The New Irrigation Era

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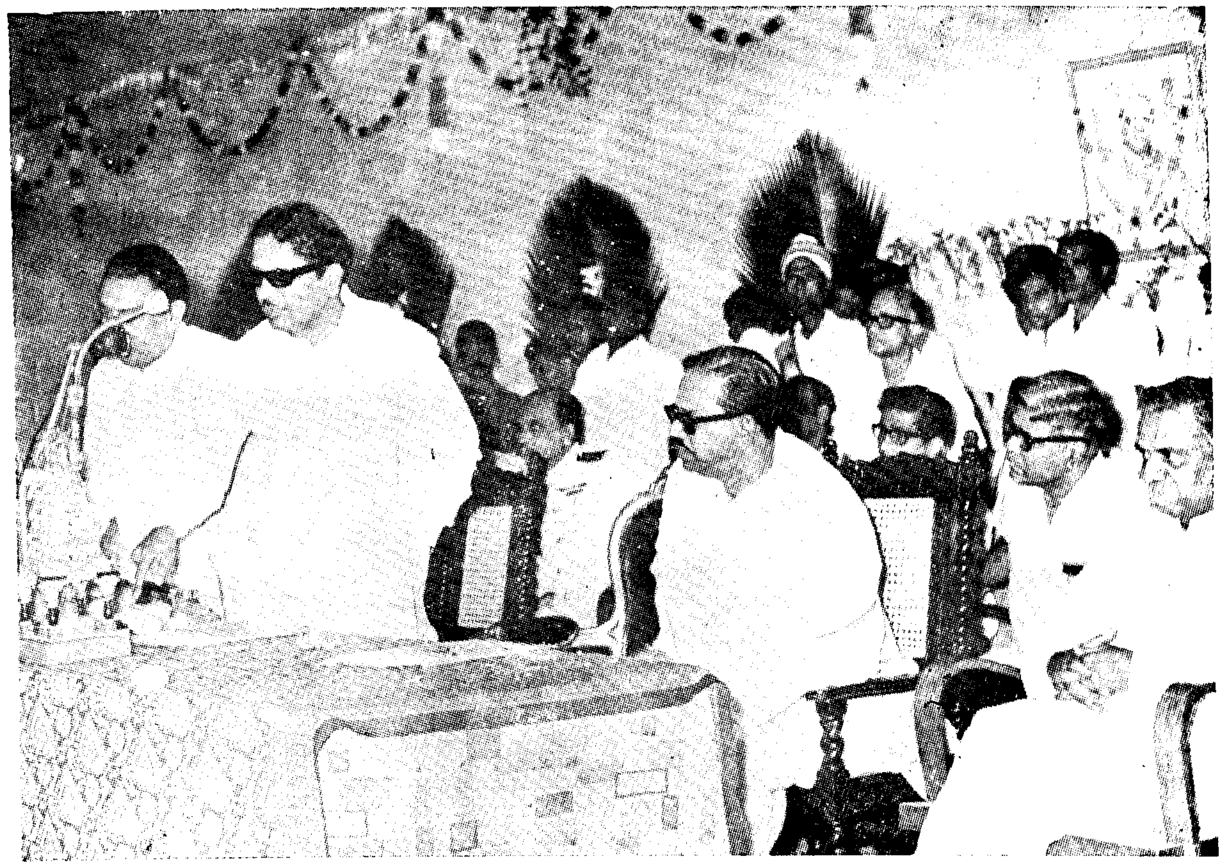
THE NEW IRRIGATION ERA

Ponnaniar Project—Masonry Dam in Progress.



Chief Minister Declares Open the Boat House.

Ministers in Pleasure Boat on Cooum.

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THE NEW IRRIGATION ERA

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(a) *Address delivered by Thiru S. P. Namasivayam,* B.E., F.I.E., *Chief Engineer (General and Irrigation), Tamil Nadu at the 11th Convention of the Agricultural Engineers held at Coimbatore in January* 1973.

(b) *Summary of the Proceedings of Silver Jubilee Celebrations of Irrigation Research Station, Poondi.*

(c) *Kerala-Tamil Nadu Hydrological Link.*
Thiru P. Kumaraswamy, Director, Irrigation Research Station, Poondi.

8. NEWS ROUND UP—

(a) *Cyclone and Floods in December* 1972.

(b) *A Note on Cooum Improvement Scheme.*

9. திருக்குறளில் நீர்மவியல்—

திரு. செ. ச. சீனிவாசன், பி.இ., தொழில் நுட்ப அலுவலர் (சிறப்பு), தலைமை நீர்ப்பாசன பொறியாளர் அலுவலகம், சேப்பாக்கம், சென்னை-5.

10. களிமண் நிலங்களில் கட்டிடத்தின் அடித்தளங்கள்—

திரு. செ. ஈஸ்வர மூர்த்தி, பி.இ., எம்.எஸ்சி. (என்ஜி.), இளநிலைப் பொறியாளர், மண்தன்மை ஆராய்ச்சிக்கூடம், சேப்பாக்கம், சென்னை-5.

All correspondance should be addressed to the Executive Engineer, P.W.D., Gauging Division, Chepauk, Madras-5.

EDITORIAL.

The Report of the Irrigation Commission brought out in 1972 favours as a policy Irrigation Projects designed to utilise river flows on 75 percent dependability basis. Its view on utilisation of flows of lower dependability is reproduced below :—

" The rainfall in various catchment areas varies from year to year and so does the volume of water in rivers. Irrigation projects have to be so designed that their full requirements are met in most years. At present, the practice is to design the projects to utilise river flows of 75 percent dependability. It means that in 75 years there is some surplus in the river and in 25 years some shortage ranging from marginal to substantial. It is obvious that the higher the dependability, the less the quantity of water available for utilisation. Availability can however, be improved by providing an extra capacity in the reservoir for carrying over supplies from surplus years to lean years. By adopting this device, a project can be designed on river flows of lower dependability to provide a larger volume of water to irrigators, with the same degree of assurance. But the provision of carry-over capacity in a reservoir entails additional cost and it becomes a matter of evaluating the additional supply against the additional cost. "

Planning of projects with 50 percent dependability is advocated by some because of a few distinct advantages. Such projects lend themselves easy for notifying a larger extent of area benefited by the scheme with consequent increase in the number of beneficiaries. On the otherhand, when we give hopes to many and also expect more revenue, we have to embark on a costly project investing huge amounts. With 50 percent dependability, failures may be more pronounced and large number of beneficiaries may doubt efficacy of planning and execution of irrigation works. This will lead to frustration. Those who promise high, cannot take it easy on the basis that the beneficiary will take the bad years philosophically and will feel no pain in parting with uniformly higher rates of taxes and betterment levies which he is called upon to pay.

In many of the completed projects there is a heavy shortfall in utilisation in spite of creating a higher potential. This is an eye opener to the planner to think more in terms of higher dependability and better utilisation. The higher dependability projects will achieve quick development and also will increase irrigation intensity, if the surplusses are used for the second crop.

Infine, no definite ruling can be given as to whether a new irrigation project should be designed at 75 percent dependability basis or a little higher or a little lower. Every project should be individually considered, keeping in view the potential available, the facilities required and existing commitments.

PONNANIAR RESERVOIR PROJECT.

by

THIRU M. BALASUBRAMANIAM, B.E., (HONS.)@

1. *River Ponnaniar* :—The River Ponnaniar, a tributory of the main River, Ariyar (which has two other tributories of its own namely Therkumalayar and Kannuthu Odai) takes its origin in the Kadavur Hill Ranges (the South West portion of Kulithalai Taluk in Tiruchy District) and after flowing for a distance of about 32 km. (20 miles) eastwards, it finally joins the main River Ariyar, near Anvaiyur, which is 4.8 Km. (3 miles) west of Manapparai. This tributory namely Ponnaniar, as well as the other two tributories mentioned above, are not perannial but they carry enormous quantity of water during the monsoon period.

2. *Scope of the Project* —The Project contemplates on the formation of a reservoir across the Ponnaniar River in D. Edayapatti village limits by harnessing monsoon flows and utilise the stored water for Irrigation purposes in the Mughavanur village downstream. The total yield available at the proposed site is about 10.2 M. cum (362 M. Cft.), the catchment area being 87·02 sq. Km. (33·60 sq. miles) and the yield from the catchment being 0.7537 M. cum/hectare (10·77 M.Cft./acre) taking Manapparai as the nearest rainfall station for this scheme. The surplus at the last anicut now existing in the river is found to be in the order of 33·98 M. Cum. (1200 M. Cft.). Hence the reservoir across Ponnaniar to utilise 9·91 M. Cum. (350 M. Cft.) of water can be safely formed, leaving 0·34 M. Cum. (12 M. Cft.) of water as evaporation losses. At present an extent of about 110 hectares (271 acres) of wet land is under cultivation from the water of Ponnaniar river, through two Korambus, namely Sekkanam Korambu on the right and Muthalmedai Korambu on the left and through an anicut, namely Karungulam Anicut, all of them being situated within 2 K.M. downstream of the proposed site of reservoir. By letting down regular supplies of water from the Reservoir, through the river course this ayacut of 110 hectares (271 acres) will also get stabilised. The requirement of water for this purpose works out to 1·274 M. Cum. (45 M.Cft.) at the rate of 8·2236 hectares per M. Cum. (6 acres per M. Cft.). The available yield for Irrigation purposes after allowing for the evaporation losses is 9·91 M.Cum. (350 M.Cft.). It is assumed that the storage of 9·91 M. Cum.(350 M.Cft.) will be realised in two fillings and hence a reservoir with an effective storage of 4·83 M. Cum. (175 M.Cft.) can be constructed. From the detailed survey of the topography at the reservoir site it has been asertained that this effective storage of 4·83 M. Cum. (175 M.Cft.) is obtainable with an effective hight of about 15·545 m. (51 feet) of the reservoir i.e., between contours 234·70 m (770·00), and 258·24 m (821·0′). After meeting the requirements of the existing ayacut (i.e., 1·274 M.Cum. (45 M.Cft.) with the balance available storage of 8·51 M.Cum. (305 M.Cft) (i.e. 9·91 M.Cum. (350 M.Cft.) minus 1·274 M.Cum. (45 M.Cft) and at the same rate of 8·2236 hectares per M.Cum. (6 acres per M.Cft.) a new ayacut of about 740·57 hectares (1830 acres,) is proposed to be brought under wet cultivation.

The impounding of water in the reservoir will raise the water table in the area, and in the wells in the locality thus increasing the food production indirectly. The Collector of Trichy while giving concurrence to the Scheme has slso confirmed that the rights of the lower down ryots are not affected by this Scheme.

3. *Selection of Dam Site.*—The site for the Dam selected for the reservoir lies in the narrow gorge formed by two mountain ranges, namely Semmalai hill on the the right and Perumal Malai Hills on the left in Kadavur Basin. The site proposed for the Dam (which will comprise mainly of Earth Dam covering the entire right flank and the river portion with masonry spillway on left flank) and its general alignment were inspected and approved by late Thiru U. Ananda Rao, the then Consulting Engineer to Government and Joint Secretary in 1970. The staff of the Geological Survey of India, Southern Region who also inspected the site described the geology of the rock in the area as to comprise of prominent bands of quartzite of Archaean age conformable with the general foliation trend of the gneisses in the area. The rock of this area according to them have been subjected to Plutonic metamorphism and have suffered considerable folding etc., which are clearly brought out by the changes in the trend of the quartzites in the area. The quartzites in the area are buff to pale yellow in colour, coarse grained and highly laminated. In order to locate the existance of more reliable rock extensive exploratory drilling operation were conducted at the site of the alignment especially below the proposed spillway and non spillway (Bulk head) section which is essentially to be of Masonry. The rock cores obtained by drilling operation, revealed that the gneissic rock available below the general ground level was also not a Continuous one, being intercepted with numerous fractures and joints. The rock even at great depths, was found to be weathered to some extent though the core recovery was in the odrer of 80%. Tests were conducted in the Soil Machanics Laboratory, Madras on the rock samples for durability etc., it was apprehended that the

@ *Executive Engineer, P.W.D., Ponnaniar Project Division, Manapparai, Trichirappalli District.*

14 *Masonry dam*—

(i) Length	51·82 m (170′).
(ii) Maximum height (at river bed).	23·16 m (76′).
(iii) Spillway	9·75 × 2 Nos. = 19·5 m (32′ × 2 Nos. = 64′).
(iv) Number of vents ..	2 Nos.
(v) Size of each vent ..	9·75 m × 3·05 m (or) (32′ × 10′).
(vi) Crest level	247·19 m (811′).
(vii) Thickness of Central pier.	7·62 m (25′).
(viii) Top level of road way.	253·29 (831′).
(ix) Width of road way at top.	3·66m (12′).

15 Irrigation-*cum*-River sluice.

(i) Sill level	234·70 m (770·0)
(ii) Location	At the central pier.
(iii) Number of vents ..	1 No.
(iv) Size of vent	1·52 m × 1·83 m (5′ × 6′).
(v) Size of sluice Barrel.	1·52 m × 1·83 m (5′ × 6′).
(vi) Discharge (Max.) ..	284·3 Cumecs (1,040 cusecs).
(vii) Discharge required ..	1·18 cu.mecs (42 cusecs).

16 Earth Dam.

(i) Length	195·07 m (640′).
(ii) Max. Height at River bed.	24·39 m (80′) i.e. (751′ to 831′).
(iii) Top width of earth dam.	7·62 m (25′).
(iv) Road way on top of earth dam.	6·096 m (20′).
(v) Downstream side slope.	2:1 with 3·048 m (10′) wide berms at 6·096 m(20′) heights.
(vi) Upstream side slope.	2·5 : 1 From top up to El. +244·14 m (801′) 3 : 1 Below El. +244·14 m (801′).

17 New ayacut proposed ..	740·57 hectares at 8·224 hectares per M.Cums. (1,830 acres at 6 acres/M.Cft.).
18 Stabilisation of ayacut ..	110 hectares(271 acres)

5. *Financial Aspects of the Project*—

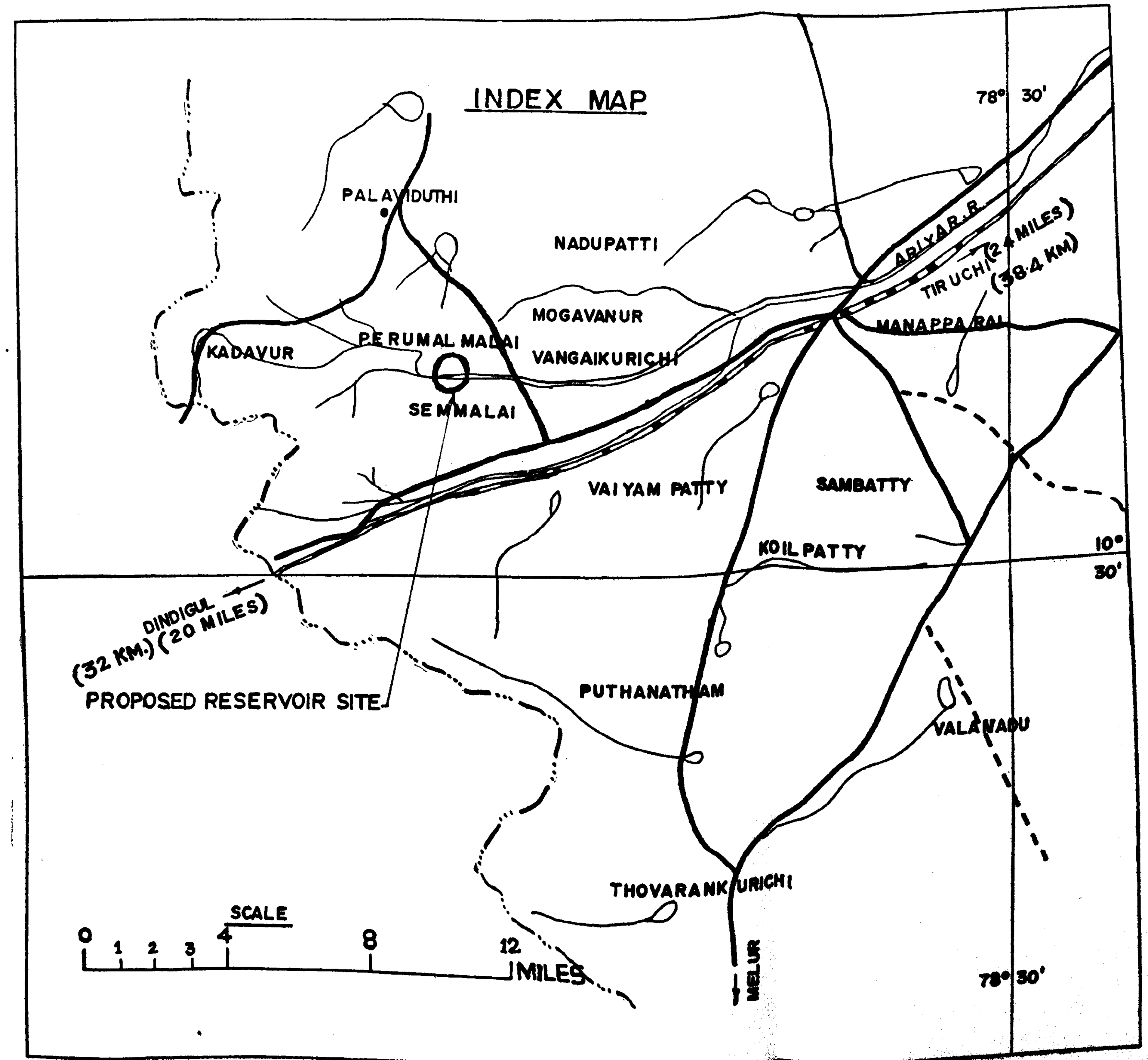
The financial aspects of the Project are abstracted below :—

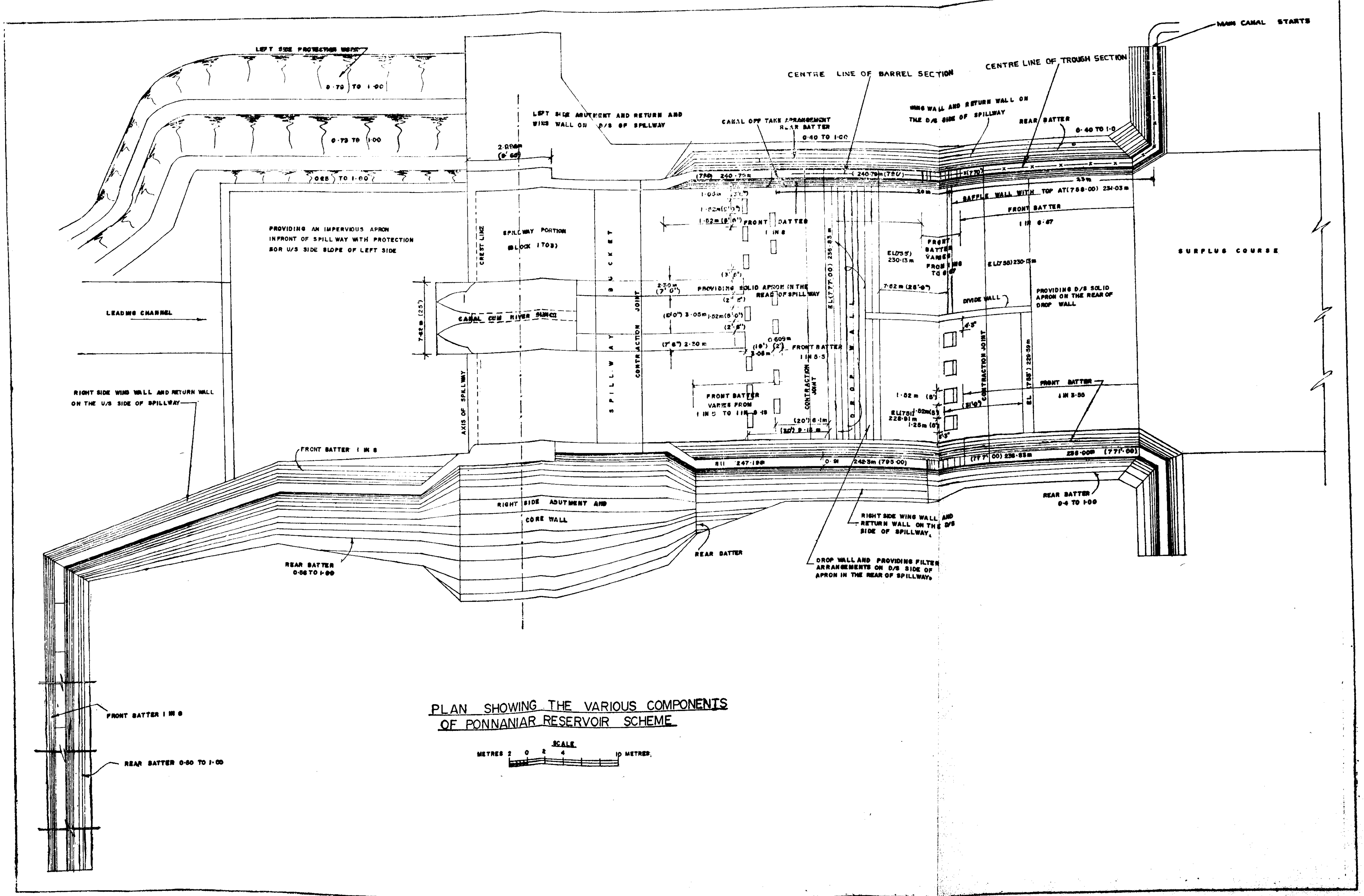
	RS.
A. *Direct Charges*—	
I. Head Works ..	81,22,540
II. Main Canal and Branches.	2,10,000
III. Distributories and field bothies.	4,03,260
IV. Spl. Tools and Plants.	1,65,000
Establishment charges including Pensionary and audit charges suspense and Receipts and recoveries.	10,86,200
	99,87,000
B. *Indirect Charges*—	
Capitalise abatement of land revenue.	10,000
Total ..	99,97,000

The percentage return from this Project is 3·7 or 4 as per the original Scheme Estimate. However due to lining of the Main Canal and Distributory the Maintenance cost is likely to go down very much, and hence the percentage return is likely to increase by 20 per cent to 30 per cent. The rate per tonne is brought down from Rs. 5,077 to Rs. 4,725, by having the lined section of canal.

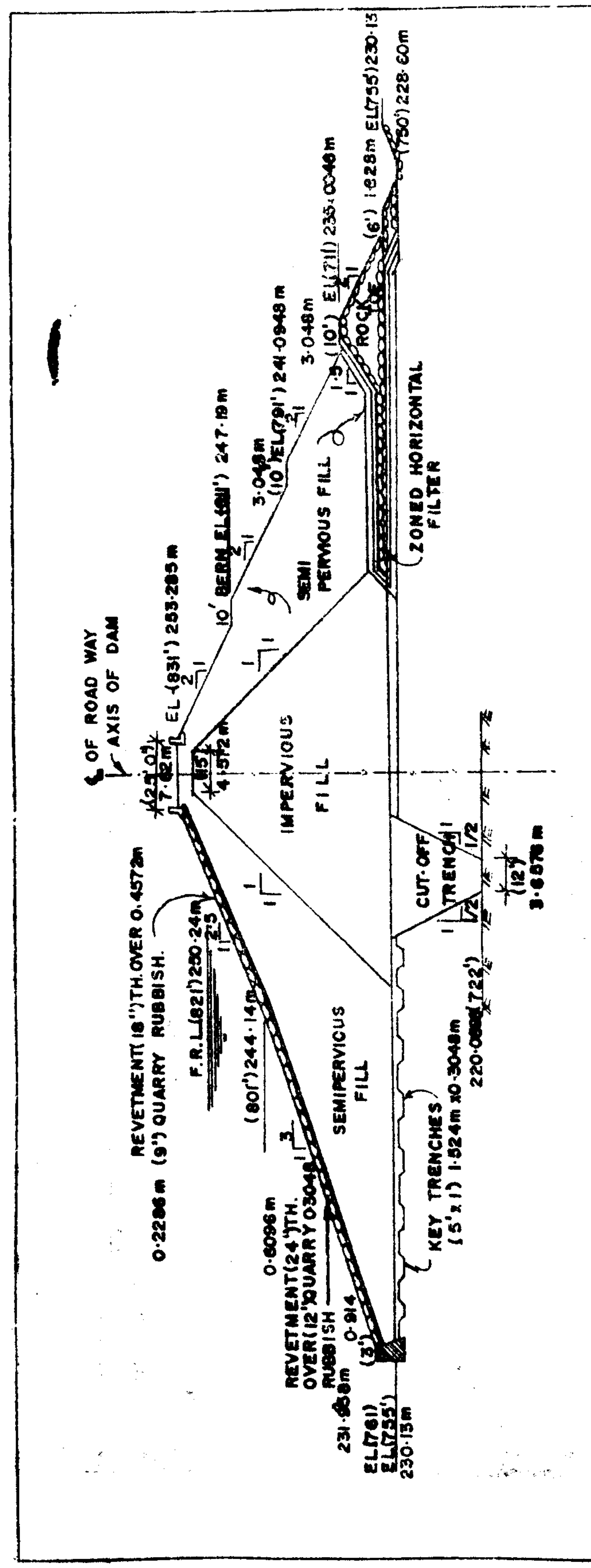
The Project works are in advanced stage of construction and is programmed to be completed in all respects by the end of February 1974.

* (Comments on the design and other aspects of the Project are welcome from readers.)



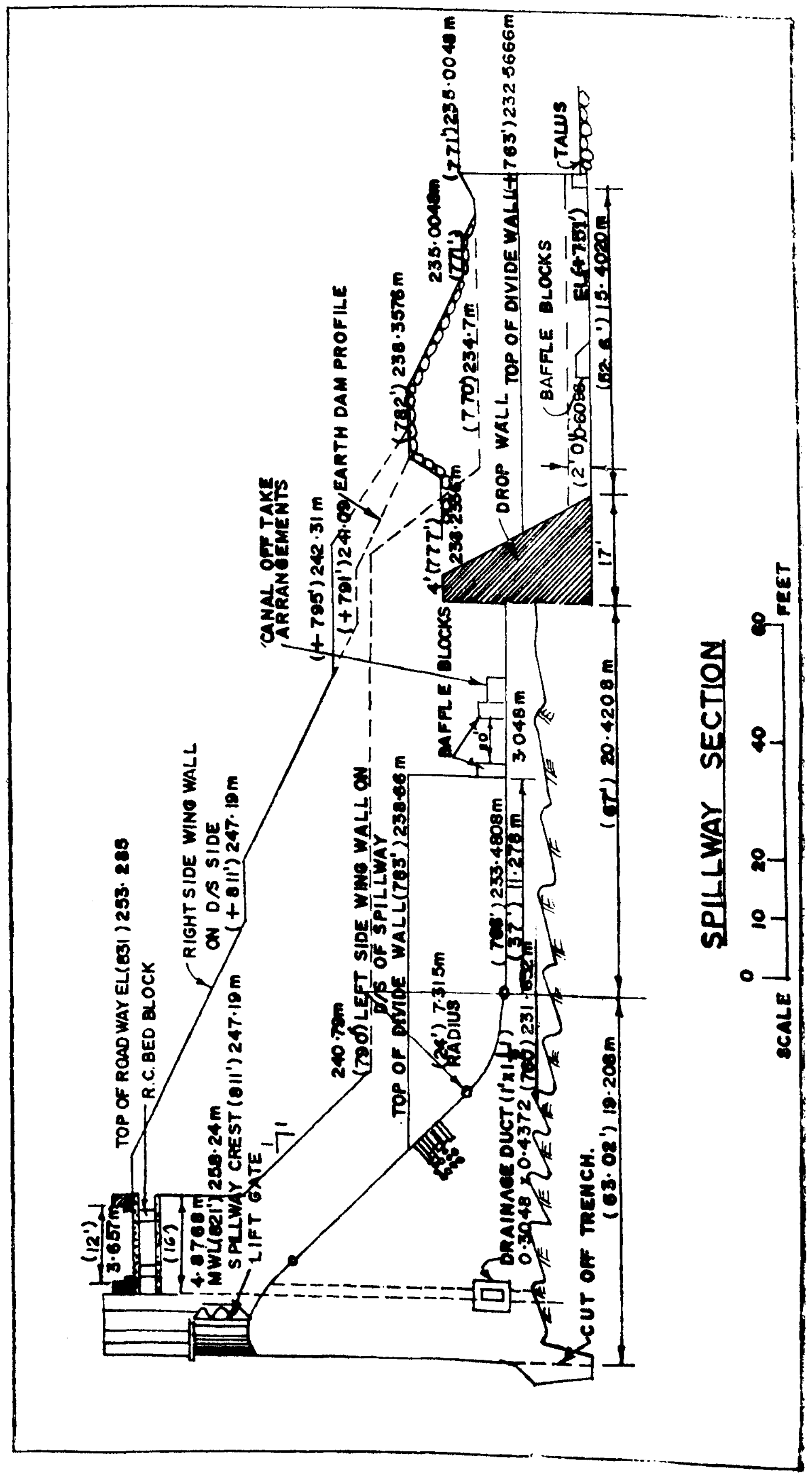


PLAN SHOWING THE VARIOUS COMPONENTS OF PONNANIAR RESERVOIR SCHEME



SECTION OF EARTHDAM

SCALE 0 20 40 60 80 FEET



GLIMPSES OF PALAR BASIN

By

Thiru C. S. Kuppuraj, B.E., A.M.I.E. (Ind.)*

Preamble :

The Palar river which is seen with its dry sandy bed throughout the year, except perhaps for a week or two in some years only when some surface flow is seen, is a classic example of the situation contemplated by Avvaiyar, the Sangam poetess, when she sang.

" ஆற்றுப் பெருக்கற்று அடிசுடும் அந்நாளும்,
ஊற்றுப்பெருக்கால் உலகூட்டும். "

An assessment of the irrigation supplies and Town water supplies drawn from this apparently dry river makes us realise the greatness of this sub-surface river which feeds the people in its basin and heralds their prosperity like the real philanthropist who donates without any publicity or fanfare. Few people could imagine that this river of white sand sustains an irrigated area of thousands of acres and feeds more than twenty towns with an aggreagate population of approximately twenty lakhs. The Palar basin, which extends into the three Southern States of Mysore, Andhra Pradesh and Tamil Nadu, lies mainly in the North Arcot and Chingleput districts of Tamil Nadu. This basin is replete with palces of historical importance religious and literary importance and in recent times of industrial and technological importance. For example India's third Nuclear Power Station and the First Reactor Research Centre are drawing their entire fresh water requirements from the sandy beds of this underground river. An attempt is made in this article to describe some of the less known aspects of this great river.

Origin and Course :

The River Palar rises from the Nandhi hills of Mysore Plateau in Chinthamani taluk of Kolar district. Nandhi Hills which is 72 kms. (45 miles) from Bangalore situated at an altitude of 1,478 m. (4,850') M.S.L. is famous for its salubrious climate, the highest temperature of which has never gone over 85F. This is a famous hill resort. This hill resort is very near Devanahalli. There are records to show that this place was used as summer resort by Tippu Sultan the Great Emperor of Karnataka. The historic standing rock in these hills about 304·8 m. (1,000') in depth is even now called "TIPPU ROCK". It is said, that captives who were courtmartialled to death by the Emperor were rolled down from the top of the rock.

There are two temples constructed by the emperors of Vijayanagaram. At the foot of the hills there is a temple called Boganandeeswara, constructed by the Queen Rathnavathi, in the later half of eighth century, subsequently remodelled, renovated by the emperors of Cholas and Vijayanagaram and Palayakars. The two famous towns, viz., Chickanahalli and Dhottanahalli are situated at the foot of the Nandhi Hills. One can understand how nice and a gifted place it is when the father of the Nation 'Mahatma Gandhi' has once selected this beautiful place with its splendrous scenes as a summer resort. It is from this famous Nandhi Hills, river Palar originates. The whole of Palar Valley is studded with tanks numbering about 1500 within the Mysore territory. The factor which deserves particular mention is that there is a series of large tanks, viz. (1) Kannamangala, (2) Attur, (3) Akkimangala, (4) Kalhalli, (5) Mailsandra, (6) Kadagathur, (7) Byrandahalli, (8) Somabudi Agrahara, (9) Jannaghatta, (10) Mudavadi, (11) Chillpalli, (12) Jodi Holali, (13) Betamagala, (14) Ramasagara constructed right across the main river Palar itself within the boundary of Mysore State. It is worth mentioning here that the last two but one of the tanks in the series (items 12 and 13 the Jodiholali and Bitamangala tanks) are intended not for irrigation but for water-supply to the Kolar Gold fields which claim to be the only gold mine in India.

After traversing a length of 92·8 Kms. (58 miles) in Mysore State, and a short length in Andhra Pradesh the river Palar enters Tamil Nadu limit near Pullur Village in Vaniyambadi taluk of North Arcot district and 16 Kms. (10 miles) further on, it falls through the gorges of the Eastern Ghats into the Vaniyambadi Valley. The Palar then winds its way Eastwards through the whole length of North Arcot District. The course of the river and the lie of the land on either side present a very picturesque appearnce. The river Palar then enters the Chingleput District, runs in a South-East direction and finally empties itself into the Bay of Bengal near Sadras, the old Dutch settlement, which is about 64 Kms. (40 miles) south of Madras.

The meandering length of the river Palar is 363 Kms. (230 miles) from its origin up to its confluence with Bay of Bengal. The flows in the river are sporadic and short lived especially in the recent decades.

* Superintending Engineer, P.W.D., Chingleput-North Arcot Circle, Vellore-1.

Tributories :

The major tributories of river Palar are (*a*) Malatar; (*b*) Koundanya Nadhi; (*c*) Poiney river; (*d*) Cheyyar.

(*a*) The river Malatar originates from the Nandhi Hills near Mulbagal and joins Palar near Ambur, which is situated on the right bank of river Palar.

(*b*) The tributory Koundanya Nadhi rises from Punganuru forest in Andhra Pradesh, after traversing through Palamaner joins Palar near Gudiyatham. There is a tank called Pallavan-Eri at Palamaner, fed by this river. The legend goes that this tank was formed by a Pallava King. The very name Palamaner seems to be distortion of Pallavan-Eri.

(*c*) The chief affluent Poiney river starts from the southern slopes of the Eastern Ghats in Chandragiri taluk of Chittoor district in Andhra Pradesh and joins Palar near Tiruvalam which is about 9·6 kms. (6 miles) from Ranipet. The town Ranipet on the left bank of Palar was established by Nawab Sadullakhan in memory of the queen of Raja Desingh who lost his life in the battle. The story goes that soon after the death of Raja Desingh, the queen put an end to her life by self-immolation. In praise of her chastity and affection towards her husband, the Nawab formed and named this town as Ranipet.

(*d*) The river Cheyyar emanates from Javadhi Hills in Chengam taluk and traverses through Polur, Arni and Cheyyar taluks of North Arcot district and joins the Palar at Tirumukkudal at Wallajabad of Chingleput district.

While we think of Palar Basin the historical events that took place at Vellore, Arcot and Kancheepuram stand before our eyes.

The Vellore town is situated on the bank of river Palar. There is a famous fort in the heart of the town. This fort was constructed by Bommireddi and Thimmireddy of Badrachalam by engaging the sculptors Badrikasi and Immam from Northern India. A temple with an attractive Kalyanakudam inside the fort attracts a large number of tourists. The Vellore Mutiny of 1806 broke out owing to the infringement of the religious sentiment of the sepoys.

The town Arcot is about 24 kms. (15 miles) from Vellore. There is a dilapidated Fort inside the town. The history of the Fort dates back to British and French settlement. Robert Clive after defeating Chandha Sahib,near Tiruchirappalli,captured the entire Karnataka and Arcot Fort. This victory of Robert Clive had laid the foundation for the establishment of British Empire in India.

Kancheepuram :—

The town is located on the bank of Vegavathi river which is a minor tributory of Palar. This town was in existance even 2,000 years back. It was once the capital of Pallava empire. This town was specially mentioned in the ancient lores viz.,Porumbanarrupadai, Manimegalai, Kalingathubarani, Dandialangaram and Periyapuranam. The town is famous for its beauty and leading temples, which attract large number of tourists. Now, Kancheepuram is the district headquarters of Chingleput district.

There are records to prove that much importance was given for the irrigation by the anicient kings who were the pioneers in the construction of tanks. There were a number of tanks in this basin even before the British put their mind for according irrigation facilities in this basin. But all the irrigation works were at the mercy of Nature. The tanks thus formed were receiving their supply through the open head channel taking off from the river Palar. It was customary to form a Korambu at the head of open head channel across the river to draw supply of water. The fact, that the tanks were already in existence can be very well understood from the reports of Lt. Col. R. Baird Smith F.G.S., Director of Ganges Canal Works who inspected the irrigation works in South India in 1856 "..........the whole of this gigantic machinery of irrigation is of purely native origin, as it is a fact that not one new tank has ever been made by us and the concurrent testimony of those best informed on the subject shows that a great many fine works of the kind have been alllowed to fall into utter disrepair and uselessness ".

For readers, some of the tanks of great antiquity and supposed to have been in existance 400 years prior to British settlement are given hereunder

1. *Kaveripauk Tank :*—Constructed by Nandhivaram III the Pallava King. This is an extensive tank having a bund about 6·4 kms. (4 miles) long.

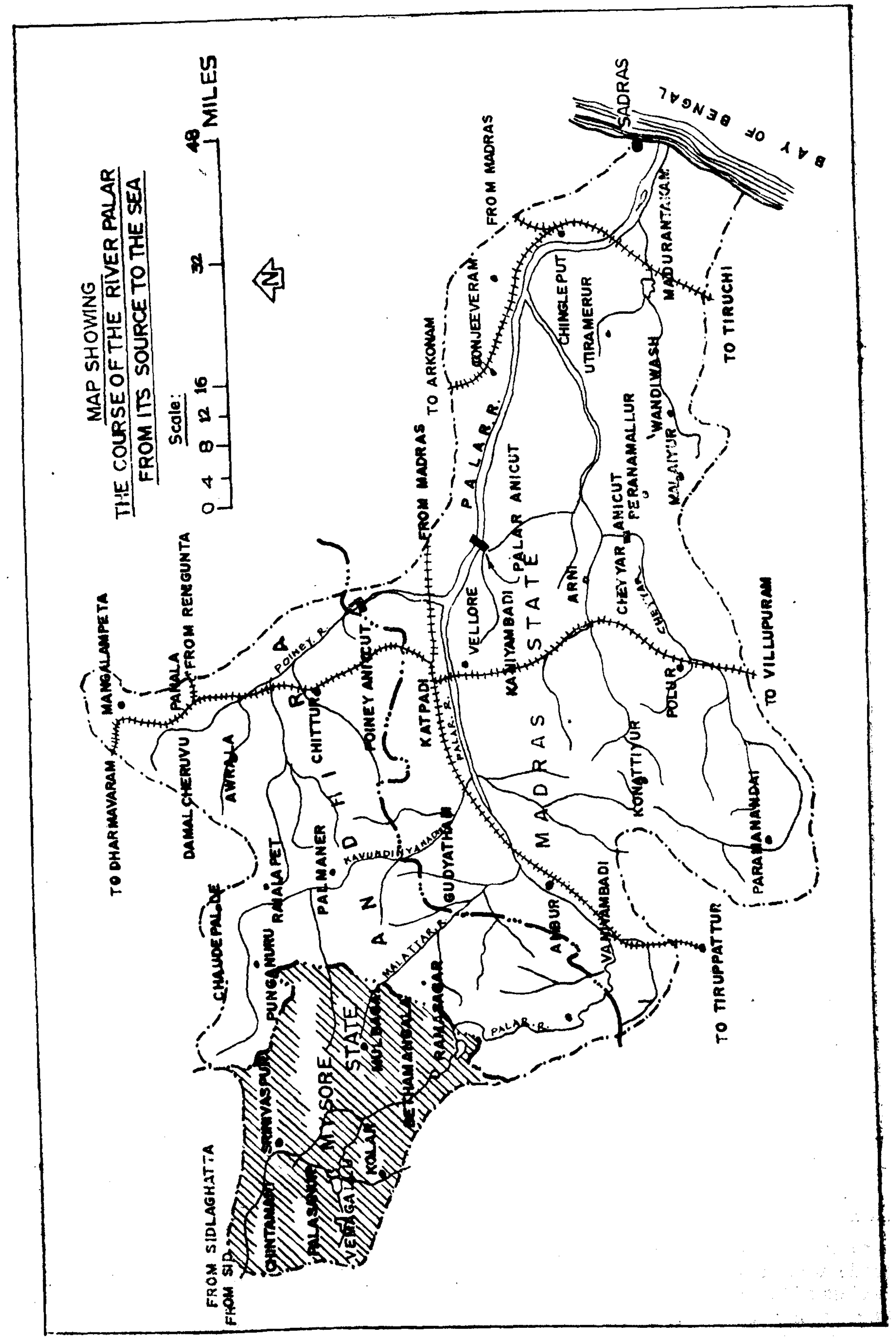
2. *Mahendravadi Tank :*—This was constructed by Mahendravarman who reigned about the first-half of the 7th century.

3. *Parameswara Tataka Tank :*—This tank is situated in the village of Kuram 14·4 kms. (9 miles) North West of Kancheepuram. It was constructed by Parameswaravarman, a great grandson of king Mahendravarman I.

4. *Thenneri Tank :*—This tank was built by Pallava King named Thirayan in the first half of eighth century. It was originally called as Thiryaneri. Subsequently, the name has been changed to Thenneri.

5. *Vinnamangalam Tank :*—The tank was constructed by Udayendran, the Great, in 920 A.D.

As stated above, these tanks were prone to the vagaries of nature. In order to have assured supply of water to the tanks and improve the irrigation facilities, several development works were taken up and one among them is the construction of anicut across tributories and the main river Palar.



If at all it is said that North Arcot stands next to Thanjavur with regard to food production, it is because of the construction of the anicut across river Palar near Wallajapet and the high ground water potential of the basin. Statistics reveal that there are more than one lakh pump-sets installed for lift irrigation in North Arcot. We are proud to say that nowhere in India so many pump-sets are installed within a single district to exploit the ground water potential.

The Palar Anicut.

The only anicut across Palar is near Wallajapet. Prior to the construction of the above anicut, a Korambu or temporary sand dam used to be constructed across the river to increase the supply in the open head channels feeding the ayacut. This temporary dam used to get seriously damaged after each flash flood. Hence to have a permanent remedy the anicut was constructed across river Palar during the year 1855-1858 at the place where the Korambu was in existance. In the year 1903, there were heavy flows in river Palar. The water level recorded then was 2·39 m. (7·85′) over the anicut crest. Due to this sudden and unprecedent inflow the anicut was extensively damaged and a portion of the anicut was completely breached. The anicut was restored in the year 1905.

The anicut as it stands now is 800·9 m. (2,628′) long and 1·52 m. (5′) high with a base width of 2·44 m. (8′0″) and top width of 1·22 m. (4′0″). It is founded on two rows of circular wells 2·44 m. (8′0″) deep and 0·91 m. (3′0″) inner diameter. There are 19 scour vents on the left flank, 9 of size 1·52 m.×1·06 m. (5′0″×3′6″) and 10 of size 1·52 m.×1·22 m. (5′0″×4′0″) and 20 of size 1·52 m.×1·22 m. (5′0″×4′0″) on the right flank. A portion of the anicut measuring 91·44 m. (300′) has been provided with 0·60 m. (2′) falling shutters on the crest.

There are four channels taking off from the anicut, two on either side with their head sluices built in continuation of the anicut.

The particulars are as below :—

Name of channel.	*Length of channel in K.Ms./ M. F.*	*Sill level of head sluice in metres/ feet.*	*Number and size of vents.*	*Designed carrying capacity in cumecs/ cusecs.*	*Number of tanks fed.*	*Designed ayacut in hectare/ acres.*
Left side channel—						
Mahendravadi	39·8 ——— 24-7	148·8 ——— 488·18	3 Nos. 1·52 m.×1·52 m. (5′×5′)	15·63 ——— 552	28	1,962 ——— 4,849
Kaveripauk	17·4 ——— 10-7	148·8 ——— 488·18	18 Nos. 1·52 m.×1·52 m. (5′×5′)	97·50 ——— 3,443	159	20,663 ——— 51,059
Right side channel—						
Sakkaramallur	32·6 ——— 20-3	149·10 ——— 489·33	3 Nos. 1·82 m.×1·52 m. (6′×5′)	17·56 ——— 624	21	2,070 ——— 5,105
Dusi or Tennambattu ..	37·0 ——— 23-1	149·40 ——— 490·18	15 Nos. 1·52 m.×1·52 m. (5′×5′)	45·25 ——— 1,598	109	8,054 ——— 19,903

The surplus course of Kaveripauk Tank is Courtaliyar which feeds, through Poondi Reservoir, the Red Hills tank from which water-supply is made to the Madras City.

In the year 1961–62, certain improvements were carried out to this anicut at an estimated cost of Rs. 53·00 lakhs.

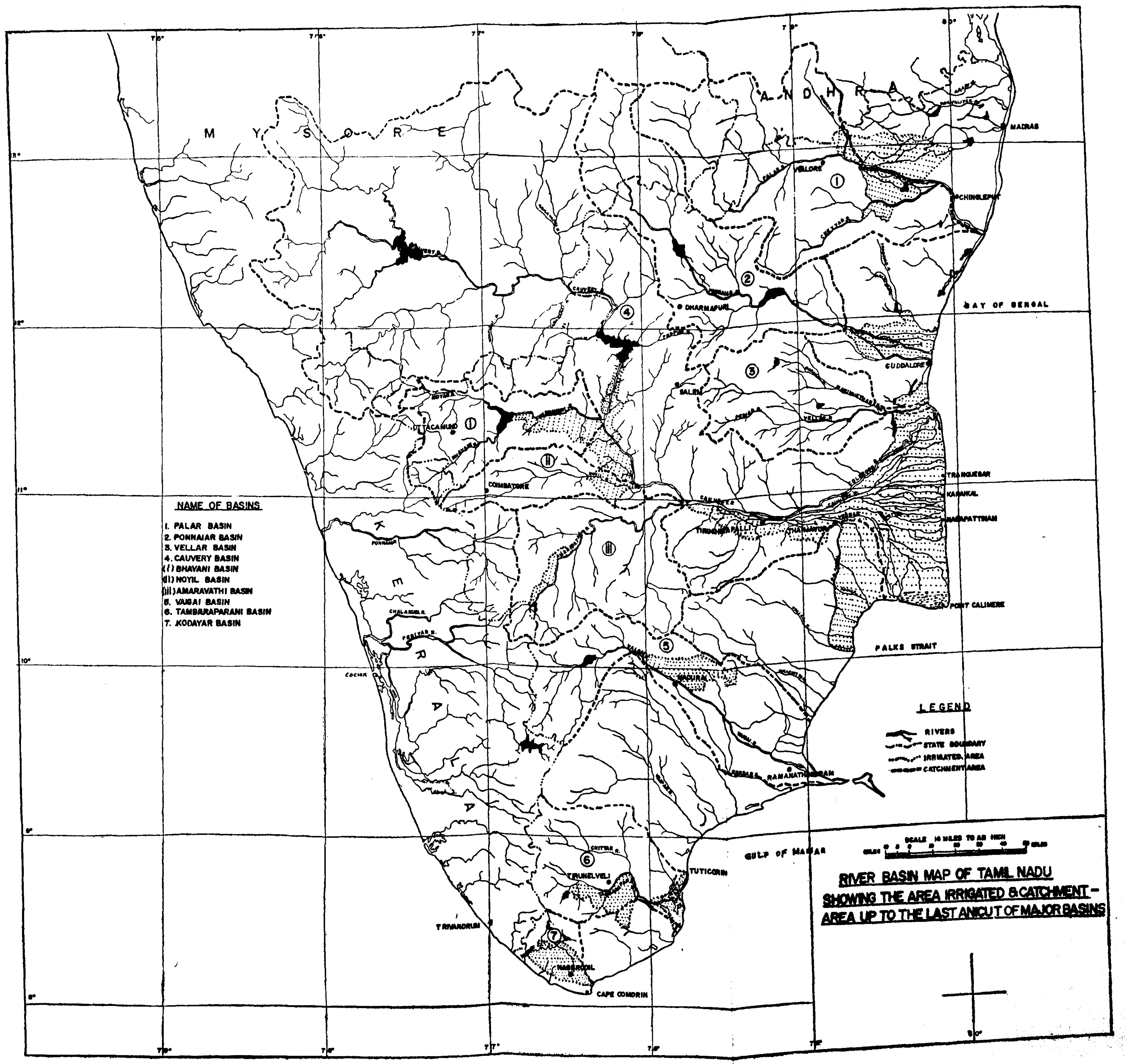
The main works carried out were in the form of providing divide walls and curtain walls, upstream of the head sluices of the left and right side channels and providing stilling basins and curved trough below anicut. The four main channels were also lined with sand cement blocks.

In North Arcot and Chingleput districts where perennial river sources are not in existence, the Palar Anicut System plays an important part in the betterment of irrigation facilities.

A map showing the Palar Basin is appended.

In these few pages, we have attempted to give a few general glimpses of the Palar Basin. In the issues to come we hope to present to the reader more detailed accounts on the following aspects of the basin :—

1. Growth of Irrigation in Palar Basin.
2. Growth of Industries in Palar Basin.
3. Water-supply schemes in Palar Basin.
4. Places of Tourists interest in Palar Basin.
5. Growth of Religion in Palar Basin.
6. Historical battles fought in Palar Basin.



SMALL AND LARGE ORIFICES

By

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SYNOPSIS

This paper presents the theoretical investigations carried out to classify small and large orifices on the basis of errors of discharge computed by the use of a small orifice formula instead of a large orifice formula. It is shown that the orientation of orifices affects these errors of discharge in the smaller ranges of the ratio of head to the height of the orifices and does not significantly affect in the higher ranges. It is also established that the orifices function as large orifices when the ratio of head to the height of the orifices is lesser than unity and as small orifices for larger values of this ratio. A discharge formula has also been presented for large circular orifices which is an extension of the formula derived for the flow through circular weirs by Stevens. The numerical tabular values given for discharge computation by Stevens were found to be incorrect and the correct values are presented in this paper.

INTRODUCTION.

Orifices are well known devices for metering flows. They are invariably classified into small and large orifices. However, very little information is available for the basis of such classification and the ranges of head under which these classifications are valid. The small orifice formula assumes that the average flow velocity across the section of an orifice is equal to the point velocity at the centre of gravity of the section and the discharge is given by the product of this point velocity and the area of the section. This assumption is valid as will be shown later for large depths of flow when compared to the height of the orifice. The large orifice formula explicitely takes into account the velocity variation over the section and the total discharge is computed by integrating the discharges through elementary strips. The classification of small and large orifices may be based on the ratio of the depth of flow to the width, height or the area of the orifice. However a rational parameter will be the one which introduces least error in discharge computations when the small orifice formulae are used instead of large orifice formulae. To compute these errors, for orifices of different shapes, a discharge formula for large circular orifice is lacking.

DISCHARGE FORMULA FOR LARGE CIRCULAR ORIFICE.

For large circular orifice, only an approximate formula exists which is based on a series expansion (1). However, a rigorous formula based on elliptic integrals can be derived following the procedure as adopted by Stevens (2) or deduced simply as below from the discharge formula presented by him for flow through a circular weir. For weir flow, the depth of flow is limited below the top of the circle and the discharge is given by

$$Q=2\int_0^H \sqrt{2gy(d-y)(H-y)}\,dy \quad \ldots\ldots (1)$$

Where Q = the discharge, d = the diameter of the circle, g = the acceleration due to gravity, y = the height of elementary area of the section and dy = the thickness of the elementary area. Equation (1) has been evaluated by Stevens (2) as

$$Q=\frac{4}{15}\sqrt{2g}\,d^{5/2}\,\Delta\left(\frac{H}{d}\right) \ldots\ldots (2)$$

Where

$$\Delta\left(\frac{H}{d}\right)=E\left\{-2+3\left(\frac{H}{d}\right)-\left(\frac{H}{d}\right)^2\right\} -2F\left\{-1+\left(\frac{H}{d}\right)-\left(\frac{H}{d}\right)^2\right\} \ldots\ldots (3)$$

in which

$$E=\int_0^{\frac{\pi}{2}}\left(1-\frac{H}{d}\sin^2\theta\right)^{-\frac{1}{2}} d\theta \text{ and}$$

$$F=\int_0^{\frac{\pi}{2}}\left(1-\frac{H}{d}\sin^2\theta\right)^{\frac{1}{2}} d\theta$$

are the complete elliptic integrals of the first and second kinds respectively. For the case of orifice flow, the depth of flow is maintained above the top of the circular section (Fig. 1) and the discharge is given by

$$Q=2\int_0^d \sqrt{2gy(H-y)(d-y)}\,dy \quad \ldots\ldots (4)$$

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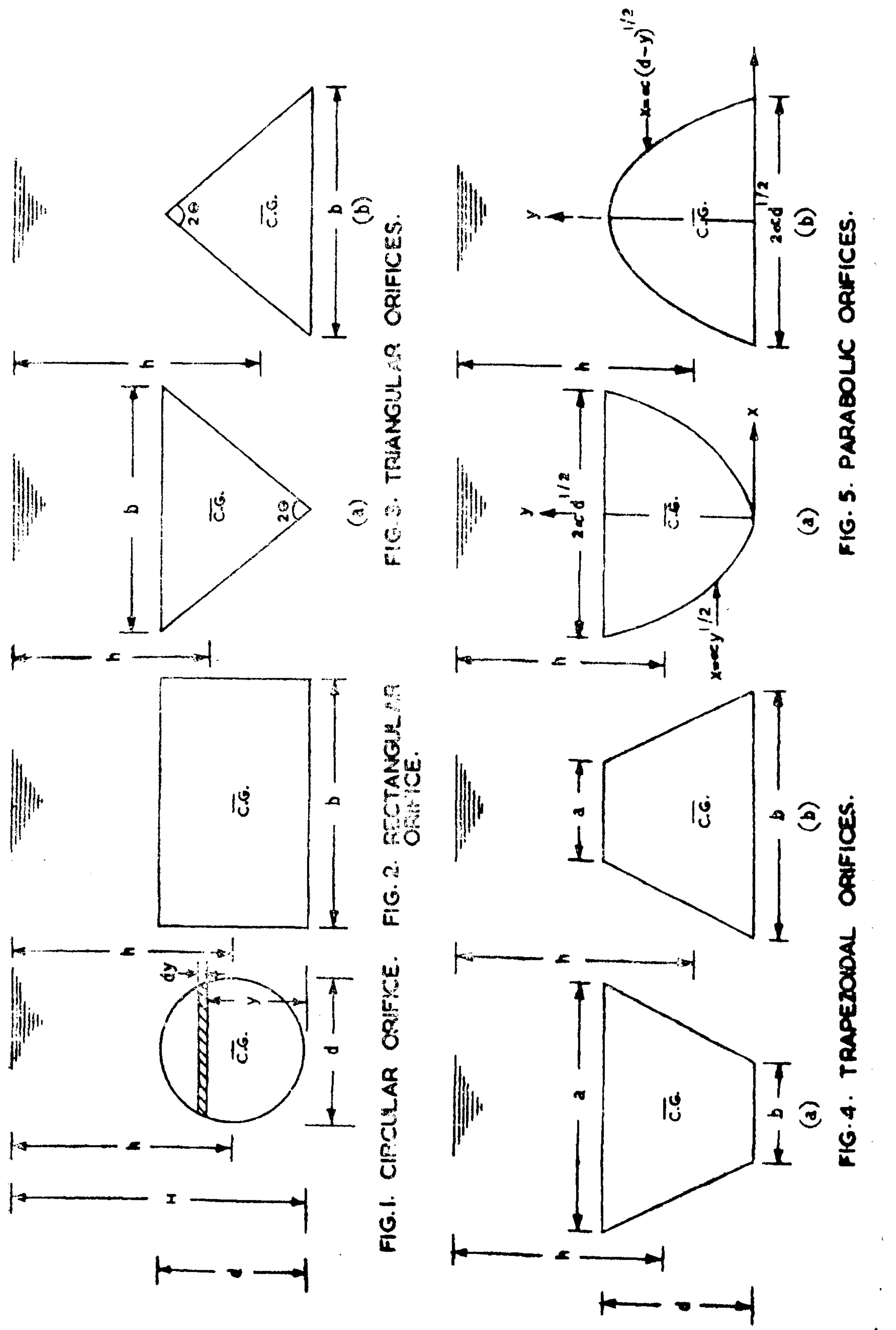

FIG. 1. CIRCULAR ORIFICE. FIG. 2. RECTANGULAR ORIFICE. FIG. 3. TRIANGULAR ORIFICES.

FIG. 4. TRAPEZOIDAL ORIFICES. FIG. 5. PARABOLIC ORIFICES.

comparison of equations (1) and (4) shows that H in equation (1) is replaced by d in equation (4) and vice versa, and hence, equation (4) can be evaluated with the help of equation (2) as

$$Q = \frac{4}{15}\sqrt{2g}\, H^{5/2}\, \Delta\left(\frac{d}{H}\right) \cdots\cdots (5)$$

In equation (5), Δ (d/H) may be obtained from equation (3) by replacing (H/d) with (d/H). To use equation (2) for weir flow, or equation (5) for orifice flow, the values of Δ for different values of H/d or d/H may be evaluated taking the values of E and F from tables on elliptic integrals (3). In either case of the flow, the numerical values of Δ will be the same for different values of (H/d) or (d/H). Stevens (2) has incorrectly tabulated the values of Δ to be used for weir flow computations. Equation (5) when used to compute the discharge through a typical orifice with the tabular values of Δ as given by him yielded a very low value of 0.45 for the co-efficient of discharge for H/d equal to or greater than 30 as against a value of 0.62 obtained for H/d less than 30 and this was the nucleus for noticing the incorrectness of the tabular values. The correct values of Δ computed using an IBM 1620 digital computer are given in the appendix and when these values were used for the particular orifice, a co-efficient of discharge of about 0.62 was obtained throughout the range of (H/d).

ERRORS OF DISCHARGE FOR DIFFERENT ORIFICES.

Since the large orifices formulae are more rigorous than the small orifice formulae, the former ones are taken as standard formulae. The errors of discharge that will be introduced with small orifice formulae are expressed as percentage of discharges computed by the large offices formulae.

The discharge throguh a circular orifice computed with small orifice formula is given by

$$Q = \frac{\pi}{4} d^2 \sqrt{2gh} \cdots\cdots (6)$$

Where h = (H — 0.5d) is the depth of flow measured from the centre of gravity of the section as in Fig. 1. The percentage error E of discharge is then given by

$$E = \left\{ \frac{\frac{4}{15}\sqrt{2g}\, H^{5/2}\, \Delta\left(\frac{d}{h+0.5d}\right) - \frac{\pi}{4} d^2 \sqrt{2gh}}{\frac{4}{15}\sqrt{2g}\, H^{5/2}\, \Delta\left(\frac{d}{h+0.5d}\right)} \right\} 100 \cdots\cdots (7)$$

Equation (7) can be simplified as

$$E = \left\{ 1 - \frac{\frac{15}{16}\pi\sqrt{h/d}}{\left(\frac{h}{d}+0.5\right)^{2.5}\Delta} \right\} 100 \cdots\cdots (8)$$

The percentage errors of discharge for rectangular (Fig. 2), triangular (Fig. 3), trapezoidal (Fig. 4) and parabolic (Fig. 5) orifices are similarly written down as shown in Table 1. In the table, h = the depth of flow measured from the centres of gravity of the different sections, d = the height of the orifices, b = the bottom width and a = the top width of the orifices. The parabolic orifice is defined by the equation,

$x = \alpha y^{1/2}$ where α = a parameter. x = half width, and y = height of any point on the parabola, The following value of h/d = 1/2, 1/3, 2/3 3.5/7.5, 4/7·5, 1/2.5, 1·5/2.5 and 1/2 respectively correspond to the top of the orifices in the cases of rectangular shape, triangular shape with apex at the bottom, with apex at the top, trapezoidal shape with shorter side at the bottom, with shorter side at the top, parabolic shape with the origin at the bottom, with the origin at the top and the circular shape. The percentage errors for trapezoidal orifices are simplified for two sets of typical values of a, b and d. The values of percentage errors, E [column(5),table 1] were computed on the digital computer for different values of (h/d) and they are given in Table 2. The first values of the percentage error shown in Table 2 for the different orifices correspond to the aforementioned values of (h/d) for the top of the orifices. From column (5) of table 1, it may be seen that the percentage errors for different orifices are independent of the area of sections and dependent only on the values of (h/d). As may be seen from the table 2, the percentage errors at smaller values of (h/d) are negative indicating that the small orifice formulae give larger values of discharge for these values of (h/d). However, no such conclusion can be drawn for higher values of (h/d) since the percentage error varies in a random manner. For sections with centres of gravity situated at comparatively larger heights from the bottom, the percentage errors of discharge as seen in the smaller ranges of (h/d) [from columns (4) and (5), (6) and (7), and (8) and (9)] are more for the same depth of flow measured above the top of the orifices.

Thus the orientations of the orifices affect the errors of discharge in the smaller ranges of (h/d) and do not materially affect in the larger ranges. For (h/d) grater than 1, the percentage errors of discharge for the different orifices are less than unity which may be taken as the allowable limit. Hence, it may be concluded that for h/d grater than 1, all orifices behave as small orifices and only for (h/d) less than or equal to 1 they behave as large orifices with the affect of their orientation on discharge being pronounced.

TABLE 1 PERCENTAGE ERRORS INTRODUCED IN DISCHARGE COMPUTATIONS

SL NO	SHAPES OF ORIFICES	SMALL ORIFICE FORMULAE FOR DISCHARGE 'Q'	LARGE ORIFICE FORMULAE DISCHARGE Q	PERCENTAGE OF ERRORS E
1.	RECTANGLE (FIG-2)	$bd\sqrt{2gh}$	$2/3\,b\sqrt{2g}\{(h+0.5d)^{1.5}-(h-0.5d)^{1.5}\}$	$\left\{1-\dfrac{1.5\sqrt{h/d}}{(h/d+0.5)^{1.5}-(h/d-0.5)^{1.5}}\right\}\times 100$
2a.	TRIANGLE WITH APEX DOWN (Fig 3a)	$d^2\tan\theta\times\sqrt{2gh}$	$8/15\tan\theta\sqrt{2g}\left\{(h+2/3d)^{2.5}-2.5d(h-1/3d)^{1.5}-(h-1/3d)^{2.5}\right\}$	$\left\{1-\dfrac{1.875\sqrt{h/d}}{(h/d+2/3)^{2.5}-2.5(h/d-1/3)^{1.5}-(h/d-1/3)^{2.5}}\right\}100$
2b	TRIANGLE WITH APEX UP (Fig 3b)	$d^2\tan\theta\times\sqrt{2gh}$	$8/15\tan\theta\sqrt{2g}\left\{(h-2/3d)^{2.5}+2.5d(h+1/3d)^{1.5}-(h+1/3d)^{2.5}\right\}$	$\left\{1-\dfrac{1.875\sqrt{h/d}}{(h/d-2/3)^{2.5}+2.5(h/d+1/3)^{1.5}-(h/d+1/3)^{2.5}}\right\}100$
3a	TRAPEZIUM WITH SHORTER SIDE DOWN (FIG.4a).	$\dfrac{(a+b)}{2}d\sqrt{2gh}$	$2/3\,b\sqrt{2g}\left\{(h+d/3\,\frac{b+2a}{b+a})^{1.5}-(h-d/3\,\frac{a+2b}{a+b})^{1.5}\right\}+4/15\sqrt{2g}(\frac{a-b}{d})$ $\left\{-2.5d(h-d/3\,\frac{a+2b}{a+b})^{1.5}-(h-d/3\,\frac{a+2b}{a+b})^{2.5}+(h+d/3\,\frac{b+2b}{b+a})^{2.5}\right\}$	$\left\{1-\dfrac{1.875\sqrt{h/d}}{(h/d+\frac{4}{7.5})^{1.5}-1.5(h/d-\frac{3.5}{7.5})^{1.5}-0.2(h/d-\frac{3.5}{7.5})^{2.5}+0.2(h/d+\frac{4}{7.5})^{2.5}}\right\}\times 100$ (WITH b=d AND a=1.5b)

SL No	SHAPES OF ORIFICES	SMALL ORIFICE FORMULAE FOR DISCHARGE 'Q'	LARGE ORIFICE FORMULEA DISCHARGE 'Q'	PERCENTAGE OF ERRORS 'E'
3b	TRAPEZIUM WITH SHORTER SIDE UP (FIG-4b)	$\dfrac{a+b}{2}d\sqrt{2gh}$	$\frac{2}{3}a\sqrt{2g}\left[(h+\frac{d}{3}\,\frac{b+2a}{b+a})^{1.5}-(h-\frac{d}{3}\cdot\frac{a+2b}{a+b})^{1.5}\right]+\frac{4}{15}\cdot\frac{b-a}{d}\sqrt{2g}\times$ $\left\{2.5d(h+\frac{d}{3}\,\frac{b+2a}{b+a})^{1.5}+(h-\frac{d}{3}\cdot\frac{a+2b}{a+b})^{2.5}-(h+\frac{d}{3}\cdot\frac{b+2a}{b+a})^{2.5}\right\}$	$\left\{1-\dfrac{1.875\sqrt{\frac{h}{d}}}{(\frac{h}{d}-\frac{4}{7.5})^{1.5}+(1.5)(\frac{h}{d}+\frac{3.5}{7.5})^{1.5}-(0.2)(\frac{h}{d}+\frac{3.5}{7.5})^{2.5}+(0.2)(\frac{h}{d}-\frac{4}{7.5})^{2.5}}\right\}\times 100$ (with a=d, b = 1.5a)
4a	PARABOLA WITH APEX DOWN (FIG 5a)	$\frac{4}{3}\alpha d^{1.5}\sqrt{2gh}$	$\alpha\sqrt{2g}\left\{(0.7d-0.5h)\sqrt{d}\sqrt{(h-0.4d)}+0.25(h+0.6d)^2\tan^{-1}(\frac{0.7d-5h}{\sqrt{h-0.4d}})+0.125\pi(h+0.6d)^2\right\}$	$\left\{1-\dfrac{\frac{4}{3}\sqrt{\frac{h}{d}}}{(0.7-0.5\frac{h}{d})\times\sqrt{\frac{h}{d}-0.4}+0.25(\frac{h}{d}+0.6)^2\times\tan^{-1}\frac{0.7-0.5(\frac{h}{d})}{\sqrt{\frac{h}{d}+0.6}}+0.125\pi(\frac{h}{d}+0.6)^2}\right\}100$
4b	PARABOLA WITH APEX UP (FIG 5b)	$\frac{4}{3}\alpha d^{1.5}\sqrt{2gh}$	$\alpha\sqrt{2g}\left\{(0.7d+0.5h)\sqrt{d}\sqrt{h+0.4d}-0.5(h-0.6d)^2\tanh h^{-1}\sqrt{\frac{d}{h+0.4d}}\right\}$	$\left\{1-\dfrac{\frac{4}{3}\sqrt{\frac{h}{d}}}{(0.7+0.5\frac{h}{d})\sqrt{\frac{h}{d}+0.4}-0.5(\frac{h}{d}-0.6)^2\times\tan h^{-1}\frac{1}{\sqrt{\frac{h}{d}+0.4}}}\right\}100$
5	CIRCLE (FIG-1)	$\frac{\pi}{4}d^2\sqrt{2gh}$	$\frac{4}{15}\sqrt{2g}\;d^{2.5}\,\Delta(\frac{d}{h+0.6d})$	$\left\{1-\dfrac{(\frac{15}{16})\pi\sqrt{\frac{h}{d}}}{(\frac{h}{d}+0.5)^{2.5}\Delta}\right\}100$

CONCLUSIONS.

1. A discharge formula for large circular orifices has been deduced.

2. The basis as to when exactly an orifice can be classified as small or large has been theoretically established.

3. The orifices of orientations having centres of gravity at comparatively smaller heights from the bottom introduce lesser errors of discharge in large orifice range.

APPENDIX 1.

To get the computerised solution for the values of $\triangle$ (d/H) from equation (3), the complete elliptic integrals of the first kind, E and the second kind F can be written as polynomials as follows (4).

$$E = (a_0 + a_1 k_1 + a_2 k_1^2 + a_3 k_1^3 + a_4 k_1^4) + (b_0 + b_1 k_1 + b_2 k_1^2 + b_3 k_1^3 + b_4 k_1^4) \ln (1/k_1)$$

Where, $k_1 = (1 - d/H)$, $a_0 = 1{\cdot}38629\ 436112$,

$a_1 = 0{\cdot}09666\ 344259$, $a_2 = 0{\cdot}03590\ 092383$,

$a_3 = 0{\cdot}03742\ 563713$, $a_4 = 0{\cdot}01451\ 196212$,

$b_0 = 0{\cdot}5$, $b_1 = 0{\cdot}12498\ 593597$,
$b_2 = 0{\cdot}06880\ 248576$

$b_3 = 0{\cdot}03328\ 355346$, $b_4 = 0{\cdot}00441\ 787012$

and $$F = (1 + a_1 k_1 + a_2 k_1^2 + a_3 k_1^3 + a_4 k_1^4) + (b_1 k_1 + b_2 k_1^2 + b_3 k_1^3 + b_4 k_1^4) \ln (1/k_1)$$

Where, $a_1 = 0{\cdot}44325\ 141463$,
$a_2 = 0{\cdot}06260\ 601220$,

$a_3 = 0{\cdot}04757\ 383546$, $a_4 = 0{\cdot}01736\ 506451$,

$b_1 = 0{\cdot}24998\ 368310$, $b_2 = 0{\cdot}09200\ 180037$

$b_3 = 0{\cdot}04069\ 697526$, $b_4 = 0{\cdot}00526\ 449639$

These polynomials for E and F introduce in their computations as small errors as less than or equal to 2×10^{-8}. The values of $\triangle$ computed using these polynomials are given in Appendix 2.

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NOTATIONS.

Top width of the orifice	a
Bottom width of the orifice	b
Height of the orifice	d
Elliptic integrals	E, F
Acceleration due to gravity	g
Depth of flow from the bottom of the orifice.	H
Depth of flow from the centre of gravity of the orifice.	h
Half width of the orifice at any heighty ..	x
Discharge through the orifice ..	Q
Discharge function for circular orifice ..	$\triangle$
Parameter for the profile of parabolic orifice	α
Semi apex angle of triangular orifice ..	θ

TABLE 2—PERCENTAGE ERRORS OF DISCHARGE.

Serial number.	$\frac{h}{d}$	Rectangular Orifice. (Fig. 2).	Triangular Orifice. Apex at the bottom. (Fig. 3a)	Triangular Orifice. Apex at the top. (Fig. 3b).	Trapezoidal orifice. Shorter side at the bottom (Fig. 4a).	Trapezoidal orifice. Shorter side at the top. (Fig. 4b).	Parabolic orifice. Origin at the bottom (Fig. 5a).	Parabolic orifice. Origin at the top. (Fig. 5b).	Circular orifice. (Fig. 1).
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	0·4	..	− 4·85830	..	..	..	− 7·36893	..	..
2	0·5	− 6·06601	− 2·87889	..	− 7·28168	..	− 3·90953	..	− 4·13095
3	0·6	− 3·45952	− 1·93810	..	− 3·70576	− 3·32168	− 2·55409	− 3·27955	− 2·55025
4	0·7	− 2·44051	− 1·40272	− 1·82249	− 2·50446	− 2·31688	− 1·82041	− 2·16502	− 1·78071
5	0·8	− 1·80053	− 1·06558	− 1·30731	− 1·83268	− 1·72233	− 1·36991	− 1·57307	− 1·32420
6	0·9	− 1·38970	− 0·83841	− 0·99634	− 1·40743	− 1·33579	− 1·07096	− 1·20321	− 1·02698
7	1·0	− 1·10788	− 0·67762	− 0·78744	− 1·11810	− 1·06857	− 0·86154	− 0·95323	− 0·82181
8	2·0	− 0·26424	− 0·16980	− 0·18177	− 0·26339	− 0·25807	− 0·21221	− 0·22213	− 0·19764
9	3·0	− 0·11649	− 0·07599	− 0·07922	− 0·11569	− 0·11416	− 0·05438	− 0·09721	− 0·08723
10	4·0	− 0·06536	− 0·04265	− 0·04414	− 0·06478	− 0·06420	− 0·05315	− 0·05434	− 0·04896
11	5·0	− 0·04175	− 0·02751	− 0·02827	− 0·04138	− 0·04100	− 0·03408	− 0·03460	− 0·03076
12	6·0	− 0·02900	− 0·01929	− 0·01922	− 0·02868	− 0·02861	0·02780	− 0·02397	− 0·02134
13	7·0	− 0·02121	− 0·01377	− 0·01398	− 0·02109	− 0·02031	− 0·01743	− 0·01755	0·01531
14	8·0	− 0·01636	− 0·01075	− 0·01132	− 0·01609	− 0·01615	− 0·01332	− 0·01338	− 0·01156
15	9·0	− 0·01290	− 0·00870	− 0·00905	− 0·01273	− 0·01257	− 0·01060	− 0·01058	− 0·00901
16	10·0	− 0·01035	− 0·00793	− 0·00590	− 0·01022	− 0·01024	− 0·00856	− 0·00846	− 0·00541
17	15·0	− 0·00459	− 0·00459	− 0·00087	− 0·00459	− 0·00500	0·00380	− 0·00698	− 0·00188
18	20·0	− 0·00264	− 0·00184	− 0·00064	− 0·00207	0·00398	− 0·00213	0·00184	− 0·00699
19	25·0	− 0·00173	− 0·00426	− 0·00319	− 0·00053	− 0·00277	− 0·00129	− 0·00108	0·01169
20	30·0	− 0·00118	− 0·00094	− 0·00386	0·00004	− 0·00132	0·00077	− 0·00019	0·01533
21	35·0	− 0·00089	− 0·00945	0·01669	− 0·00044	0·00046	0·00083	0·00018	0·00494
22	40·0	− 0·00087	− 0·01805	0·00388	0·00220	− 0·00455	− 0·00083	0·00084	0·01748
23	45·0	− 0·00024	− 0·00700	0·01685	− 0·00064	0·00175	− 0·00023	0·00077	0·00618
24	50·0	− 0·00048	− 0·00189	− 0·02452	0·00037	− 0·00415	− 0·00015	0·00075	0·04761
25	60·0	− 0·00059	− 0·03916	0·18281	− 0·00747	0·01869	− 0·00052	0·00132	0·00306
26	70·0	− 0·00008	− 0·02151	0·03585	− 0·00557	0·00526	0·00031	0·00165	0·07281
27	80·0	− 0·00094	− 0·15233	− 0·11646	0·02678	− 0·02092	− 0·00079	0·00223	0·11430
28	90·0	− 0·00040	0·11899	− 0·17350	0·02355	0·02355	0·00191	0·00396	0·10892
29	100·0	− 0·00065	− 0·35860	0·47773	0·05865	0·01601	− 0·00174	0·00434	0·04302

APPENDIX 2.

Values of $\triangle$ for different Values of $k = \frac{d}{H}\left(\text{or } \frac{H}{d}\right)$

k	0·000	0·001	0·002	0·003	0·004	0·005	0·006	0·007	0·008	0·009
0·00	0·00000	0·00000	0·00001	0·00003	0·00005	0·00007	0·00011	0·00014	0·00019	0·00024
0·01	0·00029	0·00036	0·00042	0·00050	0·00057	0·00066	0·00075	0·00085	0·00095	0·00106
0·02	0·00117	0·00129	0·00142	0·00155	0·00169	0·00183	0·00198	0·00213	0·00229	0·00245
0·03	0·00263	0·00280	0·00299	0·00318	0·00337	0·00358	0·00378	0·00399	0·00421	0·00444
0·04	0·00467	0·00490	0·00514	0·00539	0·00564	0·00590	0·00616	0·00643	0·00670	0·00698
0·05	0·00727	0·00756	0·00786	0·00816	0·00847	0·00879	0·00911	0·00943	0·00976	0·01010
0·06	0·01044	0·01079	0·01114	0·01150	0·01186	0·01224	0·01262	0·01300	9·01338	0·01378
0·07	0·01417	0·01458	0·01499	0·01541	0·01583	0·01625	0·01668	0·01712	0·01756	0·01801
0·08	0·01847	0·01892	0·01939	0·01986	0·02034	0·02082	0·02131	0·02180	0·02230	0·02280
0·09	0·02331	0·02383	0·02435	0·02487	0·02540	0·02594	0·02648	0·02702	0·02758	0·02814
0·10	0·02870	0·02927	0·02985	0·03043	0·03101	0·03160	0·03220	0·03280	0·03341	0·03402
0·11	0·03464	0·03526	0·03589	0·03653	0·03716	0·03781	0·03846	0·03912	0·03978	0·04044
0·12	0·04111	0·04179	0·04247	0·04316	0·04385	0·04455	0·04526	0·04596	0·04668	0·04740
0·13	0·04812	0·04885	0·04959	0·05033	0·05107	0·05182	0·05258	0·05334	0·05411	0·05488
0·14	0·05566	0·05644	0·05723	0·05802	0·05882	0·05963	0·06043	0·06125	0·06207	0·06289
0·15	0·06372	0·06456	0·06540	0·06624	0·06709	0·06795	0·06881	0·06967	0·07054	0·07142
0·16	0·07230	0·07319	0·07408	0·07498	0·07588	0·07679	0·07770	0·07862	0·07954	0·08046
0·17	0·08140	0·08234	0·08328	0·08423	0·08518	0·08614	0·08710	0·08807	0·08904	0·09002
0·18	0·09100	0·09199	0·09298	0·09398	0·09499	0·09599	0·09700	0·09803	0·09905	0·10008
0·19	0·10111	0·10215	0·10319	0·10424	0·105296	0·10635	0·10742	0·10849	0·10956	0·11064
0·20	0·11172	0·11281	0·11390	0·11500	0·11610	0·11721	0·11832	0·11944	0·12056	0·12169
0·21	0·12282	0·12396	0·12510	0·12625	0·12740	0·12856	0·12972	0·13089	0·13206	0·13323
0·22	0·13442	0·13560	0·13679	0·13799	0·13919	0·14040	0·14160	0·14282	0·14404	0·14526
0·23	0·14649	0·14773	0·14897	0·15021	0·15146	0·15271	0·15397	0·15523	0·15650	0·15777
0·24	0·15905	0·16033	0·16162	0·16291	0·16420	0·16551	0·16681	0·16812	0·16943	0·17075
0·25	0·17208	0·17341	0·17474	0·17608	0·17742	0·17877	0·18012	0·18148	0·18284	0·18421
0·26	0·18558	0·18695	0·18833	0·18972	0·19111	0·19250	0·19390	0·19530	0·19671	0·19812
0·27	0·19954	0·20096	0·20238	0·20382	0·20525	0·20669	0·20813	0·20958	0·21104	0·21249

APPENDIX 2.

Values of $\triangle$ for different Values of $k = \frac{d}{H}$ (or $\frac{H}{d}$)

k	0·000	0·001	0·002	0·003	0·004	0·005	0·006	0·007	0·008	0·009
0·28	0·21396	0·21542	0·21689	0·21837	0·21985	0·22134	0·22283	0·22432	0·22582	0·22732
0·29	0·22882	0·23034	0·23186	0·23337	0·23490	0·23643	0·23796	0·23950	0·24105	0·24259
0·30	0·24414	0·24570	0·24726	0·24883	0·25040	0·25197	0·25355	0·25513	0·25672	0·25831
0·31	0·25990	0·26150	0·26311	0·26472	0·26633	0·26795	0·26957	0·27120	0·27283	0·27446
0·32	0·27610	0·27774	0·27939	0·28104	0·28270	0·28436	0·28602	0·28769	0·28937	0·29104
0·33	0·29272	0·29441	0·29610	0·29779	0·29949	0·30120	0·30290	0·30461	0·30633	0·30805
0·34	0·30977	0·31150	0·31323	0·31497	0·31671	0·31846	0·32020	0·32196	0·32372	0·32548
0·35	0·32724	0·32901	0·33079	0·33256	0·33434	0·33613	0·33792	0·33971	0·34151	0·34332
0·36	0·34512	0·34693	0·34875	0·35057	0·35239	0·35422	0·35605	0·35788	0·35972	0·36156
0·37	0·36341	0·36526	0·36712	0·36898	0·37084	0·37271	0·37458	0·37645	0·37833	0·38021
0·38	0·38210	0·38399	0·42460	0·38778	0·38969	0·39159	0·39350	0·39542	0·39734	0·39926
0·39	0·40118	0·40311	0·44453	0·40699	0·40893	0·41087	0·41282	0·41478	0·41673	0·41869
0·40	0·42066	0·42263	0·42460	0·42657	0·42855	0·43054	0·43253	0·43452	0·43651	0·43851
0·41	0·44051	0·44252	0·44453	0·44654	0·44856	0·45058	0·45261	0·45464	0·45667	0·45871
0·42	0·46075	0·46279	0·46484	0·46689	0·46894	0·47100	0·47307	0·47513	0·47720	0·47927
0·43	0·48135	0·48343	0·48552	0·48760	0·48969	0·49179	0·49389	0·49599	0·49810	0·50021
0·44	0·50232	0·50443	0·50656	0·50868	0·51081	0·51294	0·51507	0·51721	0·51935	0·52150
0·45	0·52364	0·52580	0·52795	0·53011	0·53227	0·53444	0·53661	0·53878	0·54096	0·54314
0·46	0·54532	0·54751	0·54970	0·55189	0·55409	0·55629	0·55849	0·56070	0·56291	0·56513
0·47	0·56734	0·56956	0·57179	0·57402	0·57625	0·57848	0·58072	0·58296	0·58520	0·58745
0·48	0·58970	0·59196	0·59421	0·59648	0·59874	0·60100	0·60328	0·60555	0·60782	0·61011
0·49	0·61239	0·61468	0·61697	0·61926	0·62156	0·62386	0·62616	0·62847	0·63078	0·63309
0·50	0·63541	0·63773	0·64005	0·64238	0·64471	0·64704	0·64937	0·65171	0·65405	0·65640
0·51	0·65874	0·66109	0·66345	0·66580	0·66816	0·67053	0·67289	0·67526	0·67763	0·68001
0·52	0·68239	0·68477	0·68715	0·68954	0·69193	0·69432	0·69672	0·69912	0·70152	0·70393
0·53	0·70634	0·70875	0·71116	0·71358	0·71600	0·71842	0·72085	0·72328	0·72571	0·72814
0·54	0·73058	0·73302	0·73546	0·73791	0·74036	0·74281	0·74527	0·74772	0·75018	0·75265
0·55	0·75511	0·75758	0·76005	0·76253	0·76501	0·76749	0·76997	0·77245	0·77494	0·77743

0·56	0·77992	0·78242	0·78492	0·78743	0·78993	0·79244	0·79495	0·79746	0·79998	0·80250
0·57	0·80502	0·80754	0·81007	0·81260	0·81513	0·81766	0·82020	0·82274	0·82528	0·82783
0·58	0·83037	0·83292	0·83548	0·83803	0·84059	0·84315	0·84571	0·84828	0·85084	0·85341
0·59	0·85599	0·85856	0·86114	0·86372	0·86630	0·86889	0·87148	0·87407	0·87666	0·87926
0·60	0·88185	0·88445	0·88705	0·88966	0·89227	0·89488	0·89749	0·90010	0·90272	0·90534
0·61	0·90796	0·91058	0·91321	0·91584	0·91847	0·92110	0·92374	0·92638	0·92902	0·93166
0·62	0·93430	0·93695	0·93960	0·94225	0·94490	0·94756	0·95022	0·95288	0·95554	0·95820
0·63	0·96087	0·96354	0·96621	0·96888	0·97156	0·97424	0·97692	0·97960	0·98228	0·98497
0·64	0·98766	0·99035	0·99304	0·99573	0·99843	1·00113	1·00383	1·00653	1·00924	1·01194
0·65	1·01465	1·01736	1·02008	1·02279	1·02551	1·02823	1·03095	1·03367	1·03639	1·03912
0·66	1·04185	1·04458	1·04731	1·05004	1·05278	1·05552	1·05826	1·06100	1·06374	1·06640
0·67	1·06923	1·07198	1·07473	1·07749	1·08024	1·08300	1·08575	1·0885	1·09128	1·09404
0·68	1·09680	1·09957	1·10234	1·10511	1·10788	1·11065	1·11343	1·11620	1·11898	1·12176
0·69	1·12454	1·12733	1·13011	1·13290	1·13569	1·13815	1·14127	1·14406	1·14685	1·14965
0·70	1·15245	1·15525	1·15805	1·16085	1·16365	1·16646	1·16926	1·17207	1·17488	1·17769
0·71	1·18050	1·18332	1·18613	1·18895	1·19177	1·19459	1·19741	1·20023	1·20305	1·20588
0·72	1·20870	1·21153	1·21436	1·21719	1·22002	1·22285	1·22569	1·22852	1·23136	1·23420
0·73	1·23704	1·23988	1·24272	1·24556	1·24840	1·25125	1·25409	1·25694	1·25979	1·26264
0·74	1·26549	1·26834	1·27119	1·27405	1·27690	1·27976	1·28262	1·28547	1·28833	1·29119
0·75	1·29405	1·29692	1·29978	1·30264	1·30551	1·30838	1·31124	1·31411	1·31698	1·31985
0·76	1·32272	1·32559	1·32846	1·33134	1·33421	1·33708	1·33996	1·34284	1·34571	1·34859
0·77	1·35147	1·35435	1·35723	1·36011	1·36299	1·36588	1·36876	1·37164	1·37453	1·37741
0·78	1·38030	1·38319	1·38607	1·38896	1·39185	1·39474	1·39763	1·40052	1·40341	1·40630
0·79	1·40920	1·41208	1·41498	1·41787	1·42076	1·42366	1·42655	1·42945	1·43234	1·43524
0·80	1·43813	1·44103	1·44393	1·44682	1·44972	1·45262	1·45552	1·45842	1·46132	1·46421
0·81	1·46711	1·47001	1·47291	1·47581	1·47871	1·48161	1·48451	1·48741	1·49031	1·49322
0·82	1·49612	1·49902	1·50192	1·50482	1·50772	1·51062	1·51352	1·51643	1·51933	1·52223
0·83	1·52513	1·52603	1·53093	1·53383	1·53673	1·53963	1·54253	1·54543	1·54834	1·55124
0·84	1·55414	1·55704	1·55993	1·56283	1·56573	1·56868	1·57153	1·57443	1·57733	1·58022
0·85	1·58312	1·58602	1·58891	1·59181	1·59470	1·59760	1·60049	1·60339	1·60228	1·60917
0·86	1·61207	1·61496	1·61785	1·62074	1·62363	1·62652	1·62941	1·63230	1·63518	1·63807
0·87	1·64096	1·64384	1·64673	1·64961	1·65249	1·65587	1·65826	1·66114	1·66402	1·66689

APPENDIX 2.

Values of $\triangle$ for different Values of $k = \frac{d}{H}\left(\text{or } \frac{H}{d}\right)$

k	0·000	0·001	0·002	0·003	0·004	0·005	0·006	0·007	0·008	0·009
0·88	1·66977	1·67265	1·67552	1·67840	1·68127	1·68415	1.68702	1·68989	1·69276	1·69563
0·89	1·69849	1·70136	1·70422	1·70709	1·70995	1·71281	1·71567	1·71853	1·72139	1·72424
0·90	1·72710	1·72995	1·73280	1·73566	1·73850	1·74135	1·74420	1·74704	1·74989	1·75273
0·91	1·75557	1·75841	1·76124	1·76408	1·76691	1·76974	1·77257	1·77540	1·77822	1·78105
0·92	1·78387	1·78669	1·78951	1·79233	1·79514	1·79795	1·80076	1·80357	1·80638	1·80918
0·93	1·81198	1·81478	1·81758	1·82038	1·82317	1·82596	1·82875	1·83153	1·83432	1·83710
0·94	1·83988	1·84265	1·84543	1·84820	1·85096	1·85373	1·85649	1·85925	1·86201	1·86476
0·95	1·86751	1·87026	1·87301	1·87575	1·87849	1·88122	1·88396	1·88669	1·88941	1·89214
0·96	1·89486	1·89757	1·90028	1·90299	1·90570	1·90840	1·91110	1·91379	1·91648	1·91917
0·97	1·92185	1·92453	1·92720	1·92988	1·93254	1·93520	1·93786	1·94051	1·94316	1·94580
0·98	1·94844	1·95108	1·95371	1·95633	1·95895	1·96156	1·96417	1·96677	1·96937	1·97196
0·99	1·97454	1·97712	1·97969	1.98226	1·98482	1·98737	1·98991	1·99245	1·99498	1·99749
1·00	2·00000									

NOTE ON TECHNIQUES OF HYDRAULIC MODEL STUDIES

By Thiru P. Kumaraswamy*

1.0 *Introduction*—

1.1. Hydraulics is yet to become an exact science and its methodology is necessarily empirical especially in the fields of river and coastal hydraulics. Rigorous theories of flow of water and of other phenomena to be useful in field application do not exist. Most of the designs relating to hydraulic structures are based on a few basic laws, and a large amount of judgement gained by field experience. In this context the hydraulic model has filled up the knowledge gap to some extent. Nowhere in the world to day is a major hydraulic structure designed and built without prior analysis through model experiments.

1.2. The hydraulic laboratory is primarily a service organisation and most of the work is done at the request of the design department. Preliminary designs are usually submitted to the laboratory with the request that specified features be studied from the standpoint 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.

1.3. Model studies are conducted according to established rules of hydraulic similitude whereby there is a definite known relationship between performance of the model and that of the corresponding field structure. Comparison of model and prototype has clearly demonstrated that, with few exceptions, there is a correspondence of behaviour within and usually well beyond, expected limitations. In many cases agreement between model and prototype performance has exceeded expectations. In some cases where agreement at first appeared to be lacking, it was found that failure to recognise or interpret model results correctly was cause for the disagreement . Correspondence between the model and prototype has been especially complete for overfall spillway crests, valves, gates, outlet features, and energy dissipators. It is customary to provide calibration curves based on model results in lieu of field calibration. Energy dissipators, including stilling basins and buckets of various types, designed on the basis of model findings, have been successfully operated in substantial agreement with model indications. River improvement plans of tremendous magnitude have worked out successfully according to predictions based on model tests. The high efficiencies and smooth operating characteristics of the large modern turbines and pumps can also be attributed to model experiments. In practically all cases it will be found that improvements indicated by models have been substantiated when the prototype structures have been constructed.

1.4. This note stresses the necessity for understanding the view point of the designer of hydraulic structures and equipment, and thus gives prominence to the conviction that the laboratory worker must appreciate the criteria, and limitation which affect the designer's work. Familiarity with design considerations is essential also to the proper conduct of field studies. Field studies are considered to be an extension of the Hydraulic Laboratory investigations to the prototype level.

2.0. *Principles of Hydraulic Similitude*—

2.1. *Laws of Similitude*—

2.1.1. Hydraulic Laboratory practice is based on the application of a set of relationships which are known as the laws of hydraulic similitude. These laws, which are developed from the basic relations of fluid mechanics, express the inter relations of the various fluid parameters such as velocity, pressure, shear, etc., under similar boundary conditions. A knowledge of hydraulic similitude is essential to an enlightened use of hydraulic models.

2.1.2. Similitude, as applied to hydraulic models, goes considerably beyond the superficial aspects of geometric similarity with which it is sometimes erroneously identified. Similitude canbe defined as a known and usually limited correspondence between the behaviour of a model and that of its prototype, with or without geometric similarity. The correspondence is usually limited because it is impracticable to construct and operate a model in which all the conditions required for complete similitude (geometric, kinematic and dynamic similarity) are satisfied, eventhough these conditions are known . The term " similitude " should be qualified to indicate the general limits of correspondence, or, one might speak of several similitudes, each of which has a definite set up limitations.

2.1.3. The application of similitude in hydraulic model testing is based on recognition of the fact that although complete similitude is impracticable, there are several imperfect similitudes which can be exploited as required. Experience has demonstrated that any given problem can be simplified into the interplay of two major forces. It is therefore possible to develop the pertinent similitude by theoretical means. Each similitude consists of a set of transference ratios which may be applied to model findings in predicting prototype behaviour.

2.1.4. In addition to similitudes having an analytical basis, there are others which are developed, by experiment with the model, for application in special cases, such as the prediction of changes in bed configuration in erodible channels. Although attempts are being made to develop a more rational basis for transference of experimental results in such cases, the current procedure is based primarily on empirical relations. In other cases, the only basis for predicting prototype behaviour from model experiments is one compounded of experience and hydraulic sense. The techniques

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which have been mastered by engineers experienced in this field permit them, by adjusting and applying model results in a nonformalized manner, to arrive at reasonably accurate predictions of prototype action. In general, the value of a model depends on the degree of accuracy with which it demonstrates that phase of the behaviour of its prototype being investigated.

2.2. *Geometric Similarity.*

2.2.1. Two objects or systems are geometrically similar if the ratios of all corresponding linear dimensions are equal. This is independent of motion of any kind and involves only similarity in form. Similarity of length can be expressed as follows :

$$\frac{L_m}{L_p} = L_r$$

Where L_m and L_p are corresponding linear dimensions in model and prototype, respectively, and L_r is the length ratio or simply scale ratio.

Then

$$\text{Area ratio } A_r = \frac{L_m^2}{L_p^2} = L_r^2$$

$$\text{Volume ratio } V_r = \frac{L_m^3}{L_p^3} = L_r^3$$

and the hydraulic radius ratio

$$R_r = \frac{A_m P_p}{P_m A_p} = \frac{L_m^2 L_p}{L_m L_p^2} = \frac{L_m}{L_p} = L_r$$

where P represents wetted perimeter. The scale ratio will then read 1/50, 1/100, etc.

2.3. *Kinematic Similarity and Dynamic Similarity.*—

2.3.1. Kinematic similarity is similarity of motion. When the ratios of the components of velocity at homologous points in two related or geometrically similar systems are equal, the two states of motion are kinematically similar, and the paths of homologous particles will also be geometrically similar.

2.3.2. Dynamic similarity between two geometrically and kinematically similar systems requires that the ratios of all homologous forces (including the force of inertia) in the two systems be the same. This follows Newton's second law of motion which can be written :

$$Ma = \text{Vector sum } F_p + F_g + F_v + F_t + F_e \quad \ldots\ (1)$$

where

Ma = Mass reaction to the forces acting, considered herein as the inertial force.

F_p = pressure force connected with, or resulting from, the motion.

F_g = force imposed on the liquid mass by gravity, as represented by its weight.

F_v = viscous shear force.

F_t = force due to surface tension, and

F_e = forces produced by elastic compression of the fluid.

For over-all similarity in two fluid motion occurrences, the ratio of the inertia forces, model to prototype, must equal the ratio of the vector sum of the active forces, model to prototype.

Or

$$\frac{M_m a_m}{M_p a_p} = \frac{(F_p+F_g+F_v+F_t+F_e)_m}{(F_p+F_g+F_v+F_t+F_e)_p} \quad \ldots\ (1a).$$

where the subscripts m and p represent model and prototype, respectively. Perfect similitude requires in addition that

$$\frac{M_m a_m}{M_p a_p} = \frac{(F_p)_m}{(F_p)_p} = \frac{(F_g)_m}{(F_g)_p} =$$

$$= \frac{(F_v)_m}{(F_v)_p} = \frac{(F_t)_m}{(F_t)_p} = \frac{(F_e)_m}{(F_e)_p} \quad \ldots\ (1b)$$

All but one of these ratios may be regarded as independent, that one being fully determined once the others are established. The pressure ratio is usually regarded as the dependent quantity, and therefore it does not play a controlling part in the following discussion of similitude techniques.

2.3.3. No model fluid is known which has the requisite viscosity, surface tension, and elastic modulus to satisfy the conditions of Equation (1b.) Moreover, hidden in the term for viscous forces is the effect of boundary roughness which is equally difficult to adjust for complete similitude.

2.3.4. Experience has shown that it is not so difficult to apply Equation (1a,) as it would at first appear, because one or more of the forces may not act in the flow occurrence in question, while some may act only to a negligible amount or may be related to the most prominent force. In fact, for all practical purposes, a particular state of fluid motion can

usually be simulated in the model by considering only one of the forces on the right of Equation (1a.) In at least 90 percent of all hydraulic model studies, the forces connected with surface tension and elastic compression are relatively small and can be neglected safely. From a practical standpoint, a particular fluid motion can be represented in a model by considering that either gravity forces or viscous forces predominate.

2.4. *The Froude Law.*

2.4.1. When gravitational effects predominate, a practical basis for similitude can be established by equating the ratio of gravitational forces to that of inertia forces and neglecting the other forces in Equation (1a.). Before doing so, the inertia force of Equation (1) will be expressed in dimensional terms

$$\text{Since mass} = \frac{W}{g} \text{ or } \frac{\gamma L^3}{g}$$

where γ is the specific weight of the liquid, and

$$\text{acceleration} = \frac{L}{t^2},$$

$$Ma = \frac{\gamma L^3 L}{g t^2} = \frac{\gamma L^4}{g t^2}$$

Substituting ρ for $\frac{\gamma}{g}$ and V for $\frac{L}{t}$ $Ma = \rho L^2 V^2$

Where ρ is density of fluid, L is length, and V is velocity.

Dimensionally, the gravity force is $F_g = \gamma L^3$

Considering only the gravity force on the right of Equation (1).

$$Ma = F_g = \gamma L^3 \text{ or } \rho L^2 V^2 = \gamma L^3$$

Substituting ρg for γ in the right-hand member

$\rho L^2 V^2 = \rho g L^3$ Expressing this in model prototype ratios as suggested in Equation. (1a) :

$$\frac{L_m^2 V_m^2}{L_p^2 V_p^2} = \frac{g_m L_m^3}{g_p L_p^3}$$

$$\text{or } L_r^2 V_r^2 = g_r L_r^3$$

$$\text{and } \frac{V_r^2}{g_r L_r} = 1, \text{ or } \frac{V_r}{\sqrt{g_r L_r}} = 1 \ldots (2)$$

The expression $\frac{V}{\sqrt{gL}}$ applied to either the model or the prototype, is known as the Froude number. The equality of the number in model and prototype as expressed by Equation (2) is known as the Froude law.

2.5. *The Reynolds Law.*

2.5.1. When viscous forces predominate, the basis for similitude is obtained by equating the ratio of viscous forces to that of inertia forces and neglecting the other forces in Equation (1a.)

By definition $F_v = \mu L V$ Where μ is the coefficient of viscosity of the fluid.

From Equation (1a) $Ma = \mu L V$.

Expressing the above in model-prototype ratios,

$$\rho_r L_r^2 V_r^2 = \mu_r L_r V_r$$

and

$$\frac{\rho_r L_r V_r}{\mu_r} = 1 \ldots\ (3)$$

which is usually written

$$\frac{L_r V_r}{\nu_r} = 1 \quad \ldots\ (3a)$$

Where ν is the kinematic Viscosity defined as μ/ρ

The Reynolds number is simply $\frac{LV}{\nu}$ Equation (3a) states that for complete similarity with respect to viscous effects the Ryenolds numbers should be equal in model and prototype.

2.6. *The Cauchy or Mach Law.*

2.6.1. In those special cases where either surface tension forces or forces resulting from elastic compression predominate, the basis for similitude is obtained in a similar manner by equating the ratio of surface tension forces or the ratio of elastic forces to the ratio of inertial forces. In the case of surface tension forces, one obtains the Weber law.

$$\frac{\rho_r L_r V_r^2}{\sigma_r} = 1 \ldots (4)$$

Where σ

is a measure of surface tension. In the case of elastic forces one obtains the Cauchy or, as it is sometimes known, the Mach law :

$$\frac{\rho_r V_r}{E_r} = 1 \quad \ldots\ldots (5)$$

where E is the bulk modulus of the fluid. An alternative form of this expression is obtained by introducing the celerity or velocity of a pressure wave:

$$C = \sqrt{\frac{E}{\rho}}$$

and defining the Mach number as :

$$M = \frac{V}{C} = \sqrt{\rho \frac{V^2}{E}} \quad \ldots\ldots (5a)$$

2.7. *Other Similitude Aids.*

2.7.1. Two additional parameters useful in hydraulic model testing are the cavitation number and the Karman number. The cavitation number serves as an index by which the experimenter can predict the effects of cavitation in prototypes. The Karman number offers a means of estabilishing the same type of turbulent flow in the model as exists in the prototype and is useful in the design of river models.

2.8. *Alernative Methods for Developing Similitude Relationships.*

2.8.1. A somewhat different approach to the development of the dimensionless parameters such as the Froude and Reynolds numbers, while not new, has been increasing in popularity in recent years. The method appears to be more difficult in a mathematical sense then the one presented and for this reason it is not given in detail. It will be sketched, however, for those who may be interested because it is held to be a more fundamental and general method. It consists in general of the following procedure.—

2.8.2. The fundamental equations of the fluid motion are written first. Then the variables are replaced by dimensionless variables formed by relating each variable to the value of the variable at some reference point in the system under consideration. For example, linear variables may be expressed by their ratios to some reference length such as pipe diameter, channel depth, etc. Velocities may be expressed by their ratios to a known velocity such as the mean velocity of transport (discharge divided by area of section). The fundamental equations are restated in terms of the dimensionless variables. The transformed equations can be shown to be identical with the original equations except for the insertion of one or more coefficients. These coefficients are the Froude, Reynolds, Mach, and Weber numbers.

2.8.3. When the values of the coefficients are the same for two flow occurrences, the details of flow are determined by the same equations. Although in the usual case the equations cannot be solved, the conditions required to establish the applicability of the equations to the two flow occurrences also establish the conditions for similitude. The concept of similitude used here differs slightly from that previously discussed. Instead of defining similitude as equal ratios of flow variables at corresponding points in model and prototype. it is defined as the condition wherein the ratio between the variables at two locations in the model is equal to the corresponding ratio in the prototype. One concept can be derived from the other by an algebraic transformation. Partial similitude is obtained by neglecting one or more of the terms in the fundamental equations as the situation permits. For example, the neglect of all terms except those for the gravity forces results in the Froude law as the basis for the similitude.

2.9. *Significance and Application of Similitude Laws.*

2.9.1. To satisfy more than one law in any given case would require that the physical properties of the testing fluid be variable over rather broad limits. For example, to satisfy the Froude and Reynolds laws simultaneously it would be necessary that :

$$\frac{V_r}{\sqrt{(gL)_r}} = \frac{L_r V_r}{\nu_r}$$

Since the gravitational constant is nearly always the same in model and prototype, that is gr=1, the kinematic viscosity ratio would have to be related to the length scale as follows :

$$\nu_r = L_r^{3/2} \quad \ldots\ldots (6)$$

Satisfaction of this criterion for a hydraulic structure model with a length scale of 1 to 25 would require a testing fluid with a kinematic viscosity 1/125 that of water. Such a fluid is not available and if water is used as the testing fluid as is customary, the kinematic viscosity ratio is nearly unity.

2.9.2. Since the physical properties of practical testing fluid are such that only one similitude law can be satisfied in a given instance, the accepted procedure for applying similitude relations consists of selecting and applying the dominant law.

2.9.3. The conditions assumed in formulating the Froude law (Equation 2) are approximated in the case of turbulent flow with a free water surface, since the

effects of gravity outweigh those of viscosity, surface tension and elasticity. *However, since the viscous forces are neglected, every effort should be made to minimize them by using large models and smooth boundaries.* Usually, when the Reynolds number of the model exceeds a value of 10,000 and depth of flow is substituted for L in the Reynolds number, the viscous forces are relatively unimportant. If viscous effects cannot be ignored, it is possible to make compensating adjustments. Since most hydraulic problems involve free surface phenomena in one form or another, the Froude law of similitude is used more frequently than any or the other laws.

2.9.4. Steady flow in a pressure conduit, or flow around a deeply submerged body, approximates the conditions assumed in formulating the Reynolds law of similitude. Since there is no free surface connected directly with the flow pattern and the flow is steady, the forces of surface tension and elasticity are eliminated. Further, it can be proven that the gravity forces balance out and therefore do not affect the flow pattern. The Reynolds number in the model rarely can equal that of the prototype, but fortunately this is not necessary. If the Reynolds number of a model with smooth boundaries exceeds 1,000,000 for all pert'nent test flows, the boundry resistance forces become essentially independent of the Reynolds number. This can be shown by referring to Moody's diagrams where the dimensionless friction factor "f," in the Darcy

Weisback formula $h_f = f \frac{L}{D} \frac{V^2}{2g}$ is plotted with

respect to the Reynolds number for flow in pipes with various relative roughness co-efficients, denoted as

$$\frac{\varepsilon}{D}$$

For values of $\frac{\varepsilon}{D}$ less than 0·001 the friction factor

becomes nearly constant when the Reynolds number is greater than 1,000,000 and the ratio of resistance forces, model to prototype, depends only on the ratio of relative roughnesses. To obtain a satisfactorily large Reynolds number in the model, it is often necessary to increase the velocity by using exaggerated heads.

2.9.5. When surface tension is the predominating factor in a problem to be studied, the other three forces are usually insignificant and the Weber law applies. In engineering, surface tension is manifest in capillary waves on a water surface, capillarity in monometer tubes, capillary wetting action in filter beds and earth dams, tendency of a jet from an orifice to assume a circular cross section regardless of shape of orifice, and definite changes in flow over weirs at low heads. Surface tension seldom plays an important role but its effects are ever present and troublesome in model testing.

2.9.6. Except for cases of unsteady flow, especially water-hammer problems, the similitude based on the Mach number has little application in hydraulic model testing. Since most water-hammer problems yield readily to analytical methods, model studies are rarely needed. On the other hand, aerodynamic testing has led to an extensive development of the use of the Mach law to deal with problems involving the flow of gases at velocities exceeding the speed of sound, and water entry problems of ballistics have recently brought the analysis of liquid flow to the same stage.

2.10. *Similitude Ratios*

2.10.1. On deciding which of the four dimensionless parameters is relevant to the problem at hand, the similitude ratios can be developed from that parameter. For a case of open channel flow where the Froude law applies, the model prototype relationships are obtained directly from Equation 2.

$V_r = \sqrt{g_r L_r}$ Since

$g = \frac{\gamma}{\rho}$ the velocity ratio can also

be written $V_r = \left(\frac{\gamma L}{\rho}\right)_r^{1/2}$

Other kinematic relationships are as follows:

Time ratio $t_r = \frac{L_r}{V_r} = \frac{L_r}{\left(\frac{\gamma L}{\rho}\right)_r^{1/2}} = \left(\frac{L\rho}{\gamma}\right)_r^{1/2}$

acceleration ratio

$a_r = \frac{L_r}{t_r^2} = \left(\frac{\gamma}{\rho}\right)_r$

discharge ratio $Q_r = \frac{L_r^3}{t_r} = L_r^{5/2}\left(\frac{\gamma}{\rho}\right)_r^{1/2}$

The dynamic relationships are obtained in the same manner

Mass ratio $M_r = L_r^3 \rho_r$

force ratio $F_r = M_r a_r =$

$(L^3\rho)_r \left(\frac{\gamma}{\rho}\right)_r = (L^3\gamma)_r$

These and additional relationships are listed for reference in Table 1. When the fluid in model and prototype is the same, both γ_r and ρ_r become unity.

2.10.2. Even though the model boundary surfaces are made as smooth as possible to minimize viscous effects, such as resistance, it is often necessary to make adjustments in the model design or operating technique to compensate for them. There are several methods by which such compensations are effected and they will be discussed in connection with the type of model to which each is applicable.

Table 1

Characteristic	Dimension	Scale ratios for the Laws of	
		Froude	Reynolds
Geometric properties			
Length	L	L_r	L_r
Area	L^2	L_r^2	L_r^2
Volume	L^3	L_r^3	L_r^3
Kinematic Properties			
Time	t	$\left(\frac{L\rho}{\gamma}\right)_r^{1/2}$	$\left(\frac{L^2\rho}{\mu}\right)_r$
Velocity	Lt^{-1}	$\left(\frac{L\gamma}{\rho}\right)_r^{1/2}$	$\left(\frac{\mu}{L\rho}\right)_r$
Acceleration	Lt^{-2}	$\left(\frac{\gamma}{\rho}\right)_r$	$\left(\frac{\mu^2}{\rho^2 L^3}\right)_r$
Discharge	L^3t^{-1}	$\left[L^{5/2}\left(\frac{\gamma}{\rho}\right)^{1/2}\right]_r$	$\left(\frac{L\mu}{\rho}\right)_r$
Dynamic properties			
Mass	M	$(L^3\rho)_r$	$(L^3\rho)_r$
Force	MLt^{-2}	$(L^3\gamma)_r$	$\left(\frac{\mu^2}{\rho}\right)_r$
Density	ML^{-3}	ρ_r	ρ_r
Specific weight	$ML^{-2}t^{-2}$	γ_r	$\left(\frac{\mu^2}{L^3\rho}\right)_r$
Pressure intensity	$ML^{-1}t^{-2}$	$(L\gamma)_r$	$\left(\frac{\mu^2}{L^2\rho}\right)_r$
Impulse and momentum	MLt^{-1}	$\left[L^{7/2}(\gamma\rho)^{1/2}\right]_r$	$(L^2\mu)_r$
Energy and work	ML^2t^{-2}	$(L^4\gamma)_r$	$\left(\frac{L\mu^2}{\rho}\right)_r$
Power	ML^2t^{-3}	$\left[\frac{L^{7/2}\gamma^{3/2}}{\rho^{1/2}}\right]_r$	$\left[\frac{\mu^3}{L\rho^2}\right]_r$

2.10.3. In closed conduit problems where the Reynolds law applies, another set of similitude ratios may be developed from Equation 3 as follows.

$$\text{Velocity ratio } V_r = \left(\frac{\mu}{L\rho}\right)_r$$

$$\text{Time ratio } t_r = \frac{L_r}{V_r} = \frac{L_r}{(\mu/L\rho)_r}$$

$$= \left(\frac{L^2\rho}{\mu}\right)_r$$

$$\text{acceleration ratio } a_r = \frac{L_r}{t_r^2}$$

$$= \left(\frac{\mu^2}{L^3\rho^2}\right)_r$$

$$\text{discharge ratio } Q_r = \frac{L_r^3}{t_r} = \left(\frac{L\mu}{\rho}\right)_r$$

These and other relationships for flow in which viscous forces predominate are tabulated for ready reference in Table 1.

2.10.4. Except for problems dealing with lubrication and a very limited number of hydraulic structure and equipment problems for which the prototype Reynolds number is low, these ratios cannot be used. For example, in a hydraulic model having a scale ratio of 1 to 20 it would be required that the velocity ratio be 20 to 1 to satisfy the Reynolds law. In most problems the prototype velocity will exceed 5 feet per second and model velocity of over 100 feet per second would be required. Such velocities are beyond the capacity of most hydraulic laboratories. Usual procedure is to recognise the applicability of the Reynolds law of similitude and approximate it by operating the models at the highest velocity attainable in the laboratory.

2.10.5. In a similar manner, the similitude relationships governed by the Weber and Mach parameters can be developed from Equations 4 and 5, respectively.

2.11. *Distorted Models*

2.11.1. Models of river channels, estuaries, floodways, and canal structures are often distored geometrically when the length is great in proportion to width and depth, otherwise the channels would be of insufficient cross section to obtain representative flow conditions unless the model were extremely large. In a distorted model, the depth scale ratio is not the same as the length and width scale ratios, and in an occasional model different scale ratios are used for length, width, and depth. This necessitates revision of the foregoing similitude relationships. The principal limitation, when considering a distorted model, is that the model should be proportioned so that conversions of kinetic to potential energy, or vice versa, will not be distorted beyond a tolerable degree. When a model is distorted the distortion is planned to accomplish a definite objective. The results obtained from such a model are definitely limited to this objective.

2.11.2. The advantages of distorted models are (1) sufficient tractive force can be developed to produce bed load movement with a reasonably small model and available model sediment; (2) water surface slopes are exaggerated and therefore easier to determine, (3) the width and length of the model can be held within economical limits for the required depth; and (4) operation is simplified by use of a smaller model.

2.11.3. Several disadvantages are: (1) velocities are not necessarily correctly reproduced in magnitude and direction; (2) some of the flow details are not correctly reproduced; (3) slopes of cuts and fills are often too steep to be moulded in sand or erodible material and (4) there is an unfavourable psychological effect on the observer who views distorted models.

2.11.4. The similitude ratios for distorted models based on the Froude law are:

$$\text{Horizontal length ratio} = L_r$$

$$\text{Depth ratio} = D_r.$$

$$\text{Horizontal area ratio} = L_r^2$$

$$\text{Vertical area ratio} = L_r\, D_r$$

$$\text{Slope ratio} = D_r/L_r$$

$$\text{Velocity ratio} = D_r^{1/2}$$

In problems requiring the use on distorted models, resistance often plays a significant part. Then the ratio of model to prototype resistance must be made the same as the ratio of gravity forces. The resistance may be related to the velocity by means of the well-known Manning formula.

$$V = \frac{1{\cdot}49 R^{2/3} S^{1/2}}{n}$$

where S = the resistance slope

R = hydraulic radius

n = roughness coefficient

In oder that the resistance slope equal the channel slope

then

$$S_r = \frac{D_r}{L_r} = \frac{n_r^2 V_r^2}{R_r^{4/3}} = \frac{n_r^2 D_r}{R_r^{4/3}},$$

then

$$n_r = \frac{R_r^{2/3}}{L_r^{1/2}} \text{ or } n_m = n_p \frac{R_r^{2/3}}{L_r^{1/2}}$$

The value of the hydraulic radius ratio (Rr) does not bear any fixed relation to the length and depth scales but varies with the shape of the cross section. Then, for any given reach, the Rr and the Lr are known and nr. can be computed, and applied to the prototype n to obtain the required value of n for the model. Usually the required value of n in the model must be obtained by artificially roughening the model channels. Unless basic data on the values of n for different types of roughness are available the appropriate roughness must be obtained by trial and error. The model with rough ness adjusted for a particular depth will yield dependable results for flow at or near that depth. If problems of several depths occur, model roughness should be adjusted to give an average friction that is approximately right for each depth, or the roughness may be varied with depth for a closer approximation at all depths. The fact should not be overlooked however, that the Manning n is strictly a rughness characteristic, and that the manning formula applies only at sufficiently high values of Reynolds number where resistence forces are proportional to the square of the velocity.

2·11·5. Generally speaking, distorted models are not adopted to situations where curvatures in the water surface are involved. For example, the flow through spillways or between bridge piers will not be correctly represented in a distorted river models.

2·12. *The Karman Number*—

2.12.1. An important precaution in testing distorted river models is to be sure that flow in the model is of the same type as that in the prototype. Flow is invariably turbulent in rivers and, as friction forces predominate, it appears that a criterion for model design would be the Reynolds parameter or some variation of the factors which constitute this number. A practice in some laboratories has been to adjust the scales in the model so that when the hydraulic radius is substituted for length in the Reynolds number, the value will be not less than 2,500. The Waterways Experiment Station of the U.S. Corps of Engineers has used the criterion VR = 0·02 to insure turbulent flow, where V is average velocity in the channel and R is hydraulic radius. This corresponds to a limiting value of 1,800 for the Reynolds number for a temperature of 66°F.

2.12.2. The Neyrpic laboratory at Grenoble, France, goes a step further and undertakes to insure that the proper type of turbulent flow will be established; that is, a flow that may be described as the "rough well type" or "rough turbulent type". This type is required in models for all cases except for very sluggish rivers. It is claimed that the Reynolds number, in the usual application, has never been a completely satisfactory parameter for open channel flow, and is not an adequate criterion for river model turbulence. In place of the Reynolds number the following parameter is proposed and called the "roughness Reynolds number" or the "Karman number," in honour of Professor von Karman of the California Institute of Technology:

$\bar{K} = \frac{KV_*}{\nu}$ Where $\bar{K}$ is the karman number, k is the roughness parameter proportional to the sand grain size and

$V_* =$ Shear velocity $= \sqrt{RSg}$

3·0. DESIGN CONSTRUCTION AND OPERATION OF MODELS.

3.1. *General considerations*—

3.1.1. If the prototype structure is in the design stage the laboratory investigation includes only analytical and model studies and there are definite limitations. The theory of fluid mechanics is far from complete and models possess definite scale effects which are not always predictable. So it is essential that an early descision be reached regarding the nature of the basic hydraulic problem. At this stage the problem should be throughly examined and discussed by design and laboratory engineers. The laboratory engineer will thus acquire a familiarity with design considerations and construction limitations which will aid in the selection of the appropriate technique and will generally assure the development of a practical solution. The discussion will also serve to familiarize the designer with the potentialities as well as the limitations of a model study of the problem.

3.1.2. Although it is desirable to define the problems involved in any study, it is not always possible to do so in advance of the actual construction of a model. In such cases it is necessary to proceed with the selection of a model scale, and with actual construction, on the basis of experience with similar problems. The designer's request for study may specify an over-all check of hydraulic performance of a design which may be stimulated more by uncertainty with respect to general arrangement than by recognition of specific sources of malfunctioning. Operation of the model will generally disclose, to a trained model operator. the need for design changes.

3.1.3. Before starting construction of a model, account must be taken of limitations imposed by funds, time, and availability of personnel, in relation to the size or type of model. The desirable accuracy of final results will materially affect the type or extent of the investigation. For example, an extremely small spillway model may be used to determine the effect of upstream or downstream river flow or erosion tendencies but establishment of discharge co-efficients for gates or an overflow crest requires one many times as large. Limitations of space or pumping facilities are often controlling factors. For example, a large model of a river system can be tested satisfactorily only in the larger laboratories. However, it may be possible to construct and test a section at a time if a satisfactory method of combining the results can be devised.

3.1.4. It is well to remember that, no matter how carefully a model is designed or constructed it will not contribute an automatic solution. It does provide information which must be interpreted in the light of a knowledge of basic mechanics and hydraulics, as well as experience. Although a model serves to demonstrate fundamental principles that apply equally well to the prototype, it is operated according to a similitude law which is seldom satisfied. Thus, any direct translation of results must be carried out with understanding and restraint.

3.2. *Prototype Information required.*

3.2.1. Information for use in planning a model should include an over-all plan and section of the proposed or existing structure, with sufficient detail to determine the shapes and characters of all surfaces over which the flow will pass in some cases, topography of the site and surrounding area, results of foundation test boring, and details of any other related structures (particularly the hydraulic features) may be necessary. Knowledge of the type and current state of material composing the riverbed and the canyon walls may be useful. A reliable curve showing water stage with respect to discharge for the river downstream is important. In the case of river or estuary problems, historical data, such as discharge, water stage, tide salinity, cross sections of channels, and soundings are essential. For spillways and outlet works, a complete description of proposed operating conditions and the range of discharges to be expected should be available.

3.2.2. Problems relating to a model study should be discussed with the designers or other responsible persons familiar with the project. Full knowledge of local and general design conditions that may affect hydraulic performance will often save considerable work and time. The importance of becoming thoroughly familiar with design requirements early in the study cannot be overemphasized.

3.3. *Design and Construction of Model.*

3.3.1. Success or failure in achieving desired results with least work and cost depends largely on the design of the model. The first step is to select such a scale that similarity of the occurrence being studied will be preserved in the model. The model should be made as large as practicable, considering cost and benefits that may be derived. Increasing a model size may desirably enhance its usefulness and improve accuracy of results. At some point, however, depending on the type, the increase in cost and difficulty in operation will offset the advantage in size. Economy would dictate that the model be as small as possible and still yield valid results. But proving that the results from the small model are valid can be troublesome and time-consuming. Current practice is to follow precedent when available and to err on the side of largeness so far as this is permitted by limitations of available space and water-supply.

3.3.2. The following scales have been used successfully in model studies in the past and may be useful as a guide. Spillways for large dams have been constructed on scales ranging from 1:30 to 1:100. For medium size spillways, the model usually should not be smaller than that determined by a scale of 1:60. Outlet works having gates and valves are constructed to scale ratios from 1:5 to 1:30. Canal structures, such as chutes and drops, are usually constructed on scales from 1:3 to 1:20. The range of horizontal scales for river models is usually between 1:100 and 1:1000. The vertical scale for distorted river models is usually between 1:20 and 1:1000.

3.3.3. Minimum sizes have been set for certain types of studies. Individual models of valves, gates, or conduits are specified to be equivalent to at least 4 inches in diameter. The scale of an outlet model often is determined from this minimum valve diameter. Models of canal structures should exceed 4 inches in bottom channel width, and spillway models should be scaled so that normal heads over the crest exceed 3 inches. If models are constructed to dimensions smaller than these they are difficult to build, and there is a possibility of introducing similitude defects. Model valves and gates for use in development work or for rating of prototype control devices should be equivalent to atleast 6 inches in diameter. Time and expense may also be saved by choosing the scale for tunnel or conduit models to accommodate a standard pipe size or a piece of pipe already on hand. Also, advantageous use can be made of the parts of previously tested models. The model scale, which may affect the entire study, is dependent on many factors and should be carefully chosen.

3.3.4. A model need not be made strictly like the prototype. If the surfaces over which the water flows are reproduced in shape and the roughness of the surfaces is approximately to scale, the model will usually serve its purpose. Models of spillways are commonly constructed of sheet metal soldered to metal-ribbed framing, or they are made of smooth concrete mortar, screeded to a rigid frame of metal ribs. Spillway crests have also been constructed of wood. Wood exposed to the water is not recommended for the more permanent portions of models, because of shrinking and swelling. Well oiled wood, however, is satisfactory for piers and training walls of spillways, as these parts are usually subjected to many revisions.

3.3.5. Models representing tunnels or closed conduits are constructed of sheet metal or sheet plastic. Sheet metal is less expensive and can be used where it is not necessary to observe the flow. Where visibility of flow is desired or where warped surfaces are involved, plastic conduits are much to be preferred.

3.3.6. Valves and gates of the smaller sizes may be fabricated of plastic. However, for development work, where high heads are involved, they are constructed of bronze. Models of open canal structures are constructed of wood, or of wood covered with sheet metal lining. Where wood only is used, it is well oiled and the joints calked with white lead.

3.3.7. Models of the stable portions or riverbeds often consist of a cement and sand mortar, plastered over brick masonry. Sand or gravel is placed on this mortar to represent the loose or movable river material. Models of river systems may be constructed of concrete, where a fixed bed is employed, or entirely of sand or coal dust where movement of sediment is to be studied.

3.3.8. A head tray and a tail box are usually built for each model. The head tray serves as the lake, or reservoir, ahead of the structure and insures quiet approach conditions. It may be constructed of wood, lined with sheet metal, or may consist of a box built of brick or masonry blocks. In either case, it must be watertight, so that the discharge through the model can be measured accurately. The head tray contains a baffle arrangement, consisting of gravel, a series of screens, or a set of vanes, to still the flow before it enters the model. The tray contains the head gauge which is used to measure the elevation of the water surface in the reservoir.

3.3.9. The tail box is at the down stream end of the model. It contains a gate, or other device, by which the tail water level can be varied. Sand or gravel is often placed in the tail box to aid in the study of scour patterns and designs are judged to some exent by their scouring tendencies. The tail box need not be watertight but time and trouble are often saved by making it that way. For this reason the fabrication of the tail box usually resembles that of the head tray. The tail box contains a tail water gauge to indicate the water stage in the river downstream from the structure.

3.3.10. Instruments are essential in model testing. The importance of their proper installation and use is strongly emphasized, because comparison of measurement is the deciding factor in many hydraulic designs. Provision should be made for suitable instrumentation while the model is in the design stage. Instrument connections and piezometers are relatively easy to install early in the construction, whereas their installation may be very difficult after the model is completed. Sufficient piezometers should be placed in the more critical portions of the model that measurements from them will completely define the local action. Piezometers are relatively cheap; they should be provided in generous numbers to offset any oversight in defining the critical points.

3.3.11. Accurate and clear model drawings prevent time-consuming errors in construction. Drawings should contain sufficient information to enable the model makers to build a structure which conforms to the designer's specifications. Where model makers are experienced in hydraulic model construction, many drawing details may be omitted, and much of the method of construction may be left to the shop foreman. Important part which will affect the performance of the structure, or special features that are new in concept, should be detailed completely. Working drawings should include all information necessary for installation of instruments, such as piezometers, pressure cells, velocity measuring devices, etc., and thereby obviate the necessity for extensive changes after the model is completed. Since it is necessary to obtain greater accuracy in building a model than that to which most craftsmen are accustomed, care should be taken to see that the required tolerances are met, particularly in the critical parts. Greatest accuracy should be maintained where there will be rapid changes in direction of flow and were high velocities will prevail. The stage of the testing program will affect the type and accuracy of construction. In early tests, where many schemes are tried to determine their overall feasibility, the construction need not be as carefully finished as in the final stages of testing where the data required are to be used for operating the prototype structures.

3.3.12. There are many short cuts and tricks of the trade to be learned in the construction and testing of models. These can best be learned by experience. For example, the construction of the model should be made sufficiently flexible to allow considerable modification with a minimum of rebuilding. A model with a spillway stilling basin set on or close to the laboratory floor will require extensive rebuilding if the basin is to be lowered. Provision in the original plan to keep the model sufficiently high to permit the basin to be lowered without major rebuilding will make this revision comparable to a routine adjustment.

3.4. *Operation of Model.*

3.4.1. The operating program should be carefully planned to evaluate the worth of the design under study. In general, the evaluation consists of proving, by qualitative and quantitative tests, that the design

meets the operation requirements. For an undistorted model, the operating program can be divided into two phases; adjusting, and testing. In the case of a distorted, movable-bed model, a third phase, verification, must also be considered. Adjustments in the model consist of preliminary trials to reveal defects and inadequacies. *This phase should not be hurried*; it is important and ecomomical, in the long run, to spend the time required to make certain that the model behaves as intended and that the instrumentation is satisfactory. Often, minor alterations in design are suggested by these tests, such as partial re design or revision or the shifting of some measuring instrument. It is during the adjustment phase that the experimenter becomes acquainted with the peculiarities of the model and becomes adept in its operation.

3.4.2. When the model is of the movable-bed type, the adjustment phase involves verification tests in which past recorded action of the prototype must be duplicated in the model, otherwise, reliable quantitative results cannot be obtained. Because such tests are often the only basis for similitude, verification is an extremely important part of the operation of movable-bed models.

3.4.3. Testing should consist of systematically examining each proposed design with regard to possibilities for improvement, reduction in cost of construction, and reduction in maintenance costs for the prototype. The experimenter must exercise patience, imagination, and ingenuity and be capable of interpreting, the model results correctly. We should work in close co-operation with the designing engineers, since they should be continually informed as to progress and consulted concerning future testing. Data should be analysed concurrently with the testing to prevent accumulation of unnecessary data. When possible, functional relationships among the different variables should be applied to the data to aid in detection of erroneous measurments.

3.4.4. In addition to the regular testing, the experimenter should be alert to obtain general information which will be of value eventually (for compilation), even though it is not of interest to the designers at the time. Results of experiments on many models are usually required in assembling general design information, and it is a waste of time and money to construct models and not obtain all the useful information. For example, the following general information is desired on spillway and outlet works design: (1) calibration curves for all spillway crests; (2) complete calibration curves for all types of gates (3) water surface profiles through gate sections of spillways and other open channel structures; (4) information on the spreading of jets in flat chutes; (5) stilling pool actions for various types of sloping aprons and baffle pier arrangements; and (6) effects of negative pressures on the stability and discharge of overfall dams. Some of these data are obtained in routine testing, but constant vigilance will be necessary to assure that the remainder of the information is obtained where possible. Such information can be very valuable in connection with verification studies by actual field measurements.

3.4.5. Since the end product of any model study is the report which transmits the findings and recommendations, the experimenter must maintain a complete and accurate set of notes, including a diary. Dates may be of special significance in the future. Negative as well as positive results should be recorded. A complete photographic record of all important tests is indispensable; it often eliminates the necessity of repeating tests. The importance of presenting a clear, concise, well organized, and well illustrated account of the experimentation can best be appreciated by attempting to assimilate tests of a few existing reports on subjects unfamiliar to the reader.

4·0. *River Models* :—

4·0·1. The term 'river model" is used in a broad sense, and applies to all models in which the slope of the water surface throughout the model is relatively flat. These models usually involve studies of flow patterns or movements of sediment and can be classified in two types: fixed bed models and movable-bed models.

4·0·2. Typical problems studied with river models include (1) effects of closing or opening certain channels in a multiple channel river, (2) hydraulic losses at constructions such as bridge piers or cofferdams, (3) probable erosion around cofferdams or other structures, (4) characteristics of flow in and around diversion schemes, (5) effects of currents and waves on river navigation, and (6) tidal and estuary problems.

4·1 *Fixed-bed models.*

4·1·1. Problems involving relatively long stretches of a canal or river, wherein actual changes in bed configuration are not critical, are usually studied with the aid of fixed-bed models. Such problems include study of backwater effects in a river due to obstructions or the changes in backwater conditions due to channel improvements, flood routing studies, determination of flow distribution in estuary channels, etc. Although the Froude law of similitude is applicable, special attention must be given to the similarity of boundary resistance, which is of major importance and cannot be ignored or adjusted when the results are interpreted. In the case of a large model the velocity may be high enough to insure turbulent flow and the minimum model roughness may give smooth enough boundaries to represent the prototype properly. Then a geometrically similar model may be used.

4·1·2. Unless the model is unusually large, a distortion in slope is required (1) to offset the disproportionately high resistance of the model boundaries and (2) to obtain a sufficiently high value of Reynolds number to insure turbulent flow.

4·1·3. Distortion in slope required to satisfy condition (1) can be computed with sufficient precision by the Manning formula.

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

Since the Froude law required that $V_r = \sqrt{D_r}$

formula may be written.

$$V_r = \sqrt{D_r} = \frac{R_r^{2/3} S_r^{1/2}}{n_r}$$

Substituting D/L for S results in the expression :

$$D_r = \frac{R_r^{2/3} \sqrt{D_r}}{n_r \sqrt{L_r}}$$

This expression may be re-arranged to the following :

$$D_r / L_r = \frac{n_r^2 D_r}{R_r^{4/3}} \quad \ldots\ldots (8)$$

If roughness co-efficient n is known for model and prototype, n_r known and the distortion $\frac{D_r}{L_r}$ can be computed for the hydraulic radius ratio of the mean section.

4·1·4. When the slope distortion required to satisfy condition (2) is greater than that required for condition (1) the model must be made rougher by artificial means to compensate for the exaggerated slope.

4·1·5. The required model roughness can be computed from Equation (8) if the distortion and the prototype n are known. Then the model boundaries are roughened by various means such as distributing bits of mortar, grains of sand or gravel until the required value of n is obtained. When general data regarding the equivalent roughners k of various devices such as screens or expanded metal lath are available the model roughness can be designed. If such information is not available, the process is one of cut and try. The model with roughness adjusted for a particular depth will yield dependable results for flow at or near this depth. When a problem involves several depths, the model roughness should be adjusted to give an average resistance which is approximately correct over the desired range, or the roughness may be varied with depth for a closer approximation.

4·1·6. In many problems the roughness of the prototype is not known and therefore cannot be reproduced in the model. In such cases studies are made for a range of resistance which is estimated to ncompass the prototype resistance.

4·2. *Movable-bed models.*

4·2·1. There are many open channel problems involving scouring, deposition, and transportation of channel-bed material. Such problems are studied by means of movable bed models. In some cases, however, limited studies of problems of this type can be made by investigating current direction and velocity in fixed-bed models. Despite the limitations of the similitude obtainable with movable-bed models, they have proved to be a valuable aid in the solving of complex problems involving the shifting of stream bed materials. Similitude in movable bed models defies the mathematical analysis which can be applied to models involving hydraulic structures or other fixed boundary studies. Instead of arranging the various hydraulic forces involved to meet definite requirements laid down in any law of similitude, the successful prosecution of a movable-bed model study requires that the combined action of the hydraulic forces bring about similitude with respect to the all-important phenomenon of bed movement, the essence of this type of model study.

4·2·2. The general design approach is that of select-in scales and bed material which will result in bed movement of a nature generally similar to that in the prototype, taking into account the relative effects of the various discharges from minimum to maximum. There are two prerequisites to such a design approach : first, thorough knowledge of the characteristics of the prototype based on the collection and study of hydraulic and hydrographic data ; and second, experience in the field of movable-bed hydraulic models.

4·2·3. When the dimensions of a water course, including the particle size of the bed material, are scaled down to model size a discontinuity is encountered. The usual model scales result in sediment particles which are so small that they no longer act like bed material, but tend to become either suspended or compacted into an unyielding bed. The necessity for using bed material of particle sizes greater than required on a dynamic basis necessitates the use of geometrical distortion in order to obtain the essential movement of the bed material. Their vertical scale ratio is therefore made larger than the horizontal scale ratio. Thus, all slopes in the model are exaggerated. The distortion (vertical scale divided by horizontal scale) may vary from 2 or less, to as high as 7, or even more in special cases. Generally speaking, distortion should be kept as low as possible without reducing bed movement appreciably.

4·2·4. In problems involving the study of sediment action relating to a structure such as the canal intake and sluiceway of a diversion dam, distortion which would include the structure, is not advisable. In such cases, the, model must be large enough to insure movement of available model sediment.

4·2·5. Although vertical exaggeration is usually necessary from the standpoint of obtaining sufficient bed movement, it does introduce certain undesirable effects, some of which have been metioned previously but which warrant additional discussion at this point. The exaggeration may increase the slopes of the model banks beyond their angle of repose so that they will no longer stand. Distortion also increases the longitudinal slope of the stream, thus tending to upset the flow regimen to a point where artificial model roughness is required to restore it The vertical exaggeration also causes distortion of the lateral distribution of velocity and kinetic energy.

4·2·6. The difficulties of compromising resistance and bed movement have been minimized by using various model bed materials of lower specific gravity, so that less scale distortion is required to produce proper movement. Among such materials are coal, pumice, sawdust, ground plastics, etc.

4·2·7. A bed load feeding apparatus is usually desirable for a movable-bed model. It is provided at the channel entrance to supply bed load to the model. The rate of feeding should be adjustable so that it can be varied.

4·3·. *Verification of river models.*

4·3·1. The verification of a movable-bed model is an intricate, cut and try process of progressively adjusting the various hydraulic forces and varying the model operating technique until the model will reproduce, with acceptable accuracy, changes in bed configuration which are known to have occured in the prototype between certain dates. In this way, the accuracy of the functioning of the model is established, and certain of the scale ratios, such as time and discharge ratios, are determined experimentally. Their verification procedure may consist of the following steps:—

(*a*) Two prototype bed surveys of past dates are selected (The time between these two dates being known as the verification period). The model bed is molded to conform to the earlier survey.

(*b*) The hydraulic phenomenon which occured inthe prototype during the verification period is simulated in the model to the proper time scale (which is estimated to begin with), to see that all of the regulative measures undertaken by nature during that period are reproduced in the model at the proper time.

(*c*) The model bed is surveyed at the end of the period, and the model is considered to be satisfactorily verified only when this survey checks the prototype survey with acceptable accuracy.

4·3·2. During the cut and try verification process, it may be found necessary to manipulate the time scale, discharge scale, rate and manner of bed load feeding, or the slope scale of the water surface, and perhaps the gradation of the bed materials. Often, it is necessary to use scales for time and discharge which vary with stage so that each model stage will effect its proper share of movement. It is evident, therefore, that the verification phase of a movable-bed model calls for an intimate knowledge of the prototype as well as experience with this type of model study.

4·3·3. After a satisfactory verification of a river model, the degree of similitude attained remains largely a matter of judgment. The similitude is based on this general reasoning : if the model accurately reproduces changes which are known to have occured in the bed of the prototype, it can be relied on in predicting changes of similar nature which can be expected to occur in the future. These dynamic distortions inherent in the verification of a movable-bed model place an important limitation on the type of tests which can be made and thus on the type of results which can be obtained from the study. Since the verification is achieved on the basis of an adjusted simulation of recorded prototype phenomena, the model cannot be expected to respond accurately to conditions which involve a drastic departure from those involved in its verification. In the final analysis, the validity of the results of a movable-bed river model study and the interpretation of its results are largely dependent on general judgment and reasoning, the basis on such reasoning being the verification of the model, a knowledge of the prototype, and familiarity with the general characteristics of such models.

5·0. Coastal Models.

5·1. *Models of Waves.*

5·1·1. If the phenomenon to be studied is the form, pattern, and travel of waves, for instance, in a study involving breakwater location for the purpose of reducing wave height, an undistorted model is required. This type of model is designed in strict accordance with the scale relations based upon the Froude law. Ordinarily, models with fixed surfaces (usually concrete) are used for this type of study. It may be necessary to adjust the roughness of the model-bed to insure correct velocity of travel of the waves. However, it is usually found that the surface roughness has only a small affect, if any, on the wave simulation.

5·1·2. The scale to which this type of model should be constructed depends almost entirely upon the absolute magnitude of the prototype waves and the accuracy of measuring equipment on the model.

5·2. *Models of Tidal Areas.*

5·2·1. *Models for the study of these phenomena are most accurate when constructed to undistorted scales.*— However if the prototype is reasonably wide and shallow, or if the study is more concerned with average velocities than with precise velocity distributions, models of small goemetric distortion, say eight or ten, may be used. Scale relations are determined in accordance with principles already described for distorted or undistorted models, respectively.

5·2·2. *Beaches.*—Models of this type combine the characteristics of the movable-bed type with those of the tidal type. *Since they almost invariably involve the action of waves on the beaches, the models must be constructed with little or no geometric distortion.*—The amount of permissible distortion depends upon two factors—the slope of the beach, and the amount of bending of the waves as they approach the problem area. *The distortion must not be so great as to cause the beach to be steep, if the prototype beach is relatively flat; otherwise the point of breaking of the waves will be misplaced.* If the waves approach the beach without much change of direction, a small distortion might not affect their direction of approach.

On the other hand, if the waves bend appreciably as they approach the beaches, geometric distortion would alter the wave pattern, tending to cause the waves to travel in a straight line without bending.

KOSI PROJECT—FLOOD CONTROL IN BIHAR.

By

Thiru R. GHOSH, B.E. *

1·0 *The River Kosi.*

1·1 Kosi is one of the important rivers coming from the Himalayan range. Emerging from the Tibetan Plateau, it crosses the Himalayan range and flows through hill gorges of Nepal up to Chatra, where it debouches into the sandy plains of Nepal. Passing through the Saptari district of Nepal, and Saharsa, Darbhanga and Purnea districts of North Bihar, it falls into Ganga, near Kursela, after traversing a distance of about 190 miles from Chatra. In 1731, Kosi used to flow by the side of Purnea town and meet the Ganga near Manihari Ghat, on the south of Purnea. By the time the work for controlling the river was started in 1955, it had moved from east to west, covering a distance of about 70 miles from Purnea to Supaul in the district of Saharsa. From Chatra down to a distance of about 26 miles, the slope is as steep as 4·54 ft. per mile and from there up to Dagmara covering a distance of about 16 miles, the slope drops down to about 2·35 ft. per mile. Thereafter, over a distance of 41 miles up to Bhaluahi, the slope further reduces itself to 1·48 ft. per mile. Beyond Bhaluahi, the slope gradually flattens to about 0·34 ft. per mile.

2·0 *Kosi Project.*

2·1 The Kosi Project is a multipurpose scheme comprising Flood Control, Irrigation and Power. The present scheme consists of the following three units :—

(i) A barrage across the river at Bhimnagar with appurtenant works.

(ii) Embankments on either side of the river for a length of 168 miles protecting an area of about six lakh acres in the districts of Darbhanga, Madhubani and Saharsa of Bihar.

(iii) Eastern Kosi Canal System, envisaged to irrigate an area of 14·05 lakhs acres annually.

2·2 Subsequently Rajpur Canal System taking-off from the Eastern Kosi Main Canal was sanctioned to provide annual irrigation to an area of 3·97 lakhs acres in the district of Saharsa. A Power House with an installed capacity of 20,000 k.w. in the Eastern Kosi Main Canal to utilise a fall of 13′-0″ was also sanctioned.

2·3 The barrage on the river Kosi and the flood embankments have been designed for a maximum flood discharge of 950,000 cusecs against the flood discharge of 850,000 cusecs observed during 1954 floods. The distance between the embankments varies between four to ten miles, depending upon a number of factors mostly relating to the topographical features of the area. The river was visited with a discharge of 913,000 cusecs in October 1968 and the barrage and the flood embankment stood safe. Thus the barrage and the embankments have already withstood virtually the onslaught of the designed flood.

2·4 The construction of the barrage at Bhimnagar has provided stability to the river courses up to few miles upstream of the barrage and few miles on the downstream by effecting control of the river gradient in the reach from Chatra to Bhimnagar Barrage, about 26 miles long. With the construction of the embankments, the river has been confined within the embankments and the vast area to the tune of about six lakhs acres on the countryside on both banks of the river has been rendered flood-free.

3·0. *Problems encountered in maintenance of flood embankments.*

3·1 The Kosi is a meandering river and real problem of this river is its inherent tendency to change its course frequently. This is attributed largely due to its excessive course silt content, which the deficient river slope in the lower reaches is not able to carry. After construction of the embankments, the river has been confined within the embankments, but the shifting of the course and formation of new channel, locally known as dhars are constantly going on and the river configuration changes every year after the floods. The protection of the flood embankments from the fury of the river has been one of the major problems engaging the attention of the project Engineers.

3.2. In order to appreciate the stupendous task of the upkeep of these flood embankments, it would be relevant to observe the channel characteristics of river Kosi vis-a-vis the marginal flood embankments. The concept of embankments was evolved in the case of the Kosi with a two-fold objective. One was to prevent spills and the other to arrest the lateral movement of the river and confine the same to flow into a central channel. While it may be reasonable to expect the embankments to prevent spills fairly, satisfactorily subject to certain limitations, the same thing cannot be said of the lateral movement, particularly of a river like the Kosi. Confinement of flood flow between the embankment is ikely to introduce changes in channel characteristic in regard to width, depth, slope and velocity with consequent modifications into river sections. Naturally, therefore, the relationship between sediment load and channel characteristics particularly slope, would get altered resulting in positional changes of bends and meanders, which inturn would

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result in scour or fill. The scour will exhibit itself in the shape of sets due to undercutting and the fill in shoal or island coupled with a general and gradual raising of the river bed. Thus the effect of the embankment will be to produce river sets on the one hand and a gradual rise in the river bed levels with a corresponding rise in the river flood levels on the other. In as much as the embankments in the Kosi have been built far apart, the distance varying from about 4 miles in the upper reach to some 10 miles down below, the rate of rise in levels will be greatly retarded. Such a rise however small it may be will naturally bring about a progressive encroachment on the freeboard. While the designed freeboard is 6 feet, its gradual reduction will tend to reduce the safety of the enbankments, until the reduction reaches the optimum limit of, say a 3 feet freeboard. Reduction beyond this limit will considerably enhance the element of risk. At such a stage, would arise the necessity to raise enbankments if they are to serve another period of usefulness and their life extended thereby.

3·3 While the problem of rise in the bed level is comparatively easier to solve from an engineering point of view, the problem of scour caused by undercutting and formation of sets is difficult to tackle. It is also not possible to predict the points of attack, although it may not be unreasonable to anticipate the minor valley crossings as the possible vulnerable spots, where the river may be expected to take a set. A number of permeable spurs are required to be constructed at such vulnerable points along the embankment and at the most critical sites a battery of long and short impermeable spurs with length varying from 1,500′ to 300′ have to be provided to protect the embankments from the river attack. The nose of impermeable spurs are provided as launching aprons comprising of boulder crates. In the case of lingering danger to the embankment at some such vulnerable points, even retired lines may have to be provided to act as second line of defence. For formulating suitable protection measures for the safety of the embankments against the fury of the river, every year post flood cross-sections of the river are prepared after river survey and sent to Central Water and Power Research Station, Poona for model tests. Permanent model of river Kosi is maintained at the Research Station and the river conditions are similuted in the model year to year. and specific recommendations are given for protection measures to be undertaken for the safety of the embankments against the onslaught of the river. Roughly on an average, the annual cost of maintenance of the flood embankments has worked out to the tune of 70 to 80 lakhs of rupees.

4·0 *Review of important flood events.*

It would be interesting in this context to briefly enumerate the significant flood events during which the Kosi flood embankments were attacked by the meandering river. In the flood of 1961, a channel developed about four miles downstream of the barrage site, which touched the eastern embankment near 15 k.m. and flowed parallel to it up to 28 k.m. A severe attack on the eastern embankment was again experienced during the flood of 1962, which was successfully met. On the western bank, erosion was experienced for the first time during the flood of 1962, when the river came as close as 120 ft. from the toe of the embankment, by cutting the right bank to the extent of 1,500 ft. opposite village Dalwa in Nepal territory, eight miles drownstream of the barrage site. The main river was diverted through the barrage in March 1963. During the subsequent flood of that year, the river renewed its attack on the west bank near Dalwa. In 1964, the western embankment near Kanauli about two miles below Dalwa became the target of attack by the river. River conditions at Kanauli remained normal up to the end of July 1964, when the river was about 1,000 ft. away from the toe of the embankment. During the first week of October 1968, there was unprecedented flood in the river Kosi when the maximum discharge observed was of 9·13 lakhs cusecs which was close to the designed discharge of 9·5 lakhs cusecs. The eastern and western Kosi flood embankment withstood the danger posed by the high flood, save and except some minor breaches at the tail end of the western embankment. During the floods of 1971, however the river Kosi developed an eastern swing and the reach of the eastern embankment from 10 k.m. to 30 k.m. has become seriously vulnerable to the onslaught of river Kosi. Special protection measures are being undertaken with a view to forestall the attack of the river to the embankment in this reach. Over and above the schemes of providing impermeable spurs, extension of the existing spurs and resectioning of the embankment section, a proposal of providing a second line of defence by raising the existing right bank of Supaul Branch Canal to act as a retired line is under serious consideration of the Government.

5·00 *Necessity of construction of a Second Control Structure on river Kosi.*

5·1 In view of the heavy recurring cost incurred for the protection of the flood embankments against the fury of the river, it is imperative that some effective long term flood control measures are undertaken. The reach from the present Bhimnagar Barrage to Dagmara a distance of 16 miles has been vulnerably exposed to the hugging tendency of the river from west to east all these years. The slope of the river in this reach is as steep as 2·34 ft. per mile. The construction of a second control structure i.e. Dagmara Barrage would go a long way in controlling the gradient of the river in this reach and would eventually pin down the river within the embankments and cajole the same to flow more or less in a centralized channel. The importance of early construction of a second control structure as an effective and long-term check of the fury of the meandering

river can hardly be over emphasized, especially so in view of the exprience of the floods of 1971 when the reach 10 K.M. to 30 K.M. of Eastern Flood embankment was vulnerably exposed to the onslaught of the eastern swing of the river course.

5·2 Presently model tests are being conducted at Central Water and Power Research Station, Poona with a view to examining the effect of Dagmara Barrage in the gradient control of the river and subsequent stability to the river course.

6·0 *Conclusion.*

The Kosi Project was conceived essentially as a flood control scheme. Since a barrage was coming up as a component of the flood control works, the incidental works of irrigation were also added subsequently. For all practical purposes, it is an experimental scheme, designed to serve as a full-scale model for determining suitable measures to control the vagaries of this unique river. It is also not yet complete in itself. The main problem being the heavy silt charge carried by the river, the idea as envisaged originally, as an integral part of the present scheme, was to construct a series of check-dams in the upper catchment of the river and also to introduce soil conservation measures in that area after completion of flood embankments and the Barrage. But, for various reasons, those schemes could not materialise and as a result problem of siltation in the Canals has assumed a menacing dimension.

ADDRESS DELIVERED BY THIRU S. P. NAMASIVAYAM, B.E., F.I.E. CHIEF ENGINEER (GENERAL AND IRRIGATION), TAMILNADU AT THE 11TH CONVENTION OF THE INDIAN AGRICULTURAL ENGINEERS HELD AT COIMBATORE IN JANUARY 1973.

I must first thank Mr. Sankaranarayana Reddy, Convenor of this Convention for giving me this opportunity to be in your midst. He had asked me to talk on any subject I choose. I must confess that any subject which I choose in the present circumstances of my pre-occupations would compulsively be connected only with the Thanjavur delta. But my talk cannot be a casual one because it is to be given before an audience of experts. Secondly, the subject which I have chosen, that is, " The challenge that awaits the Irrigation Engineers in the Thanjavur Delta " requires respectful handling from any speaker.

When I say " Irrigation Engineers " I use the term to include the Agricultural Engineers also. There is already a small controversy about the question " Who should be called Irrigation Engineers-Agricultural Engineers ? or the Engineers who now go by the name of Irrigation Engineers ? Some say that the latter class are really only Hydraulic Engineers and it is the Agricultural Engineers who really should be called Irrigation Engineers. I think this is all needless controversy. Both the classes of Engineers are interested in Irrigation and that is all there to it.

As you are all doubtless aware, the question of sharing of the Cauvery Waters is likely to be taken up shortly for negotiations among the riparian States. There is every chance of the negotiations ending in an equitable agreement. I shall now skip over the forthcoming intervening period and go beyond the point of time when an agreement would have been reached.

The total area under cultivation in Tamil Nadu is about 6 million hectares net. Of this, about 2·5 million hectares net are under irrigation by canals, tanks, wells, etc. The extent irrigated by canal systems alone, which only can be deemed to be dependable, is just about a million hectares net, out of which 0·5 million hectares lie in the Thanjavur Delta area. Such a large extent of lands irrigated by a single river system, in a compact block, is not available any where else in our State. We have, therefore to get the best out of the Thanjavur Delta in order to progressively increase our food production. We have to see that we get the maximum out of every unit of water. And we have to get the maximum out of every unit of land. Such a dual problem is rarely met with on such a scale any where else in our country. This means we have to increase the irrigation efficiency of the delta which is already high at 56 percent. And we have to increase the irrigation intensity which now stands at a low 135 percent and also increase the yield per hectare. Increase in irrigation intensity requires additional water supplies, which, on the surface of it, so to say, are simply not available.

The constraints, which now stands in the way of increase in the Irrigation intensity in the delta, are more particularly: (i) commencement of Kuruvai cultivation depends upon a fairly firm setting of the South-West monsoon later the commencement, lesser the second crop area and thus the intensity. And (ii) the extent of Kuruvai cultivation depends upon the availability of irrigation water in the rivers during June-September period. Though the anticipated Cauvery waters sharing among the riparian States is not likely to diminish the supplies to the delta, neither can we hope for any palpable increase in them. However, it might be though that the constraints I have mentioned could be overcome by resort to large-scale exploitation of ground water of which the delta is popularly believed to abound in. But recent investigations have disclosed certain difficulties in this area too. Firstly, the availability of exploitable ground water is not that much as one was ordinarily led to believe. In the Cauvery and Vennar sub-basins put together it is estimated to be only about 29 TMC. How inadequate this quantity is, to meet the Kuruvai cultivation demands will be evident from the fact that 48 TMC of water is issued to these two sub basins during the first month of the season itself. Another factor which rules out ground water being used as a substitute for the surface water even partly in the delta is that it is not a new resource; there are only ground water reservoir facilities and not resources. When these underground reservoirs are emptied even partially, they are quickly filled in by the surface waters flowing in the delta irrigation channels and rivers. The shallow aquifer is almost fully recharged within about two months after the water from Mettur Dam is released over the delta for Irrigation. The recharge component through irrigated paddy fields is negligible in the Cauvery sub-basin because of the high clay content of the soil in the irrigated tract with a small infiltration rate. Since most river channels cut through the semi-confining layer, they recharge the aquifer directly by lateral seepage immediately after the surface water is released for irrigation. The recharge of the aquifer is thus the first charge on Mettur waters. In other words, the more you exploit the aquifer, the more you have to draw from Mettur. So, ground water can, at best, be utilised only to supplement Mettur water temporarily to be recouped by it later and not substitute it once for all.

How then to utilise the ground water without having to draw any additional supplies from Mettur, to enable early commencement of kuruvai cultivation on the largest extent possible ? In other words, how to improve the efficiency of the system—at least in favourable years ? This is the problem that is facing the irrigation Engineers in the delta. It certainly is a challenging problem as the efficiency of the system is already at a high level—50 percent. The depth of delta (of irrigation water) at the river head is only 4·2 ft. compared to 4·2 to 4·8 ft. and 3·8 to 4·0 ft. obtaining in the more modern Godavari and Krishna delta systems respectively. The delta obtained in the Cauvery system is all the more efficient if we consider the fact that rainfall is well distributed through both the irrigation seasons in the Godavari and Krishna deltas compared to the uneven distribution in the Cauvery delta area—meagre during the first crop period and copious during the north-east monsoon period.

To achieve maximum production out of every unit of water, irrigation efficiency has to be increased. To achieve maximum production out of every unit of land, irrigation intensity has to be maximised which also in turn requires increase in irrigation efficiency for the reasons I have mentioned above. Achivement of maximum production out of every unit of land requires also increase in yield per hectare which demands the most productive agronomic and land management practices.

The modernisation proposals for the delta framed in 1969 was a package consisting of (i) construction of regulators, irrigation sluices, etc., on irrigation rivers and canals, to exercise full control over the irrigation waters (ii) lining of those reaches of irrigation rivers and channels where seepage was heavy, to reduce percolation losses in transit, (iii) flood control measures like construction of a barrage across the Cauvery arm at the Upper Anicut, diversion of upland drainages which were entering Vennar into Coleroon, etc., to exclude floods from the delta, (iv) straight cuts of drainage rivers to the sea to reduce drainage congestion in tail end areas of the delta during heavy north-east monsoon falls, these 4 items aimed at improving the irrigation efficiency in the delta and (v) exploitation of ground water to enable raising of early kuruvai nurseries and to supplement surface irrigation in the last 2 or 3 critical weeks of Thaladi cultivation, this item aimed at increasing the irrigation intensity in the delta; Finally item (vi) land improvement works like providing irrigation inlets and drainage outlets for individual fields, reducing field bund sizes, regrouping of field boundaries, etc., for better land management. The last item aimed at increasing the yield per hectare. The Modernisation proposals had a few more items also like village communications, to improve marketing facilities, etc. The modernisation proposals are in main expected to ensure timely water suppliers and better water and land management and thereby increase agricultural production per hectare from 2,460 kg. to 3,160 kg. paddy. The present average for the State is about 2,500 kg. of paddy per hectare. In addition, in favourable years an increased second crop extent was also expected.

The Agricultural Branch of Irrigation Engineers has a great part to play in the work that is awaiting them in the delta. The Agricultural Irrigation Engineers, if I may call them so and in which term I include for the present purposes Agronomists also would be dealing with a very important part of the modernisation scheme, that is, land development and management works. It is a very complex work too. But you have already shown that it can be done. Your work at Siddamalli has been highly praised by the Irrigation Commission who inspected this development activity in 1971. They have noted in their report that by this development the yield per hectare has increased by 0·6 tonnes and have recommended that effective steps be taken to gradually replace field to field irrigation of rice by the system of field channels with separate drains serving individual drains as in Siddamalli. There can be no higher tributes to your work.

I wish you all success in your endeavours to meet the challenge that is waiting you in the Thanjavur Delta.

SUMMARY OF THE PROCEEDINGS OF SILVER JUBILEE CELEBRATIONS OF IRRIGATION RESEARCH STATION, POONDI. INSTITUTION OF SRI M. VISWESVARAYYA MEMORIAL ENDOWMENT LECTURE AND INAUGURATION OF 42ND ANNUAL RESEARCH SESSION OF CENTRAL BOARD OF IRRIGATION AND POWER.

The Irrigation Research Station, Poondi, after its twenty-seven years of useful, successful and meritorious service, celebrated its Silver Jubilee on 22nd June 1972 at Poondi.

This year, the 42nd Annual Research Session of the Central Board of Irrigation and Power was also inaugurated at Poondi, along with the Silver Jubilee Celebrations.

During the celebrations, an Annual Endowment Lecture in memory of Sir M. Viswesvarayya, was instituted. This lecture will be held annually at Poondi, delivered by eminent engineers and will pertain to problems connected with Hydrology and Hydraulics including Ground Water, hydraulic structures and other matters connected with irrigation.

The proceedings of the celebrations are briefly reported here :

There were about 400 delegates from all over the country who attended the celebrations including members of the Central Board of Irrigation and Power, Directors of Research Stations, Engineers from various States and the centre engaged in the design, construction and maintenance of Irrigation Structures and representatives of public and private undertakings and Educational Institutions.

Er. S. P. Namasivayam, Chief Engineer (General and Irrigation), welcomed the Honourable guests and delegates. In his Welcome Address he mentioned that Tamil Nadu is entering into a period of scarcity of water resources at present, but with every hope to tide over this difficulty.

The function was presided over by His Excellency Thiru K. K. Shah, Governor of Tamil Nadu. The celebrations were inaugurated by Honourable Dr. M. Karunanidhi, Chief Minister of Tamil Nadu.

Honourable Thiru S. J. Sadiq Pasha, Minister for Works distributed the gold medals for the staff members who had put in twenty years and more of service at Poondi, in recognition of their meritorious and lengthy service. Honourable Thiru O. P. Raman, Minister for Electricity, released the I.R.S. Silver Jubilee Souvenir.

Honourable Dr. K. L. Rao, Union Minister for Irrigation and Power, Government of India, inaugurated the Sir M. Viswesvarayya Memorial Endowment. Er. K. V. Sirinivasa Rao, President, Central Board of Irrigation and Power, delivered the annual address of the 42nd Annual Research Session of the Central Board of the Irrigation And Power.

Honourable Dr. M. Karunanidhi, in his Inaugural Address hoped that the Annual Research Session would create a good climate for co-operation between Tamil Nadu and its neighbouring States in solving irrigation problems. He pointed out that 95 percent of surface water resources of Tamil Nadu had already been exploited. He appealed to engineers to submit new techniques for tapping water for irrigation. He pointed out how in ancient times in Tamil Nadu, Irrigation system was perfected by means of Anicuts and Tanks. He also announced that the Project Report to upgrade the Poondi Research Station, into an Institute of Hydraulics and Hydrology had been approved by the State Government and action would be taken to implement the recommendations in the Project Report.

His Excellency Thiru K.K. Shah, in his Presidential Address said that if the waters of the major rivers in the country were harnessed to the benefit of the entire country, a new wave would sweep the country binding its various parts closer. If this effort was combined with an efficient water transport system, a new band of unity would be forged. The Governor said that the Central Board of Irrigation and Power, had been acting as a unifying factor. He commended the high standards of research work at Irrigation Research Station, Poondi.

Honourable Thiru S. J. Sadiq Pasha, Minister for Works, while distributing the gold medals to those who have put in long service at Poondi, mentioned that I.R.S. Poondi had done very useful work in the fields of Hydraulics and Hydrology. He also briefly mentioned the events that led to the proposal to establish a higher Institute of Hydraulics and Hydrology. The new institute for which approval has been announced by Chief Minister, earlier will conduct research in weather modification, ground water exploration, new techniques of drilling wells, desalination of sea water and brackish water, etc.

Honourable Thiru O.P. Raman, Minister for Electricity, released the I.R.S. Silver Jubilee Souvenir and commended the studies conducted at Poondi, for the projects undertaken by the Tamil Nadu Electricity Board. He pointed out that I.R.S. Poondi has developed the first Direct Simulation Continuous Resistance Electronic Analogue Computer in the world for ground water studies.

Er. K. V. Srinivasa Rao, President, Central Board of Irrigation and Power, in his Address suggested water budget for each basin and urged supervision and control by a single authority of the inter-related activities of the two sources-surface water and ground water. He also briefly mentioned the contributions to research made by all Research Stations and other

organisations during the preceding year and the role played by the Central Board of Irrigation and Power.

The Union Minister for Irrigation and Power, Honourable Dr. K. L. Rao, who inaugurated the Sir M. Viswesvarayya Endowment pointed out that natural resources belonged to the entire community and the aim should be their optmium utilisation benefiting all regions. He said that "Regional Passions and rivalries" should be eschewed in the utilisation of natural resources. He said, that because of inter-state rivalry, there were 50 river water disputes pending before him. It was engineers rather than politicians who were parochial. In one dispute while ministers of contending States accepted the compromise suggested by the Centre, Engineers did not agree to that, he pointed out.

He pleaded for new techniques of water management on recharging the ground water for conservation and use in times of scarcity. He mentioned that such a techinique was being adopted in U.K. He called upon the Central Board of Irrigation and Power and its allied institutions to devote their attention to applied research instead of abstract research.

A lunch was given to all the delegates and guests in the Periyar Park. After lunch in the after noon, Er. J. I. Coilpillai, Retired Chief Engineer of Tamil Nadu, Public Works Department, delivered the first Sir M. Viswesvarayya Memorial Endowment Lecture. In his lecture he highlighted the contributions made by Sir M. Viswesvarayya for the country in general and the engineering profession in particular. He traced the history of development of hydraulic research in Tamil Nadu and at I.R.S., Poondi and the contribution made by it during the last twenty seven years of existence.

This was followed by two Symposia on the following subjects: (i) Research on Water resources ; (ii) Research and Education. After the Welcome Address by Er. P. Kumaraswamy, Director, Irrigation Research Station, Poondi, the Symposium, "Research on Water Resources" commenced under the Chairmanship of Dr. G. Rangaswamy, Vice-Chancellor, Agricultural University, Coimbatore. There were two papers for this symposium from S. Banerji, Member, Secretary for the Indian National Committee of the International Hydrological Decade and V. B. Lal, Scientist, I.H.D. Institute, C.S.I.R., New Delhi, and Er. D. Doddiah, Advisor, Central Board of Irrigation and Power. The General Report for this symposium was read by Dr. R. Sakthivadivelu, Assistant Professor of Hydraulic Engineering, College of Engineering, Madras. The two papers invited lot of discussion from the delegates. The delegates actively participated in the discussion and mentioned the contributions made by their states in the development of water resources. The Chairman, in his concluding remarks suggested for a co-ordinated research work between Agronomist and Irrigation Engineers to harness and conserve the rain water. He called for continuous research work in that direction.

After the introductory remarks by Er. P. Kumaraswamy, Director, Irrigation Research Station, Poondi, the symposium, "Research and Education", commenced under the Chairmanship of Thiru N. D. Sundaravadivelu, Vice-Chancellor of Madras University. There were three papers for this symposium from Dr. Malcolm S. Adiseshiah. Director, Madras Institute of Development Studies, Er. P. Sivalingam, B. E. (Hons.), MS (Calif), MIE (Ind.) Director of Technical Education, Tamil Nadu and Dr. V. C. Kulandaiswamy, Dean, Post-Graduate Studies, College of Engineering, Guindy, Madras. The General Report was prepared and read by Prof. A. Mohana Krishnan, Professor of Civil Engineering, College of Engineering, Madras.

During the discussion that followed the presentation of the General Report, Dr. V. C. Kulandaiswamy, pointed out that the most important fact that faces engineering education today is the responsibility of developing men who will often realise, understand and comprehend the important aspect of interdisciplinary approach which has to be promoted consciously, and that this was one of the most important responsibilities of our educational institution. The delegates who took part in the discussion wanted a proper dialogue between man in the field and man in the laboratory and there should be a constant exchange of ideas. They also stressed for more co-ordination between the two. Padmashri N.D. Sundaravadivelu, Chairman, in his concluding remarks deplored the tendency of present day student population depending on the "notes" available in the market for their text books. He opined that this does not make the student self confident in his life. He also pointed out that it should be possible for anyone to study any course he wants at any time instead of the present procedure which does not admit people to enter the college at a time they want to prosecute their studies. He compared this to going in an escalator where people get into the escalator and get out only at the end.

Er. P. Kumaraswamy proposed the Vote of Thanks.

An exhibition was organised by Messrs. Miscrotek Instruments, Madras at the time of Silver Jubilee Celebrations. Various types of measuring and indicating instruments, manufactured by Messrs. Philips (India) Limited and a closed Circuit—TV were some of the important equipments exhibited for the benefit of delegates. Honourable Dr. K. L. Rao, visited the exhibition and was very much impressed.

In the evening a dance performance by Selvi Sasikala Devi, was arranged for the delegates. The delegates left Poondi to Madras after a sumptuous dinner served within the laboratory precincts.

In spite of the unusually heavy downpour on the Silver Jubilee day, officers and staff of Irrigation Research Station, Poondi, managed the celebrations with efficiency and enthusiasm.

KERALA-TAMIL NADU HYDROLOGICAL LINK

BY

P. KUMARASWAMY*.

A rational method to assess the possibilities of hydrological interlinking Kerala and Tamil Nadu is outlined in this report.

According to a Report on the Kerala Water Resources prepared by Er. T. P. Kuttiammu, Chief Engineer, Irrigation and General, Kerala, on 5th July 1958 and published by the Kerala Government, a total of 42·6 billion-cum. is the available water potential of the Kerala Rivers South of Kuttiyadi River. Since it may not be possible to utilise the waters of rivers north of Kuttiyadi River for the benefit of Tamil Nadu for geographical reasons, those rivers have not been considered.

Due to copious rainfall in Kerala and large values of runoff-co-efficient of the hilly forested catchments, the Kerala rivers yield a runoff of one to three metres. The percentage of irrigable land to total area of the State is smaller than the State of Tamil Nadu. Therefore, the surplus waters available in Kerala can be utilised in the following manner to the betterment of both the States.

A storage reservoir is to be formed on the Kerala side of the Western Ghats which divides Kerala and Tamil Nadu. Hydro-electric power will be developed at this reservoir. A tunnel at an elevation slightly higher than the maximum water-level in the reservoir will be drilled through the Western Ghats at a suitable location, thus connecting Kerala and Tamil Nadu.

Wherever topography and economics permit, this tunnel can be even at a lower elevation than MWL of the reservoir. Half the power generated at the Kerala Reservoir will be utilised for pumping a certain percentage of the reservoir water through the inter-connecting tunnel to Tamil Nadu. The waters emerging through the tunnel will be stored up in a reservoir to be formed on the Tamil Nadu Side. In this manner, the power developed and the water potential are shared between Kerala and Tamil Nadu.

Two examples of such reservoirs that can be formed are given below for purposes of illustration.

A dam of about 75 m. height and 300 m. length can be formed across Periyaru approximately at latitue 9°23′ N and 77°19′ E with F.R.L. at El-1050 m. above M.S.L. A tunnel of about 1 km. length can be drilled through the Western Ghats Ridge leading to Silangadai Aru on the Tamil Nadu side.

As a second example, a dam can be built across the Kakkiaru at about 9°15′ N and 77° 13′ E forming a reservoir with F.R.L. at El—1050 m. above M.S.L. and a tunnel of about 3 km. length can be drilled through the Ghats to lead waters to Kottamalaiaru on the Tamil Nadu Side.

Before embarking on such schemes a policy decision has to be taken on the question of the fraction of the water and power potential to be shared between the States of Kerala and Tamil Nadu. A formula for sharing these assests has been developed in the next paragraph.

Suggested Formula relevant to Formation of Irrigation-*cum*-Power Reservoirs in Kerala for Interlinking Kerala and Tamil Nadu Water-sheds.

Let us assume that there is a possibility of forming a reservoir in Kerala (West of the Western Ghats Ridge line), with the bed level of the dam at El. EB (M.S.L.) metres. The annual inflow into the reservoir is assumed to be $Q\,m^3$ The following further assumptions are made.

Minimum depth of water to be maintained in the Kerala Reservoir.	$d_{min}\,(m)$
Maximum depth in Kerala Reservoir	$d_{max}\,(m)$
Average depth in Kerala Reservoir	$d_{av}\,(m)$
Kerala Share of water	$Q_K\,m^3/year$
Tamil Nadu share of water	$Q_T\,m^3/year$
Evaporation and other losses from the Kerala Reservoir.	$Q_L\,m^3/year$

$$Q = Q_K + Q_T + Q_L \cdots\cdots(1)$$

Average flow through the Kerala Power House $= \dfrac{Q_K}{365\times24\times3600}\,m^3/sec$

Average power developed

$$= \frac{Q_K}{365\times24\times3600}\times d_{av}\times 9{\cdot}8\times\phi_1\,(KW)$$

* Director, Irrigation Research Station, Poondi.

Where ϕ_1 is the gross efficiency of power generation.

Let

$$P_K = \frac{\phi_1\times9{\cdot}8}{365\times24\times3600}\times Q_K\cdot d_{av}\cdots(2)$$

Let half this power generated at the Kerala Power House be utilised to pump water over the Western Ghates in to Tamil Nadu. Let the inlet level of the Kerala—Tamil Nadu Tunnel be at

$El.\,E_T\,(m)$ (M.S.L.).

Average head through which pumping has to be done—

$$E_T-(E_B+d_{av})$$

Power required for pumping

$$= \frac{Q_T}{365\times24\times3600}\left[E_T-(E_B+d_{av})\right]\times\frac{9{\cdot}8}{\phi_2}$$

Where ϕ_2 is the gross efficiency of the pumping system.

$$\therefore \frac{1}{\phi_2}\cdot\frac{9{\cdot}8}{365\times24\times3600}Q_T\left[E_T-(E_B+d_{av})\right]$$
$$=\frac{1}{2}\times\frac{\phi_1\times9{\cdot}8}{365\times24\times3600}Q_K d_{av}$$

$$\text{i.e., } Q_T\left[E_T-(E_B+d_{av})\right]=\frac{\phi_1\phi_2}{2}\cdot Q_K\cdot d_{av}$$

$$=\frac{\phi_1\phi_2}{2}\cdot d_{av}(Q-Q_T-Q_L)$$

$$\therefore Q_T\left[E_T-(E_B+d_{av})+\frac{\phi_1\phi_2}{2}d_{av}\right]$$
$$=\frac{\phi_1\phi_2}{2}d_{av}\,(Q-Q_L)$$

$$Q_T=\frac{\frac{\phi_1\phi_2}{2}\,d_{av}\,(Q-Q_L)}{(E_T-E_B)-d_{av}\left[1-\left(\frac{\phi_1\phi_2}{2}\right)\right]}\cdots\cdots(3)$$

assuming $\phi_1=\phi_2=0{\cdot}8$ we have

$$Q_T=\frac{0{\cdot}32\,d_{av}(Q-Q_L)}{(E_T-E_B)-0{\cdot}68\,d_{av}}$$

$$\text{i.e., }\frac{Q_T}{Q}=\frac{0{\cdot}32\,d_{av}}{(E_T-E_B)-0{\cdot}68\,d_{av}}\cdot\left(1-\frac{Q_L}{Q}\right)$$

we shall also assume for the sake of estimation

$$\frac{Q_L}{Q}=0{\cdot}2$$

$$\therefore\frac{Q_T}{Q}=\frac{0{\cdot}256\,d_{av}}{(E_T-E_B)-0{\cdot}68\,d_{av}}\cdots\cdots(4)$$

The fraction of waters to be shared between Kerala and Tamil Nadu thus works out Q_T/Q on the basis that the power developed is shared half and half. For example when (E_T-E_B) is 100 m. and dav=30m, Q_T/Q works out to 9·64 percent.

Based onthe above principle and assuming an average value of $Q_T/Q = 0{\cdot}2$ an amount of about 8 billion cu.m. of Kerala waters can be utilised in Tamil Nadu

NEWS ROUND UP

(a) *Cyclone and Floods in December 1972*

A Cyclone of severe intensity struck the Tamil Nadu Coast near Cuddalore on the morning of 6th December 1972. Thereafter, it weakened into a deep depression and moving, slowly across the penisula along a trail touching Kalrayan, Jawadi, Sheveroy Hills, the Nilgiris, emerged into the Arabian Sea on the morning of 8th December. For full 48 hours the entire area was under the influence of the deep depression and experienced heavy rainfall of the order of 300 mm. to 375 mm. (12" to 15") within a period of 36 hours. The rainfall in this area between September and November 1972 had been above normal, as a result of which, Irrigation Reservoirs and tanks had received copious supplies, even before the onset of the cyclone and were not in a position to absorb further unprecedented sudden inflows. The fury of nature left in its trail heavy damages to Irrigation sources/structures, and none of the northern districts of Tamil Nadu were spared. South Arcot District bore the brunt of the Cyclone and a few important irrigation structures in this District were damaged. While the century old Tirukoilur anicut was completely washed away, in Gomukhi Dam, commissioned 3 years ago, the earthen dam was over topped and breached. In all, 1600 Irrigation sources/stucture in the State were damaged the estimated cost of repairs being about Rs. 7.00 crores.

A special circle and four Divisions were formed immediately for attending to the flood repair works in South Arcot and Thanjavur Districts. The Central Government have indicated assistance, in the nature of subsidy, to the tune of Rs. 4.00 crores for repairs to Major Irrigation Sources.

(b) *A Note on Cooum Improvement scheme.*

The commencement of Cooum Improvement Scheme envisaged to remove the stinking smell of the Cooum and to make it a pleasure resort to the citizens of Tamil Nadu was inaugurated by the Late Chief Minister of Tamil Nadu Thiru C. N. Annadurai, in the year 1967.

The scheme was taken up in three stages.

First stage of works consist of installation of a regulator with two vents each of 10.67m (35') width and a jetty 76.81 m (252') long. A sand pump capable of moving on rails, along the jetty keeps the approach channel and the regulator free from sand accumulations.

The second stage contemplates on the improvements to the river from Chetpet Railway Bridge upto Napier Bridge, a length of 8.2 k.m. (5 miles 2 furlongs) by channelising the river and protecting the sides with cement concrete slabs. A walkway 3.05 m (10') in width on either margin is an additional protection during floods. The area reclaimed by channelisation viz., about 20.23 Hectare (50 acres) will be put to various uses. With this channelisation the Cooum could be used for pleasure boating which would be an added attraction to the City. Seven boat jetties have been constructed along the margin of Cooum to facilitate boating. As an added facility snack bars are proposed as an experimental measures at a few boat jetty sites.

The area available east of North Beach Road and lying between Anna Samadhi and Cooum mouth is developed by providing public attractions and utility services. These include a swimming pool 50 m × 25 m set to international standards, a deep diving pool of size 25 m × 15 m and a boat club-*cum* canteen.

As an added tourist attraction, parks and lawns are being developed on the south arm of the river Cooum and at the site of boat jetties. Shady trees and ornamental plants have been already planted and zig zag pathways, children's playgrounds, etc., laid.

A jet fountain operated from a trough in the the basin above the Napier bridge is capable of shooting 90 metres (about 270') high is illuminated by flood lights. This fountain with its illumination will be an unique attraction.

The third stage envisages rehabilitation of slum dwellers who have to be shifted from the margin of river Cooum. 941 families are proposed to be rehabilitated elsewhere.

The scheme was completed in February 73 and Dr. Kalaignar M. Karunanidhi Hon'ble Cheif Minister of Tamil Nadu declared open the pleasure boat service in Cooum on 4—2—1973.

திருக்குறளில் நீர்மவியல்.

எழுதியவர் : செ. ச. சீனிவாசன், பி. இ., *

நீர்ம இயல் என்னும் சொல் நீரைப்பற்றிய அறிவியலைக் குறிக்கும். இஃது, நிலவுலகிலிருக்கும் நீரைப் பற்றிய தால், இவ்வியல், நில அறிவியலாகும். நில அறிவிய லெனின், இவ்வியலின் எல்லை வரம்பின்றி விரியும். ஆதலின், நீர்மவியலின் எல்லை வரையறுக்கப்பட்டுள்ளது. சான்றாக, மாகடல் இயலின் எல்லாத் துறைகளும், நீர்மவியலின் பாற்பட்டனவல்ல. மேலும், நீர்மவியல் மருத்துவத் துறைக்குப் பயன்படும் நீரின் பண்புகளைப் பற்றிய துமல்ல. நீர்ம இயல், கணக்கான அறிவியலன்று. ஏனெனில், இவ்வியலுக்கு பல நடைமுறைச் செயற்படுத் தல்கள் உள. எனவே, நீர்ம வியலை வரையறுத்துக் கூறுவது எளிதன்று. வெப்ச்டரின் அகராதியில், நீர்ம இயலைப் பற்றி, "இஃது நீரின் பண்புகள், பங்கீடு, சுழற்சி (Circulation) முதலியனவற்றைப் பற்றிக் கூறும். திட்டவட்டமாக, நில உலகின் பரப்பிலும், மண்ணுக் குள்ளும், அடியிலுள்ள கற்பாறைக்குள்ளும், விண் வெளியிலும், உள்ள நீரைப் பற்றியது; குறிப்பாக ஆவியாதல், மழை பொழிதல் முதலியனவற்றைப் பற்றியது," என்று குறிப்பிடப்பட்டுள்ளது. நீர்மவியல், "நிலவுலகின்கண்ணுள்ள நீரின் தோற்றம், சுழற்சி, பங்கீடு, வேதியல், இயக்கவியற்பண்புகள், சுற்றுப்புறங்க ளால் நீரின் இயல்பு மாற்றம், உயரினங்களுடன் பிணைப்பு முதலியவற்றைப் பற்றியது. நீர்மவியலின் வீச்சு, நிலவு லகின் நீரின் முழு வாழ்க்கை வரலாற்றையும் உள்ளடக்கும்," என்றும் அறுதியிட்டுள்ளனர். இவையன்றி, நீரியலின் நிலவுலக நீர்வளத்தின் நடைமுறை இன்றியமையாத தன் மையைப் பற்றி வலியுறுத்துவம் வறையறுப்புகள் பல உள. வைச்லர், பிரேட்டர் என்போர், "நீர்மவியல், நில உலகின் நிலப் பரப்பின் நீர் வளத்தின் வரவு, செலவு களின் செயற் பாங்குகளைப் பற்றிக் கணிப்பது. மேலும், இது, விண்வெளியினூடே, நிலப் பரப்பின் மேலே, நில உலகின் அடுக்குகளினூடே, நீரின் பயணத்தைப் பற்றியது. இவ்வறிவியல், நீர்ம வியற் சுழற்சியின் பல கலைகளை அறிவுறுத்துவது", என்று அறுதியிட்டுள்ளனர்.

நீர்ம வியலை இயன்ற அளவு அறுதியிட்ட பின்னர், தலைப்பில் கொடுத்துள்ள பொருளைப் பற்றி ஆராயப் புகு வோம். முதற்கண், இக்கட்டுரையின் தேவை என்ன? நம் நாட்டவரின் புதுமைக்கு வழி கோலாது, பழமை யைப் பற்றியே என்றும் பறைசாற்றும் பாணியில் இன்றும் ஈடுபடுவானேன்? ஆங்கில கவி சேஃபியர்ரைப்பற்றி, "உலக மாமேதை, அவர் சொல்லாது விட்ட பொருளில்லை," என்று ஆங்கிலேயர் கூறுவது போன்ற நாமும், வள்ளுவப் பெருந்தகை நீர்ம வியலைப் பற்றிக் கூறியுள்ளதை நிலை நாட்டும் பாணியில், வலிந்து பொருள் கொண்டு, குறளைத் துருவித் துருவி ஆராய்கிறோமா? அல்லது, வள்ளுவ மாலையில் மதுரைத் தமிழ் நாகனார், "எல்லாப் பொருளும் இதன் பாலுள, இதன் பால் இல்லாத எப்பொருளும் இல்லையால்", என்று வானளாவப் புகழ்ந்துள்ளதை உண்மையா என்று பல கோணங்களில் நுழைந்து பார்க் கின்றோமா? என்ற இன்னோரன்ன பல வினுக்கள் எழ லாம்.

ஆங்கிலத்தில், "பரிதியின்கீழ் புதியது என்பதே இல்லை", என்றும், "பழைய மது புதிய குப்பியில்", என்றுங் கூறுவர். எனவே, "புதிய" பிரச்சினைகள் என்று நாம் கருதுவன ஏதுமில்லை. தொன்று தொட்டு நம்மைத் தாக்கியவைகள் தாம். அன்றிலிருந்து இன்றுவரை, ஏன்? என்றுமேகூட, உழவுக்கு முதலிடந்தான். பண்டை நாகரீக இனத்தவரான தமிழினத்தார், உழவுத் தொழிலைக் கண்ணே போல் போற்றி வந்தனர். உழவுக்கு உறுதுணை யான, நீரைப் பற்றியும், நீர் நல்கும் வானத்தைப் பற்றி யும் நன்கு அறிந்திருந்தார்கள். சான்றாக, நம் நிகண்டு களில் நீரோட்டத்தைப் பற்றி அருவி, மலைச்சாரல், கலுழி, உயவை, உந்தில் நதி, துறை, வருபுனல், குடிஞை, சிந்து எனப்பல சொற்கள் உள. இவையே நம் முன்னோர் நீரைப்பற்றியும், நீரியலைப் பற்றியும் நுணுகியறிந்தோர் என்று நிலை நாட்டச்சாலும் கரிகாலன் கல்லணையை நீர்ப் பாசனக்கு, என்றும் நிலைத்திருக்கும் வண்ணம், ஈராயிரம் ஆண்டுகட்கு முன்னரே கட்டி, தமிழினத்தினர் நீர்ப் பாசனத்தில் வல்லுனர்கள் என்று நிலை நாட்டி விட்டான். எனவே, வள்ளுவப் பெருந் தகையும், மழைக்கு இறை வணக்கத்தின் பின் மழையைச் சிறப்பித்து, அது முப்பாலுக்கும் மூல காரணமாதலின் முதலதிகாரமாக, "வான் சிறப்பு", என்ற அதிகாரத்தில் பாடியுள்ளார். ஏன்? முதலதிகாரத்திலேயே இறைவனை, "அறவாழி", என்றும், பிறவியைப் பற்றி, "பிறவிப் பெருங்கடல்", என்றும், கடல், என்ற தெள்ளத் தெரிந்த பருப் பொருளைக் காட்டி தெரியாத நுண் கருத்துக் களைக் காட்டியுள்ளார்.

இனி, திருக்குறளில், நீர்ம வியலைப் பற்றிக் கூறிய வற்றையும், சிறப்பாக, நீர்ப்பாசனப் பொறியாளர்கள், நீர்ம வியற் பொறியாளர்கள், வானிலை ஆராய்ச்சியாளர் கள் முதலியோர்க்கு, வள்ளுவப் பெருந்தகை, இன்று விரைந்து செயல்பட என்ன பணித்துள்ளார் என்பன வற்றையும், பல சொல்லக் காமுறுது சில சொற்களில் கண்டறிவோம்.

திருக்குறளில் மழை, நீரின் இலக்கணம், சுற்றுப் புறத்தால் நீரியல்பு மாற்றம், நீர்த்தேக்கங்கள், ஊற்று நீர், வெள்ளம், கடல், காற்று முதலிய நீர்மவியலின் கூறுகளைப் பற்றி முப்பத்தேழு குறள்களில் வலியுறுத்தப் பட்டுள்ளது. கட்டுரையின் இறுதியில் செயல்படுத்துங் குறள்களைக் காண்போம்.

மழையைப் பற்றியும், நீரின் பயனைப் பற்றியும் பதினேழு குறட்பாக்களில் கூறப்பட்டுள்ளது. "வான் சிறப்பு", என்ற அதிகாரத்தில் பத்துக் குறள்களில், மழை பெய்ய, உலகம் வழங்கி வருதலால், உலகத்து வாழும் உயிரினங்களுக்கு அம்மழை அமிழ்தம் போன்றதென்றும்; உண்பவர்க்குத் தக்க உணவுப் பொருள்களை விளைவித்துத் தருவதோடு, பருகுவார்க்குத் தானும் ஓர் உணவாக மழை அமைகின்ற தென்றும்; மழை பெய்யாமல் 'பொய்படுமானல், கடல் சூழ்ந்த அகன்ற உலகமாக இருந்தும், பசி உள்ளே நிலைத்து நின்று உயிர்களை வருத்தும் என்றும்; மழை என்னும் வருவாய் வளம் குன்றி விட்டால் (பசி தீர்க்கும் உணவுப் பொருள்களை உண்டாக்கும்), உழவரும் ஏர் கொண்டு உழமாட்டார் என்றும்; பெய்யாமலோ, அதிகம் பொழிந்தோ பெரு வெள்ளத்தால் வாழ்வைக் கெடுக்க வல்லதும் மழை; மழையில்லாமல் வளம் கெட்டு நொந்தவர்களுக்கும் வெள்ளத்தால் பாதிக்கப் பட்டவருக்கும் துணையாய் அவ்வாறே காக்க வல்லதும் மழையாகும் என்றும்; வானத்திலிருந்து மழைத்துளி வீழ்ந்தால் அல்லாமல், உலகத்தில் பசும் புல்லின் தலை யையும் காண அறிதென்றும்; மழை பெய்யாமல் போகு மானல், இவ்வுலகத்தில் வானோர்க்காக நடக்கும் திருவிழாவும் நடைபெறுது, நாள் வழிபாடும் நடைபெறு தென்றும்; மழை பெய்யவில்லையானல், இந்தப் பெரிய உலகத்தில் பிறர் பொருட்டுச் செய்யும் தானமும், தம் பொருட்டுச் செய்யும் தவமும் இல்லை என்றும்; நீரின் சுழற்சியை, அழகாக மேகம் கடலிலிருந்து நீரைக்

* தொழில் நுட்ப அலுவலர் (சிறப்பு), தலைமை நீர்ப்பாசன பொறியாளர் அலுவலகம், சேப்பாக்கம், சென்னை-5.

கொண்டு அதனிடத்திலே பெய்யாமல் விடுமானால் பெரிய கடலும் தன் வளம் குன்றிப் போகும் என்றும்; எப்படிப்பட்டவர்க்கும் நீர் இல்லாமல் உலக வாழ்க்கை நடைபெறாது என்றால், மழை இல்லையானால் ஒழுக்கமும் நிலை பெறாது போகும் என்பதை, "நீரின்றமையாது உலகு" என்ற முக்கால உண்மையைப் பகர்ந்துள்ளார். இயற்கை பொய்த்தால், செயற்கை மழைக்காக அன்றே முயன்று வெற்றி கண்டனர் போலும்! "வாழ்க்கைத் துணை நலம்" என்னும் அதிகாரத்தில்,

"தெய்வந்தொழா அள் கொழுநன் தொழுதெழுவாள்
பெய்யெனப் பெய்யும் மழை", என்று கூறப்

பட்டுள்ளது.

இதுவா செயற்கை மழை பொழிவிக்கும் முறை? என்றோ, இன்று கற்புக்கரசியார் இல்லையா? என்ற ஐயப்பாடுகளை எழுப்பாது பெய்யென்றால் மழை பெய்விக்க வேண்டும் என்ற எண்ணம் அன்றே இருந்தது என்று கொள்வதே முறை. வளமற்ற பாலைவனத்தின் கொடுமையை "வன்பாற்கண் வற்றல் மரந்தளிர்த்தற்று" என்று எடுத்துக்காட்டாக "அன்புடைமையில்" கையாண்டுள்ளார். கைம்மாறோ, கையூட்டோ, கருதாது பொழியும் மழையின் கைம்மாறு மாண்மை, "ஒப்புரவறிதலின்" முதற்குறளில் ("கைம்மாறு வேண்டா கடப்பாடு மாரிமாட்டு என்ஆற்றுங் கொல்லோ உலகு") என்று வியந்துள்ளார். இந்தியாவின், உணவு, மின்சாரம் ஆகியவற்றின் பற்றாக்குறைக்கும் காரணமாகிய பருவ மழையை நாம் நம்பியிருக்கும் இன்றைய நிலையை, "செங்கோன்மையில்" "வானோக்கி வாழும் உலகெல்லாம்" என்றும், மழையின்றி பஞ்சத்தால் வாடுவதை "கொடுங் கோன்மையில்", "துளியின்மை ஞாலத்திற்கு எற்று", என்றும், பருவ மழை தவறுவதை, "உறை கோடி ஒல்லாது வானம் பெயல்", என்றும் படம் பிடித்துக் காட்டுகின்றார். அரபு நாட்டு மக்கள், குளிப்பதற்கும் நீர்ப் பஞ்சம். நீரால் புறம் தூய்மையாகும். அது போன்று வாய்மையால் அகம் தூய்மை யாகும் என்பதனை, "புறந் தூய்மை நீரான் அமையும்..", என்று "வாய்மை" என்னும் அதிகாரத்தில் எடுத்துக் காட்டாக நீரின் பயனைக் கூறுகின்றார்.

நீரின் இலக்கணம் என்ன? "அரணில்," நீர், மணி போல் தெளிந்திருக்க வேண்டும் என்பதனை, "மணி நீரும்", என்று குறிப்பிட்டுள்ளார். வானத்தினின்று பொழியும் மழை, நிலத்தைத் தொடு முன் மணி நீராக இருக்கும் என்பதனை அறிவோம். ஆயினும் நிலத்திற் பட்டதும் அந்நீர் தன் தூய்மையை இழக்கிறது. மேலும், கழிவு நீர் முதலியவற்றை ஆறுகளிலும், கால்வாய்களிலும் கலப் பதால், நீர் தன் தூய்மையைப் பெரிதும் இழக்கின்றது. மக்களுடைய அறிவு, இனத்தின் இயல்பினை உடையதாகும் என்ற கருத்திற்கு இதனை எடுத்துக் காட்டாக "சிற்றினம் சேராமை" என்ற அதிகாரத்தில், "நிலத்தியல் பால் நீர்திரிந்தற்றாகும்", என்று கூறியுள்ளார்.

நீரைத் தேக்க அணைகள் தேவையல்லவா? அணை யின்றி நீர்த் தேக்கமா? கரையற்ற நீர் நிலை, எவ்வாறு நீரைத் தேக்க இயலாதோ அவ்வாறே சுற்றத்தால் சுற்றப் படாதவன் வாழ்க்கையின் இன்பங்களைத் தேக்கவோ, நுகரவோ முடியாது என்பதனை, "சுற்றந்தழாலில்", "கோடின்றி நீர் நிறைந்தற்று" என்கின்றார். குளங் களில் நீர் நிறைந்திருப்பின், ஊரில் உள்ள செல்வந்தர், வறியோர் என்ற பாகுபாடின்றி எல்லோரும் நீரைப் பயன்படுத்துவர். இவ்வாறே, ஒப்புரவினால் உலகம் வாழு மாறு விரும்பும் பேரறிவாளினின் செல்வமும் பயன்படும் என்பதனை, "ஒப்புரவறிதலில்" "ஊருணி நீர் நிறைந்தற்றே", என்று கூறுகின்றார்.

வெள்ளம், திடீரென்று முன்னெச்சரிக்கையின்றி பெரு மளவில் வந்து பெரிய இழப்புகளை உண்டாக்கும். "இடுக் கண் அழியாமையில்", "வெள்ளத்தனைய இடும்பை" என்று, வெள்ளத்தை பெருந்துன்பங்களுக்கு நிகராகக் கூறுகின்றார். மாந்தரின் உயர்வு, எதைப் போன்றது? இதற்கு விடையாக, "ஊக்கமுடைமை"யில், "வெள்ளத் தனைய மலர் நீட்டம்" என்கின்றார்.

நிலத்தின் புறத்தே ஓடும் நீரின் அளவு தற்போதைய தேவைக்கு குறைந்துள்ளதென்று நில நீரைத் தோண்டி, தேடி அலைகின்றோம். ஆழமாகத் தோண்டத் தோண்ட, ஊற்று நீர் அதிகமாகக் கிடைக்கும். இதைப் போன்று கற்கக் கற்க அறிவு ஊறும். இவ்வுண்மையை, "கல்வி" யில், "தொட்டனைத்தூறும் மணற்கேணி", என்றார். ஒரு நாட்டில் இன்றியமையாத உறுப்புக்களாகிய ஊற்று (ஊற்று நீர்), மழையுமாகிய இருவகை நீர்வளங்களை "நாடு" என்னும் அதிகாரத்தில் "இருபுனலும் வாய்ந்த மலையும் வரு புனலும் வல்லரணும் நாட்டிற்கு உறுப்பு", என்று ஓதுகின்றார். இறைத்த கிணறு ஊறும் என்பார். தலைவி, தலைவன் பிரிந்திருக்கும் போழ்து, தன் காம நோயை பிறர் அறியாமல் மறைக்க மறைக்க மிகுகின்றது. எதைப் போன்று? இறைப்பவர்க்கு ஊற்று நீர் மிகுவது போல? இதனை "படர் மெலிந்திரங்கல்" என்னும் அதிகாரத்தில்,

"மறைப்பேன் மன்யானிஃதோ நோயை இறைப்பவர்க்கு
ஊற்று நீர் போல மிகும்" என்று கூறுகின்றார்.

கடல் அளவிடற்கரியது; மிகப் பெரியது என்ற உண்மை, குறளில் பல இடங்களில் கூறப்பட்டுள்ளது. காமத்துப் பாலில் பல முறை காமம் கடலிற் பெரிதென கூறப் பட்டுள்ளது. விரிவஞ்சி, கடலைப் பற்றிக் கூறப்பட்டுள்ள சில குறள்களை மட்டும் குறிப்பிடப்பட்டுள்ளது. "செய்ந் நன்றி அறிதல்" என்னும் அதிகாரத்தில், இன்ன பயன் கிடைக்கும் என்று ஆராயாமல் ஒருவன் செய்த உதவியின் அன்புடைமையை ஆராய்ந்தால், அதன் நன்மை கடலை விடப் பெரிதாகும், என்னும் கருத்தை,

"நன்மை பயன்தூக்கார் செய்த உதவி நயன்
தூக்கின் கடலிற் பெரிது" என்று, கடலை மிகப் பெரிய தொன்றுக்குக் எடுத்துக் காட்டாகப் பயன்படுத்தியுள்ளார். இந்நில வுலகம், கடலால் சூழப்பட்டுள்ளதை, அழகாக, "பிறனில் விழையாமை" யில், "நாம நீர் வைப்பின்" என்று கூறியுள்ளார். காற்று, நிலவுலகில் பரவி இருப்பதை "அருளுடைமை" யில், "வளி வழங் கும் மல்லல்மா ஞாலம்", என்று குறிப்பிட்டுள்ளார்.

மேற்கண்டவற்றால் திருக்குறளில், நீர்மவியல் என்ற தலைப்பின் கீழ் சில அதிகாரங்களில் ஒதுக்கப்படாது, நீர்ம வியலின் பல கூறுபாடுகளை குறளில் பரந்திருக்கின்றன என்று புலப்படும். மழை நீர், ஊற்றுநீர், கடல் முதலியவை பழந்தமிழர் வாழ்க்கையில் நெருங்கியுள்ளன. அவற்றின் இன்றியமையாமை நீர் சுழற்சி, நீரின் பல தோற்றங்கள், முதலியனவற்றை நன்றாக உள்ளங்கை நெல்லிக்கனி போன்று அறிந்திருந்தனர். எனவே, வள்ளுவப் பெருந் தகையும் தெரியாத நுண்ணிய கருத்துக்களை விளக்கும் போது நன்றாக தெரிந்த மாரி, நீர், கடல் முதலிய வற்றை எடுத்துக் காட்டுகளாகக் கையாண்டுள்ளார். வள்ளு வப் பெருந்தகை, தாங் கூற விரும்பிய தெள்ளிதில் புலப் படாத அரிய நுண் கருத்துக்களை விளக்க எடுத்தாண்ட எடுத்துக்காட்டுகளுள், நிலத்தோடு நீர் பொருந்தி கலந்தமையை, "நிலத்தோடு நீரியைந் தன்னரகத்து" என்றும் குறளில், வள்ளுவப் பெருந்தகையின் நயம் நன்கு புலப்படும்.

"நீரின்றமையாது உலகு" என்ற முக்கால குறள் உண் மைப்படி, நீருக்காக எல்லாவிடங்களிலும் தேக்கங்களை அமைக்க விழைதல், நாடுகளுக்கிடையே உள்ள சச்சரவு கள், கங்கை, காவிரி நீரிணைப்பைப் பற்றிய தீவிர முயற்சி, ஊற்று நீரைக் கண்டுபிடிக்க பல திட்டங்கள், செயற்கை மழை பொழிவிக்க ஆராய்ச்சி, கடல் நீரை ஆவியாக்கி குடிநீராக மாற்ற ஆராய்ச்சி முதலியவற்றில் முழுமனதோடு இன்று ஈடுபட்டிருக்கும் நமக்கு, வள்ளுவப் பெருந்தகை, நீவிர் இனி "வானோக்கி வாழாது" "தொட்டனைத்தூறும் மணற்கேணி"கள், பலவற்றை அமைக்குமாறும், மழையைய் "பெய் என்றால் பெய்யுமாறும்" செய்யும் ஆராய்ச்சி களிலும் ஆக்கப் பணிகளிலும் செயல்படுமாறு பணிக் கின்றார் என்று கொண்டு அவர் ஆணைகளை சிரமேற்கொண்டு ஆவனவற்றை விரைந்து ஆற்றுவோமாக.!

களிமண் நிலங்களில் கட்டிடத்தின் அடித்தளங்கள்.

எழுதியவர் :

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1. களிமண் நிலங்களில் கட்டப்படும் கட்டிடங்கள் தகுந்த வடிவமைப்பு முறைகளினால் கட்டப் பெறுவிடின் காலம் செல்லச் செல்ல பலவிதச் சேதங்களுக்கு ஆளாகின்றன. கட்டிடத்தில் குடி புகுந்தோர் கடும் பயம் கொள்ளுமளவில் விளையும் இச்சேதங்கள் யாவை? சுவர் நடுவில் சுண்டு விரல் அளவு வெடிப்புகள், சமமாயிருந்த கூடத்தளத்தில் அலை போன்ற மடிப்புகள், வெடிப்புகள், நேராயிருந்த சுவர் நிலை குலைந்து ஒரு பக்கம் சாய்ந்து போதல் ஆகியவை களாம். சென்னையில் அண்ணா நகரிலும், நந்தனத்திலும், நுங்கம்பாக்கத்திலும் இத்தகு நிலைமைகளுக்கு ஆளான கட்டிடங்களைக் கண்கூடாகக் காணலாம்.

1.1. பொதுவாகக் கரிசல் மண் நிலங்களிலும் (Black Cotton Soils) களிமண் நிலங்களிலும் கட்டப் படும் கட்டிடங்கள் இத்தகு நிலைமைகளுக்கு ஆளா கின்றன. கடைகால் எடுத்து கட்டிடத்திற்கு அடித்தளம் அமைக்குங்கால் தகுந்த வடிவமைப்பு முறைகளைப் பின் பற்றாததனாலும், சிறப்பான கவனம் செலுத்தாததனாலுமே இந்நிலைமைகள் பெரும்பாலும் ஏற்படுகின்றன. நெடுங் காலமாகவே இந்தப் பிரச்னை பொறியாளர்களின் கவனத்தை ஈர்த்து வந்துள்ளது. கட்டிடப் பொறியியல் துறையில் நமது நாடு இந்நாள் சிறப்பான முன்னேற்றம் கண்டுள்ளது. கரிசல் மண் நிலங்களிலும் களிமண் நிலங்களிலும், இலகு வகை கட்டிடக் கோப்புகள் (Light weight structures) எழுப்புவதற்கான தெளிவான வடிவமைப்புகள், தற்காலப் பொறியியல் நூற் களஞ்சியங்களில் காணக் கிடைக்கின்றன.

அவைகளை விரிவாகக் காணுமுன்னர், கட்டிட அடித் தளத்தின் தேவைகள் என்ன என்பதைக் காணுவோம்.

2. அடித்தளத் தேவைகள்:

2.1. எந்த ஒரு கட்டிடத்தின் அடித்தளமும் கூறப் போகும் அடிப்படைத் தேவைகளை நிறைவு செய்வதாய் இருக்க வேண்டும்.

1. அடித்தளத்தின் உறுதியும் நிலைப்பாடும், அடித்தள மண்ணின் தாங்கும் திறன் (Bearing Capacity) மற்றும் அதன் படிவு (Settlement) ஆகியவைகளைப் பொறுத்தே உள்ளன.

2. ஆகவே எந்த ஒரு அடித்தளமும் மேற்கூறிய நிலை தன்மைத் தேவைகளை ஒரே நேரத்தில் நிறைவு செய்ய வேண்டியதாய் உள்ளது.

2·2. இத்தேவைகளின் விவரம் வருமாறு :

1. அடித்தளம் செலுத்தும் அழுத்தத்தினால் மண்ணில் ஏற்படும் கத்தரிப்பு நிலைகுலைவினின்றும் (Shear failure) போதிய காப்பு வேண்டும். அதாவது அடித்தளம் செலுத்தும் அழுத்தம் அடித்தள மண்ணின் அனுமதித் தாங்கு திறனுக்கு (Allowable bearing capacity) உட்பட்டதாக இருக்கவேண்டும்.

அடித்தள மண்ணின் தாங்கும் திறனானது, அடித் தளத்தின் வடிவம் அடித்தள மண்ணடுக்குகள் (Soil Profile) மண்ணடுக்குகளின் ஆழம், அலகு எடை, கத்தரிப்புத் திறன் (Shear strength) நிலநீர் மட்ட உயரம் ஆகியவைகளைப் பொறுத்தே உள்ளது.

2. அடித்தளம் செலுத்தும் அழுத்தத்தினால் அடித்தள மண்ணில் ஏற்படும் சீரற்ற படிவம் (differential settlement), உயர் படிவம் (maximum settlement) அனுமதிக்கப்பட்ட காப்பு எல்லைகட்குட்பட்ட அளவினதாக இருக்க வேண்டும். களிமண் நில அடித்தளங்களுக்கு 4 செ. மீ. உயர் சீரற்ற படிவம் (maximum differential settlement) மணற் பாங்கான நில அடித்தளங் களுக்கு 2·5 செ. மீ. உயர் சீரற்ற படிவம் இந்தியத் தர நிர்ணய நிறுவனம் (I.S.I.) பரிந்துரைக்கின்றது.

3. வருங்கால விளைவுகளினின்றும் அழிவு ஏற்படாவகையில் அடித்தளம் அமையத் தெரிவு செய்யும் இடமும் வடிவமைப்பும் ஏற்றதாக இருக்கவேண்டும். வருங்கால விளைவுகள் யாவை? அவை நில நீர் மட்டத்தின் ஏற்றம் இறக்கமாக இருக்கலாம்; மண்ணின் நனைப்பும் உலர்வு மாக இருக்கலாம்; குளிர் சாதனக் கருவிகளாலும் வெப்ப அடுப்புகளாலும் மண்ணில் நிகழும் ஈர நிலை மாற்றங்களாகவும் இருக்கலாம். கட்டிட மருங்கில் நடை பெறும் புதிய கட்டுமான வேலைகளால் அடித்தளத்தில் ஏற்படும் நிலையற்ற தன்மையாகவும் இருக்கலாம்.

அடித்தளத் தேவைகள் என்ன என்பதை இதுகாறும் கண்டோம். இனி, கட்டிடங்களில் விரிசல்கள் ஏற்படுவதற் கான காரணங்களை ஆராய்வோம்.

3. விரிசல்கள் ஏற்படுவதற்கான காரணங்கள் :

இலகு வகைக் கட்டிடங்களில் ஏற்படும் விரிசல்களுக்கு பல வகையான காரணங்கள் கூறலாம்.

3.1. பெரும்பாலும் அடித்தள மண்ணின் மேல் நோக்கிய அல்லது கீழ் நோக்கிய அல்லது பக்கவாட்டு நகர்வுகளே கட்டிட விரிசல்களுக்குக் காரணமாய் உள்ளன. இவைபற்றிய தொடர் காரணங்கள் வருமாறு :

1. அடித்தள மண்ணின் மேல் நோக்கிய அல்லது கீழ் நோக்கிய நகர்வுகள் மண்ணின் பெருக்கத்தினாலும் (அதா வது உப்புதலினாலும்) சுருக்கத்தினாலும் ஏற்படுகின்றன.

2. மண்ணின் பெருக்கமும் சுருக்கமும் மண்ணில் ஈர நிலை மாற்றங்களால் விளைகின்றன.

3. மண்ணின் ஈரநிலை மாற்றங்கள், அருகிலுள்ள குடி நீர் அல்லது வடிகால் குழாய்களினின்றும் ஒழுகும் நீரா லும், அருகிலுள்ள நீர் உறிஞ்சும் மரங்களினாலும், ஓடை கள் போன்றவை அருகில் இருப்பின் அவற்றில் உள்ள நீரோட்டத்தினாலும் ஏற்படும்.

3.2. மண் பெருக்கத்தினாலும் சுருக்கத்தினாலும் ஏற் படும் வெடிப்புகள், அடித்தளத்தின் அழுத்தம் மண்ணின் பெருக்க அழுத்தத்தை விட அதிகமாக இருப்பின் மண் பெருக்கத்தினால் கட்டிடத்திற்கு எவ்வித சேதமும் விளை வதில்லை. ஆனாலும் இலகுவாக அழுத்தப்பட்டுள்ள கூடத் தின் தளங்கள் முதலியவை மண் பெருக்கத்தால் சேதம் அடையலாம்.

இலகு வகைக் கட்டிடங்களில் ஏற்படும் வெடிப்புகளுக்கு சில சமயங்களில் கட்டிடத்தின் அடியில் எப்பொழுதும் ததும்பி நிற்கும் ஈரமே காரணமாய் உள்ளது. ஈரம் சேரச் சேர மண் பெருக்கமடைந்து கொண்டே வருகிறது. அதனால் கட்டிடங்கள் வெடிப்புக்கு ஆளாகளாம்.

* இளநிலைப் பொறியாளர், மண் தன்மை ஆராய்ச்சிக்கூடம், சேப்பாக்கம், சென்னை-5.

சில இடங்களில் பருவ காலங்களுக்குத் தகுந்தாற் போல் மண்ணில் நிகழும் ஈரநிலை மாற்றங்கள் கட்டிடத்தின் சேதங்களுக்குக் காரணமாய் உள்ளன. மாரிக் காலங்களில் கட்டிடத்திற்கு மருங்கில் உள்ள ஈரத்தின் ஊடுருவலால் கட்டிடத்திற்கு அடியில் ஈரம் ததும்பி நிற்கின்றது. கோடை காலங்களில் கட்டிடத்திற்கு மருங்கில் உள்ள மண் உலர்ந்து போகின்றது. அதை ஈடு செய்ய கட்டிடத்திற்கு அடியி லுள்ள ஈரம் வெளியேறிக்கொண்டே இருக்கின்றது. இத் தகைய நிகழ்ச்சிகளால் மாரிக் காலத்தில் மண் பெருக்க மும் கோடைக் காலத்தில் மண் சுருக்கமும் உண்டா கின்றன. அதனால் மண் உயரவும் படியவும் செய்கின்றது.

மண்ணின் உயர்விலும் படிவிலும் கட்டிடத்தின் பகுதிகளில் இழுவிசை (Tensile force) உண்டாக்கப் பட்டு கட்டிடம் விரிசல்கள் காணத் தொடங்குகிறது. இந்த விரிசல்கள் செங்குத்தாகவும் கிடையாகவும் மூலைவாக்காக வும் இருக்கும். கோடைக் காலத்தில் காணத் தொடங்கிய வெடிப்புகள் மழைக் காலத்தில் மீண்டும் கூடத் தொடங் கும். ஆனல் முழுவதும் கூடுவதில்லை. காரணம், சுருங்கிய அளவுக்கு மண் மீண்டும் பெருக்கம் அடைவ தில்லை. சுருங்கும் பொழுது கட்டிடத்தின் பளு துணை புரிகிறது. ஆனல் மண் பெருக்க மடையும் பொழுது கட்டிடத்தின் பளுவை மீறித்தான் பெருக்க மடைய வேண் டும். கட்டிடம் ஒரு முறை விரிசல் கண்டால் உடைந்த செங்கல் போன்ற கட்டிட நுண் பொருள்கள் விரிசல் களுக்குள் சிக்கிக் கொள்வதால் மீண்டும் விரிசல்கள் முழு மையாகக் கூடுவது தடைபடுகின்றது.

கடும் வெடிப்புகள் கண்டுள்ள கட்டிடங்களின் வரலாற்றை ஆயுங்கால் பெரும்பாலும் இவ்விரிசல்கள் கோடை மாதங் களில்தான் காணத் துவங்குகின்றன என்பது கண்டறியப் பட்டுள்ளது. அடித்தள மண்ணின் சமமில்லாத சுருங்கங் களினல் விளையும் சீரற்ற மண் படிவுகளால் தான் (Differential Settlement) கட்டிடத்தில் வெடிப்புகள் ஏற்படுகின்றன என்று பொறியாளர்கள் கருதுகின்றனர்.

கோடைக் காலங்களில் கட்டிட மருங்கில் உள்ள மண் உலர்ந்து போகின்றது. இதனல் அடித்தள மண்ணில் உள்ள ஈரமும் உலரும் பொழுது அடித்தள முனைப் பகுதி யில் உள்ள மண் அதிகமாகவும் நடுப்பகுதியில் உள்ள மண் குறைவாகவும் சுருங்குகின்றது. மேலும், அடித்தள வெளிப்புற மண் சுருங்குங்கால் முனைப்பகுதி மண்ணை யும் சேர்த்துப் பிடித்திழுக்கின்றது. அடித்தள முனை களினடியில் உள்ள மண் இவ்வாறு நகர்வதால் அடித்தள முனைப்பகுதி நடுப்பகுதியைவிட அதிகமாகப் படியத் துவங்கு கிறது. அடித்தளத்தின் சீரற்ற படிவுகளால் கட்டிடம் விரிசல்கள் காணத் துவங்குகின்றது.

3.3. கட்டிடத்திற்கு அடித்தளம் அமைத்த பருவ காலம் : கட்டிடத்திற்கு ஏற்படும் சேதத்தின் வகைகள் அது எந்தப் பருவத்தில் கட்டப்பட்டது என்பதையும் பொறுத்து உள்ளது. அடித்தள மண் பெருக்கமடைந்து உதித்துப் போயிருக்கும் நிலையில் மாரிக் காலங்களில் எழுப் பப்பட்ட கட்டிடங்கள் அடுத்து வரும் வேனிற் காலத்தில் விளையும் சீரற்ற சுருக்கங்களினல் வெடிப்புகள் காணத் துவங்குகின்றன. மாறுகக் கோடைக் காலத்தில் மண் சுருங்கி அடர்ந்துள்ள நிலையில் எழுப்பப்பட்ட கட்டிடங்கள் அடுத்து வரும் மழைக் காலத்தில் ஏற்படும் அடித்தள மண் பெருக்கத்தால் தூக்கப்படுவதால் விரிசல்கள் காணத் துவங்கு கின்றன. முதலில் குறிப்பிட்ட கட்டிடங்களில் விரிசல்கள் கோடையில் தோன்றி விரிவடைகின்றன. இரண்டாவதில் மழைக் காலங்களில் தோன்றி விரிவடைகின்றன.

3.4. வறட்சிக் கால விளைவுகள் : **1969**-ம் ஆண்டில் சென்னை நகரம், காலம் எப்பொழுதும் கண்டிராத வறட்சியில் பீடிக்கப்பட்டது. நில நீரைக் குடி நீருக்குப் பயன்படுத்தி நிலைமைதனைச் சமாளிக்க வேண்டியிருந்த தால் நில நீர் மட்டம் மிகவும் கீழிறங்கிவிட்டது. அதனல் அடிமண் மிகவும் சுண்டிச் சுருங்கிப் போய்விட்டது. இத்தகைய அளவற்ற அடிமண் சுருக்கங்களினல் கட்டிடம் கட்டி முடிந்து இருபது ஆண்டுகள் வரையிலும் ஓர் பழுதும் நேராதிருந்த கட்டிடங்களில்கூட விரிசல்கள் ஏற்படத் துவங்கின. மண் சுருக்கங்களினல் கட்டிடங்களில் ஏற் படும் வெடிப்புகளுக்கு இவ்வரலாறு தக்க ஒரு சான்றுகும்.

4. பெருக்கமடையும் தன்மை வாய்ந்த மண் வகைகள்.

4·1. பெரும்பாலும் பெருக்க மடையும் தன்மை வாய்ந்த மண் வகைகளில் கட்டப்படும் கட்டிடங்களே பின்னுளில் இடர்பாடுகளுக்கு ஆளாகின்றன. தொல்லை பயக்கும் இம் மண் வகைகள் யாவை ? அவைகளை எப்படி அறிவது ? ஆகியவை பற்றி இனிக் காண்போம்.

4.2. நீரும் உலர்ந்த களிமண்ணும் நெருங்கிக் கலந்தால் களிமண் தன் கன அளவில் பெருக்கம் அடைந்து காட்சி யளிப்பது கண்கூடு. இதையே பெருக்கமடைதல் என்கிருேம். சில களிமண் வகைகள் ஈரநிலை மாற்றங்களால் வெகு எளிதில் பாதிக்கப்பட்டு மிகுந்த கனப் பெருக்கம் அடை கின்றன. இத்தகைய களிமண் வகை தமிழ்நாட்டில் மிகுந்து காணப்படுவது கரிசல் மண் (Black Cotton Soil) ஆகும். இம்மண் கறுப்பு நிறமாக இருக்கும். மிகுந்த பசைத்தன்மை பெற்றிருக்கும் பருத்தி இம்மண்ணில் நன்கு விளையும்.

4.3. பெருக்க மடையும் தன்மை வாய்ந்த மண் வகைகள் எல்லாம் மிகுந்த பிளாஸ்டிக் தன்மை உடையனவாய் இருக் கும். இந்திய தராதரக் குறியீடுகளின்படி இம்மண் வகை கள் CH என்ற கூட்டத்தில் அடங்குகின்றன. திரவ எல்லை யும் (Liquid limit), பிளாஸ்டிக் எல்லையும் (Plastic limit) இம்மண்களுக்கு மிக அதிகம். தொல்லை பயக்கும் மண் வகையின் குறைந்த அளவு திரவ எல்லை 40 சதவீதம் ஆகவும் பிளாஸ்டிசிடி இன்டெக்ஸ் 20 சதவீதம் ஆகவும் சாதாரணமாக உள்ளது.

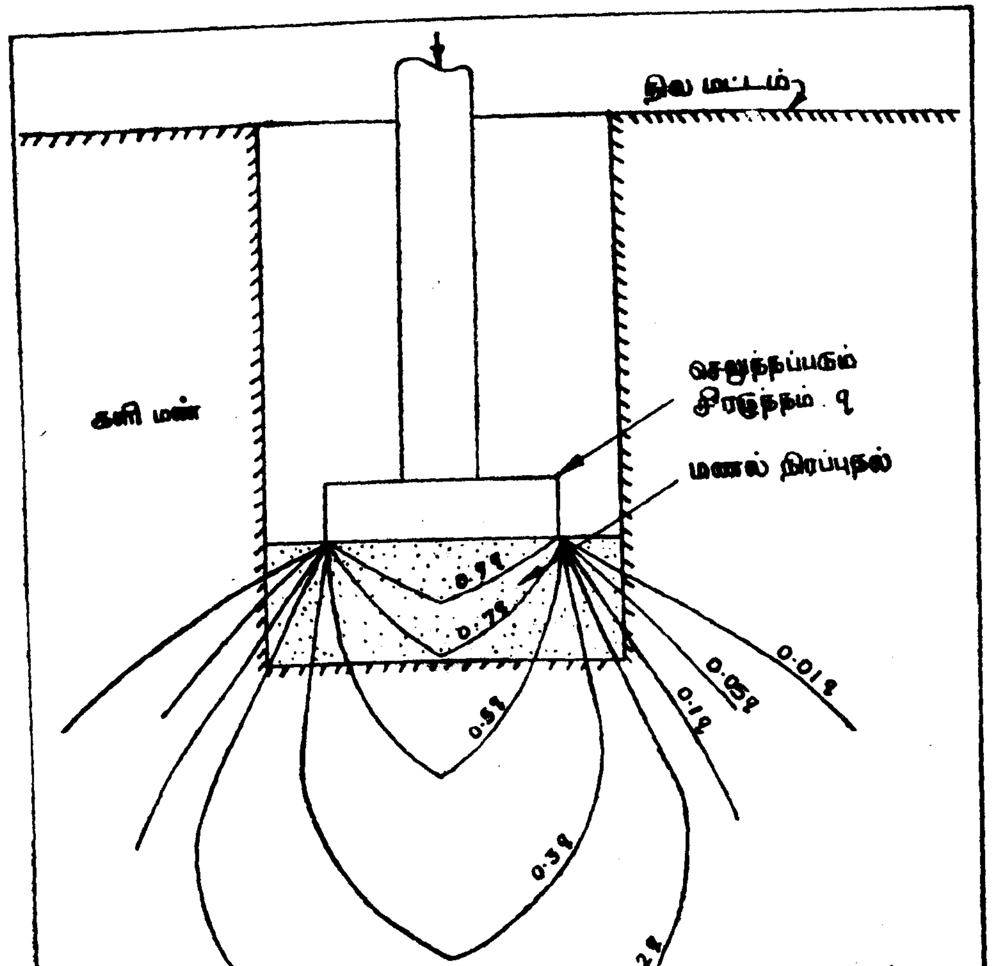
4.4. மண்ணின் தனிப் பெருக்கம் (Free Swell) 50 சதவீதத்துக்கும் மேலிருந்தால் அம்மண் வகைகள் தொல்லை வாய்ந்தவைகள் என்று பொறியாளர்கள் தெரிவிக் கின்றனர். மண்ணில் கலந்துள்ள களிமண் கனிமங்களின் (Clay minerals) அளவு, வகை ஆகியவைகளும் மண்ணில் பெருக்கமடையும் ஆற்றலை அறிய உதவுகின்றன. மாண்ட் மோரில் லோனைட், இல்லைட் ஆகிய இரண்டு களிமண் கனிமங்களும் மண்ணில் இருப்பின் அம்மண் வகைகள் உயர்ந்த கனப் பெருக்கம் அடையக் கூடும் என்பதைக் குறிப்பிடுகின்றன.

4.5. மண்ணில் துவக்கத்தில் உள்ள ஈரத்தின் அள வைப் பொறுத்தே அதன் நீள் பெருக்கமும் பெருக்கமடை யும் விசையும் இருக்கின்றன என்பது கண்டுபிடிக்கப் பட்டுள்ளது. மேலும், களிமண்ணில் ஈரத்தின் ஊடுரு வலோ அல்லது வெளியேற்றமோ மிகவும் மெதுவாக நடை பெறுவதால் அதன் பெருக்கமடைதலும், சுருங்குதலுக்கு மான நிகழ்ச்சிகளுக்கும் காலம் ஒரு முக்கிய கூறுக உள்ளது.

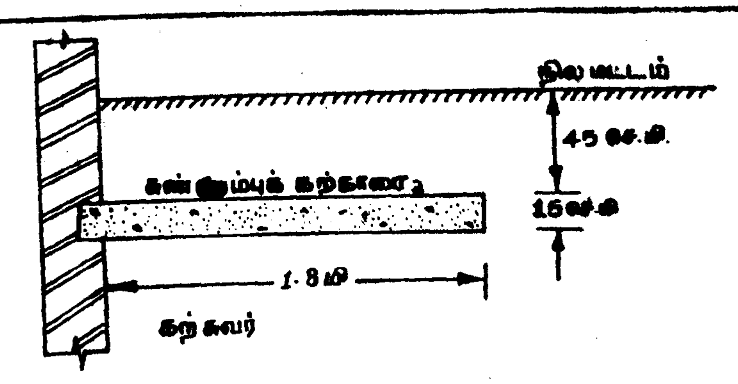
4.6. பொதுப்பணித்துறையின் சேப்பாக்கம் மண்ணியக்க வியல் ஆய்வுக் கோட்டத்திலும், நெடுஞ்சாலைத் துறையின் கிண்டி நெடுஞ்சாலை ஆய்வுக் கூடத்திலும் மண் பாகுபாடு கள் (Soil classifications) மற்றும் களிமண் பெருக்க அழுத்தம் (Swelling pressure of clays) பற்றிய சோதனைகள் நடத்தப்படுகின்றன.

5. விரிசல்கள் ஏற்படுவதைத் தடுக்கும் முறைகள்.

5.1. பெருக்கமடையும் தன்மை வாய்ந்த மண் வகை களின் மேல் கட்டப்படும் இலகு வகைக் கட்டிடங்களில் ஏற்படும் விரிசல்களைத் தடுக்கும் வண்ணம் எல்லா மண் வகைகளுக்கும் பொருந்துமாறு ஓர் வடிவமைப்பு முறை யைப் பரிந்துரை செய்வதென்பது இயலாத ஒன்றுகும். பருவகால மாற்றங்கள் மண்ணில் ஈரநிலையைப் பாதிக்காத

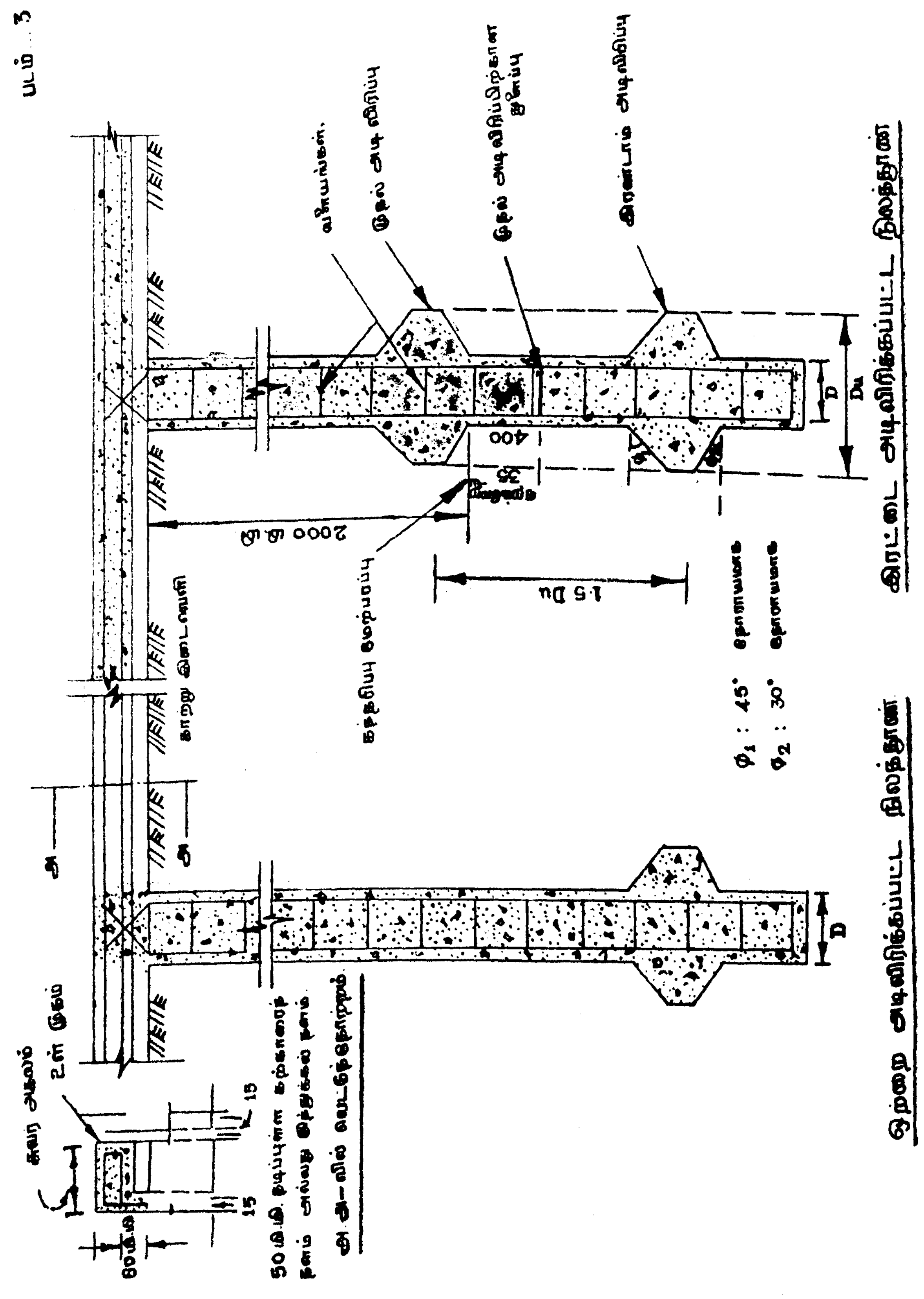


படம்... 1



நிலமட்டத்தின் கீழ் ஒட்டுத்தளம்.

படம்.2.



ஆழத்தில் அடித்தளத்தை அமைக்க முயல்வதும், அடித்தள மண்ணின் ஈரத்தைச் சீரான நிலையில் வைத்துக் கொள்ள முயல்வதும், அடித்தளப் பகுதிகள் சுற்றியுள்ள மண்ணைத் தொடாவண்ணம் மணல் இடைநிறைப்புகள் செய்வதுமே இச்சிக்கலை அணுகும் முறைகளாகும்.

5.2. விரிசல்களைத் தடுப்பதற்கான சில முறைகளை இனிக் காண்போம் :

5.2.1. அடித்தளம், அடித்தள மண்ணிற்குச் செலுத்தும் விசையானது அம்மண்ணின் பெருக்கமடையும் விசையை விடக் கூடுதலாக இருக்கவேண்டும். அதே நேரத்தில் அடித்தளம் செலுத்தும் விசையில் ஏற்படும் மண் இறுக்கத்தினால் விளையும் மண் படிவு 4 செ.மீ. மேல் இருக்கக்கூடாது. இதற்கேற்ற வகையில் கட்டிடமும் அடித்தளமும் வடிவமைக்கப்பட வேண்டும்.

5.2.2.1. பருவ நிலை மாற்றங்களை யொட்டி அடிமண்ணில் ஈரநிலை மாற்றங்கள் ஏற்படுகின்றன என்பது முன்பே கூறப்பட்டது. இம்மாற்றங்கள் ஒரு குறிப்பிட்ட ஆழம் வரையில் தான் ஏற்படுகின்றன. நமது மாநிலம் பொருத்த வரையில் பொதுவாக 12 அடி ஆழம் வரையிலுள்ள மண் தான் பருவ நிலை மாற்றங்களால் பாதிக்கப்படுகின்றது. அடித்தளத்தை இந்த ஆழத்தின் கீழ் அமர வைப்பது விரிசல்களைத் தடுப்பதில் மிகச் சிறந்த முறையாகும். ஆனால் பரப்பிய படி அடித்தளங்களை (Spread footings) இவ்வாழத்தில் அமைப்பது சிக்கனமற்ற முறையாகும். ரூர்க்கியில் உள்ள மத்திய கட்டிடப் பொறியியல் ஆய்வு நிறுவனத்தில் இதற்கெனப் புதியதோர் வடிவமைப்பு முறை உருவாக்கப்பட்டுள்ளது. இம்முறைப்படி, பொதுப் பணித்துறையின் கட்டிட மத்தியக் கோட்டத்தில் ஆய்வுச் சோதனைகள் நடத்தப்பட்டு, இம்முறையின் உறுதிப்பாடு உறுதி செய்யப்பட்டுள்ளது.

5.2.2.2. களிமண் நிலங்களில் தற்பொழுது அடி விரித்த நிலத் தூண்களின் (Under reamed piles) மேல் கட்டிடங்கள் எழுப்பப்படுகின்றன. இந்த நிலத் தூண்கள் 10 அடி முதல் 15 அடி ஆழம் வரை இடக் கட்டுமானம் (In-situ cons'ruction) செய்யப்படுகின்றன. இதற்குத் தனிப்பட்ட கட்டுமானச் சாதனங்கள் தேவையாகும். இச்சாதனங்கள் பொதுப்பணித்துறையின் சேப்பாக்கம் கட்டிட மத்தியக் கோட்டத்தில் உள்ளன. இச்சாதனங்கள் பொதுமக்களின் வேண்டுகோளுக்கிணங்க நியாயமான வாடகைக்கும் தரப்படுகின்றன. (படம் 1).

எல்லா நிலத் தூண்களையும் கூடத்தள மட்டத்தில் (Plinth level) ஓர் ஒருங்கிணைப்பு உத்திரம் இணைக்கிறது. இந்த உத்திரத்திற்கும் நில மட்டத்திற்கும் இடையில் 6" இடைவெளி விடப்படுகின்றது. இந்த உத்திரத்தின் மேல் கட்டிடம் எழுப்பப்படுகின்றது. ஒரு மாடி முதல் இரண்டு மாடிக் கட்டிடங்கள் வரை இம்முறையினால் கட்டலாம். வழி வழியான வகையில் கட்டப்படும் "பரப்புப்படி அடித்தளங்களுக்கு" ஆகும் செலவில் 40 சதவீதம் செலவுதான் இந்த முறையில் செலவாகின்றது. மேலும் இம்முறை கட்டிடத்தின் உறுதிப்பாட்டிற்கு உறுதியளிக்கின்றது.

5.2.2.3. தமிழ் நாட்டில் பல இடங்களில் பொதுப்பணித் துறை இம்முறையைப் பயன்படுத்தி பெருக்கமடையும் தன்மை வாய்ந்த களிமண் நிலங்களில் கட்டிடங்கள் கட்டியுள்ளது. களிமண் நிலங்களில் கட்டிடம் கட்டுவோர், அறியாமை காரணமாய் கடைகால் எடுத்த மண்ணையே மீண்டும் அடித்தளம் எழுப்பிய பின்னர் இடைவெளிகளில் நிறைக்கின்றனர். இது மிகப் பெரும் தவறாகும். களிமண் நிலங்களில் கட்டப்படும் அடித்தளங்கள் எக்காரணம் முன்னிட்டும் சுற்றியுள்ள மண்ணைத் தொட்டுக் கொண்டிருக்கக் கூடாது. அடித்தளக் கான்கிரிட்டுக்கு இரு பக்கமும் 1' 6" முதல் 2' 0" வரை இடைவெளி இருக்கும் வகையில் கடைக்கால் தோண்டப்பட வேண்டும். கடைக்காலின் அடியில் இரண்டடி ஆழம் மணல் நிரப்பி அதை நன்றாக ஒருங்கு சேர்த்து இறுக்கிய பின்னர் அடித்தளக் கான்கிரிட் அதன் மேல் போடப்படவேண்டும். அடித்தளக் கான்கிரிட்டும் பரப்புப் படிகளும் கட்டி முடிந்த பின்னர் இடைவெளிகளில் மணல் நிறப்பி ஒருங்கு சேர்த்துக் கெட்டிப்படுத்தவேண்டும்.

5.2.2.4. அடித்தளக் கான்கிரிட்டுக்கு அடியில் அமைக்கப்படும் மணல் படுக்கை மற்றொரு விதத்திலும் துணை புரிகின்றது. அடித்தள மண்ணிற்குக் கட்டிடத்திலிருந்து வரும் அழுத்தத்தின் பெரும் பகுதியை அது எடுத்துக் கொள்வதால் அடித்தள மண்ணிற்கு வரும் அழுத்தம் குறைகின்றது. மேலும் மாரிக் காலங்களில் மணலில் நீர் ததும்பி நிற்பதால் அது குறைந்த இடத்தையே அடைத்துக் கொண்டு அடித்தள மண் உப்புவதற்கு இடமளிக்கின்றது. வேனிற் காலங்களிலும் மணல் ஈரமாகவே இருப்பதால் அடித்தள மண் சுருங்குவதால் ஏற்படும் இடத்தை அடைத்துக் கொள்கிறது. ஆகவே, கோடையிலும் மாரிக்காலத்திலும் ஏற்படும் அடித்தள மண்ணின் நகர்வுகளை மணல் செவ்வனே சரிக்கட்டி விடுகிறது. (படம் 2.)

இம்முறையில் எழுப்பப்படும் அடித்தளங்களில் அடித்தளக் கான்கிரிட் உறுதிப்படுத்தப்பட்ட கான்கிரிட் ஆகவும் இருக்கலாம். அப்பொழுது பிளிந்த் மட்டத்தில் ஒருங்கிணைப்பு உத்திரம் தேவையில்லை. ஜன்னல் மட்டத்தில் ஓர் ஒருங்கிணைப்பு உத்திரம் இருப்பது போதுமானது. மேலும், கடைக்காலின் இரு பக்கங்களை யொட்டி ஒரு கல் செங்கல் சுவர் எழுப்பலாம். இது சற்றே சிக்கனமற்ற முறைதான். இருப்பினும் களிமண் நாளடைவில் மணலோடு கலந்து விடுவதை இச்சுவர் தடுக்கிறது. மேலும் மணல் மண் பொந்துகளில் சென்று விடுவதையும் இது தடுக்கின்றது.

5.3. அடிமண்ணின் ஈர நிலையைச் சீரான நிலையில் வைத்திருப்பது கட்டிட விரிசல்களைத் தடுப்பதில் பெரிதும் துணை புரிகின்றது. ஈர நிலையைச் சீரான நிலையில் வைத்திருக்கக் கையாளும் முறைகளை இனிக் காண்போம்.

(1) கட்டிடத்தைச் சுற்றி நிலமட்டத்திற்கு 1' 6" ஆழத்திற்குக் கீழே 6 அடி அகலமும் 4 அங்குலம் கனமும் உள்ள சுண்ணாம்பு கான்கிரிட் சுற்றுத் தளம் அமைக்கலாம். (படம் 3.)

(2) கட்டிடத்திற்குப் பக்கத்தில் புல் தரைகள் வளர்த்து மண்ணை எப்பொழுதும் ஈரத்துடன் வைத்திருக்கலாம்.

(3) நீர் நாட்டமுடைய மரங்கள் கட்டிடத்திற்குப் பக்கத்தில் வளர்ப்பது தவிர்க்கப்பட வேண்டும். இம்மரங்கள் அடிமண் ஈரத்தை அதிகப்படியாக உறிஞ்சுமாதலால் மண்ணில் ஈரம் சுண்டி சுருக்கங்கள் ஏற்படலாம்.

(4) கட்டிடத்திற்குப் பக்கத்தில் நீர் தேங்கி நிற்பது தவிர்க்கப்படவேண்டும். வடிகால்களுக்குப் பக்கத்தில் கட்டிட அடித்தளம் அமைப்பதையும் தவிர்க்க வேண்டும்.

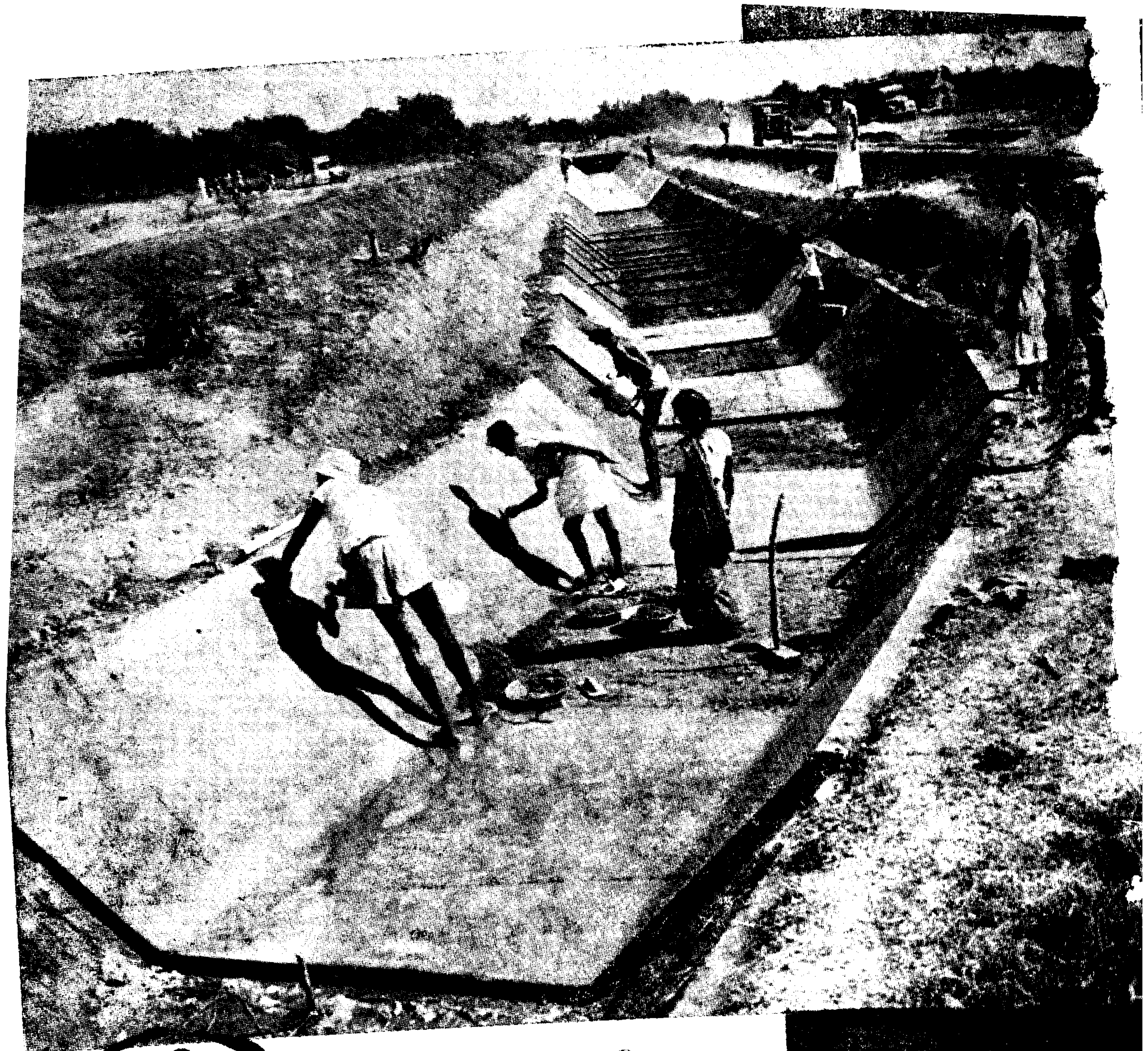
6. முடிவாகக் கூறுமிடத்து களிமண் நிலங்களில் கட்டிடம் கட்ட விரும்புவோர் :—

1. தங்கள் மனையிலுள்ள மண்ணின் பாகுபாட்டை முதலில் கண்டறிய வேண்டும். இதற்குப் பொறியியல் துறைகள், நிறுவனங்கள் ஆகியவற்றின் உதவியை நாடலாம்.

2. மண்ணின் பாகுபாடு கண்டறிந்த பின்னர் இதுகாறும் கூறிவந்த வடிவமைப்புகளைப் பின்பற்றி அடித்தளங்கள் அமைக்கலாம். மண்ணியக்கவியல் பொறியாளர்களின் (Soil Engineers) ஆலோசனைகளைப் பெறுவதும் இன்றியமையாததாகும்.

SILVER JUBILEE CLEBRATION IN IRRIGATION RESEARCH STATION-POONDI.

Works Minister Presenting Gold Medal to a Worker in Irrigation Research Station Poondi.



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