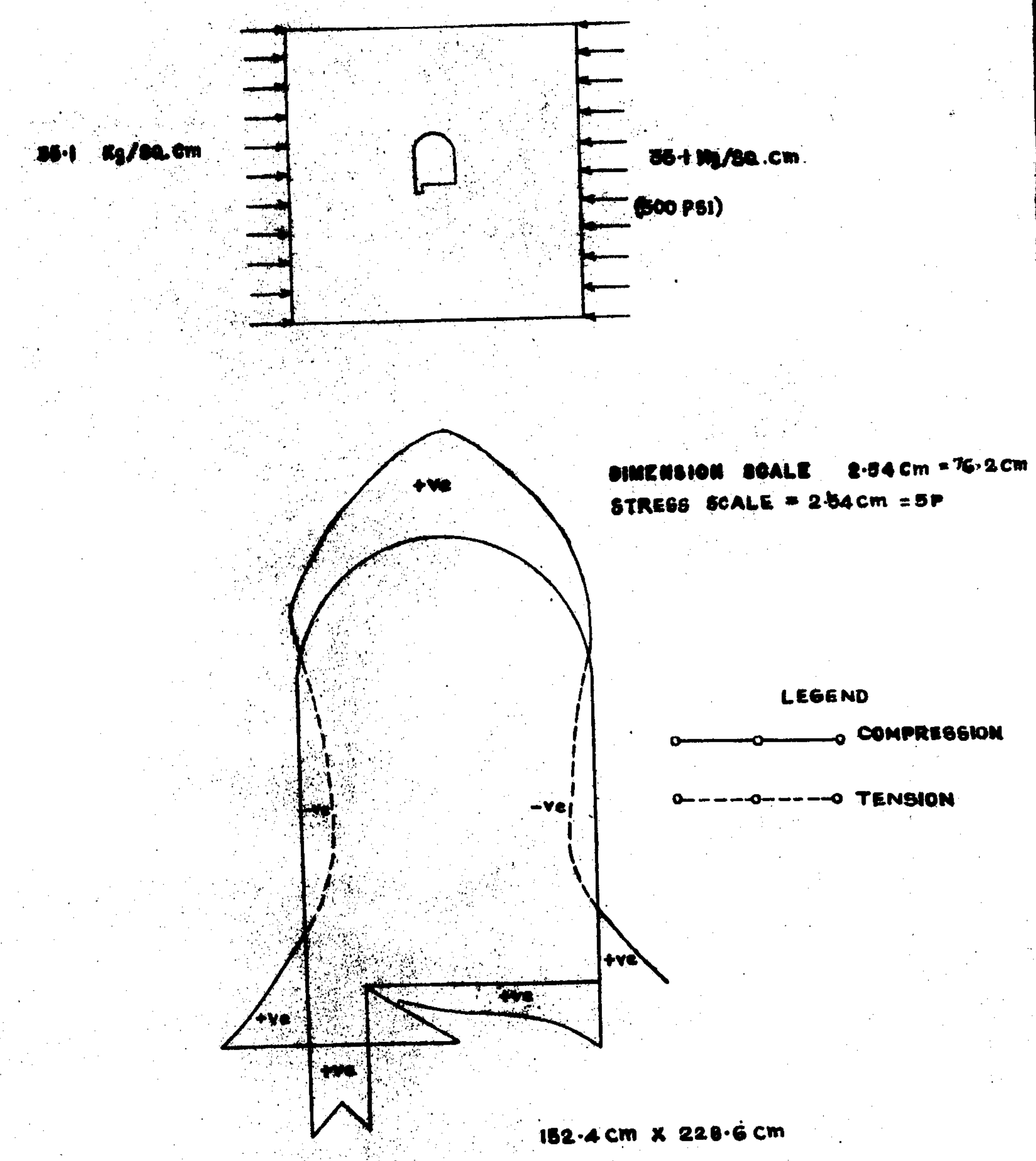
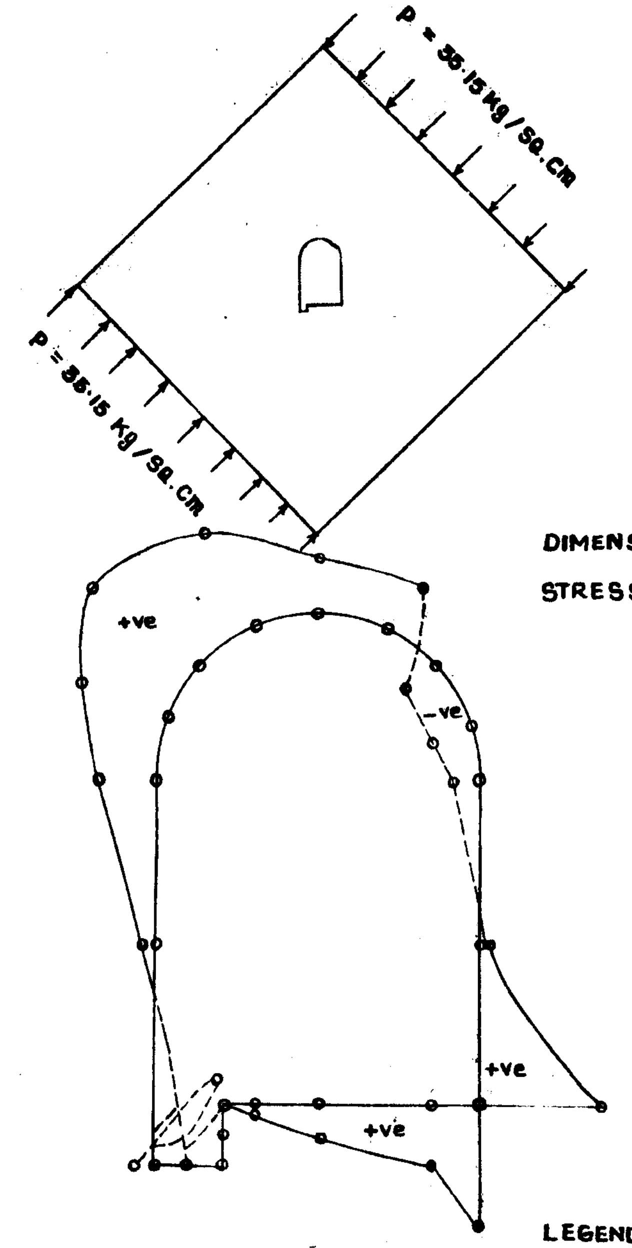


FIG.3 PHOTOELASTIC STUDIES DRAINAGE GALLERY

TRD. BY : N.N.G

GDN/NIE/8/1-75



DIMENSION SCALE = 2·54 Cm = 76·2 Cm.

STRESS SCALE = 2·54 Cm = 5 P

LEGEND.

o—o—o COMPRESSION

o- - -o- -o TENSION

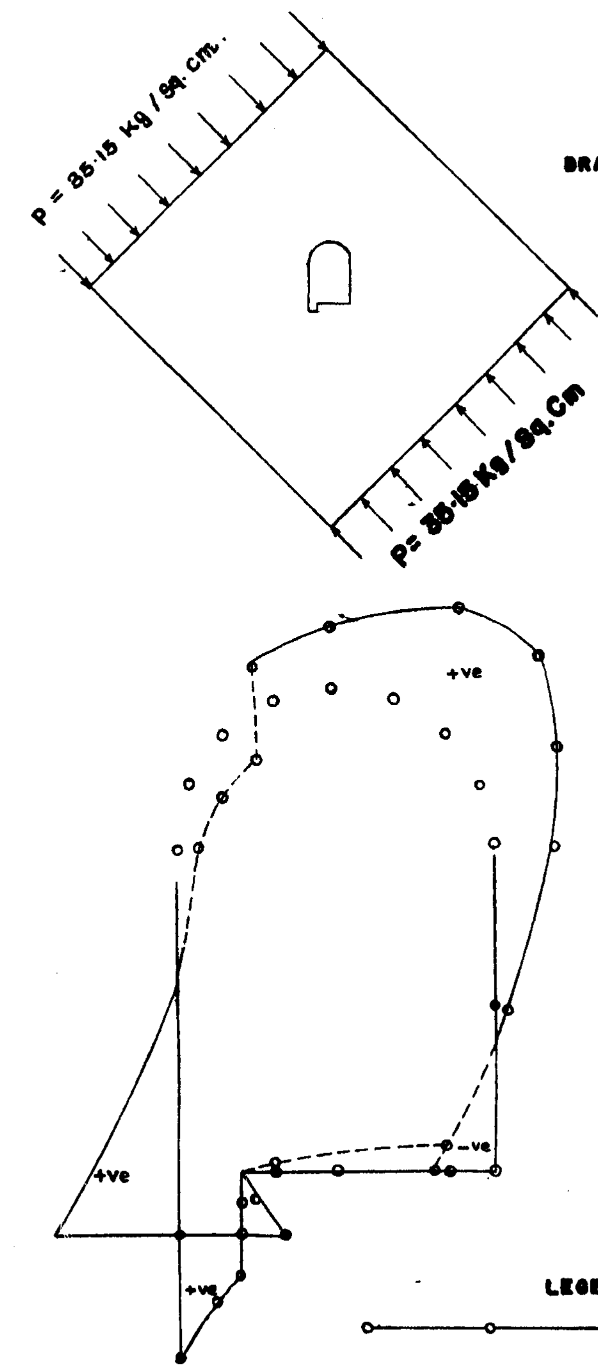
152·4 Cm. X 228·6 Cm.

FIG.4. PHOTO ELASTIC STUDIES DRAINAGE GALLERY

TRD. BY: N.N.G.

DRAINAGE GALLERY

DIMENSION SCALE
2.54 cm = 76.2 cm.



LEGEND

COMPRESSION

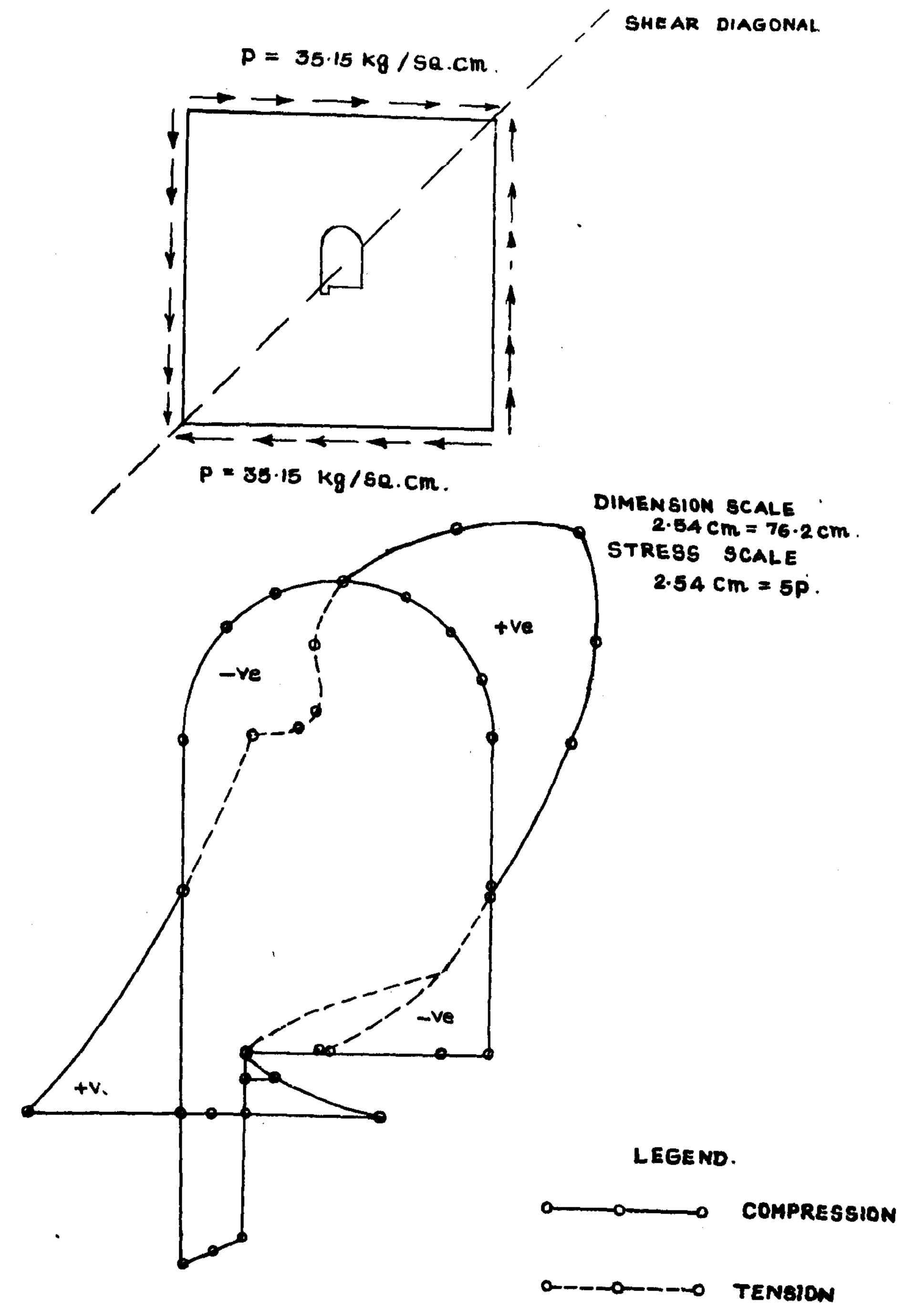
TENSION

152.4 Cm X 228.6 Cm

PHOTO ELASTIC STUDIES DRAINAGE GALLERY

FIG.5

RD. BY: N.N.G.



152·4 cm x 228·6 cm

FIG.6 PHOTOELASTIC STUDIES DRAINAGE GALLERY

TRD. BY: N.N.G.

WATER REQUIREMENTS OF CROPS IN TAMIL NADU—A REVIEW.

By

Thiru. R. K. SIVANAPPAN[1], Thiru. K. R. SWAMINATHAN[2] AND Thiru. K. R. KARAI GOWDER.[3]

1. *Introduction*:

Irrigation for increased crop production has been practiced in all parts of the world from very ancient times. Rainfall constitutes the most single factor to determine the scope for increasing the irrigation potential. The estimated total utilizable surface water in the State is 357·60 mil. hec. cm. of which 347·75 mil. hec. cm. has been harvested which is a record in any part of the world. There are more than 11 lakhs of wells in Tamil Nadu and there is scope for developing and utilizing more water from underground. Even then the area of irrigation may not exceed 45 to 50 per cent of the sown area. Many areas are being reclaimed and brought under cultivation. There is much interest in the possibilities of multiple cropping and intensive cropping to make use of our year round growing season. For all these, we have to give water so as to increase the production for ever increasing population.

Average rainfall is a constant one and is fixed. We can not get more water for our increased needs. The only way to extend the present area of irrigation is by conserving the water and by suitable water management practices in the field. Further indiscriminate use of water in excess of the requirement leads to wastage as well as turning the land into problem lands. Therefore it if necessary to arrive at a correct water requirement of crops so that only the required water can be given to the land to produce the maximum.

The term 'water requirement of crops' is the quantity of water required to produce the best growth of crops. The quantity includes the effective rainfall which has fallen during the growing period of the crop. The water requirements of crops depend upon the climatic conditions, nature of the soil, the type of crop, the method of irrigation and the agronomic practices adopted.

2. *Work done*:

In India water requirement studies have not been conducted even though majority of the crops are irrigated or rainfed. In Tamil Nadu, canal irrigation, well irrigation and tank irrigation have been practiced; yet no scientific approach has been followed either for irrigation or for assessment of water requirements. In the past, emphasis had been made only on the intervals between the two successive irrigation irrespective of the status of soil moisture depletion in the root zone. For paddy depth of water was maintained without knowing the quantity. Several methods adopted by research workers in the determinations of water requirements of crops nclude;

(a) Pot or tank culture (i) Gravimetric.
(ii) Volumetric.

(b) Field studies .. (i) Soil moisture studies.
(ii) Water balance.

(c) Climatological studies

Since pot or tank culture isolates the plant from 'Community effect' and ground-water influence, the results obtained will not represent the field water requirement because of the inherent 'Oasis effect' and advective influences. In the field conditions, studies have not been conducted on quantitative basis even though the moisture status and water balance methods have been indirectly followed by irrigating the fields at regular or pre-determined intervals. The reported results of the past have the following drawbacks:—

(a) While reporting the water requirements, no mention has been made about the varieties used, duration of crops, effective rainfall and the quantities of water applied at each irrigation, (b) The units of water requirements, used are innumerable and they include duty, acre-inches, acre-per cusec, inches, gallons and cms. (c) The reported data are devoid of the soil moisture status before and after irrigation. (d) No information is available if any intervening rainfall between the successive irrigation has replenished the moisture within the root zone. Reviewing the available records of the water requirement studies of Tamil Nadu, and re-calculation, comprehensive summary of water requirement of various crops has been given in Table I.

3. *Recent Research*:

The four important terminologies involved in the water requirements of crops are:

3.1. *Water requirement*.—It is the sum total of the water applied to the field through irrigation and precipitation. It is the additive total of water conserved in the root zone, percolated root beyond zone unavoidably and given to the atmosphere through evapotranspiration during the specified period.

3.2. *Irrigation requirement*.—The depth of water per unit area required to be delivered to a field inlet for a crop in a given period of time. It does not include direct precipitation.

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3.3 *Consumptive use.*—The quantity of water transpired by plants, retained in plant tissues, evaporated from plant foliage from surrounding surfaces and from adjacent soil in a specified period is defined as consumptive use and expressed as depth of water per unit area in any specified time in cm. or inches.

3.4 *Percolation.*—It is quantity of water that flows vertically down beyond the root zone without directly available to the root extraction.

Since the interpretation of water requirements data involves the distinct differentiation of the above terms, the recent research methodology incorporate systematic techniques in such studies. The Indian Council of Agricultural Research research programmes lay emphasis on the influance of agro-climatic parameters on these aspects of water requirements. Recently water management of different crops is gaining recognitions in view of the need for judicious utilisation of water resources. Table 2 gives the summary of water requirements of crops of recent varieties conducted recently in the State. From Table 2 it can be seen that the water requirements of crops vary in the same soil for the same variety in different season. Assuming the hydraulic conductivity and infiltration rate are same for the same soil it may be inferred that the variations in percolation rate is constant. The climatic variations influence, consumptive use. The recent studies of the improved varieties at Coimbatore show significant variations in consumptive use during the same season for different varieties. (Table 3). This leads to indicate, besides the climatological and soil influences, varietal influence also. Table 4 gives water requirements and consumptive use of paddy in Thanjavur region for different seasons.

4. *Conclusion :*

The water requirement studies of past have not given conclusive results on account of lack of quantitative measurements and of omission of several relevant parameters. The recent studies under taken on water management aspects reveal that the water requirements of crops vary from variety to variety, season to season, soil type to soil type and one climatic influence to the other. The soil moisture depletion in the root zone as affected by the crop consumption and—ground water, contribution cannot be ignored while estimating the water requirements. Hence the rooting pattern and depths and the corresponding moisture depletion, are also to be taken into account in future studies, as this would indicate the capillary contribution from the subsurface or the influence of effective rainfall. The moisture content of the soil affects the permeability of the soil which, in turn, affects the percolation losses.

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TABLE No. 1—WATER REQUIREMENT OF EARLY DETERMINATION.

Serial number and crop.	*Location.*	*Variety.*	*Season.*	*Duration.*	*Water required.* *cm.*	*inches.*	*Soil.*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1. Paddy	Coimbatore	Samba	September-January	135	15	63	Clay/Clay loam.
Do.	Do.	Co. 38 Karuna.	July -October	105	110	44	Do.
Do.	Do.	Samba	September-January	135	203	81	Do.
Do.	Bhavanisagar	ADT. 27	September-January	135	95	38	Red gravel.
Do.	Do.	TKM. 6	September-January	120	72.5	29	Do.
Do.	Do.	IR. 20	January September	120	110	44	Do.
Do.	Do.	IR. 20	October-January	120	145	58	Do.
Do.	Aduthurai	Not given	June-September	110	98	39	Sandy clay loan
Do.	Do.	Do.	October-March	150	113	45	Do.
Do.	Do.	Do.	June-September	110	92	37	Do.
Do.	Kattuthottam	Do.	Do.	110	92	37	Do.
Do.	Nagrmangalam	Do.	Do.	110	83	33	Do.

TABLE – 1 (contd)

Serial number and crop.	*Location.*	*Variety.*	*Season.*	*Duration.*	*Water required.* *cm.*	*inches.*	*Soil.*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1 Paddy	Pattukkottai	Not given	June-September	110	142·5	57	Sandy loam.
Do.	Do.	Do.	Do.	110	200	80	Do.
Do.	Do.	Do.	October-March	150	112·5	45	Do.
Do.	Do.	Do.	September-January	135	107·5	43	Do.
Do.	Sikkil	Do.	June-September	110	83	33	Do.
2 Cotton	Coimbatore	Not given	August-March	180	40	16	Clay/Clay loam.
Do.	Do.	Do.	October-April	180	68	27	Do.
Do.	Bhavanisagar	Do.	September-March	180	72·5	29	Redgravel.
3 Sugar cane	Coimbatore	Not given	March-March	360	240	96	Clay/Clay loam.
Do.	Palar	Do.	Do.	360	182·5	73	Do.
Do.	Cuddalore	Co. 419	Do.	360	195/262·5	78/105	
4 Ragi	Coimbatore	Not given	May-September	90	47·5	19	Clay/Clay-loam.
Do.	Bhavanisagar	Do.	Do.	90	52·5	21	Redgravel.
5 Bajra	Bhavanisagar	Co. 3	Not given	104	21	8·5	Do.
Do.	Do.	X3	Do.	106	25	10	Do.
6 Sorghum	Coimbatore	Not given	April-July	90	48	19	Clay/Clay loam.
Do.	Bhavanisagar	Do.	September-March	90	32·5	13	Redgravel.
7 Maize	Bhavanisagar	Ganga 101	Not given	105	92·5	37	Red gravel.
8 Tobacco	Bhavanisagar	Vazhaikappal	Do.	..	28	11	Red gravel.
9 Groundnut	Bhavanisagar	Do.	September-March	150	60	24	Redgravel.

TABLE No. 2—WATER REQUIREMENTS OF RECENT VARIETIES.

Serial number and crop.	*Location.*	*Variety.*	*Season.*	*Duration.*	*Water required.* *cm.*	*Inches.*	*Soil*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1 Paddy	Coimbatore	IR. 22	October-February	120	83	33	Clay loam.
Do.	Do.	IR. 22	April-July	120	135	54	Do.
Do.	Do.	IR. 20	August-January	120	112·5	45	Do.
Do.	Do.	IR. 20	February-January	120	105	42	Do.
Do.	Bhavanisagar	IR. 20	September-July	120	110	44	Redgravel.
2 Cotton	Coimbatore	MCU. 5	September-October	165	70	28	Clay loam.
3 Groundnut	Coimbatore	TMV. 7	December-April	120	64	25·7	Do.
Do.	Bhavanisagar	TMV. 2	January-May	110	72	28·5	Redgravel.
4 Bajra	Coimbatore	Co. 4	April-June	85	25	10	Clay-loam.
5 Ragi	Coimbatore	Co. 10	July-October	90	45	18	Do.
Do.	Coimbatore	Co. 7	December-January	90	50	20	Do.
6 Sorghum	Coimbatore	CHS. 1	March-June	90	35	14	Do.
7 Maize	Coimbatore	Decan hybrid.	January-May	115	38	15	Do.
8 Tomato	Coimbatore	Co. 1	June-August	70	35	14	Do.

TABLE No. 3—WATER REQUIREMENT, IRRIGATION REQUIREMENT AND CONSUMPTIVE USE IN PADDY FIELD AT COIMBATORE.

Variety.	Water required	Irrigation requirement	Consumptive use
	cm.	cm.	cm.
IR. 20	105·75	91·18	77
IR. 22	134·56	118·20	52

TABLE No. 4—COMPONENTS OF WATER REQUIREMENTS OF PADDY IN THANJAVUR REGION.

Season.	Consumptive use.		Percolation.		Irrigation requirements.		Water requirement.	
	cm.	Inches.	cm.	Inches.	cm.	Inches.	cm.	Inches.
Kuruvai	60	24	20·7	8·3	107·5	43	128	51·3
Thaladi	61	24·4	32·3	12·9	79	31·5	115·7	46·3
Samba	70·5	28·2	**33·8**	**13·5**	105·8	42·3	**156·7**	**62·7**

UNCERTAIN MONSOONS

Thiru. S. SETHURATHINAM *

In India, when its economy is basically agricultural, Water resources are its greatest and most important asset. To say that it is also with many other countries is saying the obvious. The primary source of this resource is rain and snow. Barring the snowfed Himalayan rivers, most of the Indian rivers yield 90 to 95 percent of the annual run-off through flood flows in two or three days or at times in one or two months.

As far as India is concerned, the rain feeding resources is mainly caused by the two monsoons namely " The South-West Monsoon " and " The North-East Monsoon ". It has been borne out, by experience of the past, that these monsoons—the most important factors for rains of required amount in required seasons—have been failing at crucial periods resulting in scarcity conditions and unsuccessful crop growth. Thus the rainfall becomes capricious in its incidence and precipitation, leaving the agriculture to be a gamble in monsoon. Such a failure of monsoons has also been explained by Dr. Devek Winstanley, in his meteorological report, published in the British journal " Nature ", an article which the has appeared in the columns of "The Hindu", dated 1st July 1973, under the head "Freakish monsoon" the change or the failure in monsoons, has been explained to be due to a big climatic change sweeping Africa, West-Asia and India. The change, according to Dr. Winstanley, consists in the atmospheric circulation systems, responsible for the monsoons in the northern hemisphere having shifted to the southern side, thus accounting for a decline in rainfall by half of what it was in 1957. The important question of anxiety and concern of the common man is, whether these failures in monsoons will be brought under control and whether these problems even if brought under control now will recur. To this question, Dr. Winstanley feels that they will surely recur, even if brought under control now. Prof. Lamb, Director of Climatology Centre at the University of East Anglia, supports him by affirming that " the world is experiencing what may be the greatest and most sustained shift with climate since 1,700". Though, there is considerable agreement about the cause of the Freakish Monsoons, namely a shift in the atmospheric circulation system, what has occasioned that shift itself is still a matter which remains to be settled. Some believe that this shift is due to the expansion and extension if the Polar ice cap while others relate it to the fluctuations in the Sun's heat. The layman is baffled naturally by all these high talks. What he would like to know is whether a climatic change has actually occured and if so whether it is temporary or is likely to become permanent. Dr. Winstanley seems inclined to the later view because he says that " One may have to live with the change rather than wait for the rain return to the normal pattern". It has also been asserted that it is incumbent on the world Meteorological Organisation, which has the reports of both these experts, to probe the problem intensively and expediciously to alert the countries concerned in time to reorient the farm strategy and schedules according to the changes in rainfall and climate. An appeal has also been made to advanced nations Food and Agricultural Organisation to assist the less developed countries with expertise and equipment that may be required for insulataing their Agricultural production against climatic vagaries. That article has been concluded with an advice to the countries concerned, to keep themselves well prepared expecting the worst.

The Nature, guarding its secrets, against Mankind with its accurate Meteorology assisted by sophisticated technology, is still, brought to light by another artcile " Monsoon riddle " in " The Sunday Standard ", dated 6th January 1974. In that, Dr. B. N. Desai, retired Deputy Director General of observatories (Forecasting), while discussing the results of Monex 1973, has come to the conclusion that the origin of the South-West Monsoon current over Arabian Sea is in the Southern hemisphere, contrary to the former belief that the origin of South-West Monsoon is in the Northern hemisphere of the Globe. Dr. B. N. Desai is afraid, if the interpretations are accepted and our forecasting is changed accordingly, the accuracy in forcasting the onset and performance of the monsoon so far achieved by us would not be maintained. He is also of the opinion that if correct forecasting of the monsoons 2 to 4 days ahead is to be made, it is absolutely necessary to have observations directly from selected observatories of Ethiopia, Somalia, Kenya, Tanganyka, Madagascar and land observatories in the Indian Ocean and that the expenditure would be insignificant.

As far as Tamil Nadu is concerned, maximum precipitation occurs during the North East monsoon. By the middle of October the belt of low pressure is transferred to the centre of the Bay of Bengal. Under it is influence, the retreating South-West monsoon current curves round as it is deflected towards the Peninsula from the North-East. This current is usually called the " North East Monssoon ". The retreating monsoon winds cause occasional showers on the east coast, the a mount of rainfall decreasing from the coast towards the interior. However, during October and November, cyclonic storms form in the Bay of Bengal and when they strike the Northern circars or the Coromandel coast they bring heavy rain to these areas.

The districts of the Deccan and Tamil Nadu receive rainfall in these months almost entirely because of these cyclonic storms or depressions, and hence the distribution of this rainfall is irregular, both in incidence and quantity. Before the end of December the North East Monsoon winds fully establish themselves over the whole region.

But, this North-East Monsoon, supposed to hit Tamil Nadu from the north-east direction, due to depressions in the Bay of Bengal, seems to have been changing its directions from time to time. It comes from Eastern, South-Eastern and Southern directions, due to the troughs formed in Bay of Bengal at various zones, so that its directions and its benefits to Tamil Nadu are highly uncertain. Of course, various reasons are attributed to the unreliable courses of this monsoon, such as earth's rotation, Sun's heat and the supposed expansion and extension of Polar ice cap and all that. The North-East monsoon having changed its course, could as well be called " South-East Monsoon ", " East Monsoon ", or even " South Monsoon ". This has well been indentified by the weather charts published in the " Hindu " in the early part of December 1973. The cuttings of such charts are enclosed at the end.

In all, we are placed at a situation that the monsoonic yields are no more reliable, either in quantity or in incidence due to climatic vagaries, which are out of human

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control. These vagaries result in unequal seasonal distribution, its still more unequal geographical distribution and the frequent departures from the Normal. It is not the actual amount of deficiency and the internal between two occurances of rainfall, but the period during which it occurs, which is of crucial importance. Even moderate deficiencies in the total rainfall can be disastrous during the critical periods of crop growth. It is the uncertainity borne of these factors which makes agriculture precarious and complicates planning for irrigation. Thus agriculture, our main source as forest to Finland and Hydro power to Sweden-becomes nothing but a gamble.

Hence, we should not be standing in the sorry state of affairs any *more*. For us, the main source of maintaining our economic individuality is the agricultural potential that we are having. This country is capable of producing far more food, not only to feed the nation but also be able to export and feed some of our fellow human beings in the world. But the possession of the vast lands is no achievement. A rational implementation is necessary and therefore, it is, that we have got to convince our fellow men, whether they be in the Planning Commission or in the Government or elsewhere, that Irrigation, the basic input is most essential for the country and has to be developed on a greater scale, than at present, whether it be through minor or major or whatever type of irrigation. We should not cry halt to this programme of irrigation, because the basic input for the agricultural potential is water, the supply of water to the parched lands and therefore, we should continue bringing out the importance of proceeding with our irrigation projects.

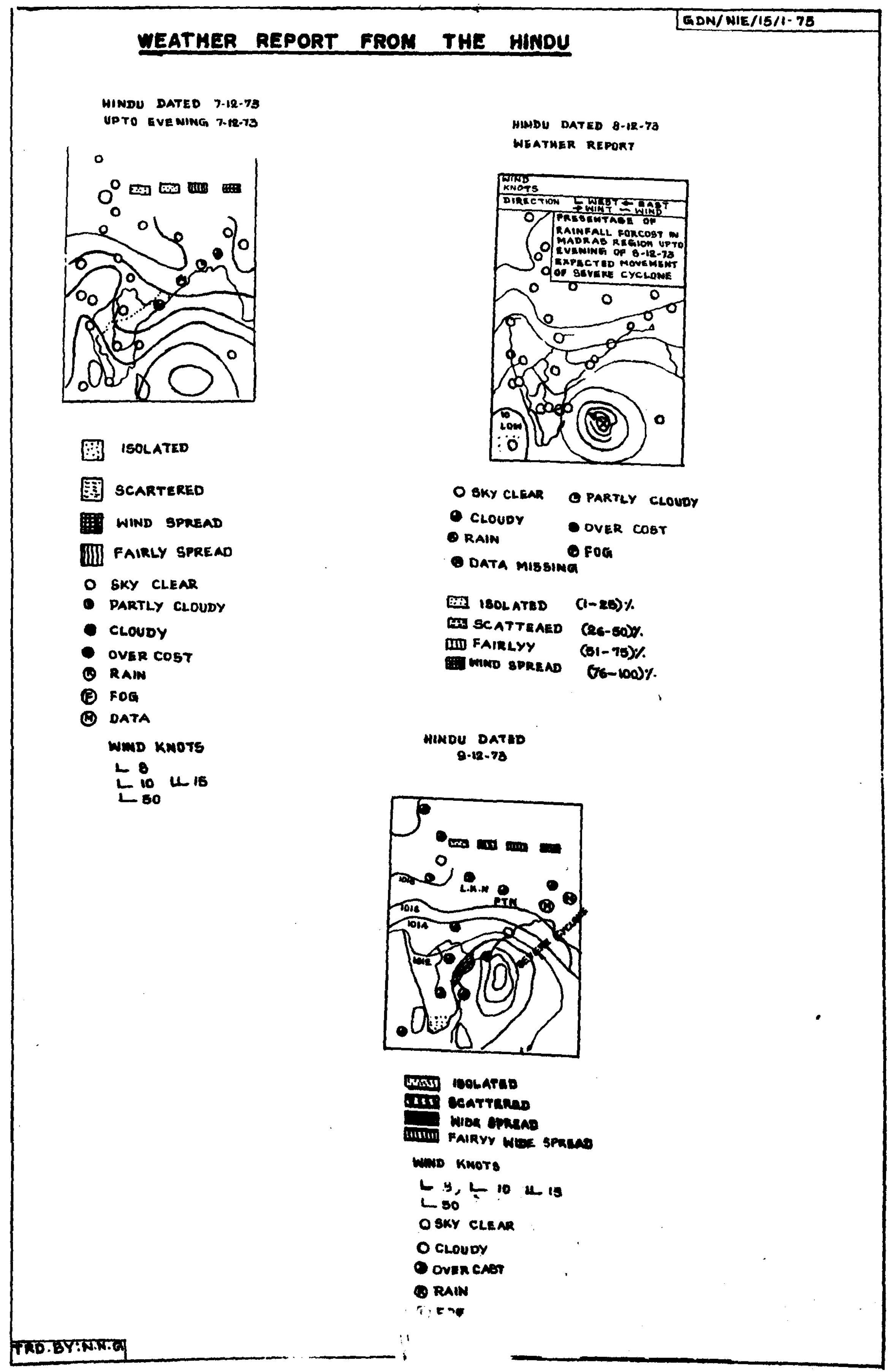
Though the alternations to the monsoons for original set up is out of *human control, the conservation of available* water and its proper use are within human power. Hence, such a knowledge such a prudence demands us to form small and medium type of reservoirs at maximum number of places, across streams, rivulates and rivers, allowing nothing to go as waste. These reservoirs will certainly conserve whatever is available which will otherwise go as waste and be useful *for serving the agriculture* whenever and wherever; for the basic function of a storage reservoir in water resources development is the altering of the time and place of availability of water in nature to suit the needs of man, i.e., conversion of a natural inflow hydrology into a desired outflow hydrology. These reservoirs bring immediate benefits to the lands, have a local appeal, help *in raising* the water table in adjoining wells. Their ability to control floods, especially in the times of changing monsoon, is not a thing to be under estimated. Thus the floods causing loss of human lives and property could be well controlled. Hence to assure food to the population, growing in geometrical progression and to avoid loss of life and property due to floods, the formation of *small* and *medium* reservoirs is the only solution whatsoever. Hence, it is the urgent need to take up investigation of possibilities of formation of such reservoirs, well in advance, to avoid delays in execution of the projects when funds become available. The yard sticks and norms cost benefitratios should all be liberalised for these schemes, as appealed by the Honourable Chief Minister of Tamil Nadu, *during his address in the foundation* stone laying function of the 'Kodaganar Reservoir Scheme', as elimination of droughts and flood protection go in a long way, in the betterment of the Community as a whole saving crores of rupees.

* THE FREAKISH MONSOONS.

The failure of the monsoon in Africa, West Asia and India in two sucessive years has been explained by Dr.Derek Whinstanley in his Meterological report published in the British Journal, "Nature", as having been due to a big climatic change sweeping these region. The change, according to him consists in the atmospheric circulation sytems responsible for monsoons in the Northern hemisphere having shifted southward thus accounting for a decline in the Annual Rainfall by half of what it was in 1957. The severity of the drought that this change is said to have brought about is already common knowledge in India but not the existence of worse conditions in some African countries bordering on the Sahara desert as, for instance Niger, Upper Volta, Chad, Dahomay, Mali, Senegal, and Mauritania. The position there is that if heavy rains do not begin in a week or two the Autumn harvest will fail again leading to serious trouble. Knowledgeable people have stressed in this connection, the possibility of the heavy rains washing off mud roads through which relief supplies are moved to the drought stricken belts thus compelling the authorities to arrange for air-lifts. The important question to be answered is whether these problems even if they are brought under control now will recur, which calls for continuous precautionary measures.

Dr. Winstanley feels that they surely will. Prof. Lamb, Director of the Climatology Centre at the University of East Anglia, supports him by affirming that "the world is experiencing what may be the greatest and most sustained shift in its climate since 1700". Though there is considerable agreement about the cause of the Freakish monsoon, namely a shift in the atmospheric circulation system, what has occasioned that shifts itself is, still a matter which emains to be settled. Some believe the shift is due to the expansion and extension of the Polar Ice Cap while others relate it to the fluctuation in the Sun's heat. The Layman is baffled naturally by all these high talk. What he would liketo know iswhether a climatic change has actually occurred and if so whether it is temporary or is likely to become permanent. Dr. Winstanley seems inclined to the later view because he says that "One may have to live with the change raher than wait for the rain to return to the old pattern". It is incumbent on the world Meteorological Organisation which has the reports of both these experts to probe the problem intensively and expeditiously to alert the countries concerned in time to reorient the farm strategy and schedules according to the changes in rainfall and climate. Over the recent years, Meteorolgy assisted by highly sophisticated technology has no doubt become capable of greater accuracy in its forecasts. But nature guards its secret so jealously that the margin of error is still wide. The margin is very wide indeed in countries like India which are yet to imbibe all the latest in the sophisticated techniques employed in the advanced nations. The Freedom from the Hungber campaign of the Food and Agriculture Organisation will have to ensure that the less developed countries are assisted with the expertise and equipment that may be required for insulating their Agricultural production against climatic vagaries. It is of course possible that the gloomy forebodings may be disproved by the weather reurning to normal. Yet until that actually happens, prudence demands that the countries concerned keep themselves well prepared expecting the worst.

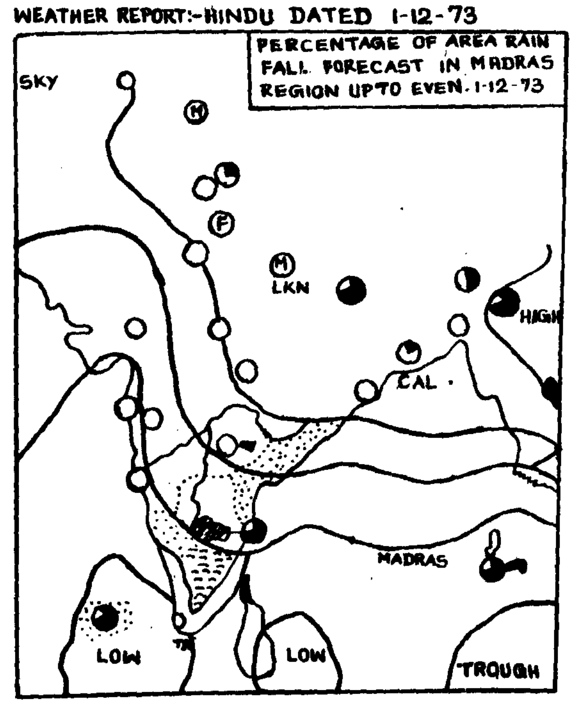
* [*Extract from "The Hindu", dated* 1st *July* 1973.]



GDN/NIE/16/1-75

"WEATHER REPORTS FROM THE HINDU"

WEATHER REPORT:-HINDU DATED 1-12-73



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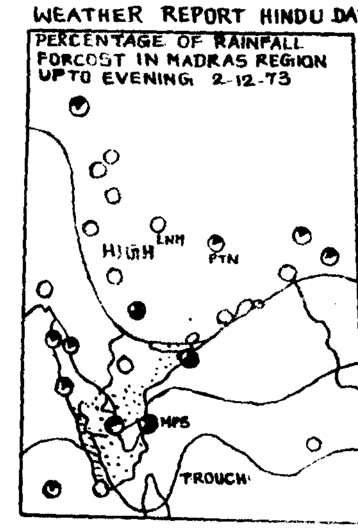

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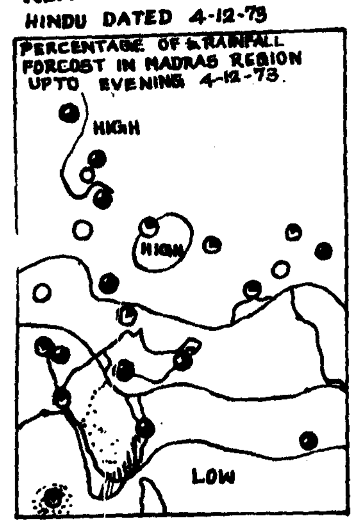

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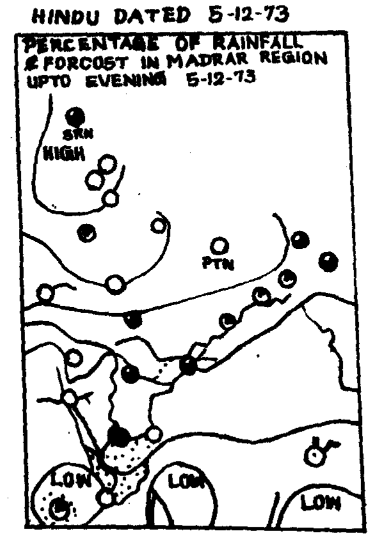


WIND KNOTS 5, 10, 15, 20

DIRECTION:- WEST · EAST.

WIND, WIND.

ISOLATED SCATTERED FAIRLY WIND SPREAD

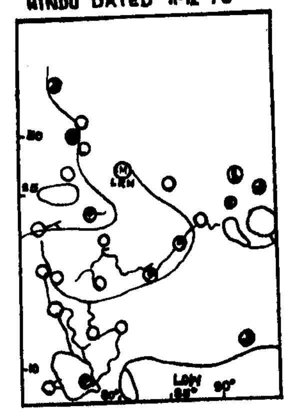
O SKY CLEAR, PARTLY CLOUDY, CLOUDY, OVER COST.

RAIN, FOG, DATAMISING

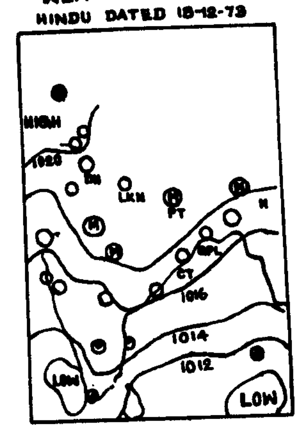
TRD.BY.N.N.G

WEATHER REPORTS FROM "THE HINDU"

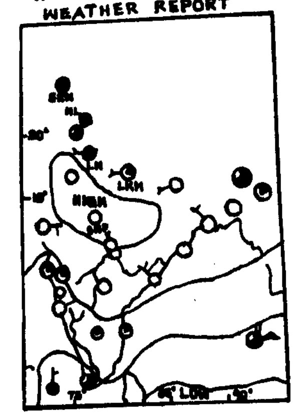
WEATHER REPORT
HINDU DATED 11-12-73



WEATHER REPORT
HINDU DATED 13-12-73



HINDU DATED- 12-12-73
WEATHER REPORT



WIND KNOTS:- L 5 , L 10, L 15 , L 30 , L 50

- SKY CLEAR
- PARTLY CLOUDY
- OVER CLOUDY
- OVER COST
- RAIN
- FOG
- DATAMISING.

- ISOLATED 1-25%
- SCATTERED 26-50%
- FAIRLY WIND SPREAD 51-75%
- WIND SPREAD 76-100%

MONSOON RIDDLE*

Thiru. DR. B. N. DESAI. †

While discussing the results of Monex 1973 in the Magazine Section of "The Sunday Standard" on August 12 last, it is stated: "over the equator itself were found westerly winds blowing from the Arabian land mass at a height of 15,000 feet. Contrary to the text-book theory, there is very little crossover from south of the equator, except near Africa."

During the International Indian Ocean Expedition, P.R. Pisharoty found only about 50 per cent of the moisture between longtitude 42 degrees and 75 degrees over the equator of that across the West Coast of India. He did not consider data in the equatorial area (within 5 degrees of the euqator on either side) between longtitude 38 degrees and 42 degrees east where the flow across the equator is maximum; he thus got low value of moisture flow across the euqator.

K.R. Saha calculated the flow of moisture for July 1964 across the euqator between longitude 42 and 75 degrees using also the data of Seychelles Island—that is,latitude 40 degrees 36 minutes, and longitude 55 degrees 30 minutes east.

Saha's value of moisture flow is about 75 per cent. If data between longitudes 38 and 42 degrees east were also used, the moisture value would not probably be less than what had been computed over the West Coast. Thus Saha's computation would show that the origin of the south-west monsoon current is in the southern hemisphere.

J. Findlater has found that the bulk of air flow across the equator between longitudes 35 and 75 degrees occures between about longitudes 37 and 45 degrees east, the maximum flow being between longitudes 38 to 42 degrees east.

Unfortunately, the observations taken by the Russians during Monex 1973 did not extend west of longitudes 45 degrees east over the equatorial area, a vital factor in considering the origin of the south-west monsoon current as shown by Findlater.

It is thus clear that unless one uses observations over the equatorial areas between longitudes 37 and 45 degrees east and particularly between longitudes 38 and 42 degrees east, one will be drawing conclusion which are untenable and cannot be accepted.

In the accompanying map (reproduced from Findlater's 1969 paper with the permission of the Royal Meteorological Society of London) are given the maximum wind speeds known to have been recorded in the layer 2,000 to 8,000 feet during the south-west monsoon of India.

The jet-speed (50 knots or more speed) winds over the Indian Peninsula would appear to have their origin in the southern hemisphere, southern by winds crossing the equator between longitudes 37 and 45 degrees east and blowing north-east to eastward to the Peninsula across the Arabian Sea.

It will also appear from the map that winds over the Arabian coast had spread of less than 37 knots and they cannot under any circumstances explain the jet-speed winds over the Peninsula. One will, therefore, conclude from that the south-west monsoon cannot have its origin in the northern hemisphere and the flow of air takes places across the equator west of longitude 45 degrees east.

The Russian ships did not take observations west of longitude 45 degrees east and no observations were made available from the island observatories in the western Indian Ocean and from observatories in Madagascar and eastern parts of Africa. However the jet speed winds reported by the Russian ships over the Arabian sea between 57 degrees east and the west coast of India are very important, as they fit in very well with the jet speed flow seen in the map.

In conclusion, it can be stated that the origin of the south-west monsoon current over the Arabian Sea is in the southern hemisphere. In fact, the conslusions of Monex 1973 stabilise the theory of the origin of the monsoon current in the southern hemisphere. I am afraid, if the interpretations are accepted and our forecasting is changed accordingly, the accuracy in forecasting the onset and performance of the monsoon so far achieved by us would not be maintained.

If correct forecasting of the monsoon 2 to 4 days ahead is to be made, it is absolutely necessary to have observations directly from selected observatories of Ethiopia, Somalis Kenya, Tanganyka, Madagascar and island observatories in the Indian Ocean. The expenditure will be insignificant.

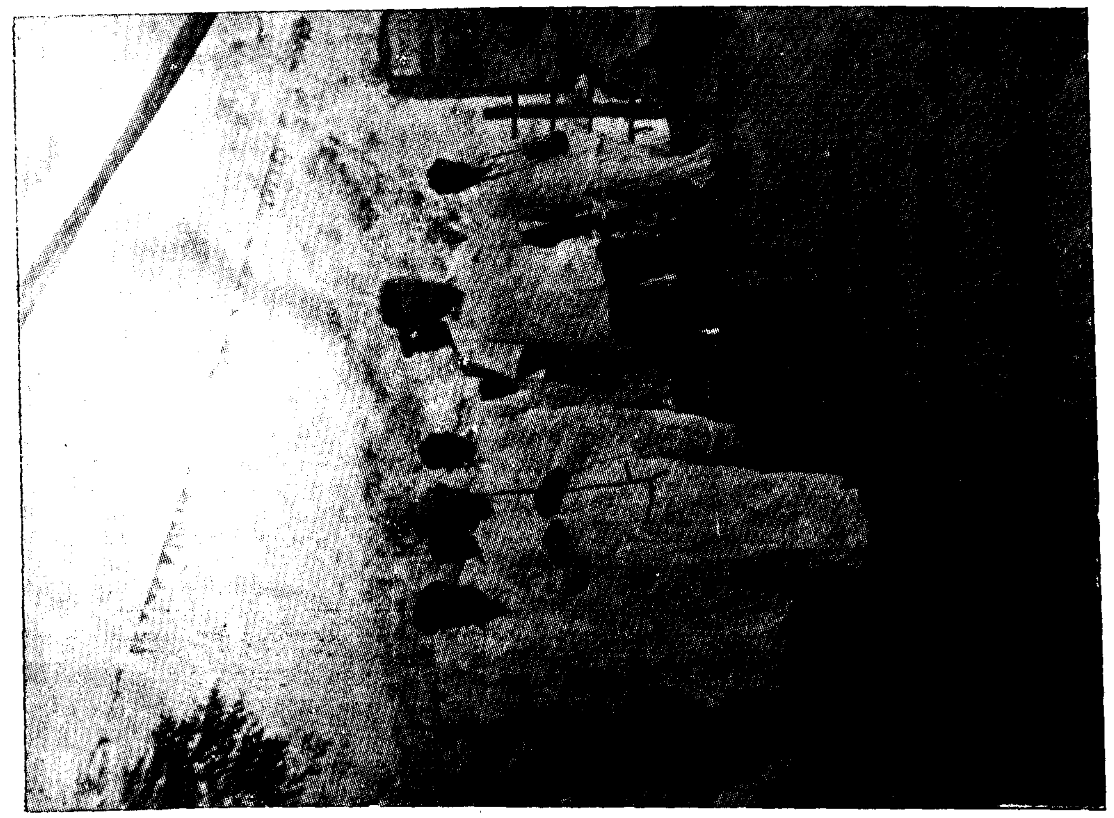
† *Retired Deputy Director general of Observatories (forecasting).* FIG IN PAGE 41

**Extract from "The Sunday Standard", dated 6th January 1974.*

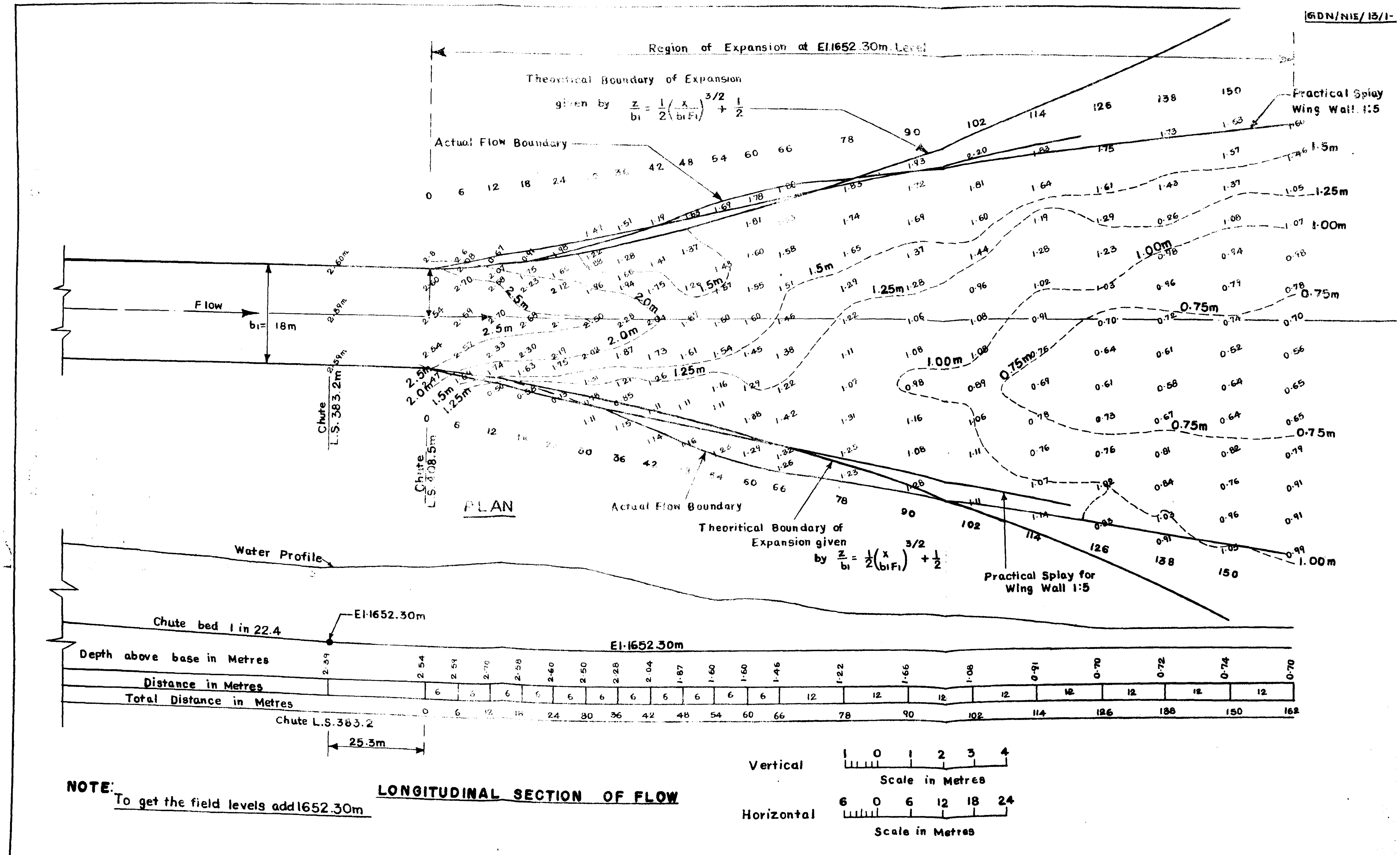
NEW IRRIGATION ERA

INDEX FOR VOLUME XIII

PARAMBIKULAM ALIYAR PROJECT—*Upper Nirar tunnel entry face inspect of mining of Tunnel*—BY HON'BLE MINISTER FOR WORKS TAMIL NADU



INSPECTION OF PARAPPALAR PROJECT BY HON. MINISTER FOR WORKS.



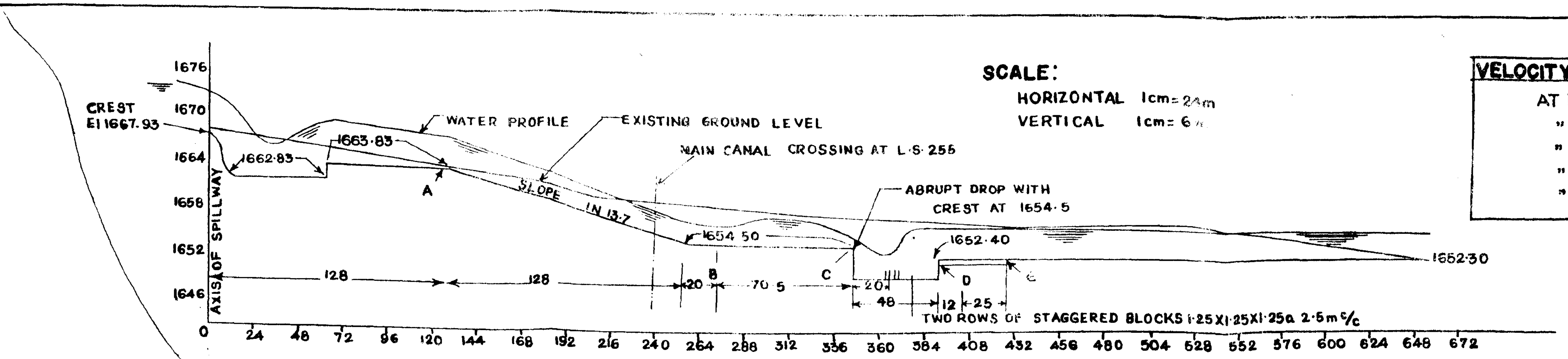
EXPANSION OF SUPERCRITICAL FLOW

GDN/NIE/14/1-75.

VELOCITY OF FLOW FOR 27000 CUSECS

AT POINT A = 3·82 m/sec
" " B = 12·70 m/sec
" " C = 13·60 m/sec
" " D = 5·05 m/sec
" " E = 1·83 m/sec

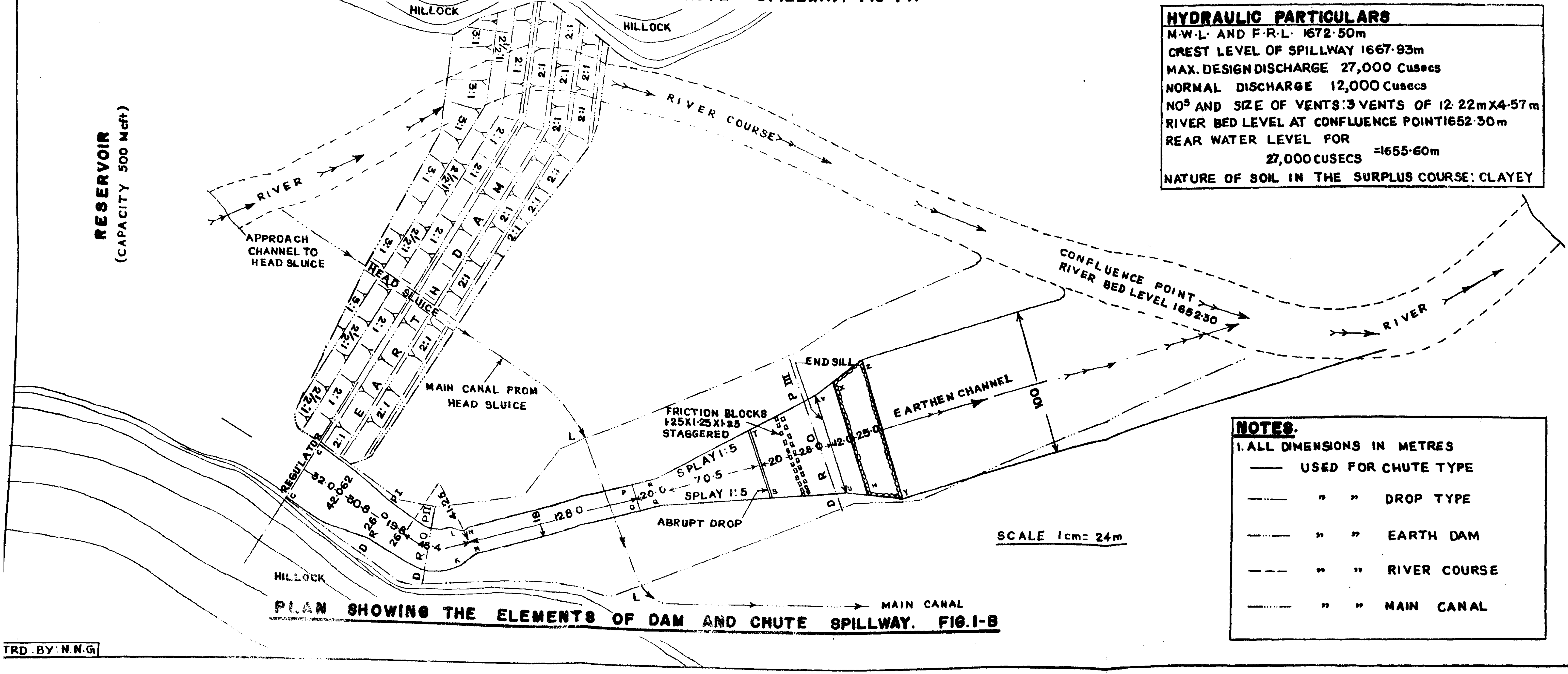
SCALE:
HORIZONTAL 1cm = 24m
VERTICAL 1cm = 6m



LONGITUDINAL SECTION OF CHUTE SPILLWAY. FIG-1-A

HYDRAULIC PARTICULARS

M·W·L· AND F·R·L· 1672·50m
CREST LEVEL OF SPILLWAY 1667·93m
MAX. DESIGN DISCHARGE 27,000 Cusecs
NORMAL DISCHARGE 12,000 Cusecs
NO^S AND SIZE OF VENTS: 3 VENTS OF 12·22m X 4·57m
RIVER BED LEVEL AT CONFLUENCE POINT 1652·30m
REAR WATER LEVEL FOR 27,000 CUSECS = 1655·60m
NATURE OF SOIL IN THE SURPLUS COURSE: CLAYEY



NOTES.

1. ALL DIMENSIONS IN METRES

——	USED FOR CHUTE TYPE
—·—	" " DROP TYPE
—··—	" " EARTH DAM
— — —	" " RIVER COURSE
·····—	" " MAIN CANAL

PLAN SHOWING THE ELEMENTS OF DAM AND CHUTE SPILLWAY. FIG.1-B

TRD. BY: N.N.G.

வேண்டுகோள்

பாசனமுறையின் மறுமலர்ச்சி செய்தி

தமிழ் மொழியே தமிழ்நாட்டின் ஆட்சி மொழியாக வேண்டும் என்று தமிழ்நாட்டு அரசாங்கம் முடிவு செய்திருப்பது வாசகர்களுக்குத் தெரிந்திருக்கும். இந்த நோக்கத்துடன் நமது பாசனத்துறையிலும் கடிதப் போக்குவரத்தைப் படிப்படியாகத் தமிழிலேயே மாற்றிக்கொண்டு வருகிறோம். ஆனால் தற்பொழுது தொழில்–நுட்பச் சொற்கள் வராத கடிதங்கள் மட்டுமே தமிழில் எழுதி வருகிறோம். மற்றவையெல்லாம் ஆங்கிலத்திலேயே எழுதப்பட்டு வருகின்றன. இன்னும் சிறிது காலத்திற்குள்ளாக அனைத்தும் தமிழிலேயே நடத்தப்பட வேண்டும் என்பதே அரசாங்கத்தின் நோக்கமாகும்.

இந்நோக்கம் சீரிய முறையில் வெற்றிபெறத் துணை செய்கின்ற வகையில், பொதுப்பணித்துறையில் பணிபுரிகின்ற தமிழார்வம் கொண்ட தொழில் நுட்ப வல்லுநர்கள், பொறியாளர்கள் ஆகியோர் தொழிற்சம்பந்தமான கட்டுரைகளையும் செய்திகளையும் தமிழில் எழுதி அனுப்பினால் அவை நமது புதுபுகப்பாசன வெளியீட்டில் பிரசுரிக்கப்பெறும் என்னும் நற்செய்தியினை அறிவிக்கின்றோம்.

எனவே, பொதுப்பணித்துறையில் பணிபுரியும் பொறியாளர்களும், மற்றும் ஏனையோரும், தொழில் நுட்பக் கட்டுரைகளைத் தமிழில் எழுதியனுப்ப வேண்டும் என்று கேட்டுக்கொள்ளுகிறோம். தற்பொழுது பொதுப்பணித்துறையில் சிறியனவும் பெரியனவுமான பல பாசனத்திட்டங்கள் நிறைவேற்றப்பட்டு வருகின்றன. இத்திட்டங்களைச் செயற்படுத்தும் போது எத்தனையோ இடர்பாடுகளும் பிரச்சினைகளும் எழுகின்றன. அவற்றையெல்லாம் நமது பாசனத்துறைப் பொறியாளர்கள் நல்முறையில் தீர்த்து வெற்றி காணுகின்றனர். இத்தகைய வெற்றியிலே கிடைத்த அனுபவம் எல்லாப் பொறியாளர்கட்கும் பயன்பட வேண்டும் என்ற நோக்கத்துடன் இக்கட்டுரைகள் அமைந்திருக்கவேண்டும். இதைத் தவிர கடந்த ஐந்தாண்டு திட்டங்களில் அநேகப்பாசனத் திட்டங்கள் நிறைவேற்றப்பட்டுள்ளன. அவற்றைப்பற்றிய கட்டுரைகளும்; அவற்றைப் பராமரிப்பதில் எழுகின்ற பிரச்சினைகளை விளக்கும் கட்டுரைகளும் வரவேற்கப்படும். மற்றும் பாசனத்துறைக்குத் தொடர்புற்ற ஏனைய பொருள்களைப் பற்றிய கட்டுரைகள், பாசனத்துறைப் பொறியாளர்களுக்கு பயன்படும் வகையில் அமைந்திருந்தால் அவைகளும் ஏற்றுக் கொள்ளப்படும்.

பொதுப்பணித்துறையிலுள்ள எல்லாப் பொறியாளர்களும் ஏனையோரும் மேற்குறிப்பிட்ட பொருளடங்கிய கட்டுரைகளை எழுதியனுப்பி, இத்துறையில் தமிழின் வளர்ச்சிக்குத் தங்களது பரிபூரண ஒத்துழைப்பையும், நல்லாதரவையும் அளிப்பார்கள் என நம்புகிறோம்.

ஆசிரியர்,
பாசனமுறையின் மறுமலர்ச்சி செய்தி.

PRINTED BY THE DIRECTOR OF STATIONERY AND PRINTING, MADRAS ON BEHALF OF GOVERNMENT OF TAMIL NADU.

(1)	(2)	(1)	(2)

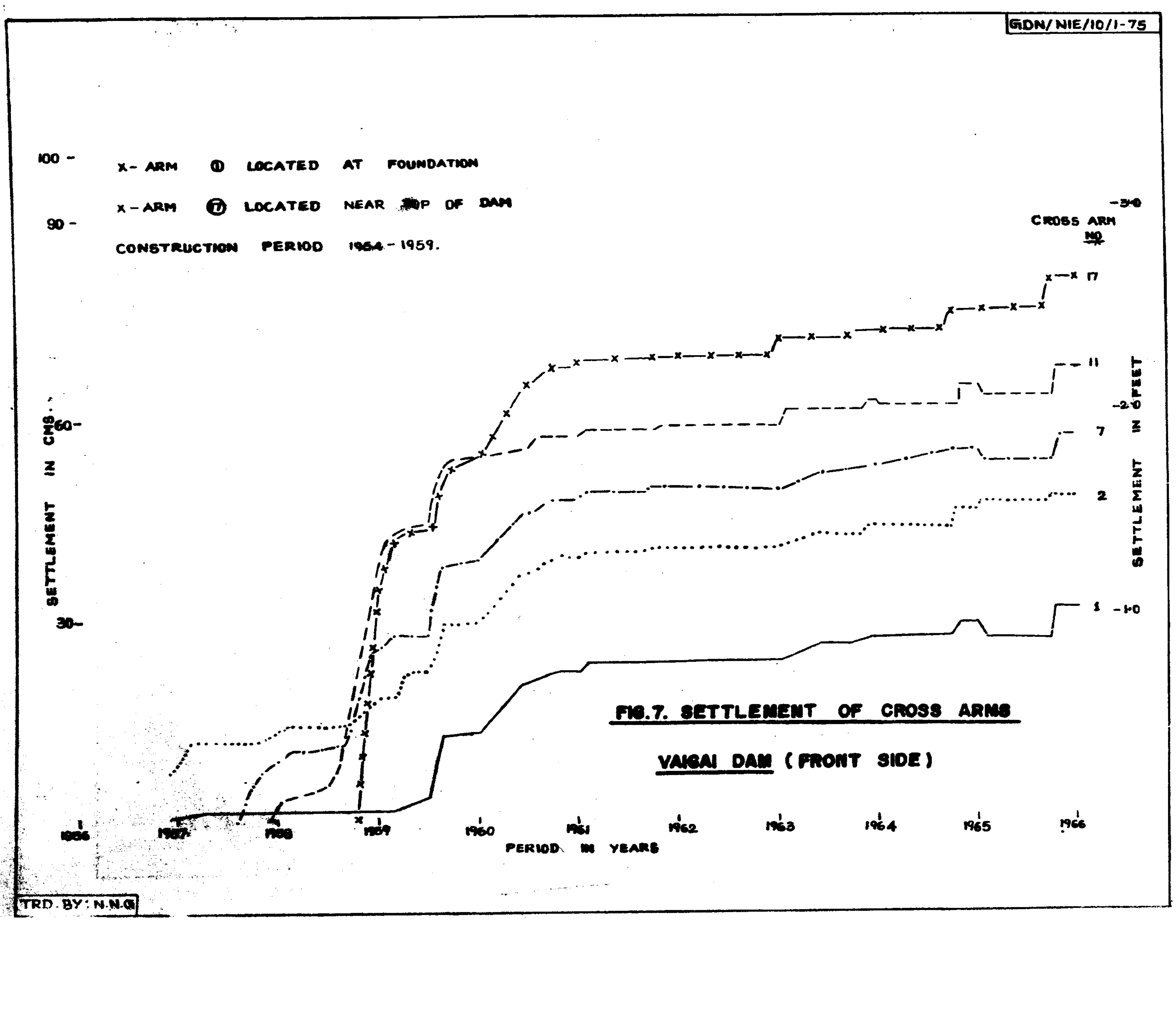
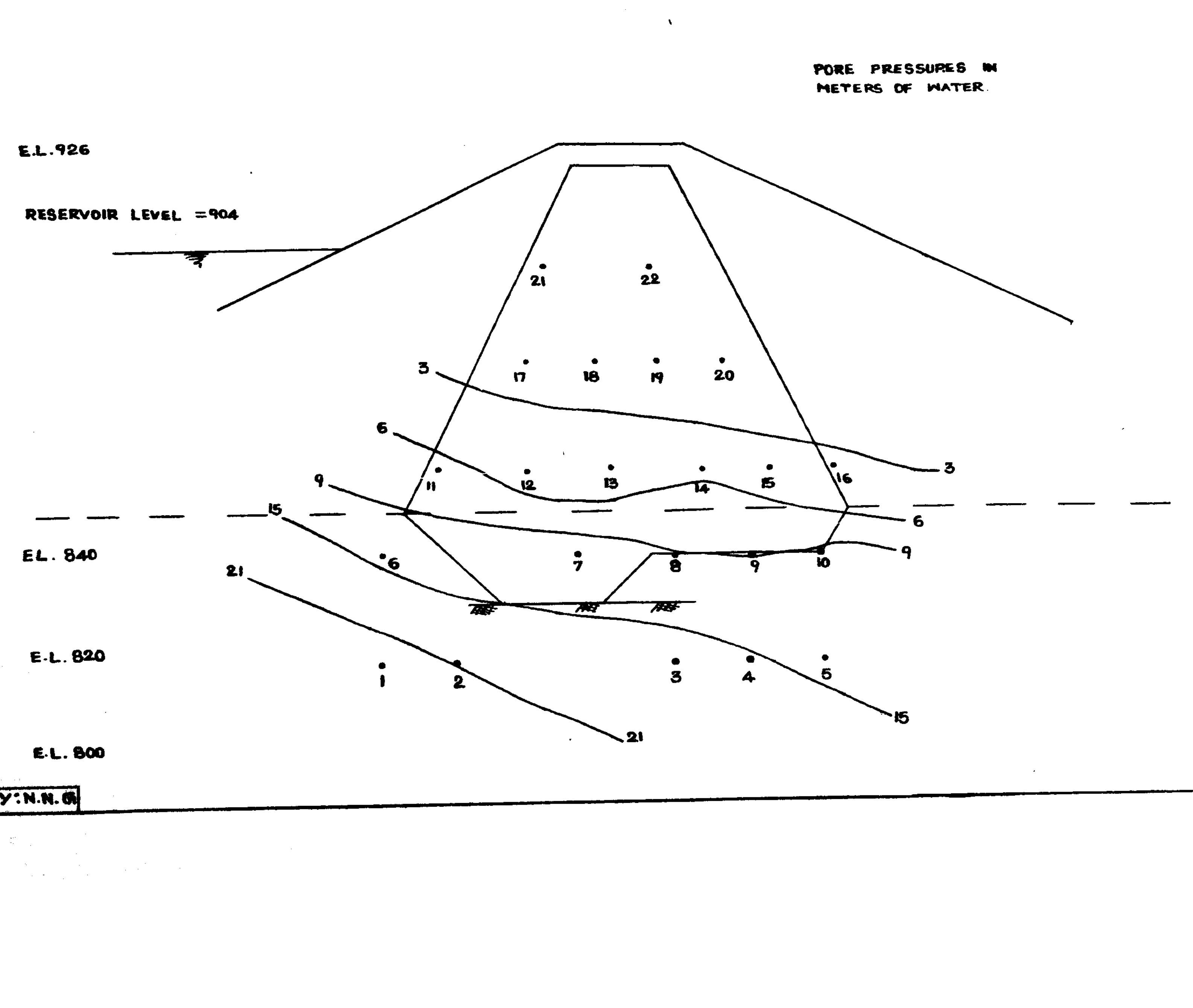


FIG.7. SETTLEMENT OF CROSS ARMS

VAIGAI DAM (FRONT SIDE)

GDN/NIE/11/1-75

FIG.8 PORE PRESSURE CONTOURS IN CORE OF DAM



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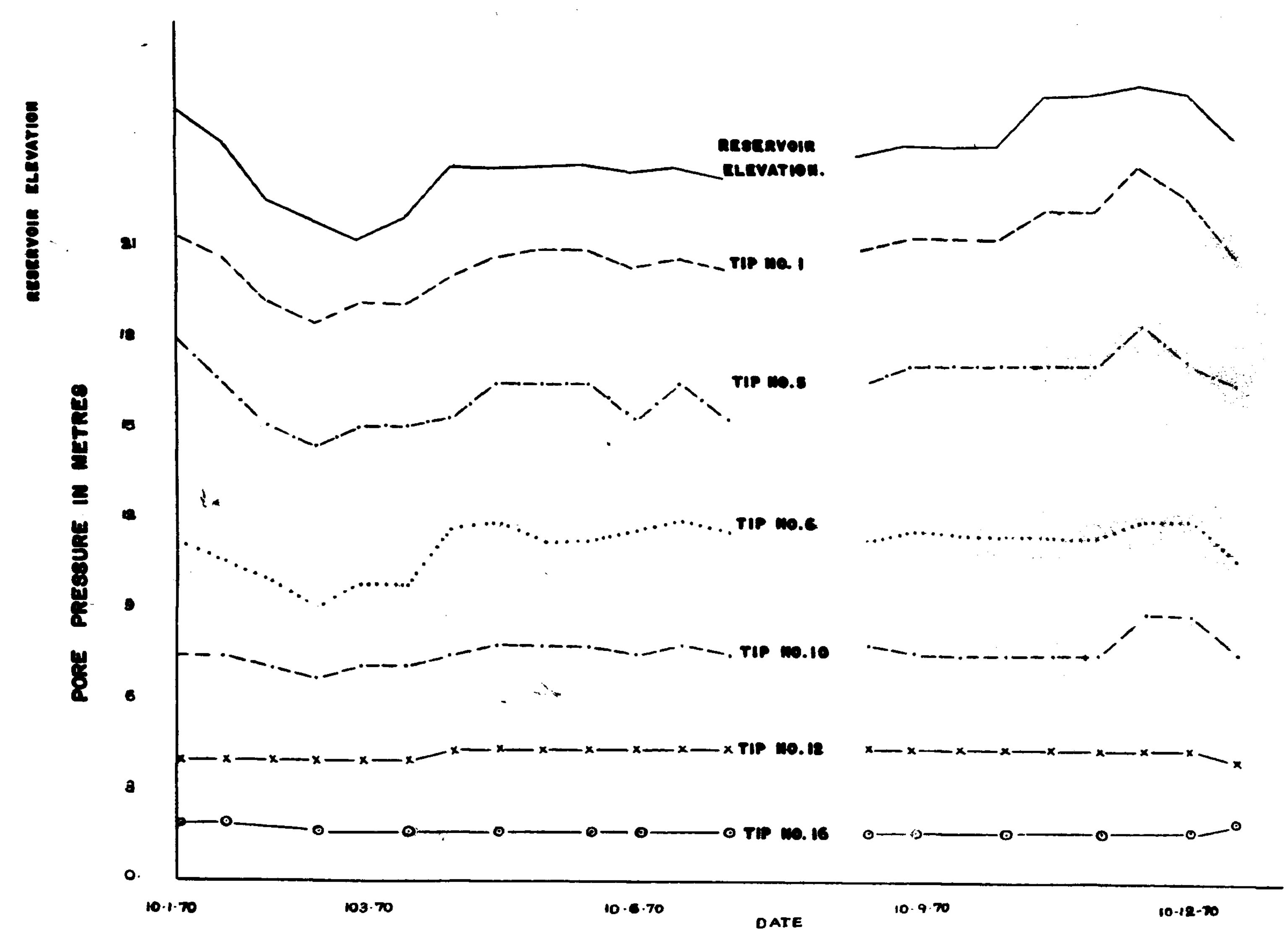


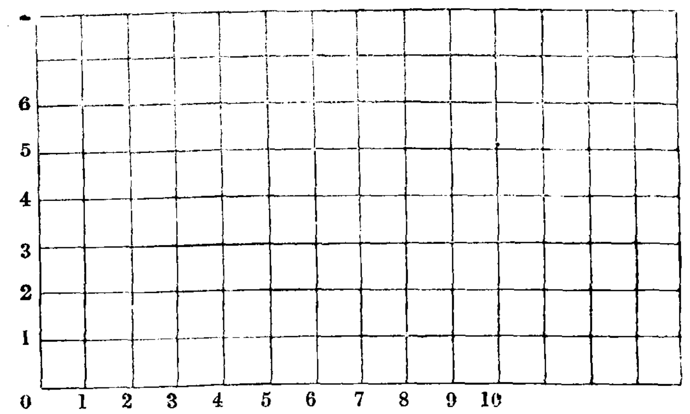
FIG.9 PORE PRESSURE VARIATION AT INDIVIDUAL PIEZOMETER.

TRD. BY: N.N.G.

4·2. From the above table it is seen that for a scale of 150/ the discharge is only 1·7 cfts which is easily available at the laboratory, and also the area required is within the available limits. Hence a scale 1/50 is selected for constructing the model.

4·3. For the area of 20 square metre a tray with level bed should be formed. The bench mark may be fixed in such a way that the lowest contour in the general plan is just 5 cm. above the bed. Hence in this case it is fixed at El. 1645·00 m. This reference elevation has to be painted on the top of the bench mark and kept in safe condition. A suitable shed may be provided for protecting the model from sun and rain.

4·4. For transferring the contours from the plan to the tray prepared, 1·00m × 1·00 m Grid lines may be drawn and painted on the tray. The Grid lines may be numberd as shown in the figures given below:—



4·5. Now, equivalent grids of 1·00 m × 1·00 m size has to be drawn in the general plan which represents 50m × 50m in the prototype. After the completion of the grid lines in the general plan all the contours may be transferred to the tray and painted with different colours with their corresponding levels. Then the axis of the earth dam, centre line of surplus course and river course may be fixed and drawn in the tray.

4·6 M.S. rods of 6 m.m. diametre has to be fixed along the contour lines to the respective heights of the contour levels. For example if we consider El. 1660·00 m. contour the total height from the bench mark 1645·00 m. is 15 metres. This is reduced to 30 cm. of height for the model. Hence 30 cm. length of 6 mm. M.S. rods may be cut and fixed along the El. 1660·00,m. contour, with an interval of 30 cm. to 35 cm. When the sharp curves of contour is to be represented, the distance between the M.S. rods can be still reduced and fixed on the tray by cement mortor. (*vide* Photo No. 1)

4·7. To start with, a single brick wall may be constructed along the axis of earth dam upto the road level leaving the space for spillway portion and canal sluice. The canal sluice portion may be made in teakwood from entry to the exit hood and the shutter with perspex sheet. As the teakwoods are all well seasoned there will be no bulking effect during the submersion in water for any amount of time. Countrywood can never be used for all these purposes.

4·8. Regarding the spillway block the profile of the spillway has to be drawn to the scale 1 in 50 in the graph sheet using the given equation of the spillway profile. Based on this curve the female template may be prepared in teakwood and used for constructing the spillways. Similarly a drawing for pier has to be drawn to the scale 1 in 50 and after making the male template the piers may be constructed using cement mortor. The radial shutters may be made in G.I. Sheet and fixed in all the vents with the help of grooves provided on the sides of piers. The opening may also be marked at 30 cm. interval along the curvature of the shutters. Next the proposed chute type surplus course may be started for construction (*vide* figure 2).

4·9. After fixing all the rods along the contours, the entire portion may be moulded by filling with brick jelly and sand. The upstream and downstream of the earth dam, Water spread area, Stilling basin arrangements, river course and surplus course, etc., have to be made leak proof with cement plaster finished smooth (*vide* figure 3).

4·10. For allowing water to the model from the main flume suitable arrangements have to be made such as leading flume, stilling tray, and honey combs, etc. (*vide* figure 4). For measuring the quantity of the water allowed into the model a 90° 'V' notch has to be fixed across the leading flume. It has to be constructed so that the water from the 'V' notch freely falls into the stilling bay and to avoid the turbulance and uneven flow on the basin honey combs may be provided (*vide* figure 4). The honey combs may be circular shaped immediately after 'V' notch and a straight honey comb across the stilling basin may also be constructed next to the circular honey comb.

4·11. In the downstream portion tail gate arrangements may be provided to build up the rear M.F.L. Water discharged through the surplus arrangement may be measured in downstream portion with 90° 'V' notch. This is to verify whether the actual quantity of water allowed in upstream flows without any leakage in the model and also to maintain a stable and steady condition of discharge for making observations, in the model. In the downstream portion also a small basin has to be constructed and suitable honey comb arrangements may be provided to get an even flow (*vide* figure 5).

4·12. For measuring the discharges in upstream and downstream side (*vide* figure 7) two gauges have to be used. For this gauge wells should be constructed at a distance equal to four times the maximum head over 'V' notch by the side of the leading flume and 2·5 cm. dia G.I. pipemay be used to allow water into the gauge well (*vide* figure 6). Similarly gauge wells have to be provided to measure the Head over crest of the spillway, and rear water levels for the surplus course.

5·0 *Conclusion*—

5·1. The same procedure may be adopted for the Construction of all the Comprehensive or three dimensional models.

Reference :—

" Note on Techniques of Hydraulic Models " by Er. P. KUMARASWAMI, Director, Institute of Hydraulics and Hydrology, Poondi.

PANORAMIC VIEW OF COMPREHENSIVE MODEL

Fig. 1 MIDDLE STAGE OF CONSTRUCTION

Photo No. 3

Photo No. 4

Fig. 5 D/s 'V' NOTCH

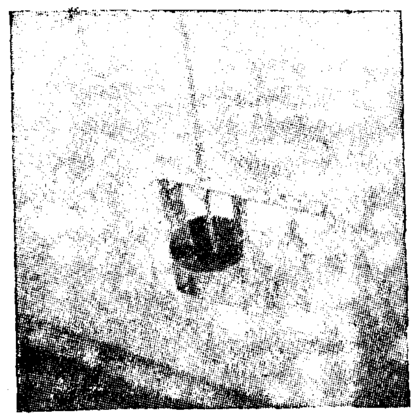


Fig 6.--U/s. GAUGE WELL WITH GAUGE

Fig. 7 D/s. GAUGE WELL WITH GAUGE

FLOW FREQUENCY ANALYSIS—CURRENT METHODS.

By

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1. *Synopsis* :

Prediction of frequency of a given magnitude of flow is important for the design of water use or control projects. Many methods are available for flow frequency analysis. Each one assumes the flow data to follow a particular statistical distribution. Statistical distributions generally used are reviewed.

The flow data of Vaigai River at Peranai has been used to illustrate the application of Gumbel's method of flow frequency analysis. The fitted Gumbel's frequency curve along with the plotting positions are shown in the figure indicate the fit of the computed line.

2. *Introduction* :

Flow frequency analysis is used by engineers as an aid in the evaluation of water use and design of water use or control projects. In some cases such an analysis gives the final solution for a flow problems but in most cases the analysis is only one of the steps in a logical engineering study in which the projects evaluation or design may advance far beyond the scope of flow frequency analysis.

A flow frequency analysis consists of a study of past records of flow to estimate frequencies of future flows. It is used for the determination of flood storage in flood control projects, estimation of runoff from airports and urban areas, design of water ways, cross drainage structures, spillways, diversion dams, bridge openings, culverts and in the study of floods flows and damages.

Frequency analysis can be directly applied to the data of flood peaks, flood volumes, mean flows, minimum flows and river stages.

3. *Commonly used methods of flow frequency analysis* :

The purpose of frequency analysis is generally the prediction of what will happen in the future. In using past records for predicting the future, it must be assumed that there is no change in the nature of factors causing floods. The past record is then considered a sample of the total statistical population consisting of past and future floods. However when predicting the future, the sample must be considered as only an approximation.

There are quite a few methods of frequency analysis differing only in the chosen statistical distribution that the recorded magnitudes of a hydrologic event are assumed to follow.

Most of the theoretical distribution used for the frequency analysis are based on three main assumption :

(i) Number of observations used in the analysis are indefinitely large.

(ii) Observations are pure random.

(iii) Factors producing hydrologic information are time independent or probabilistic rather than stochastic.

The first assumption is seldom satisfied in the real world since the record is mostly of small size. Hydrologic information are likely to be inter dependant to some extent and the factors producing hydrologic observations seldom remain unchanged. In reality, all hydrologic processes are more or less stochastic but assumed probabilistic for frequency studies to simplify the analysis (7).

Hydrologists have long sought one theoretical distribution that would describe flood events. If there were such a universal distribution, the observed distributions of flood events at various sites would differ only in parameters of that universal distribution and in sampling error. Basin characteristics do influence the flood events so that it is very unlikely that any one theoretical distribution would be generally applicable. It is also known that a frequency curve obtained from a small sample differs greatly from the population frequency curve. As a consequence of the variability of characteristics of one basin to another basin and also the sampling variability in time, several theoretical distributions are used in hydrology. In most cases, the selection of probability distribution functions is based on experience. Often two or more probability distribution functions may fit the data very well.

In general, the functions can be one, two, three and even four parameter ones, depending on the number of parameters with which they are described. The more parameters in a function, the more flexible it is in fitting any set of data. But the number of moments needed for estimating parameter increases as the number of parameter increase. The reliability of moments estimated decreases with an increase in their order. Hence a via media is to be chosen between these two conflicting requirements.

The parameters of the distribution define properties of location (central tendency, boundary values), scale (dispersion or concentration), shape, asymmetry, peakedness, asymptotes and others. The goodness of fit depends on the sample size, proper selection of the fuention and the use of best methods of estimating parameters (4).

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