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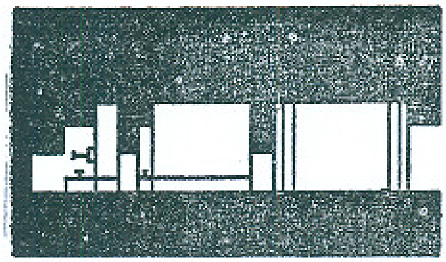
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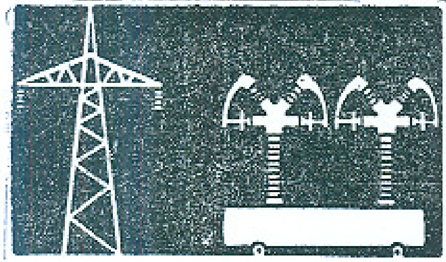
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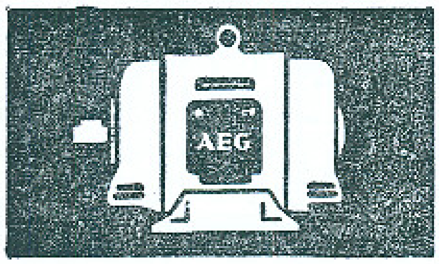

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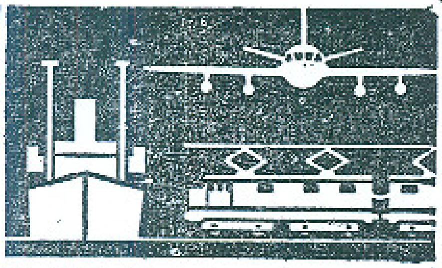
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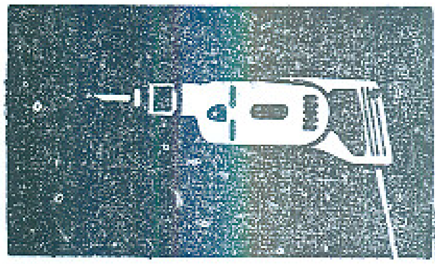
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• Contributions to this journal in the form of articles, news of engineering works, news about engineers, photographs and technical data etc. are cordially invited.

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TWELFTH YEAR OF PUBLICATION

ENGINEERING NEWS

Quarterly Journal of the West Pakistan Engineering Congress

Vol. XII DECEMBER 1967 No. 4

In this issue

MANGLA DAM

A MARK OF NATIONAL PROSPERITY

THE 23rd of September, 1967, will ever be remembered as a great national day in Pakistan's progress towards prosperity. With the completion of Mangla Dam, costing Rs. 322 crores, the nation is proud to possess one of the world's largest Earth filled dams. Within seven years, a marvelleous feat of engineering has been completed. We will now be able to store 4.75 million acre feet of water which corresponds to a perennial flow of about 6600 cusecs. This volume of water does not include the dead storage of 1.75 maf. kept to entrap the silt of the river. Nearly one quarter of the total flow of 22 million acre feet of the Jhelum will be conserved for use during the low flow periods. This dam now generates 200 MW of power which will be raised to double this capacity by the end of 1969. By 1973 the total power generated at Mangla will be 800 M.W. At that time the thermal generation in addition will be only 691 M.W. At least for the next 120 years, up to the expected useful life of the storage, the country will continue to have cheap hydro-power. It will be available even after this period, though with some reduced power factor.

When the Indus Basin Treaty was finalised in 1960, the public opinion in Pakistan was critical on the transfer of three Eastern rivers of Pakistan to India. If this treaty be now considered in the light of achievements and gains for West Pakistan, even the severest critic will find that the Treaty was in fact a major achievement of the present regime. Without this Treaty it would not have been possible to complete the Mangla project within a short period of seven years. Bhakara Dam in India has taken fourteen years for its completion.

The Treaty provides 9.3 million acre feet of storage at Tarbela on the Indus. It will generate about 2000 MW of power which will be available practically for all times to come, even though the dam is filled up, 40 to 60 years after its operation.

A flow of one cubic foot of water, in a year, adds about Rs. 25,000 to the wealth of the nation. We are losing annually nearly 80,000 cusecs, roughly 40% of our total surface water resources of West Pakistan. The construction of Mangla and Tarbela Dams will conserve about 15 maf. of water which at the present rate will add nearly Rs. 52.5 crores to the national exchequer every year.

The Indus annual flow at Tarbela is about 84 maf., and the major loss is out of this river. Construction of Tarbela will not only save wastage of water but will open out a way for diversion of the water of the Indus into new storages in the Soan Valley.

The utilization of ground water is being undertaken on a large scale. This is a result of the development of cheap design of tubwells. Their number is increasing so rapidly that very soon we will find shortage of ground water. The present major source, feeding the ground water aquifer, is the unlined canals. New links are unlined and flow across the doabs. These will thus be adding a large volume to the ground water.

These are the three outstanding gains to the nation. We will save some part of the present wastage of water worth Rs.52 crores per year, will store a large volume in the underground, and will have cheap hydro-electric power.

The Treaty has made West Pakistan independent of an aggressive neighbour bent upon utilizing every opportunity to create mischief. Depending upon such a country for our water requirements would have been a perpetual headache.

Wealth begets wealth. An expenditure of about Rs. 322 crores on Mangla and Rs. 450 crores on Tarbela will open many ways for the future prosperity of the country. Within 15 years the Nation will be wealthier by conserving 15 maf of surface run off, 3.0 maf of addition to ground water from new links and about 10 maf additional recharge due to the increase in the agricultural intensity. The Hydro-electric power is yet another corollary of this venture. The Nation should rejoice at the successful completion of a huge programme of prosperity initiated with the dedication ceremony at Mangla on 23rd of September, 1967.

Mr. S. S. Kirmani

Chief Engineer Indus Basin Works
(A biographical Sketch)

The Irrigation system of West Pakistan is unique in the world. Large rivers have been completely controlled and trained and their flow diverted through several barrages and canals to agricultural lands. Triple canals, Sutlej Valley canals, and Sukkur Barrage canals are a few examples of the marvellous system completed in record times. Such huge Engineering Works are the outcome of day and night labour of a few genius among the Engineers. Unfortunately printed biographical sketches of the Engineers who devoted their energies to complete such marvels of Engineering are not maintained for inspiration of the young Engineers. One such example of the present-day genius is that of Syed Salar Kirmani, Chief Engineer, Indus Basin Works, who heads a big organization. He has immortalised the 'Story of Mangla Dam' through the article which ensues.

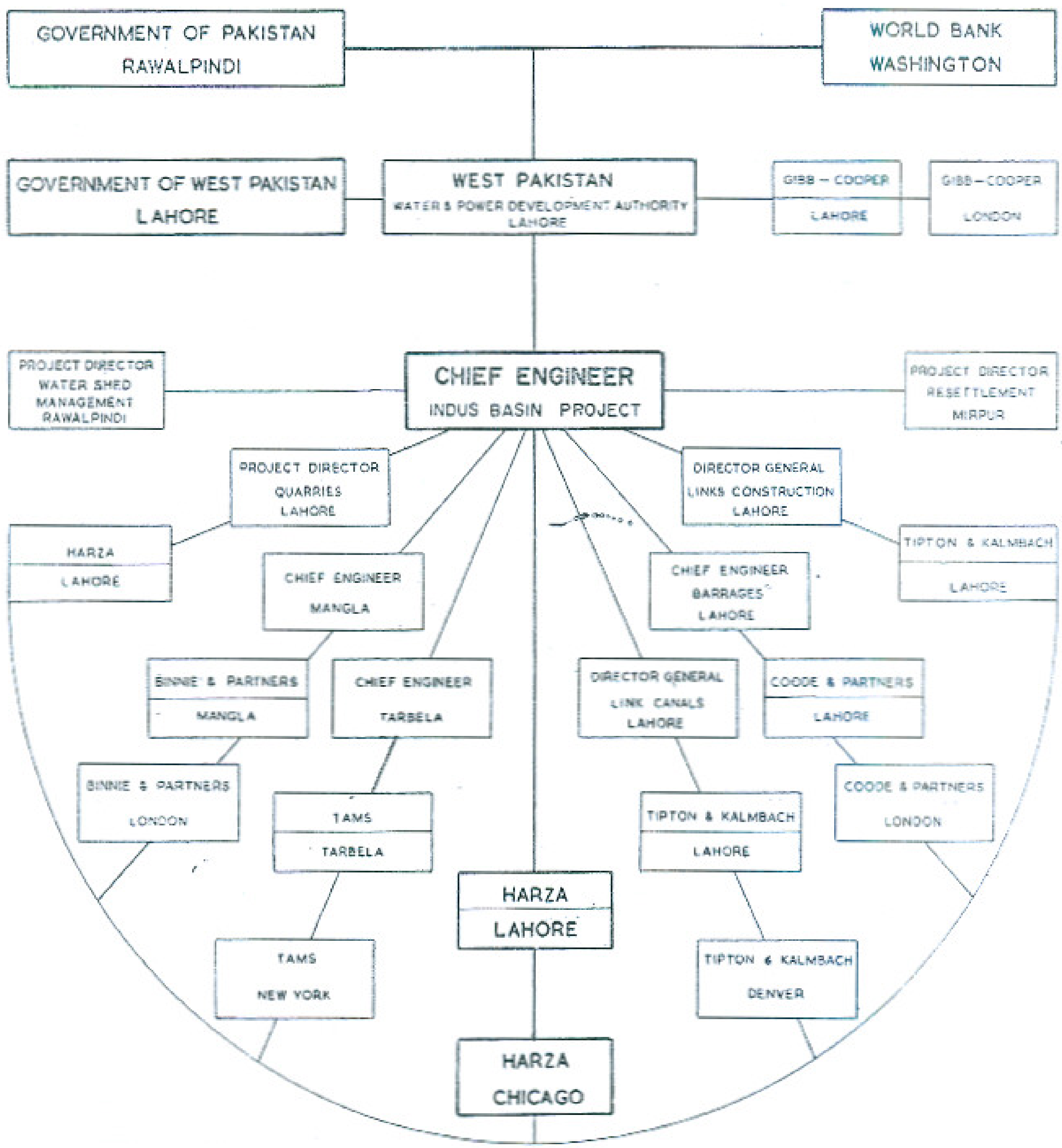
It will be a great service to the country when Mr. Kirmani pens down an account of the stresses and strains of the last eight years when making decisions for such important works.

We have reproduced briefly the rise of the great Engineer which may be an envious goal to many young in the profession.

Mr. Kirmani was born on 1st July, 1921 in Cuntur District of Madras and was educated at Andhra Christian College. He graduated in Civil Engineering with honours from the Engineering College of Guindy, Madras and joined the Irrigation Branch, Punjab Public Works Department in 1944 as an Assistant Engineer. Luckily he was assigned to the Central Design Office where his special talent and intellect began to take shape. His first important assignment was the design of Rasul Hydro-electric Project and the Ravi Syphon for the design of which he worked for about a year in 1949. At the young age of 29 he was promoted as Director, Design, for the planning of Irrigation and Hydro-electric Power in the Former Punjab.

In the year 1954 he represented Pakistan as an Official Delegate to the Third International Irrigation and Drainage Conference and the same year he was promoted as Superintending Engineer. This was a chance for him to show his intellect and the

INDUS BASIN PROJECT ORGANISATION



deep understanding of the problems of Irrigation, so that in December, 1954 he was sent to America as a Member of Pakistan Water Delegation in connection with the Indus Water Dispute, being resolved under the good offices of the International Bank of Reconstruction and Development. On return to Pakistan after a brief stay in the Dam Investigation Circle, he was sent by the Irrigation Department to West Pakistan, WAPDA as Director, Planning and Investigation. He was again called upon for a brief visit to U.S.A. as a Member of Pakistan Water Delegation for Indus Water Dispute, and on return in 1960, was appointed Director Indus Basin Projects.

In March, 1961 he was awarded the title of Sitar-e-Quaid-i-Azam and the next year in May, 1962 he was put in as Director-General Indus Project and later on in August of the same year promoted as Chief Engineer.

In November, 1967, *he was again awarded the* Sitara-e-Imtiaz.

As Chief Engineer Indus Basin Works, his responsibilities include the completion of two large storage dams, eight Inter-river link canals, six barrages and remodelling of a large number of Irrigation Works.

We look forward to the story of the man who made the story of Mangla Dam and the Indus Basin Work a reality of the dream. Let all praise be for the great genius of Mr. Kirmani!

INDEX TO ADVERTISERS

The Story of Mangla Dam

By

S. S. KIRMANI

November 23, 1967 will always be remembered as a landmark in the history of development of Water and Power Projects of West Pakistan. This day marks the completion of one of the marvels of construction in this country. Mangla Dam costing Rs. 322 crores has been completed in less than seven years, one year ahead of schedule.

Mr. S. S. Kirmani has put forth the full story of Mangla Dam from its inception to finish in November, 1967 issue of the Indus. We have abstracted the main features for the information of our readers.

India's unilateral action

Pakistan was hardly organized to face the new responsibilities when, all of a sudden on April 1, 1948, India shut off water supplies from Ferozepur Headworks to the Dipalpur Canal and from Madhopur Headworks to the fourteen irrigation channels of the UBDC serving Lahore and parts of the Sahiwal districts, alleging that Pakistan had failed to renew the "Standstill Agreement" signed on December 18, 1947, which provided, among other things, that the pre-Independence allocation of water in the Indus Basin Irrigation System would be maintained. By India's action about 10 percent of the sown area in West Punjab (almost 6 percent of the irrigated area in the whole of West Pakistan) at that time found itself without water at the beginning of the Kharif sowing season. India also cut off simultaneously the power supply from the Mandi Hydro-electric scheme which was the major source of power to Lahore, Sheikhupura and Lyallpur districts, paralysing what little was left of the industrial activity in the Province.

Negotiations with the Bank

In May, 1952, the first round of negotiations started in Washington between the Indian and Pakistan delegations under the good offices of the World Bank to find a solution of the water dispute on the basis of Mr. David Lilienthal's proposal, which visualised the solution in the creation of a surplus of water over and above Pakistan's existing uses by the construction of storage facilities within India where the best dam sites were located, with Pakistan contributing to their cost and with both countries agreeing on the design and operation of an integrated system.

The World Bank, therefore, suggested that each delegation prepare and submit a comprehensive plan. The respective plans were submitted in October, 1953. Pakistan's plan allocated the three western rivers plus 70 percent of the eastern rivers to Pakistan. Under the Indian plan all the eastern rivers would go to India along with 7 percent of the waters of the western rivers. The two approaches were so divergent that the Bank found it impossible to find a solution acceptable to both and in February 1954 announced its own plan allocating eastern rivers to India and the western rivers to Pakistan. A comparison of the three plans is given below :—

	Total uses excluding losses and unusable supplies (MAF).		
Plan	For India	For Pakistan	Total Usable
World Bank	22	97	119
Indian	29	90	119
Pakistani	15.5	102.5	118

India immediately accepted the Bank's proposal, but Pakistan contested its basis and assumptions.

The Settlement Plan

In May 1959 the Bank prepared its own plan called the "Settlement Plan" which included a 5.35 MAF reservoir at Mangla, a 2 MAF reservoir at Rohtas and a system of link canals as proposed by Pakistan in her London plan. Pakistan accepted the Settlement Plan after the Bank had agreed to modify it to include Tarbela Dam in place of Rohtas. Mangla and Tarbela provided a gross storage of 10.45 MAF for complete replacement plus additional new development on the basis of which Pakistan agreed to waive her claims on sailab and channel deterioration works as well as the recurring maintenance and operation costs. The Indus Waters Treaty was signed in September, 1960 after the Bank and the Governments of U.S.A., Canada, U.K., West Germany, Australia and New Zealand agreed to underwrite the cost of the plan which was then estimated at Rs. 400 crores. The Settlement Plan is more familiarly known as the Indus Basin Project or the I.B.P.

Mangla Contracts

Tender documents for the main contract were issued on June 1, 1961 to eight prequalified consortia. The following four consortia submitted their tenders which were opened in London on November 15, 1961 :—

Sponsor	Country	Tender Price (crores of Rs.)
Guy F. Atkinson	USA	168.55
Morrison-Knudsen	USA	178.33
Utah Construction	USA	181.25
J. A. Jones	USA	291.65

The Engineer's estimate was Rs. 185 crores. The contract was awarded on January 20, 1962 to the consortium sponsored by Guy F. Atkison who subsequently adopted the name of Mangla Dam Contractors (MDC).

The Mangla Contract is the largest single contract in the history of civil engineering.

It required a performance guarantee of 75 million dollars, the largest sum ever specified in any construction contract. When US and UK banks and insurance companies expressed their inability to provide a bond of this amount to MDC, the amount was reduced to 25 million dollars.

In order to ensure effective competition, separate mechanical and electrical contracts were let out by subdividing the work into supply contracts which permitted bidding by prequalified specialist firms. The civil contractor was made responsible for taking delivery of the equipment from the suppliers' factories, transporting it to site and installing, testing and commissioning the plant.

The following were the successful bidders for the supply contracts :—

under which they were resolved but by their net effect and results and thus atribute them to lack of foresight and competence or to inefficiency and negligence. What is amazing is not so much the complexity of the problems encountered but the ingenuity, speed and courage with which WAPDA and its Consultants resolved them and still completed the job one year ahead of schedule.

The Transport Crisis

The first major construction effort consisted of building the Baral Colony, the labour camps, community facilities, warehouses and workshops. By May 1962 huge consignments of heavy equipment stores and commissary goods started arriving at Karachi. A serious crisis developed on transportation of these goods as similar

Contract	No. of Tenders	Successful Contractor	Contract Price (Million Rs.)	Date of Award
Turbines	7	Mitsubishi, Japan	14.66	Jan. 3, 1963
Generators	9	Hitachi, Japan	10.13	Feb. 6, 1963
Transformers	28	Savigilano, Italy	4.17	June 1, 1963
HV Switchgear	9	Brown Boveri, Switzerland	13.66	May 21, 1964
Cranes	13	Hitachi, Japan	1.90	Sept. 18, 1963
Intake Gates	7	Voest, Austria	6.75	Feb. 21, 1963
Spillway Gates	6	Krupp, W. Germany	6.74	Apr. 20, 1963
Gates of Bong Escape	13	Hitachi, Japan	0.86	Jan. 10, 1964
MV and LV Switchgear	9	Pak Electron, Pakistan	1.10	Sept. 16, 1964
Cables	4	BICC, UK.	4.50	Sept. 11, 1964

The Crises at Mangla

Mangla had its share of problems some of which assumed the proportions of a serious crisis. People are apt to judge them in retrospect, not so much by the circumstances which created them or by the limitations

demands of five other major contractors of the TSMB System had to be simultaneously met. Fortunately WAPDA anticipated the crisis 18 months in advance and made arrangements with the PWR to import 23 locomotives and 2000 wagons through a credit from the World Bank to handle the IBP traffic. For Mangla alone some 55,000 wagon-loads of goods and plant were transported and during the early period three goods specials were operated daily from Karachi to the site. The co-operation extended to WAPDA by the PWR, the CCIE and the Customs was most gratifying.

Free Meals to Labour

A novel practice of providing free meals to the labour in addition to full wages was introduced. Feeding an army of over 10,000 labourers three times a day was no small task. WAPDA wondered about the prudence of this practice which was unprecedented on construction jobs in Pakistan. No other foreign contractor followed it. MDC's philosophy was that if the labour was not properly fed, their output would be adversely affected. There was logic in the philosophy but the practice could have undesirable repercussions on other jobs. WAPDA also wondered whether MDC could continue this practice throughout the job and satisfy the varied food habits and tastes of thousands of people. Even if they did the labour might feel dissatisfied in course of time with the quality of food and demand variety and improvements. It was a delicate issue for WAPDA as it would have been unwise to interfere with the Contractor's methods.

In course of time WAPDA could see the advantage of MDC's philosophy although it could not follow it nor advocate it to other IBP contractors. The labour was healthier and their stamina and performance superior to that on other projects.

The Mechanical Mole

The completion of the combination diversion and power tunnels at Mangla was perhaps the most critical phase of the whole job. The five tunnels, each 36 ft. in diameter and spaced 75 feet apart, centre to centre, and splaying to 130 feet apart at the power house portal, had to be driven, each 1,650 feet through the hill involving a total tunnel excavation of 340,000 cubic yards. A delay slowing down in the schedule for excavation and lining of the tunnels could conceivably set the entire project back by a full year. Original design contemplated tunnel excavation by explosives and conventional drifting techniques. The Contractor, however, selected to undertake the job with a giant mechanical "mole", manufactured specially for this job by the firm of James S. Robins and Associates in the USA.

After a learning period in the first tunnel during which crews were selected and trained, modifications were made to the "mole" and its overall operation perfected, the tunnel excavation moved ahead impressively with a record daily run of 105 feet of the 36 feet diameter tunnel and on several days an average progress of 60 feet per day was achieved. The first tunnel was driven in 16 weeks, the second in eight weeks, the third in seven and the fourth and fifth in five weeks each.

The Crisis of the Sheared Clays

In 1963 when excavations of the dams were exposed, it became evident that the shear strengths of the clay bed-rocks were lower than had been found or assumed for the contract design. Undisturbed block samples were taken from these areas and an extensive programme of sampling and testing was started. In mid-1964, the Consultants, in conjunction with Harza Engineering

International and Professor Skempton, a leading world authority on soil mechanics, worked out new design parameters from the recent test results and carried out a series of stability studies to check the factors of safety which were found to be seriously less than those required by the design criteria. The Consultants recommended major changes in the contract designs which included steel lining of Tunnels; additional drainage adits with wells, extra gravel fill, upstream toe-weight and blanket, and flattening the slope of excavation in the intake Dam area; change of clay core to rolled sandstone core, provision of wide upstream berms, excavation of clay bed-rock from foundations to depths up to 50 feet and back-filling with rolled sandstone for the Sukhian Dam; adits with drainage wells at the headworks, extra excavation of bed-rock on both sides of approach channel, extra excavation on the right side slope of upper and lower stilling basins, vertical deep pumped wells and a drainage system in the lower stilling basin of the Main Spillway; and upstream and downstream toe-weights for the Main Dam. WAPDA was perturbed at the belated revelations of the test results, the enormous additional costs of the measures proposed and their possible effects on the construction programme. On the other hand, the safety of the dam was of paramount importance and the need for change in designs at any stage in the construction of works in the light of new evaluation of the strength and properties of the soil foundations could not be compromised regardless of the cost and its other effects.

Before accepting the Consultants' Recommendations, WAPDA wanted to have a second opinion and invited Professor A. Casagrande and Mr. F. B. Slichter of U.S.A. and Dr. R. Peterson of Canada to review the proposals. Due to the non-availability of Dr. Casagrande and Mr. Stichter, however, a Review Board consisting of Messrs. P. T. Bennett, D. I. Bleifus, T. M. Leps and R. Peterson was appointed to review all site investigations and tests, the adjustment proposed to the design parameters and the remedial measures suggested by the Consultants. The Board visited the Mangla site during November 2—15, 1964 and submitted a detailed report which *inter alia* concluded that :—

"It is our judgement that design changes are necessary in view of the lower foundation strengths recently discovered. We agree that construction should proceed on the general basis of the design changes proposed."

In view of the categorical recommendation of the Board, WAPDA proceeded with the design changes. The effect of these changes on the cost of the project was phenomenal. The total increase in cost amounted to nearly Rs. 12 crores of which the major increases were in the costs of Intake Dam, Tunnels and Sukhian Dyke. Fortunately the additional works did not interfere with the "critical path" of construction schedule and the Contractor's superb construction planning and performance helped in completing them without adversely affecting the final targets.

River Diversion and the War

On September 6, 1965, India suddenly attacked Pakistan and a major war followed. Wapda was faced with a difficult decision.

If the diversion was delayed even by a few weeks it may have been impossible to raise the closure dam quickly enough to be safe against the winter flood.

In these circumstances, WAPDA, in consultation with the Government, made a very bold but difficult decision to proceed with the diversion. The diversion started on 14th September and by 16th September the river was successfully closed. Fortunately the war ended on 23rd September and raising of the closure dam continued without interruption. All the targets for raising the closure dam to elevation 930 by 1st November, 980 by 1st February, 1,020 by 1st March and 1,080 by 1st June were achieved ahead of schedule and the Contractor earned full bonuses for all stages amounting to one crore of rupees. WAPDA's bold approach avoided crores of rupees of potential claims from the contractor and made possible a year's storage and power benefits to the country worth over 60 crores of rupees.

The Crisis on Fabridam

Model studies on the hydraulic operation of the diversion tunnels indicated that although the tunnel exits were deeply submerged at all stages and water level in tailrace controlled by river level immediately downstream, yet hydraulic jumps formed in the tunnels for a range of river discharges between 25,000 to 100,000 cusecs under conditions of part-full to full bore flows. The air entrained in the jump collected below the soffit of the tunnel at the downstream end and there was intermittent "blow-back" through the jump or "blow-out" at the exit causing surges and rapid fluctuations in local pressure. These conditions, it was feared, could possibly cause cavitation and damage the concrete lining.

The Consultants thus recommended the construction of flexible dam in the tail-escape channel at a cost of Rs. 45 lakhs. It was put into operation in September 1965 and in March 1966 it failed. Immediate arrangements were made to construct a rock weir at a cost of Rs. 17 lakhs. These protective measures to control the hydraulic jump were found satisfactory.

Reservoir Impounding

The impounding of water in the reservoir was started as soon as the low level bulk head tail of the channel was closed. For fixing the period and date of impounding the reservoir, considerable studies with regard to the stages of the river had to be carried out. The previous 32 years' records of the river were studied and the help of Meteorological Department was utilized to determine the best date for the start of the impounding. It was started on 21st February. The discharge of the river and the rise of water level in the reservoir was anxiously awaited. At some periods it was decided to open the Irrigation valve to release 9000 cusecs for the Irrigation but the huge silt deposited in the tail reach made it impracticable. Finally on 21st March the water started flowing over the spillway. In all, the river was closed for 28 days.

Post impounding Problems

The 40 feet long turbine shafts had tilted towards the downstream side by 50 thou (thousand of an inch). Experts thought that the tilting was caused by the huge silt deposits in the tailrace which were 80 feet deep at places. Sixty scrapers, 3 shovels and one dredger worked continuously day and night for 35 days to remove the 1.2 million cubic yards of silt deposits. The shafts returned to vertical position, but by the time silt was completely removed, they tilted again on the reverse side by 70 thou. The tilt could be adjusted but it would delay the commissioning of the sets by 3 months. WAPDA could not afford this delay as Multan Power Station was working at low capacity and the new Lyallpur Plant was behind schedule. The Consultants advised that it was safe to commission the Mangla sets as many power plants had operated satisfactorily elsewhere with tilts up to 300 thou. The first set was successfully commissioned on 3rd July and the second set on 14th July.

The Success

"Verily, never will God change the condition of a people until they change it themselves."

The Jhelum river had been tamed and was no longer a wild stream. Despite a war, extensive design change and major crisis during construction, Mangla was completed in a record time of a little over 5 years, a full year ahead of schedule. It was a great achievement. Bhakra Dam in India took 14 years to complete and the construction of the high Aswan Dam in Egypt, which started in January 1960, two years before Mangla, would not be completed until the end of 1968. Talking about the success of the IBP., Mr. S. Aldewereld, Vice-President of the World Bank, said :—

"The Bank has financed 164 projects all over the world. We have been associated with the Indus Project for the past seven years and our experience was that of all the projects which the Bank has financed and administered, the Indus Project was the most successful. This project was a model of efficiency, of good engineering and of administration. The Bank has disbursed some 900 million dollars on this project so far, but there was never a question, never a dispute and never a failing on this project. Whenever I spoke about this project to the US Government and the European countries, I told them that it was a model for all countries to follow. The Bank is proud of the achievements and wants to maintain the momentum and to apply it for Tarbela."

New Horizons

Recent operation studies have revealed the great potentialities of the Mangla reservoir in providing timely irrigation supplies to the 17 million acres of crop lands served by the Jhelum, Chenab, Ravi and Sutlej and the improvements in crop yields that will be possible even in a very dry year to a level far beyond that achieved under the most favourable river conditions in any year in the past. The total Rabi supplies in the Jhelum and Chenab amounting to 10.3 MAF in an average year and 7.3 MAF in a very poor year will be increased by 3.3 MAF, an increase of 50 to 72 percent. The most significant contribution of Mangla is the effective distribution and efficient utilization of the waters of Jhelum and Chenab during the critical Rabi and Kharif seasons which will have a tremendous impact on the agricultural economy of the country. Mangla heralds a new era in irrigated agriculture and failure of crops due to the vagaries of the weather will be a story of the past.

As the reservoir is filled during the flood months, considerable quantity of water will seep into the porous formations along its 250-mile long shoreline and will be stored as bank storage which will return in the reservoir in the winter months when the levels are drawn down. Preliminary studie carried out by the author indicate that the bank storage releases in the Mangla reservoir could be as much as 0.3 to 0.5 million acre feet. This additional source of storage will greatly enhance the benefits from the reservoir.

When Mangla is raised in the future, it will provide an additional storage of 3.7 MAF. The 8 MAF off channel storage reservoir at Rohtas will provide adequate replacement in the future for the loss of Mangla capacity due to silting. These potentialities ensure the availability of Mangla storage for several centuries.

According to the latest load forecasts, the peak power and the total energy demand in the Northern Grid will increase to 1.043 MW and 4,900 million kwh by 1970 and 1,830 MW and 8,500 million kwh by 1975. The importance of Mangla for meeting the growing needs of power in the Northern Grid is evident from the following programme for installation of power generating units in the system.

Pakistan made a great sacrifice in agreeing to a solution of the Water Dispute based on the division of the rivers in the interest of peace. Apart from this fact, the Treaty gave Pakistan freedom to plan and develop the water resources allocated to her in a manner best suited to her needs and provided a system of works which will enable her to replace the water lost to India and also maintain progressive development of the water and power resources. If Mangla Dam is viewed on the basis of what it achieves and the promise it holds for the future, there is justifiable cause for satisfaction.

Year	Power Plant	Capacity (MW)		
		Additions	Cumulative	Total Mangla
1966	Existing. ..	522	522	..
1967	Mangla Units 1 and 2 ..	200	722	..
	Lyallpur-steam	132	854	200
1968	Lahore Gas Turbines ..	52	906	..
	Mangla Unit 3 ..	100	1006	300
1969	Mangla Unit 4 ..	100	1106	400
1971	Mangla Units 5 and 6 ..	200	1306	600
	Retire Thermal Plants ..	(—)15	1291	600
1973	Mangla Units 7 and 8 ..	200	1491	800

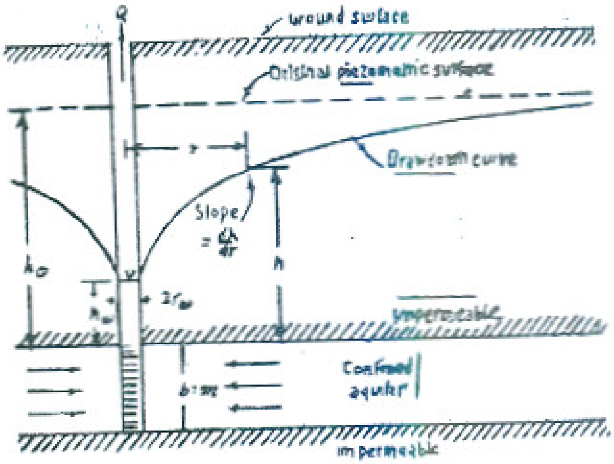
Hydraulics of Tubewells

By
Dr. NAZIR AHMAD

Artesian and Watertable Aquifer

Alluvium deposits are always stratified so that gravel, sand, silt or clay exist in layers of various thickness and extent. Sometimes a sandy material of the same grade also gets deposited in layers. These granular particles can store water in their pores. There are sites where water-bearing zone is enclosed between two relatively impervious layers of clay. If these layers are extensive and of sufficient thickness, then under certain conditions of location, the enclosed water is found to be under pressure. The water-bearing sandy medium is called an aquifer, and if confined within two impervious layers, the medium may exhibit artesian pressure. Such a condition of aquifer is shown in Fig. 1. If a pipe is installed in the aquifer, water will rise up in it equal to the pressure in the zone. Such area of artesian pressure can exist in the foothill districts of Sialkot, Gujrat, Jhelum or mountainous areas of Potwar, Rawalpindi, Haripur, Campbellpur etc.

In the Indus plains the confining layers are not very extensive and the artesian conditions of flow are rarely met with. The clay



CONFINED AQUIFER

Fig. 1

formations do not enclose the aquifer under pressure of water. A pipe installed in the formation exhibits the same position of water level as it exists in the formation. This type is called watertable condition. It exists generally in the Indus plains of the former Punjab and Sind (Fig. 2).

At certain sites where a high level source of water such as a canal exists close by, a confined watertable may exhibit a pressure of a few inches only.

The groundwater in the Indus Plains is

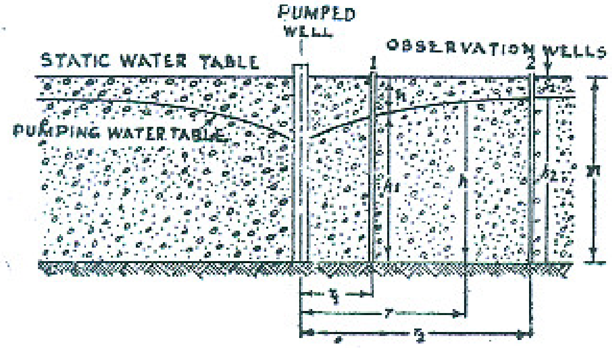


Fig. 2.—Aquifer with watertable conditions.

generally under watertable conditions.

Flow into a Well

According to Darcy's[1, 2] law the velocity of flow through any concentric cylindrical section of the water-bearing material of unit thickness is given by

$$V=P\ I$$

where I is called the hydraulic gradient and is the ratio of $\frac{h}{r}$ or $\frac{dh}{dr}$ for very small water head considered at a small distance, from a tubewell.

P or K is the permeability coefficient of the medium. For a concentric cylindrical section, the flow of water is expressed as :

$$Q = VA$$

$$= P\ I\ A \quad ..(1)$$

where A is the area of a section and is equal to

$$A = 2\pi r h$$

$$\text{or } Q = P.\frac{dh}{dr}.2\pi r h$$

$$\text{or } \frac{dr}{r} = \frac{2\pi P}{Q}.h,\ dh \quad ..(2)$$

Refer to fig. 2 in which the various parameters are explained.

Integrating between the limits r_1, r_2, h_1 and h_2, we have

$$\int_{r_1}^{r_2} \frac{dr}{r} = \frac{2\pi P}{Q} \int_{h_1}^{h_2} hdh$$

$$\text{or } \log_e \frac{r_2}{r_1} = \frac{\pi P}{Q}(h_2^2 - h_1^2)$$

now $h_2 - h_1 = s_1 - s_2$

and $h_2 + h_1 = 2m$

The above equation gives

$$Q = \frac{2\pi Pm\,(h_2 - h_1)}{\log_e \frac{r_2}{r_1}} \quad ..(3)$$

In case $h_2 = h_o$, $r_1 = r_w$, $r_2 = R$ we get

$$Q = \frac{2\pi Pm\,(h_o - h_w)}{2.303\ \log_{10}\frac{R}{r_w}} \quad ..(4)$$

R is the distance of the influence of a tubewell and $h_o - h_w$ is the lowering of water level or the depression created within a working tubewell with reference to the position of the original watertable. The thickness of water bearing medium is taken equal to m.

The above relation holds under the condition that

(*i*) the well is fully penetrating and a confining layer exists at bottom.

(*ii*) the depression $h_o - h_w$ is very small in comparison to m. In case of tubewells installed in this region, m is 200 to 300 feet and $h_o - h_w$ is 10 to 20 feet. Under these conditions the above relation shows that

(*i*) the discharge is directly proportional to the thickness of the water-bearing formation or under certain condition for full penetration equal to the length of the strainer.

(*ii*) is directly proportional to the average permeability coefficient of the formation being pumped.

(*iii*) is directly proportional to the depression head, $h_o - h_w$ and

(*iv*) is proportional to $\dfrac{1}{\log \dfrac{R}{r_w}}$ where R is the zone of influence of the tubewell. Different workers have assumed different values for R. Musket has used R equal to 500 ft. but for the formation existing in the Indus Plains the assumption of R equal to 1000 ft. for medium sand and R equal to 2000 feet for very fine sand can be adopted.

The factor $\dfrac{1}{\log \dfrac{R}{r_w}}$ is logarithmic so that a change in the value of R from 1000 feet to 2000 feet causes a change of only 8 to 10 percent in its value. The factor worked out for a few values of R and r_w is shown below.

TABLE 1

Value of the factor $\dfrac{1}{\log \dfrac{R}{r_w}}$ for

R in ft.	r_w=4 inches.	r_w=8 inches.	r_w=12 inches.
1000	0.26	0.28	0.30
2000	0.24	0.26	0.27

For a confined aquifer, the relation between yield and other Parameters is identical. In this case the thickness of the aquifer, *b* is taken equal to the water-bearing medium, *m*.

Diameter of a tubewell and its discharge

Suppose at a site two tubewells are installed having the same depth but of different diameters and these are worked under the same order of depression, then $2 \pi P m (h_o - h_w)$ is a constant. Their yield can be compared according to the relation :

$$Q_1 = \frac{X}{\log_e \dfrac{R}{r_{w_1}}} \quad \text{and}$$

$$Q_2 = \frac{X}{\log_e \dfrac{R}{r_{w_2}}} \quad \text{where } X = 2\pi Pm (h_o - h_w)$$

The relative yield of such tubewells of different diameters is as under :—

TABLE 2

Tubewell diameter in inches.

	4	6	8	12	18	24
Increase in yield	100	105	110	115	123	128
		100	105	110	117	122
			100	105	113	118
...				100	106	111
					100	104

The above table explains that increase in diameter of a tubewell keeping other factors constant does not appreciably increase the yield. A twelve inches diameter tubewell hardly gives 10 percent more discharge than a six inches diameter strainer under similar conditions of operation. When a two inches diameter tubewell is increased to 48 inches, the increase in discharge is 55 percent only. Thus increase in discharge of a tubewell by increase in its diameter give little advantage.

Increase in diameter of a tubewell increases its cost. The surface area of one foot long pipe of 6, 10 or 12 inches diameter is 1.6, 2.6, and 3.1 square feet respectively. Evidently the bigger the diameter, the more is the material used in its construction and consequent is the increase in cost. A big diameter pipe will have a thick wall also. In case of a big diameter strainer, the advantage is with regard to the decrease of the inflow velocities but the increase in cost upsets this advantage. It is thus a practice to use a small diameter screen. It is shrouded all round to increase its effective diameter.

In West Pakistan in a large number of tubewells, screens of 8 or 10 inches diameter have been used. The boring diameter had been up to 18 inches when using casing pipes or equal to 22 inches while using a rotary rig. The space in between the strainer and the boring diameter is filled with gravels of a suitable size. This increases the effective diameter. It is thus a cheap method to attain the advantages of a big diameter tubewell.

In other countries also the size of a tubewell screen is limited to 12 inches and rarely a still bigger diameter screen is used.

Length of a screen and the discharge of a tubewell

The other factors which affect the discharge directly, are the permeability coefficient, the length of screen or the water-bearing medium and the depression head.

We have given sufficient information about permeability coefficient elsewhere. We now consider the length suitable for a screen to pump a given discharge. If the permeability coefficient and diameter of a tubewell is fixed, increase in length of a strainer can result in decrease of depression head to pump the same discharge. This can make the tubewell more efficient.

If the length of a screen is short and the discharge is made up by increasing the depression head, it can result in comparative increase of inflow velocities. Many experts of tubewell design[3, 4] suggest the limitation of sub-soil flow velocities at the slits between 0.1 to 0.25 ft. per second. This brings in the factor of open area of a tubewell strainer. If slots are narrow and kept too far apart, the open area is small. It would need a long length of a screen to attain the velocity according to the above stated limits. The length of a strainer of a given diameter and with a given open space pumping one cusec under the condition of 0.1 ft. per second inflow velocity, at the slot is found as under :—

TABLE 3.

Discharge in cusecs.	Velocity limit in ft. per sec.	Diameter of strainer in inches.	Length of strainer having open space equal to :		
			5%.	8%.	10%.
One	0.1	6.0	127	80	63
,,	,,	10.0	96	48	38
,,	,,	12.0	64	40	32

The brass strainer commonly used in this country has open space equal to 5 percent. When the width of slits is increased to 1/16 inch (62/1000 inch), the open area approaches 8 to 9 percent. Under this condition, the length of a 10 inches diameter strainer pumping 3 cusecs according to the above table is equal to 144 ft. For an open area of 5 percent the length is rather long.

Length of screen on the basis of effective diameter and critical velocity of sand particles

A suitable length of a strainer to pump a given discharge can also be worked out on the basis of the effective diameter or the critical velocity at which the sand particles are set in motion.

It is a practice to provide a gravel filter around a screen. This increases its effective diameter, so that tubewell with effective diameter equal to 12, 15, 18 and 22 inches are being installed in the country.

As for the velocity of flow there is a certain critical order of velocity which sets the particles of sand in motion. The critical velocity which causes movement of sand grains can be worked out on the basis of assuming 40% pores space of the full formation. The experimentally determined value of critical velocity is given below[5].

TABLE 4

Classification of sand.	Mean diameter in mm.	Critical velocity in ft. per sec. at which particles are set in motion.	
		Full formation	Assuming 40% pores as flow area
Coarse sand ..	0.54	0.017	0.0425
Coarse grade of medium sand	0.40	0.012	0.030
Fine grade of medium sand	0.33	0.0092	0.0230
Finer grade of medium sand	0.25	0.0075	0.0188
50% mixture of medium and fine sand ..	0.22	0.006	0.0150
Fine sand ..	0.18	0.0032	0.008
Very fine sand ..	0.15	0.002	0.005
Very fine dirty sand ..	0.11	0.00069	0.00173

TABLE 5

Classification of sand	Mean dia. in mm.	Critical velocity through 40% pores of sands in ft. per sec.	Length of strainer in ft. to pump one cusec from the under-noted effective diameter in inches.			
			12″	15″	18″	22″
Medium sand ..	0.25	0.0188	66	43	29	20
Medium fine sand ..	0.22	0.015	84	54	37	25
Fine Sand ..	0.18	0.008	160	101	70	47

TABLE 6

Projects	No. of wells installed.	Dia. of strainer, (inches).	Open area & slit size in inches.	Effective dia. of boring.	Type of formation	Dis. pumped, (cusec.)	Length of strainer in ft.
Karol Project	20	Aver. 8.0 telescopic, smallest size 6.0 inches and biggest 10.0 inches.	5 and 12/1000	12—15 inches	Medium sand	1.5	120
Rasul Project	1500	8.0	5 & 12—18/1000	12—15 to 18	„	2.0	120—140
Rechna Scarp-1	2000	10.0	8.6 & 62.5/1000	22	Medium & fine sand	3—5	120—160

In fact little is yet known about the actual behaviour of different portions of a tubewell strainer when installed in a formation.

Thus the proper length of a strainer is still a debatable point but giving consideration to all the above-noted points, the following length of strainer for pumping a given quantity of discharge appears to be satisfactory. These lengths are for strainers having open area of 5 percent or more and shrouding has been used to make the effective diameter at least 12 inches.

These lengths are considered satisfactory for the formation in which both the fine and coarse grades of medium sand occur. In case the formation contains predominantly fine grade of medium sand, it will be worthwhile to increase the length of the strainer by about 10 percent.

If the formation predominates in fine sand, a 20 percent increase in the length of the strainer is recommended. It may also be kept in mind that a tubewell of 3 cusecs capacity may not be installed in a formation which is wholly composed of fine sand.

TABLE 7

Discharge in cusecs.	Diameter of strainer in inches.	Length of strainer in feet.
(*a*) *Formation, medium sand (fine or coarse type)**		
One	6.0	80
One	10.0	50—60
Two	6.0	140
Two	10.0	110—120
Three	10.0	150—160
(*b*) *Formation, only fine sand*		
One	6.0	100—120
One	10.0	100
Two	6.0	180
Two	10.0	150—160

*(*a*) In case fine grade of medium sand predominates, increase the length of the strainer by 10—20 per cent.

(*b*) Tubewells with three cusecs discharge may not be installed in fine sand.

Depression Head and the Discharge of a Tubewell

Depression head or draw-down are two terms applied to the lowering of water level inside an operating tubewell which results in the inflow of water from the outside where it is at a high level. This depression is created by the suction action of a centrifugal or a turbine pump. A suction recording gauge or a monometer fixed on the suction side of the pump, can indicate the order of low pressure created by the working of a pump.

In case of a turbine pump, the action is similar. The water level which is lowered in the housing pipe is the depression created by the working of a turbine pump.

Referring again to relation where the discharge of a tubewell is expressed as :

$$Q = \frac{2 \pi P m (h_o - h_w)}{2.303 \log \frac{R}{r_w}}$$

The discharge Q varies directly as the depression if all other parameters are constant.

In the derivation of the above formula we have assumed $h_w + h_o = 2\ m$. This is correct only if $h_o - h_w$ is very small as compared to h_o.

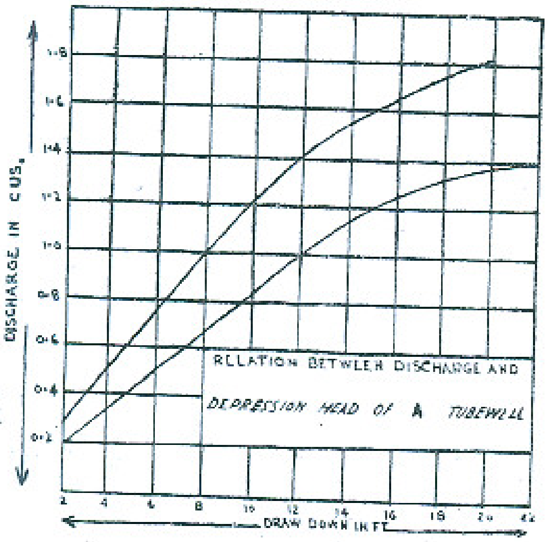


Fig. 3

The yield has a linear relation with depression so long as it is small.

In Fig. 3 two typical cases of variation of draw-down with discharge are shown for tubewells installed in the Indus Plain. It is found that in case of tubewells with depth of 200 to 350 feet and the depression between 15 and 20 feet, the yield and the draw-down follows a linear relation.

Johnson[6] has tried to explain the same point in Fig. 4. So long as the draw-down is small, the percentage of maximum yield to the percentage of depression is linear.

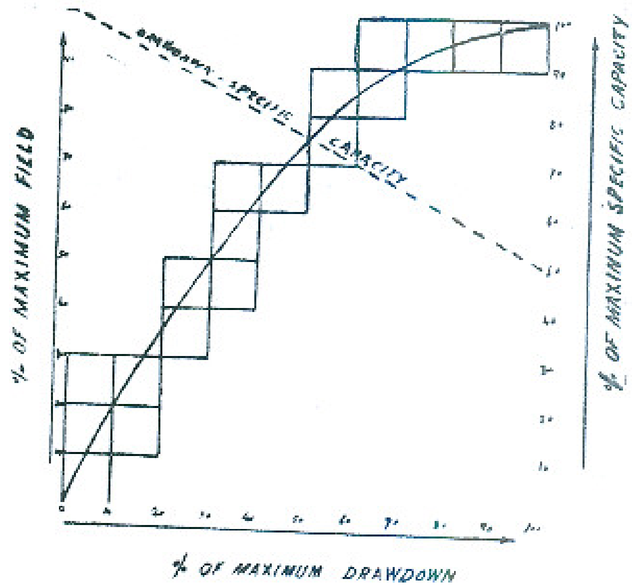


Fig. 4. Relation between Draw-down and Maximum yield.

Depression generally observed in Tubewell installed in the Indus Plains

The depression is a measure of the efficiency of a tubewell. An efficient tubewell will pump a high discharge with a low depression head. This can be a measure of the comparative efficiency of two tubewells working in a medium of the same permeability. The ratio of discharge to the depression head is called the specific capacity. It is expressed either as cusecs or gallons per minute per foot of draw-down. The size of the effective diameter of a tubewell, and the resistance to flow offered by the strainer and the shrouding affect the depression head. In table 8 the order of depression head per cusec from tubewell having variation of diameter, length and type of strainer is given.

These depressions are found in tubewells operating for the first two to five years. After a few years when incrustation start affecting the yield, the depression increases. In case of Rasult ubewells having slit type brass strainer, 8 inches in diameter and shrouded to make the effective diameter equal to 12 to 18 inches, the depression in feet per cusec of discharge after 10 years of operation increased to 10 or 13 feet and after 15 years of operation have increased from 12 to 15 feet.

Cone of Depression and its Extent

When a tubewell is put in operation, the ground-water level around it is depressed and takes the shape of a cone. (Fig. 5) After working the tubewell for about three or four days, the cone attains a stable shape. The shape of the cone is a measure of the permeability coefficient of the medium, as shown in the equation

$$\log \frac{r_2}{r_1} = \frac{2\pi m P}{Q} (h_2^2 - h_1^2)$$

now $h_2 - h_1 = s_1 - s_2$

$h_2 + h_1 = 2m$

$$\text{or} \quad \log \frac{r_2}{r_1} = \frac{2\pi P m}{Q} (s_1 - s_2) \qquad ..(5)$$

Determining the discharge Q and the lowering of watertable, s_1 and s_2 at two distances, r_1 and r_2 from the centre line of the well, P can be determined.

If the permeability of a medium is high, a given discharge is obtained at a low order of depression head, and the cone of depression is not very steep (Fig. 6*a*). In case the permeability of the medium is low, the depression is high and the cone is rather steep. (Fig. 6*b*).

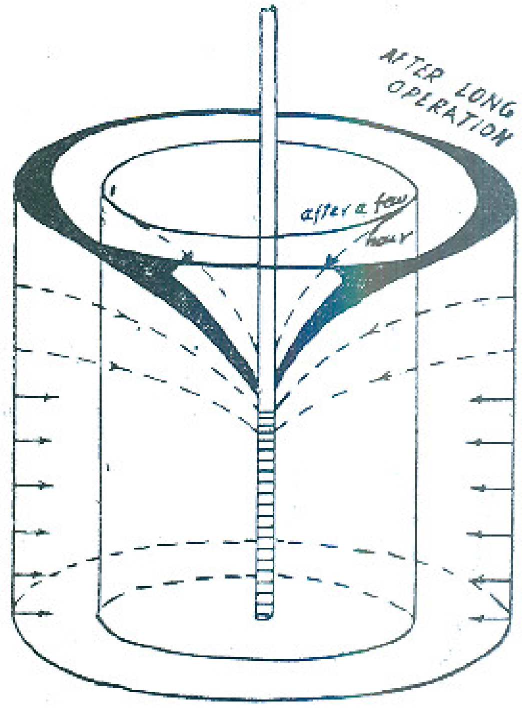


Fig. 5.—Development of cone of depression with Time

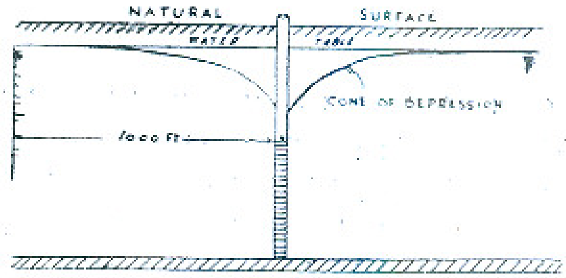


Fig. 6-*a*.—Development of cone of depression in medium of high permeability

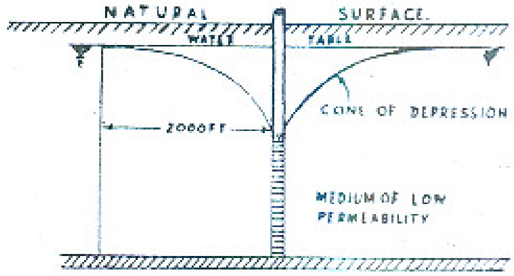


Fig. 6-*b*.—Cone of depression in medium of low permeability

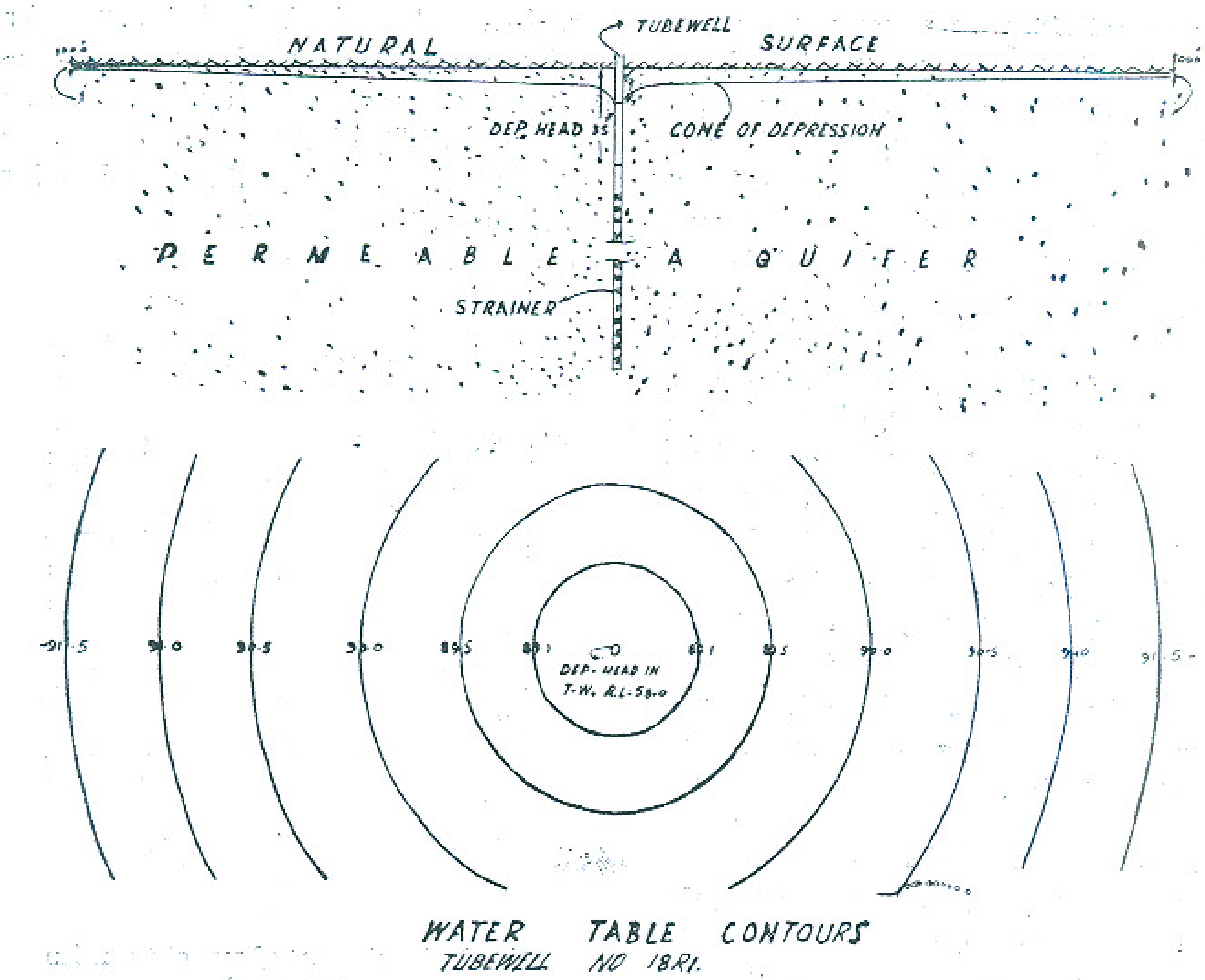


Fig. 7

In the Indus Plain the alluvial sand is either of medium or fine grade. Its average permeability coefficient for the four doabs of the former Punjab lies between 0.0026 to 0.0038 ft. per second, so that the shape of the cone in many places is nearly similar.

The main steepness of the cone occurs within 250 to 500 feet of a working tubewell. If the medium has a high order of permeability, the cone extends from 1000 to 1500 feet from a working tubewell. In many cases we find the effect of the cone extending to 2000 to 2500 ft. but beyond 500 ft. the slope of lowering is very flat, almost a straight line. Between 1000 to 2000 ft. the lowering of watertable may be a few inches only. Some examples of the extent of the cone as actually measured are put forth.

A tubewell No. $18R_1$ was operated for about a month with a depression of 35 ft. The final position attained by the cone is shown in figure 7. The effect of pumping extended to a distance of 3000 ft. The lowering of the ground-water at a distance of 2 ft. from the operating well was only seven feet.

Another similar instance is that of pumping of the tubewell No. 8 of Chuharkana Reclamation Scheme. This test was conducted for 10 days. The ultimate cone of depression formed after 10 days of pumping is plotted in Figure 8. The data of lowering recorded at different distances is given in table 9.

This tubewell was pumped with a depressio of 21 ft.

TABLE 8

Designed capacity of T.W. in cusecs.	Type of strainer.	Range of open area per-centage.	Dia. of strainer in inches.	Effective dia in inches.	Range of length of strainer in ft.	Range of depth of bore in ft.	Type of forma-tion.	Depres sion head ir feet pe cusec.
3	Slit type brass or iron	5 to 8	10	22	120—150	200—350	Med. sand.	4 to (
3	,,	,,	10	18			,,	6
2	,,	,,	8	12 to 18	120	200—250	,,	6 to
2	Coir string	10 to 15	10	10	120	200—250	,,	6 to
2	,,	10 to 15	8	8	100—120	,,	Med-fine sand.	8 to
1 to 2	,,	10 to 15	6	6	100	200	,,	10 to

TABLE 9

Length unit in feet

Well No.		Disch. in cusecs.	Depres-sion	Lowering of watertable at distances						
				2.0	50	100	250	500	1000	200
No. $18R_1$ Sheikhupura Drain R. D. 110	...	3.83	35.0	7.0	6.5	5.8	4.5	3.3	2.0	1
No. 8 Chuharkana Reclamation area	..	2.89	21.14	3.2	2.5	1.8	1.2	0.65	0.32	—
No. 4 Gaja Area Ghulam Mohammad Barrage	..	0.98	10.0	3.8	2.5	1.70	1.0	0.45	0.20	0.

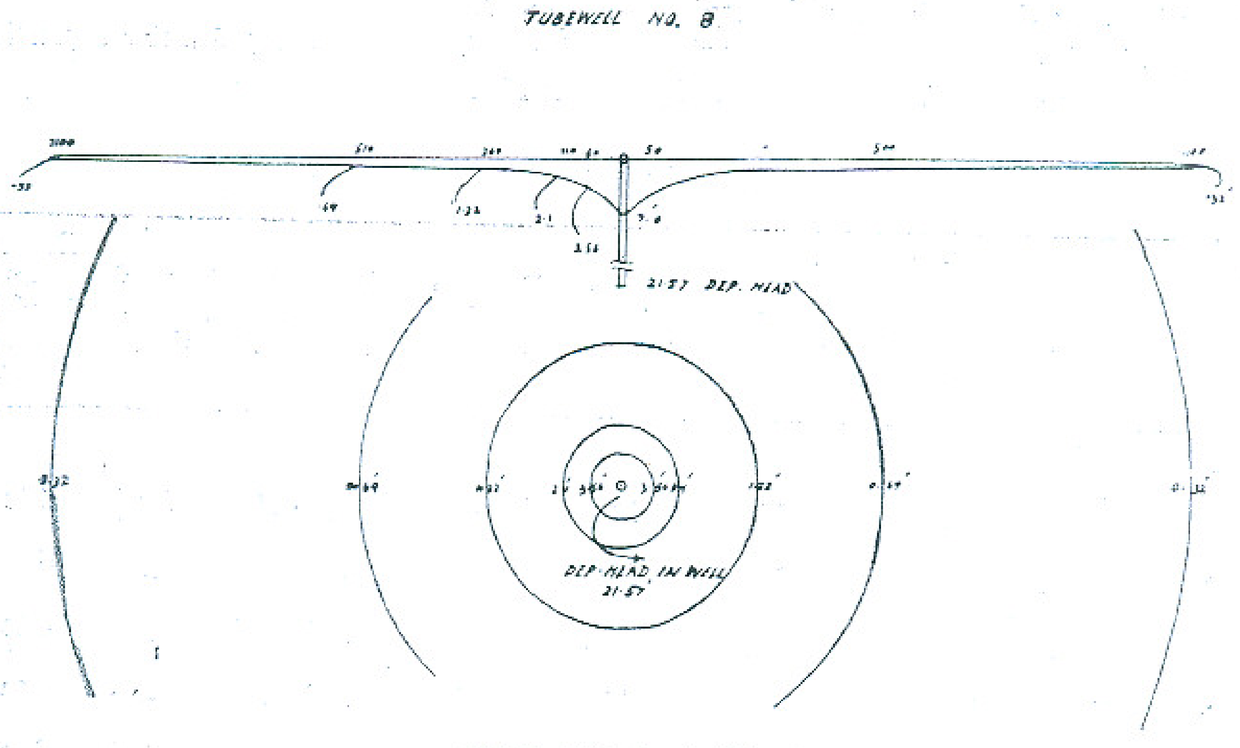


Fig. 8

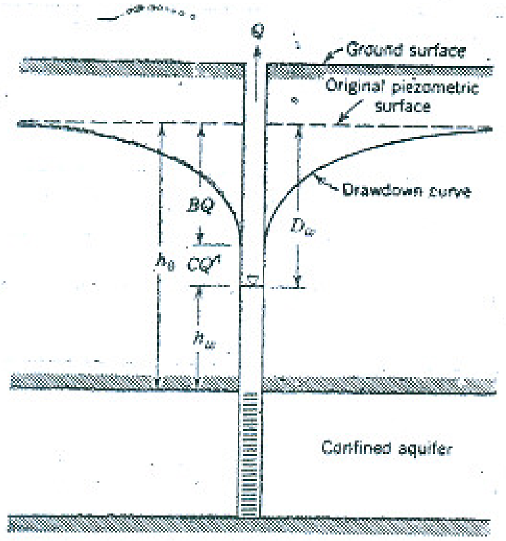


Fig. 9.—Factors of loss of head in a tubewell strainer after Jacob

The third example of pumping test put forth is that conducted in very fine sand of the Gaja area. It was performed on well No. 4 which was operated with a centrifugal pump producing a suction of about 15 tf.

The extent of the cone observed at this site is also given in table 9.

Interrelation of cone of depression with the depression inside a tubewell

The cone of depression is similar to a free surface in an earthen dam. It is the top-most flow line and it never opens out on the discharge face at the level of water inside the tubewell. The point of emergence at the face of the strainer is always some distance above the level of the depression of water inside a tubewell.

Jacob[6] has attributed this condition to the frictional losses from the formation into the strainer. He estimated two types of losses; those through the aquifer formation up to the screen and through the screen itself. Referring to Figure 9, after Jacob, the drawdown, D_w, is equal to BQ and CQ^n.

$$D_w = h_o - h_w$$
$$= \frac{Q}{2\pi K m} \log \frac{r_o}{r_w} + CQ^n$$
$$= BQ + CQ^n$$

where $B = \frac{\log r_o/r_w}{2\pi K m}$ and

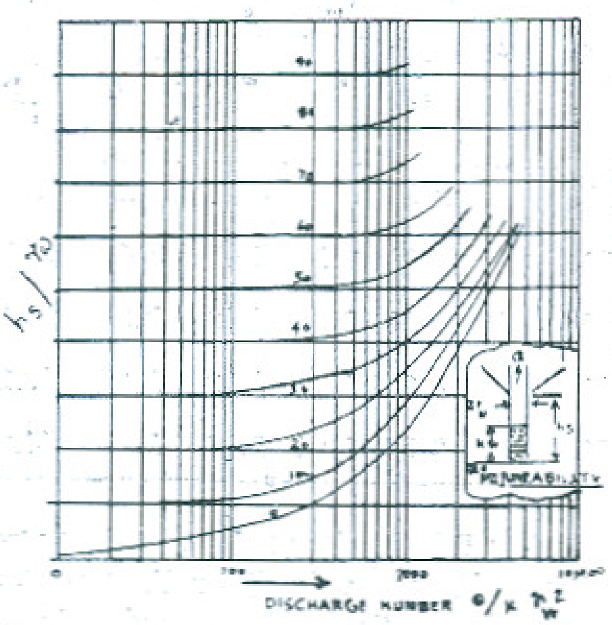


Fig. 10.—Seepage face and depth of water in the well related to discharge number for an unconfined radial well

C is a constant. The value of n is taken equal to 2.

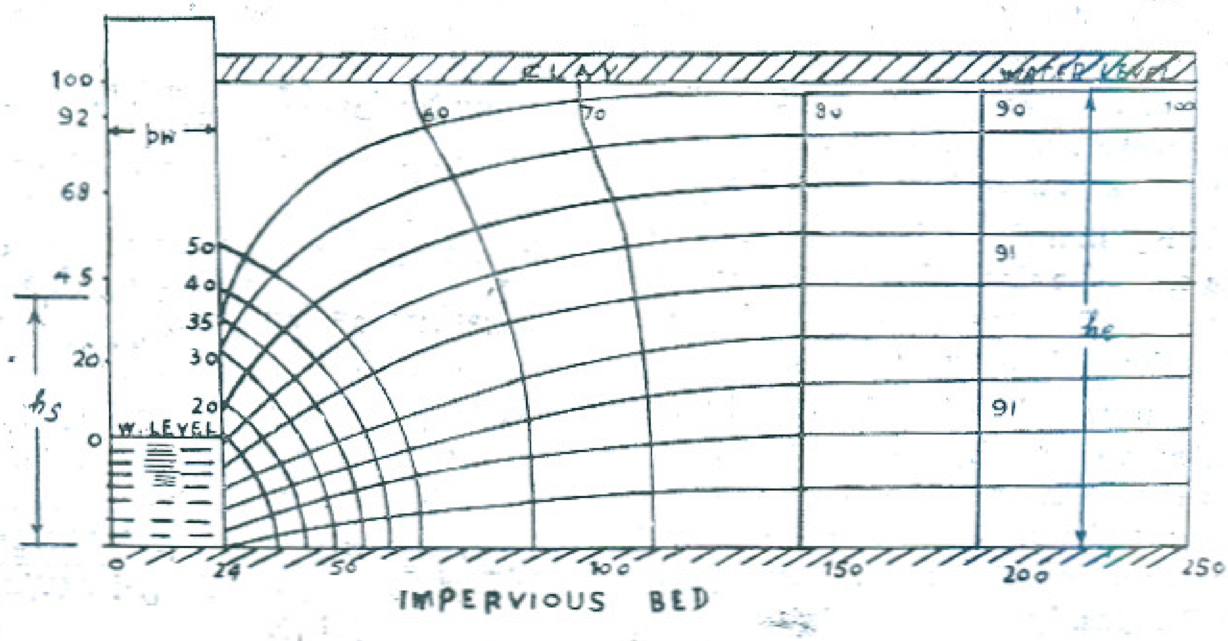
The results of some experiments conducted in the laboratory and the field to determine the head loss at the transition of the flow from the formation into the shrouding and from shrouding into the screen are available.[5]

It was noted that the head loss from the medium sand formation into the shrouding was often 3 to 5 feet and there was insignificant loss from the shrouding to the slit of the strainer.

Recently Peterson, Israelson and Hanson[7] of Utah have tried to interpret the shape of the cone on the basis of non-dimensional parameters, such as the discharge number, $Q/K r_w^2$, and the pumping level expressed as a ratio of h_s/r_w. Some of their characteristic curves are shown in Fig. 10.

This theory can be checked experimentally. A cone observed in a laboratory sand tank is shown in Fig. 11.

The position of h_w, h_s, was determined for the tubewell with strainer of r_w radius



CONE OF DEPRESSION OBSERVED IN A LABORATORY SAND MODEL

Fig. 11

when pumping a given discharges at a known depression head.

A very close agreement between the observed values of h_s/r_w and h_s with the calculated values was found out as shown in Table 10.

TABLE No. 10.—*Tubewell experiments on cone of depression.*

Observation No.	Depression head cm.	Discharge cc/sec at 20°C.	Transmission constant cm/sec.	h_w cm.	r_w cm.	h_s cm. obs.	$\frac{Q}{Kr_w^2}$	$\frac{h_w}{r_w}$	$\frac{h_s}{r_w}$	h_s/r_w taken from Peterson paper	Cal. value of h_s in cm.
1.	17.0	11.8	0.0097	90	22	93	2.51	4.09	4.22	4.2	92.4
2.	33.2	24.0	0.0097	74	22	80	5.11	3.36	3.63	3.4	74.8
3.	50.5	31.1	0.0097	57	22	70	6.61	2.59	3.18	2.8	61.6
4.	67.8	38.2	0.0097	39	22	60	8.268	1.77	2.72	2.6	57.2
5.	84.0	43.63	0.0097	23	22	46	9.86	1.04	2.09	2.0	44.0

Cone of depression and discharge from multiple tubewell

It is possible to determine the cone of depression of a single tubewell. If there is some other source close by such as another operating tubewell or a line source such as a pervious canal or a river, the shape of the cone is affected accordingly. The effect of these depends upon the location and the distance in between the working tubewells.

In Fig. 12 is shown the possible effect on the cone when three tubewells are operating together. The portion enclosed between the tubewells is partly dewatered.

When two tubewells with identical parameters such as depth of water bearing medium, diameter of strainer and permeability coefficient etc., are operated under the same depression, the discharge of each well is given by

$$Q_1=Q_2=\frac{2\pi m\,(h_o-h_w)\,K}{\log\,(R^2/r_w\,B)} \quad \ldots(6)$$

where B is the distance between the centre line of the two wells, P or K stand for permeability. Sometime *ln* is used for $\log_e$.

In case three tubewells are installed forming an equilateral triangle with distance B apart as shown in fig. 12, the discharge of each tubewell is given by

$$Q_1=Q_2=Q_3=\frac{2\,\pi\,K\,m(h_o-h_w)}{\log\,(R^3/r_wB^2)} \quad ..(7)$$

Similarly four tubewells installed on the four corners of a square of side equal to B, the discharge of each tubewell is equal to

$$Q_1=Q_2=Q_3=Q_4=\frac{2\,\pi\,K\,m\,(h_o-h_w)}{\log\,(R^4/\sqrt{2}r_w\,B^3)} \quad ..(8)$$

If three tubewells are located in a line, (fig. 13) the middle one is influenced by the two tubewells on both sides. The discharge of the two outer tubewells is given as under :

$$Q_1=Q_3=\frac{2\,\pi\,K\,m\,(h_o-h_w)\;ln\,(B/2r_w)}{2\;ln\;(R/B)\;ln\;(B/r_w)+ln\;(B/2r_w)\;ln\,(R/r_w)}. \quad ..(9)$$

The yield of the middle tubewell is influenced by the two outer ones and is

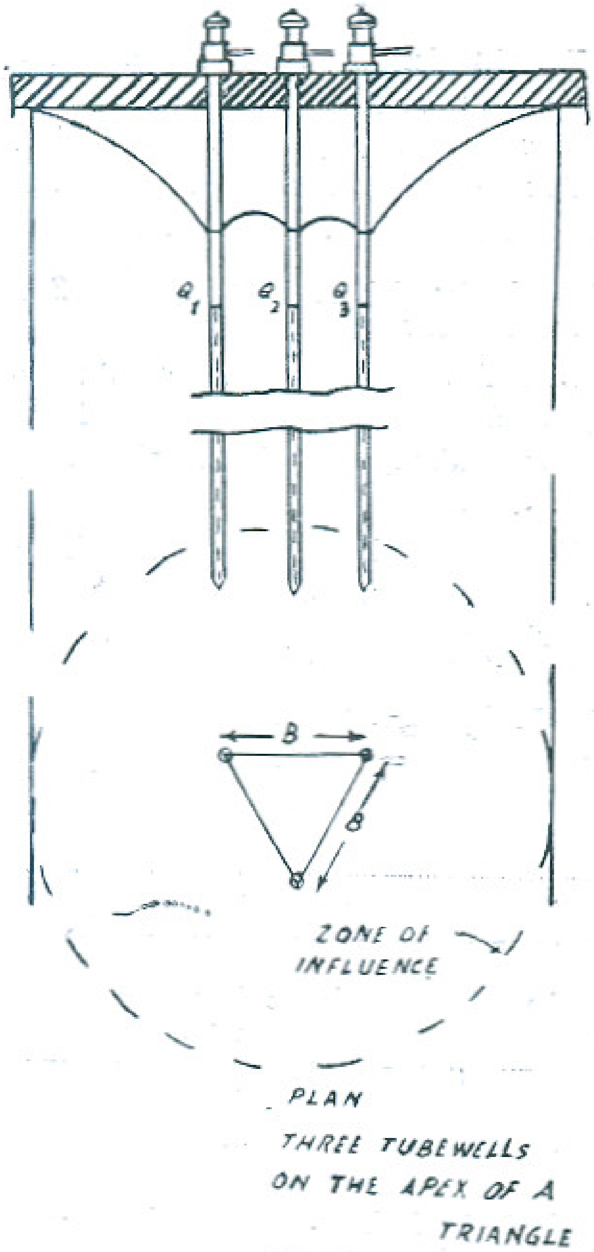


Fig. 12

given by

$$Q_2 = \frac{2\,\pi\, Km\,(h_o - h_w)\ in\ B/2r_w}{2\,ln\,(R/B)\ ln\,(B/r_w) + ln\,(B/2_w)\ ln\,(R/r_w)} \quad ..(10)$$

Other complicated cases can also be treated similarly. A case of four tubewells with a fifth tubewell in the centre as treated by Todd is as follows:

$$Q_1 = Q_2 = Q_3 = Q_4$$

$$= \frac{2\pi\, Km\,(h_o - h_w)\ ln\,(B/\sqrt{2}r_w)}{4ln\,(\sqrt{2}\,R/B)\ ln\,(B\sqrt{2}\,r_w) + ln\,(R/r_w)\ ln\,(B/4\sqrt{2}\,r_w)}. \quad ..(11)$$

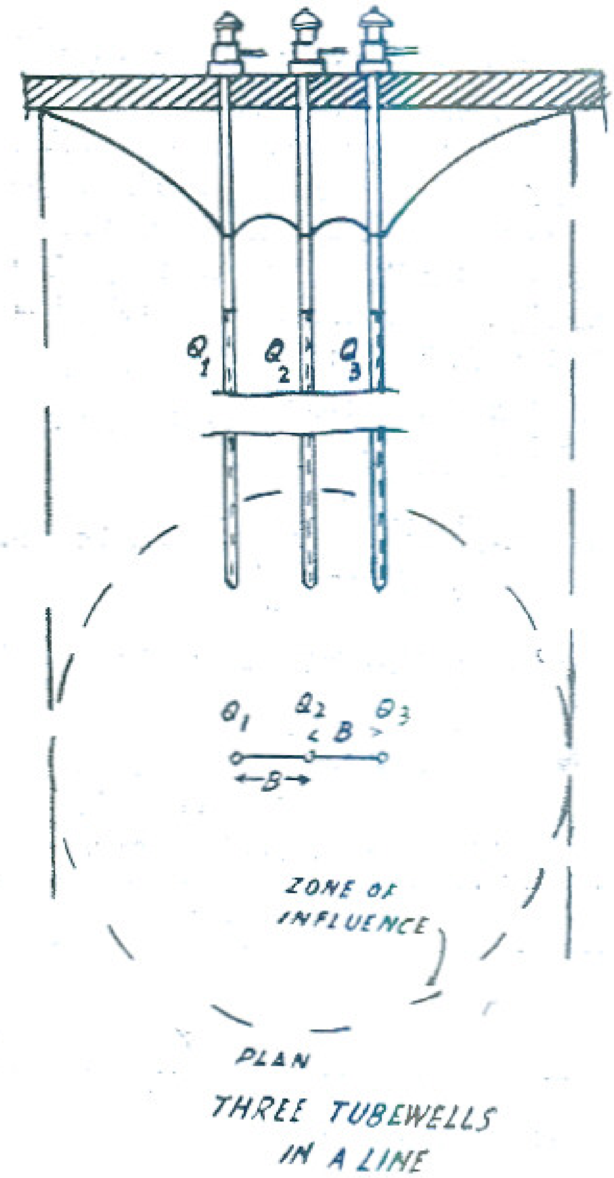


Fig. 13

$$Q_5 = \frac{2\pi Km\,(h_o - h_w)\ ln\,(B/4\sqrt{2}r_w)}{4ln\,(\sqrt{2}\,R/B)\ \ ln\,(B/\sqrt{2}\,r_w) + ln\,(R/r_w)\ ln\,(B/4\sqrt{2}r_w)}. \quad ..(12)$$

For confined aquifer replace m by b

Discharge estimation for specific cases of tubewells.

Utilizing the above-mentioned relations the discharge of tubewells for a few specified cases is worked out. Assume the depth of water-bearing formation, m equal to 200 feet, the permeability coefficient P or K equal to 0.002 feet per second and the depression head, h_o-h_w equal to 15 feet and assuming the influence of the tubewell; R equal to 1000 or 2000 feet, the discharge of tubewell located at varying distances apart is worked out. The output is compared with a single tubewell located in an extensive area with no other interferring source close by. The discharge of a single tubewell assuming R= 1000 and 2000 ft. is :

$$Q_1=\frac{2\pi \times 0.002\times 200\times 15}{2.303 \log \frac{1000}{1/2}}=5.45 \text{ cusecs.}$$

and $$Q_2=\frac{2\pi \times 0.002\times 200\times 15}{2.303 \log \frac{2000}{1/2}}=4.96 \text{ cusecs.}$$

The yield for two, three and four tubewells is worked out for the condition with distance apart varying from 25 to 500 feet.

This data together with the percentage interference are given in table 11.

Determination of interference by actual field tests on tubewells

Determination of interference of tubewells by actual performance tests is quite difficult as it is practically impossible to find identical formations.

The following is an approximate order of interferance found between two tubewells

TABLE No. 11.—*Estimation of discharge of multiple tubewell compared with a single tubewell.*

Discharge of a single tubewell assuming m= 200 ft., h_o-h_w=15 feet.
P or K=0.002. ft. per second. Q=5.45 cusecs when R=1000 ft. and
Q=4.96 cusecs for R=2000 ft.
Discharges expressed in cusecs.

	R=1000 ft.					R=2000 ft.				
	Distance in between the tubewells (ft.)					Distance in between the tubewells (ft.)				
Two tubewells working	25	50	100	250	500	25	50	100	250	500
Discharge $Q_1=Q_2$	3.57	3.80	4.09	4.55	4.96	3.12	3.26	3.56	3.82	4.19
% fall in discharge	34.5	30.29	24.94	16.91	8.99	37	34.27	28.21	22.0	15.5
Three tubewells along a △										
Discharge $Q_1=Q_2=Q_3$	2.64	2.92	3.27	3.89	4.67	2.31	2.52	2.70	3.21	3.65
% fall in yield of each well	51.6	46.39	39.98	28.61	14.31	33	49.19	45.56	37.2	28.4
Four Tubewells along the epix of a square										
Discharge $Q_1=Q_2=Q_3=Q_4$	2.14	2.42	2.79	3.52	4.36	1.41	2.05	2.32	2.84	3.40
% fall in discharge of each tubewell.	60.7	55.6	47.9	33.5	20.0	71.7	58.6	53.2	42.7	33.4

having about 100 to 120 feet length of strainer installed in a bore depth of 150 to 200 feet in medium to fine sand having permeability equal to 2 to 3×10^{-4} ft. per second and worked by a low depression of 15 feet.

TABLE 12

Interference between two tubewells

Distance between two tubewells, in feet	15	25-30	50	100	250	500
Percentage fall in discharge of each well	40-45	25-30	15-20	8-12	5	nil

Interference between wells depends on their depression head, length of strainer, depths of water-bearing formation, diameter of strainer and such other variables.

In the estimation of interference between tubewells all the variables affecting the discharge have to be taken into account.

Partially Penetrating Wells

For determining a relation for the yield of a tubewell we have always assumed the flow to be radial. This is possible only when the formation is fully screened and an impervious formation like clay or rock exists at bottom.

There are many cases where the aquifer cannot be fully screened. This is generally the case in West Pakistan where the aquifer is of considerable thickness and no extensive clay or separate rock zone exists to form separate aquifer. The clay layers which appear in the Indus formation are not extensive and actually the whole aquifer within a depth of 300 to 400 feet constitutes almost a unit. The tubewell being installed are not too deep and are very often non-penetrating. In fact this type of tubewells has to be constructed where the deep groundwater is saline and skimming tubewell has to be installed. The flow pattern for a partially penetrating tubewell is something like as shown in Fig. 14.

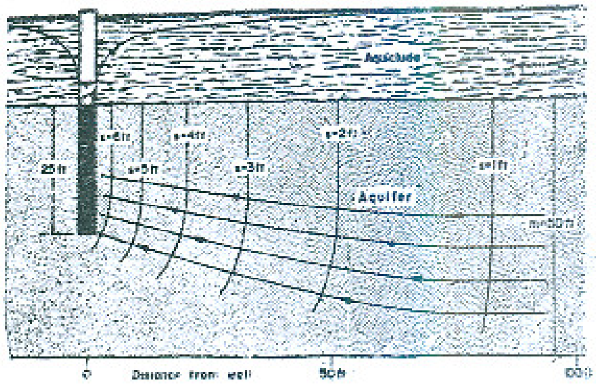


Fig. 14.—Flow lines into a non-penetrating tubewell strainer

In spite of the flow lines coming from depth greater than the length of non-penetrating strainer and making its effective length slightly longer, the non-penetrating strainer is not so efficient as the full penetrating strainer.

The bottom flow lines have to traverse a longer distance and are subjected for greater resistance, so that for the same depression head in a partially penetrating and fully penetrating, *i.e.*, $(h_o - h_w)$ partial $= (h_o - h_w)$ fully penetrating, $Q_p < Q$ where Q_p is the discharge of partially penetrating strainer. In other words for $Q_P = Q$, $(h_o - h_w)$ (Partial) $> (h_o - h_w)$ fully penetrating.

The analytical analysis of a partially penetrating well is complicated except for simple cases. The problem has been investigated by Forchheimer,[8] Kozeny,[9] Musket[10] and several others.

The equation for draw-down developed for a partially penetrating well is

$$(h_o - h_w) = \frac{Q_p}{2\pi K} \left\{ \frac{1}{h_s} \log \frac{\pi h s}{2 r_w} \right.$$

$$+\frac{0.10}{b}-\frac{1}{b}\log\frac{r_o}{2b}\Big\} \qquad (13)$$

where $b=m$ the thickness of water-bearing medium. The parameters are explained in Fig. 15 and have the usual meaning, h_o in

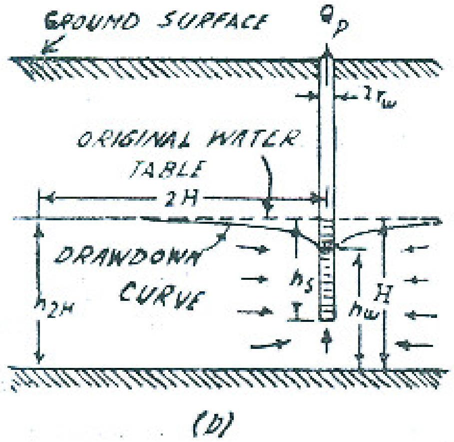


Fig. 15

particular is the head at the radius of influence, r_o from the well.

The discharge of a fully penetrating tubewells is, however, given as under :

$$Q=2\pi Kb\frac{h_o-h_w}{\log r_o/r_w}$$

$$\text{or } h_o-h_w=\frac{Q}{2\pi Kb}\log\frac{r_o}{r_w} \qquad ..(14)$$

If the value of depression is assumed the same then the ratio of the two equations 13 and 14 give

$$\frac{Q_p}{Q}=\frac{\log(r_o/r_w)}{(b/h_s)\log(\pi h_s/2r_w)+0.10+\log(r_o/2b)} \qquad ..(15)$$

This relation can thus be made use of to determine the effect of partially penetrating tubewell on its yield. Curves as shown in Fig. 16 have been plotted between the penetration fraction h_s/h or h_s/m and the discharge ratio of Q_p/Q for various values of the ratio of aquifer thickness m or b to the radius of the tubewell, r_w.

With the help of this curve when the percentage penetration is known, the percentage yield of partially penetrating tubewell can be obtained.

Further complicated cases of this nature for non-homogeneous medium have been treated by Musket.

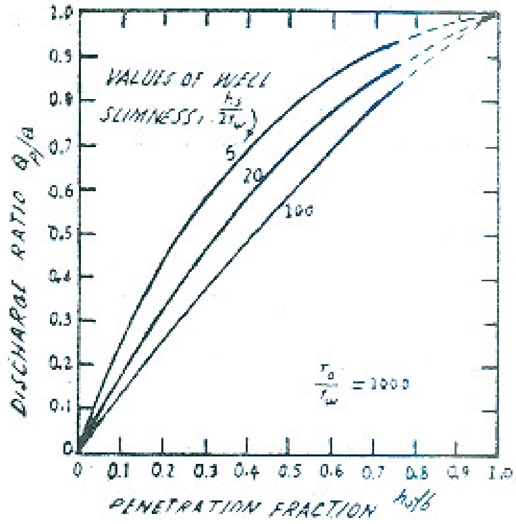


Fig. 16

Seepage from a line source

The Rasul Tubewell scheme was planned in such a way that tubewells, 200 to 350 feet deep were to be installed at the toe of the spoil bank about 60 feet from the water edge of a canal. The water pumped out was to be discharged into the canal.

It was presumed that location of tubewells, so close to canal will result in

(*i*) Shorter delivery pipes;

(*ii*) Better inspection of tubewells located along the inspection bank;

(*iii*) Lowering the cost of the scheme as the land where tubewells were to be installed belonged to the Irrigation Department and no long inspection roads were needed;

(*iv*) Mixing the pumped water into the canal was possible thus eliminating the possibility of use of unfit groundwater directly on land;

(*v*) Eliminating the problem of distribution of pumped water;

(*vi*) Dewatering of waterlogged lands adjacent to the canal which had become uncultivated due to seepage from canals was possible;

(*vii*) Reducing percolation on the assumption that cone of depression may extend below the bed of a canal creating unsaturated zone;

(*viii*) Decreasing the contribution by providing at least sixty feet of top blind pipe to give a longer percolating path to flow lines from the canal.

At the same time a deep tubewell working close to a canal was expected to collect much of the canal seepage and thus give relief to the waterlogging.

This point of increased flow from a canal was thus examined in the laboratory by the use of a sand model and electrical analogy techniques.

Musket Analysis for tubewell working along an infinite line source :—

Musket has analysed cases of single and multiple tubewells located along an infinite line source.

He has established that

$$Q = \frac{2\pi K (h_o - h_w)}{\mu \log 2\, d/r_w} \qquad (16)$$

for a case of two-dimensional flow.

The yield of a well varies according to $1/\log \frac{2d}{r_w}$. As the distance of a tubewell increases from a line source, the draw from the source decreases accordingly.

Flow lines and potential for different positions of a tubewell from a line source can be plotted experimentally with the help of electrical analogy technique. A few such cases studied are shown in Fig. 17.

If the number of tubewells along a line multiplies, the yield is effected by the closeness of the tubewells. We have already put forth cases of two or three tubewells

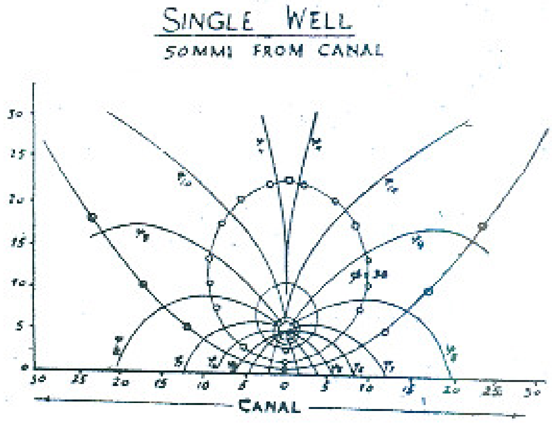


Fig. 17-*a*

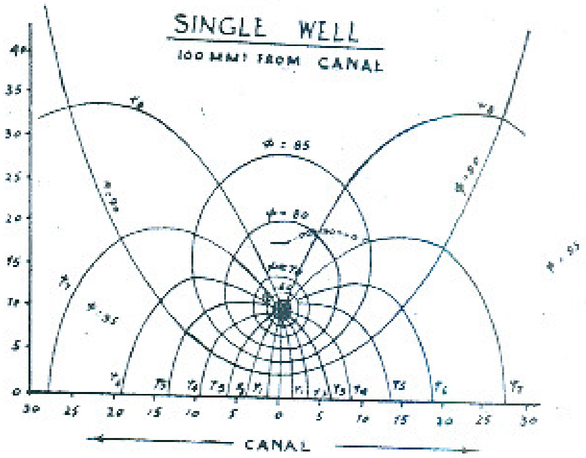


Fig. 17-*b*

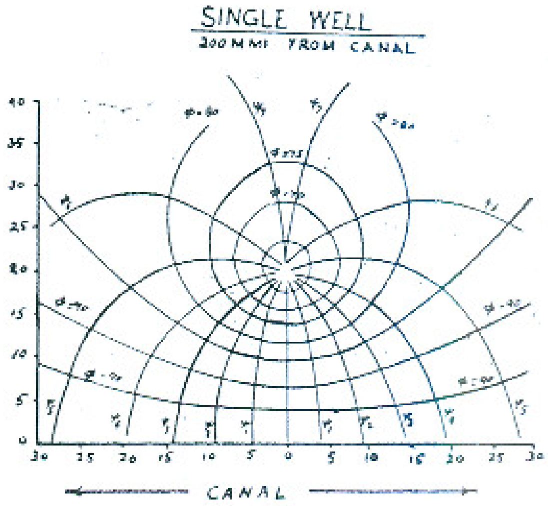


Fig. 17-*c*

working in an extensive medium.

The yield of tubewells at a distance, d from an infinite line source and a distance 'a' apart is given by

$$Q_1 = Q_2 = \frac{2\pi K (h_o - h_w)}{\mu\left[\log \frac{2d}{r_w} + \frac{1}{2}\log\left(1 + \frac{4d^2}{a^2}\right)\right]} \qquad (17)$$

If the ratio of $d/a=1$ and $d/r_w=400$ then the production capacity of each well in two well systems is only 89.26 percent. The closeness of tubewell affects the production.

Contribution from a canal to a tubewell

In the Indus plain, the watertable is high as a result of slow contribution from the seeping canals.

The condition of a canal is also variable, so that at places, a canal is located in a medium of high permeability whereas at other sites a fairly impervious formation exists under the bed of a canal.

There are cases when pumping intensively from a formation, due to low permeability of the canal bed, water is withdrawn from extensive regions.

A test was conducted by nine tubewells working at R. D. 112 along Upper Gugera Canal. After three months of operation the position of ground-water along a section was as shown in Fig. 18. The effect was extensive on both sides of the canal which had 8 feet of thick soil crust under its 10 feet water head.

A similar test was conducted two miles downstream on the same canal near Chuharkana where the bed of the canal is sandy. It showed no effect of tubewell pumping on the opposite side of the canal. The overall effect was limited to a short distance.

In Fig. 19 is shown another case of lowering of groundwater close to the Upper Jhelum Canal during the construction of Shadiwal Hydel Station.

The groundwater in a small area of 250×300 feet was taken down by about 35 feet by withdrawing about 16 cusecs of water. The site of construction was located 600 feet away from the big canal.

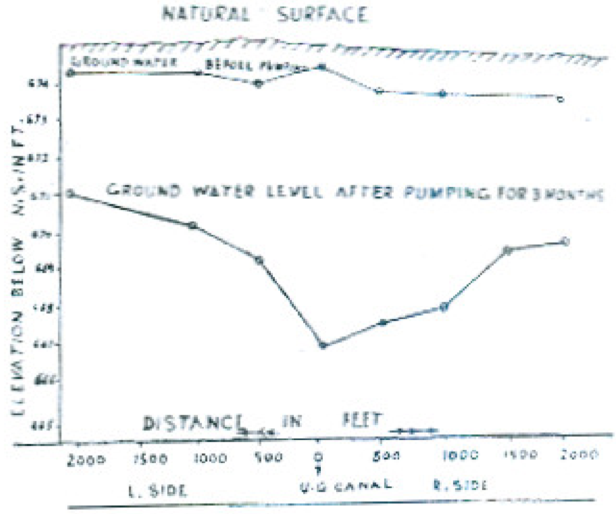


Fig. 18

The dewatered level was maintained at the position for more than a year. The effect of this dewatering was extensive on both sides of the canal. This was mainly possible as the canal had 5 to 6 feet of thick soil crust under its bed.

In a similar case of hydel construction at Chichokimallian where watertable was taken down by about 40 feet and the Upper Chenab Canal having a pervious bed, the effect of dewatering was not observed on the opposite bank of the canal.

The estimate of yield of tubewell also showed that during closure the desired water level could be maintained by pumping 1/3 less discharge. Thus the existence of water in the canal with pervious bed was responsible for adding another 1/3 of the pumped water.

Results of sand model experiments on location of tubewells away from a canal

Sand model experiments were conducted

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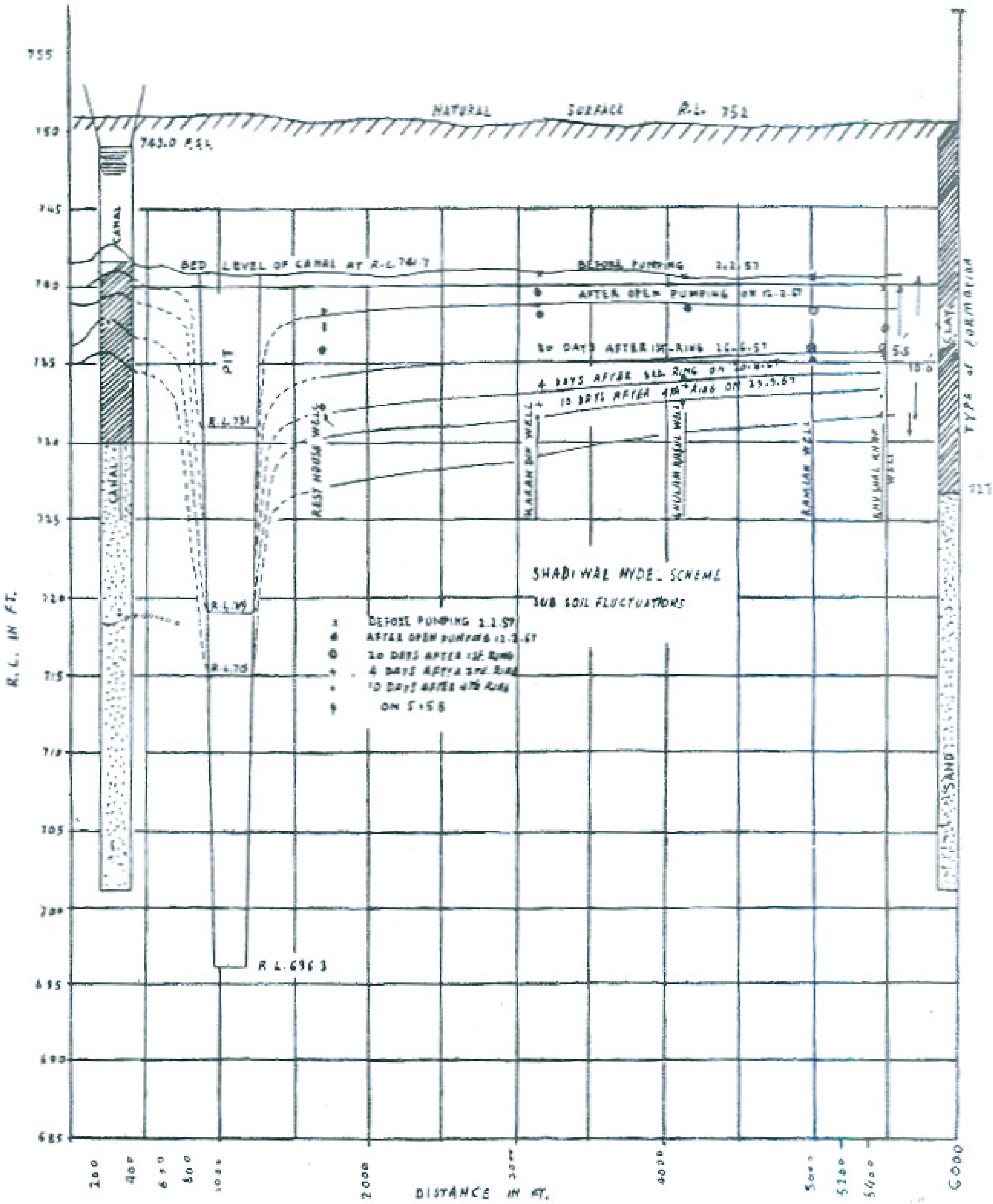

Fig. 19

to determine the contribution of a canal with a sandy bed to a tubewell located at different distances from the line source.

The ground-water-table was also maintained at a fixed level.

It was noted that a tubewell located at 50 feet from a pervious canal drew about 80 percent of its yield from the canal. When the position of the tubewell was shifted to 500 feet, the canal contribution was reduced to about 30 percent.

In case the bed of the canal had clay formation, the contribution was considerably reduced. It was a case like that of Upper Gugera Canal at R. D. 112 where the discharge of the canal is about 5000 cusecs with 10 feet depth and 200 feet in width. The canal was found to have little effect on the dewatering of the area by three months of operation of nine tubewells with drawing about 13.5 cusecs.

Similar observations conducted near Chuharkana on the same canal where the bed is pervious, showed no effect of tubewell pumping.

Multiple strainers and their advantages

There are many problems where the use of multiple strainer tubewell is essential and of advantage.

(*a*) Skimming of good quality water from shallow depth

In the Northern region of the Indus plain in the centre of each Doab of the former Punjab, there exists highly saline unsuitable water. It is overtopped by good quality water which can be withdrawn and used for various purposes. Its thickness varies between 60 and 100 feet and for this reason a strainer longer than 30 to 70 feet cannot be used. This affects the output which is un-economical unless it is increased. If the number of strainer is multiplied to say two or three and connected to the same centrifugal pumping set, the yield can be increased to the capacity of the pump.

Centrifugal pumps commonly manufactured in the country are of 2 to 2.5 cusecs capacity. It is thus possible to pump water up to this capacity by using three strainers of about 50 feet in length without disturbing the deeper saline water.

The distance apart of the strainer is adjusted keeping in view the fall in yield due to interference and the friction of the pipe. A distance of 50 feet between the two strainers results in an interference of about 15 to 20 percent.

The coupling of multiple strainer is not a problem when the watertable lies within 15 feet from the surface. An example of Niazbeg multiple tubewell is shown in fig. 20 where several tubewells of 80 feet depth are installed and connected together by pipes kept on the ground surface, about 6 feet above the water table. These can also be laid underground in case the watertable is deep.

(*b*) Cheaper installation, quick dewatering and easy renovation

Besides the skimming of good quality water from above a saline water, a multiple well system has other advantages.

A shallow bore is cheaper to install as compared to a deeper one. It can also be installed quickly by ordinary percussion technique.

A multi-strainer tubewell is everlasting and is one of the solutions for incrustation. Any completely or partly chocked strainer can be pulled out without stopping the operation of the well and is replaced. During the replacement period, the tubewell

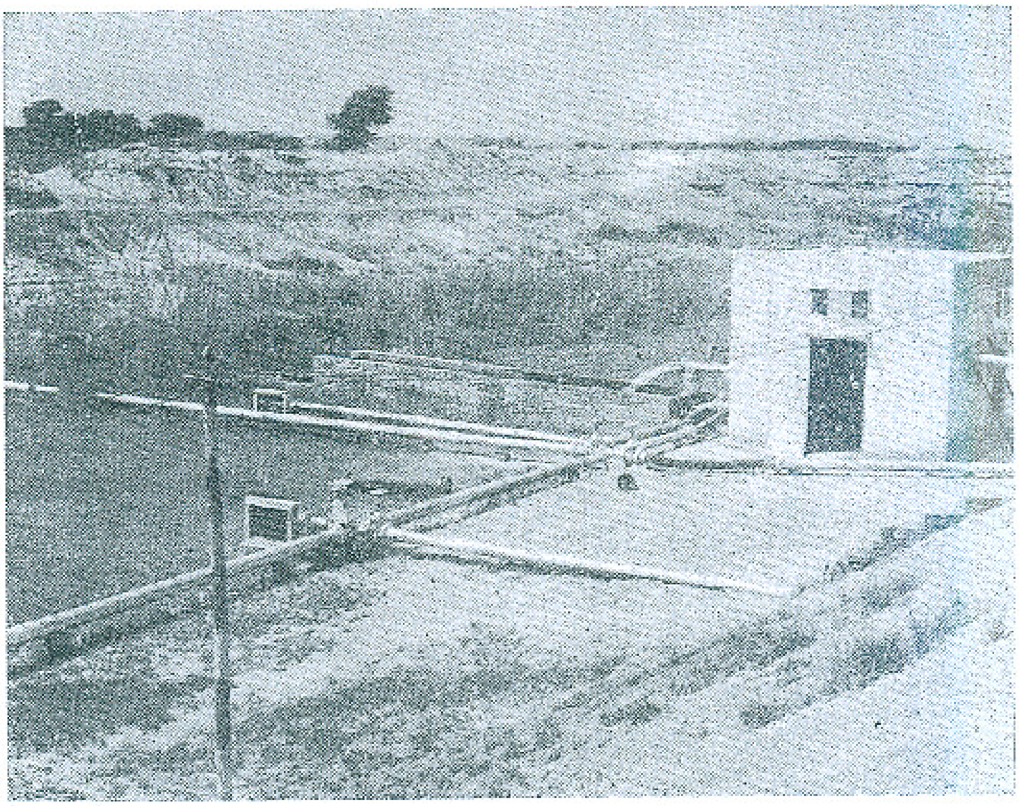

Fig. 20.—A set up of multi-strainer tubewell at Niazbeg Research Station.

will continue to yield slightly less water. Replacement of a strainer is not costly.

When a quick dewatering of a water-logged site is needed, a shallow multiple strainer tubewell is most suitable as it can dewater thin top zones.

(*c*) Multiple strainers tubewell can yield large volume at low depression head

For sites where deep multiple strainer can be installed, this technique offers a system for pumping a large volume of water by a centrifugal pump at a low depression head. It is possible to pump 4 to 5 cusecs with 12 to 15 feet of depression. If three bores 200 ft. deep with strainers equal to 120 to 140 feet long are installed 100 feet apart and coupled to the same high capacity pumping set, it can yield high order of discharge and the depression in each bore will remain within 12 to 15 feet. Thus a high yield can be obtained with low depression. This technique spreads the pumping load to multiple strainers and thus reduces the chances of quick incrustation.

(*d*) Multiple strainers tubewell is more efficient

When a strainer is non-penetrating, it forms a cone both at the bottom and at top. The curvature of flow lines at the two sites depends upon the order of suction inside a strainer. This increases the effective zone of withdrawal of a strainer as compared to the one having complete radial flow.

If there are two non-penetrating strainers then each strainer has extended zone of withdrawal at the bottom. Thus two shallow strainers become slightly more efficient when