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Interceptor Drains

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INTERCEPTOR DRAINS

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SYNOPSIS

The observations have shown that the irrigation canals have contributed predominantly towards the rise of ground water table in Pakistan. The canals were unlined and now it is not practicable to line these canals. Deep cut-off barriers on either side of the canal are also very expensive. Interception of seepage by open drains along certain link canals has been tried in Pakistan but to a limited scale. These are not only expensive to construct and maintain but also consume productive land. Subsurface interceptor drains parallel to the canals can intercept considerable percentage of seepage. Seepage so collected will have to be pumped out and used for agriculture thus serving dual purpose of preventing recharge to groundwater andproviding additional water for irrigation.

This paper gives the design concepts of interceptor drains. It describes the theoretical and analog model studies carried out for Chashma Right Bank Canal, Left Bank Outfall Drain and Fordwah Sadiqia Scrap. Provision has been made for construction of interceptor drains in the above projects but these are yet to be constructed. The construction is analogus to tile drains. The studies have proved economic feasibility of the interceptor drains.

INTRODUCTION

Canal seepage is one of the major contributers to waterlogging. Long stretches of some of the major canals have created lakes and swamp land besides the canal banks. In other cases, where seepage is somewhat less, it nevertheless has raised the adjacenet water table. The most effective method of dealing with the problem is to line the canals but the cost involved is prohibitive and the disruption caused in the operation of canals during their lining would be unacceptable.

Another possibility would be to place a vertical cutoff barrier beside the canal to reduce seepage. The seepage can be reduced by up to two thirds provided the barrier is about 50 feet deep. However, the cost of over Rs. 25,000 per foot run of the canal would be prohibitive. The third and a preferable method would be to install interceptor drains along the outer toe of the canal embankment.

Interception of seepage by means of open drains is a longstanding practice. It is commonly adopted where an outside source of recharge to groundwater is identified to be moving in a direction that would make an interception drain effective. Open drains are of limited depth and are more expensive to maintain than tile drains, and they also use productive land for their construction.

Another alternative is to install subsurface interceptor drains in the vicinity of and parallel to the source of recharge to intercept seepage before it joins groundwater. Seepage collected by the interceptor drains is pumped out and used for irrigated agriculture. Thus, interception drain serves a dual purpose of preventing recharge to groundwater and providing additional water for irrigation.

The concept of interceptor drains is relatively new in Pakistan. Interceptor drains have been

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planned for Chashma Right Bank Canal (CRBC), Left Bank Outfall Drain (LBOD), Ford-wah-Sadiqia SCARP and Lower Rechna Remaining SCARP-IV- Project. Analog model studies have been carried out for LBOD and Fordwah Sadiqia SCARP while these are underway for CRBC. A preliminary design study was made by R.S. Broughton for CRBC. Interceptor drains have yet to be constructed in Pakistan. This paper gives briefly the theoretical and analog studies carried out in Pakistan so far. It is hoped that discussion on the topic would be very helpful in improving the design and construction of the interceptor drains for the various projects.

DESIGN OF INTERCEPTOR DRAIN

The rate of interception depends upon the head in the canal, depth to watertable, permeability of the soil and the position of interceptor drain with respect to the source of recharge. Soil texture and sloughing problems have also to be taken into consideration. Interceptor drains are designed analogous to tile drains at suitable distance from the canal to avoid induced seepage discharge and at depths economical in design and easy to construct.

Location of first drain:

Data required to determine the location of the first drain below an unlined canal or lateral are: (1) channel sections and grades, (2) hydraulic conductivity of the material adjacent to the channel, (3) weighted hydraulic conductivity between permissible root zone depth and barrier, (4) depth to barrier, (5) slope of barrier and ground surface in the vicinity of the channel, and (6) distance from the centre line of channel to the irrigated land, see figure 1.

The following steps show a method of determining the distance from the canal centreline to first drain:

Step 1: Estimate the channel seepage under free drainage conditions using the following formula:

$$q_1 = \frac{K_1 (B + 2d)}{3.5} \qquad (1)$$

where

q_1 = seepage in cubic feet per linear foot of channel per day, when water table is below channel bottom (free drainage condition),

K_1 = hydraulic conductivity adjacent to the channel section, feet per day,

d = depth of water in channel at normal operation leve, feet

B = top width of water in channel at normal operating level, feet, and

3.5 = A factor used to adjust hydraulic conductivity test values to seepage losses from ponding tests.

Example: For a canal section with a base width of 100 feet and 2 to 1 side slopes, find q_1 if K_1 = 1.5 feet per day and d = 2.5 feet.

$$B = 10 + 2(2 \times 2.5) = 20 \text{ feet}$$

$$q_1 = \frac{1.5(20+2\times2.5)}{3.5} = 10.71 \text{ ft}^3/\text{ft/d}$$

For existing canals and laterals q_1 can be measured, but care must be taken to assure that there is free drainage below the canal or lateral. When a water table has developed under the canal or lateral, the depth to the water table must be measured at the same time as the seepage. Unless a thick permeable aquifer underlies the canal, a ground-water mound will rise under the channel and eventually reach the same level as the water surface in the channel. The time required for this to occur can be estimated from the formula.

$$t = \frac{K_2 y^2 D_1 S}{q_1^2} \qquad (2)$$

where

t = time in days for water table mound to rise from water depth at beginning of irrigation season to water surface in canal.

K_2 = weighted hydraulic conductivity between root zone depth and barrier, feet per day,

y = distance from water table depth at beginning of irrigation season to normal water surface in the channel, feet,

D_1 = distance in feet between water table depth (at beginning of irrigation season) and the barrier plus one-half y,

q_1 = seepage under free draining conditions, $ft^3/ft/d$, and

S = specific yield determined from hydraulic conductivity in the K_2 zone, percent by volume.

For example, if the distance between water table depth (at beginning of irrigation season) and the barrier is 20 feet, $K_2 = 1.5$ ft/d, y = 9 feet, S = 12 percent, and $q_1 = 10.71 ft^3/ft/d$ as previously calculated. Find the time, t, as defined above.

$$D_1 = 20 + 9/2 = 24.5 \text{ feet, and}$$

$$t = \frac{(3.1416(1.5)(9.0)^2(24.5)(0.12)}{(10.71)^2} = 10 \text{ days}$$

The use of q_1 in formula (2) does not account for the fact that the seepage rate begins to decrease when the water table mound reaches the bottom of the channel and will continue to decrease until the mound rises to the water surface elevation in the channel. At this point, the seepage rate becomes essentially constant and is called the terminal seepage rate, q_2. The seepage rate, q_2, can be determined by the formula:

$$q_2 = q_1 \left(\frac{B - 2d}{B + 2d}\right) \qquad (3)$$

$$q_2 = 10.71 \left(\frac{20 - 5}{20 + 5}\right) = 6.43 \text{ ft}^3\text{/ft/d}$$

As the water table mound rises above the bottom of the channel, q_1 is reduced by $\frac{10.71 - 6.43}{2.5}$ times the rise in feet above the bottom.

Often an aerated root zone must be maintained at the edge of an irrigated area adjacent to an unlined channel. This may require a drain. The seepage from the channel and the additional capacity needed in the first drain because of the seepage can be determined by the formula:

$$q_3 = \frac{K_2 \ D_2 \ hs}{X} \tag{4}$$

q_3 = Seepage in cubic feet per linear foot of channel per day when the selected root zone depth at edge of the irrigated area is maintained by a drain,

K_2 = weighted hydraulic conductivity between root zone depth and barrier, feet per day,

D_2 = one-half the sum of the distances between: (1) barrier and water surface in channel, and (2) barrier and selected root zone depth at edge of the irrigated area.

h_s = difference in elevation between selected root zone depth at edge of irrigated field and water surface in channel, and

X = distance from centreline of channel to edge of irrigated area.

Example: If h_s = 4 feet and X = 60 feet, then

$$D_2 = \frac{(20 + 9) + (20 + 9 - 4)}{2} = 27 \text{ feet, and}$$

$$q_3 = \frac{1.5 \times 27 \times 4}{60} = 2.70 \text{ ft}^3/\text{ft/d.}$$

Step 2: If the canal is on a sidehill where the ground-water movement is in one direction and where q_3 is less than q_2, use q_3 as the seepage factor in estimating the distance from the canal centreline to first drain. If movement is in two driections or from a canal on a ridge with irrigation on both sides, when q_3 is less than $q_2/2$, use q_3.

The example in this section has the canal on a sidehill with all ground-water flow in one direction and q_3 less than q_2; therefore, use the q_3 seepage of 2.70 cubic feet per linear foot of channel per day.

Step 3: Estimate the distance from the canal centreline to first required drain by the formula:

$$R = \frac{K_2(H^2 - h^2)}{2q_3} + X \tag{5}$$

where:

R = distance in feet from channel centreline to first required drain,

h = distance in feet between drain and barrier, and

H = distance in feet between barrier and maintained root zone depth at edge of irrigated area.

K_2, q_3, and X are as previously defined.

Example: if h = 20 feet and H = 20 + (9 – 4) = 25 feet, then

$$R = \frac{1.5\,(25)^2 - (20)^2}{2 \times 2.70} + 60 = 123 \text{ feet.}$$

Some irrigation recharge between the drain and the edge of the irrigated area above the drain has not been considered in the calculations. This is accomplished by using the 123 feet as the first estimate of the distance from channel centre line to first required drain. Irrigation recharge between the drain and the channel can be estimated and added to the canal seepage as follows:

(a) Deep percolation from irrigation during the peak period, 7 days between irrigations = 0.37 inch.

(b) Average daily rate of recharge during irrigation season would then be

$$i = \frac{0.37}{12 \times 7} = 0.0044 \text{ foot per day.}$$

(c) Irrigation recharge to be drained between the drain and edge of irrigated area = i(R-X) = (0.0044) (123 – 60) = 0.28 cubic foot per linear foot of drain per day.

(d) Irrigation recharge plus canal seepage q_3 = 0.28 + 2.70 = 2.98 ft^3/d. The second estimate of the distance from channel centreline to the first drain using irrigation recharge plus canal seepage can be calculated.

$$R = \frac{1.5\,(25)^2 - (20)^2}{2 \times 2.98} + 60 = 117 \text{ feet}$$

Irrigation recharge will now be i (R – X) = (0.0044) (117-60) = 0.125 ft^3/d and if added to the canal seepage, q_3 would not change the second estimate of R.

Any additional parallel drains required to keep the watertable below the acceptable level can be computed by the drain spacing methods. These methods were developed for level lands but give an acceptable spacing for slopes upto about 12 percent.

CHASHMA RIGHT BANK CANAL INTERCEPTOR DRAINS

In the area along the unlined headreach of the Chashma main canal, linear interceptor drains will be installed parallel to the main canal and distributaries to recover much of the canal leakage before it causes waterlogging of the land. It has been established that the volume of leakage from the canal and distributary far exceeds the groundwater recharge from other sources. Therefore, this interception of leakage water close to the source would provide satisfactory field drainage and increase the water available for agriculture.

The majority of the soils are loams ranging from sandy loam to clay loam. At depth 1.8 to 3.0 metres the soils are river deposited fine to medium sand. These deposits continue to depth more than 100 metres with intermittent discontinuous lenses of silt, clay and coarse sand. The soils have hydraulic conductivities high enough to permit wide spacing between drains. As the soils have little cohesion, open trench sides will slough during excavation especially in high watertable area.

Approximate Design of Interceptor and Sub-surface Drains Left of Stage – I, Chashma Right Bank Canal (R.S. Broughton);

This example is for a 5,000 ft length of interceptor drainage system parallel to the canal. One pumping station will be needed for each reach of interceptor drainage system. The longer the reach, the smaller will be the number of pumping stations, but more large diameter expensive pipe will be needed. To achieve the most economical overall designs of interceptor drainage systems, several trials will need to be made to find the least cost combination of pumping stations and drain pipe systems. Before this final design decision is taken, a seepage model study should be made to find the range of seepage amounts which can be expected to be intercepted by each line of drain pipes. The example given here shows one possible distribution of seepage interceptions in Fig. 2 to get the numbers to develop a sensible first approximation design.

Distribution of Seepages and Recharges Assumed

The assumed and calculated distribution of seepages and field recharges rates going to each drain pipe is shown in Fig. 2. These seepage rates were obtained as indicated below:

Main Canal

Total seepage losses 117,944 ac-ft/yr.

$$\frac{117,944 \text{ ac-ft/yr} \times 0.5 \text{ cfs/ac-ft/day}}{96,000 \text{ ft of canal} \times 365 \text{ days/yr}} = 0.00168 \text{ cfs/ft of canal}$$

This has been divided as follows:

seeping to right side	35% or 0.00059 cfs/ft
seeping to 5 drains left side	55% or 0.00092
deep seepage to Indus River	10% or 0.00017
Total	100% 0.00168 cfs/ft

Of the 0.00092 cfs/ft seeping to five left side drains

To First interceptor	60% or	0.000552 cfs/ft
To Second interceptor	10%	0.000092
To Third interceptor	10%	0.000092
To Fourth interceptor	10%	0.000092
To Fifth interceptor	10%	0.000092
Total:	100%	0.000920

The recharge from field irrigation losses was assumed to be distributed as follows:

Water from field irrigation losses	14,652 ac-ft/yr
minus non beneficial evapotranspiration	2,685
Net recharge from field irrigation	11,967 ac-ft/yr

Assuming that the irrigated lands are spaced out in a uniform strip 90,000 ft long, the average rate of recharge into the soil to be recovered by the subsurface drains is

$$\frac{11,967 \text{ ac-ft/yr.} \times 0.5 \text{ cfs/ac-ft/day}}{90,000 \text{ ft} \times 365 \text{ day/year}} = 0.000182 \text{ cfs/ft}$$

Assuming that the recharge rate during the peak month to be accommodated by the pipes is 1.2 times the annual average yields 0.000182 x 1.2 = 0.00022 cfs/ft.

Distribution of Field Recharge Rates to Pipes

Zone Description	Zone Recharge Cfs/ft	No. of Pipes	Rate per Pipe cfs/ft
Between Indus River and distributary	0.00006	2	0.00003
Between Distributary and Main Canal	0.00014	4	0.000035
West side of main canal	0.00002	2	0.00001
Total.	0.00022	8	NA

The summation of the canal leakages and field losses for each pipe number is given in figure2 in accordance with the above calculations. Of course these flow rates and the spacings between pipes may be modified before final design on the basis of the results from model studies and field surveys.

However, the preliminary designs will be reasonably close in terms of pipe capacity and pipe sizes since allowance has been made for the total expected leakages. It is quite possible that the total annual leakage to the Indus River will be more than has been assumed, thus reducing the amount to flow to the drain pipes.

Due to soil variability, the inflow per foot of length will vary along the length of the pipe, but field averaging will take place. The pipes will have enough total capacity. If the leakage from the cansls is much higher in some locality causing local wet areas, and if these wet areas do not reduce as silt gets deposited in the canal, then additional drain pipe laterals could be installed to improve the drainage in these localities.

Considering pump operation during off peak hours plus loss of time due to load shedding, expected pump operation would be 70% of the time. Seepage during the time when pumps are not operating, will go into temporary storage in the soil, pump sumps and the pipes. Head will build up in the soil over the pipes so that when the pumps start up, the energy gradient to cause flow towards the pumps will be greater than the pipe slope. This extra energy head will provide some factor of safety to compensate for non uniformities in frictional characteristics of pipes or soil. The actual conveyance capacity is about 10% higher than that given by using Manning's equation and the pipe bed slope, because the discharge is not constant but picks up gradually along the length of the pipes.

TABLE 1

Pipe Length and Diameter distributions for the discharges to be carried by the various component interceptors

Pipe Dia in Inches and type	slope ft/ft	Q Full Pipe cfs	Continuous seepage rates as per diagram Q_1 0.00055 cfs/ft	Q_2 0.000127 cfs/ft	Rate needed for the time Pumps operate $Q_1/0.70$ 0.000786 cfs/ft	$Q_2/0.70$ 0.000181 cfs/ft
1	2	3	4	5	6	7
4 PVC	0.0013	0.049	89	384	62	270
6 PVC	0.0013	0.18	327	1417	229	994
8 PVC	0.0013	0.34	618	2677	432	1878
10 PVC	0.0013	0.54	982	4252	687	2983
12 PVC	0.0013	0.84	1527	6614	1069	4641
12 RCC	0.0013	1.00	1818		1272	
15 RCC	0.0013	2.34	4255		2977	
18 RCC	0.0013	3.38			4300	
21 RCC	0.0013	5.08				
24 RCC	0.0013	7.26				
4 PVC	0.0001	0.043	78	338	55	238
6 PVC	0.001	0.16	291	1260	204	884
8 PVC	0.001	0.30	545	2362	382	1657
10 PVC	0.001	0.48	873	3780	611	2652
12 PVC	0.001	0.74	1345	5827	941	4088
12 RCC	0.001	0.88	1600		1119	
15 RCC	0.001	2.05	3727		2608	
18 RCC	0.001	2.97	5400		3779	
21 RCC	0.001	4.45			5662	
24 RCC	0.001	6.36				

Notes: Col. 3 from pipe discharge capacity graph.

TABLE 2

Pipe Required for Interceptor Drainage System
as per Figure-3.

Diameter Inches	Type	Length Ft	Cost Pipe (Rs/ft)	Cost Gravel (Rs/ft)	Cost Install (Rs/ft)	Cost Total (Rs/ft)	Total Installed Cost Rs.
1	2	3	4	5	6	7	8
4	PVC	2,400	5	7	12	24	57,600
6	PVC	8,200	10	9	32	51	418,200
8	PVC	9,400	13	10	36	56	67,500
10	PVC	900	20	14	41	75	67,500
12	PVC	1,700	31	16	43	90	153,000
12	RCC	300	30	20	163	213	63,900
15	RCC	2,300	45	32	163	240	552,000
18	RCC	1,700	60	34	163	257	436,900
21	RCC		75	38	163	276	
Total		26,400					Rs. 2,275,500
					Rs 455/ft of canal		

Notes:

Col. 3 From Length shown on Figure 3 main interceptor drain plus 4 parallel lateral interceptors.

Col. 4 From Mardan SCARP pipe costs.

Col. 5. From Mardan SCARP x 1.25 for poorer accessibility to gravel for diameters 4" to 12".: Rough Estimate for diameters 12" to 21" RCC.

Col. 6 From Mardan SCARP unit prices x 1.33 to account for start-up costs and other items not part of unit prices for diameters 4" to 12" PVC. For diameters 12" to 21" RCC from excavation and installation prices for Peshawar + Rs 48/ft to account for dewatering costs, as per Khaipur EAST TILE DRAINAGE PROJECT.

Col. 7 Sum of Col. 4 + Col. 5 + Col. 6.

Col. 8 Col. 7 x Col. 3

Fig. 4 shows typical design of a small pumping station for interceptor drains.

LEFT BANK OUTFALL DRAIN

(Interceptor Drains Study)

Model and desk studies have been carried out to investigate alternative means of alleviating seepage along the major canals. Seepage losses for intermediate reaches of the Jamrao canal are estimated to be of the order of 1 to 1.5 cusecs/RD which given the high value of water is worth recovering, if only in part. In this study burried collector pipes in the seepage zone beside the canal have been investigated. The objectives of the model studies were to determine how much seepage could be recovered or inhibited, the impact on the watertable in the canal strip and to estimate the economic viability of the measures.

Burried Pipe Interceptor Drain Studies

Investigations were carried out using an electric analogue and these were complemented by a considerable number of calculations using a finite element solution. Comparison of the two methods was made and found to be good. The finite element method also had the advantage of being extremely quick, once the initial grid had been set up.

Figure 5 presents a flow net plotted with a drain 30m from the centre line of the canal 2m below ground level. This shows the shape of the water table with a depression at the drain and a slight recovery going away from the canal.

The graph shown in Figure 6, gives some insight into the relationship between original and induced seepage components of the drain/canal flow system.

Figure 7 shows that over the distane range modelled, the proportion of original seepage in the drain effluent varies linearly with distance from the canal, such that at 15m it comprises about 30% of the drain discharge and rises to 50% at about 30m from the canal.

Figure 6 shows that the recovered original seepage quantity in terms of volume, is also linear with distances from the canal within the range measured. However, it is weakly related compared to the large changes induced seepage which would occur in this distance range. At larger distances the quantity of recovered original seepage will tend towards a constant value as the induced proportion disappears; the magnitude depending on the slope of the phreatic surface and the drain depth.

If the canal were to fully penetrate the aquifer, the recovered original seepage quantity would be constant at all distances from the canal, but in this case it rises near the canal because of increasing seepage gradients and increasing effective drain penetration in the region of diverging flow.

An approximation for the magnitude of recovered original seepage in drains at distance from canals can be made using standard formula for flow into trenches (or pipes). The discrepancy is less than 20% depending on distance from the canal, although it cannot be checked exactly from the model test results vailable. The formula used here is that developed by Chapman from model studies aimed to develope a partial penetration factor: :

$$Q = \left[0.73 + 0.37\left(\frac{H - ho}{H}\right)\right]\frac{K}{2L}\left(H^2 - ho^2\right) \quad \ldots$$

Where
- Q : flow into drain per unit length
- H : effective aquifer thickness
- ho : effective aquifer thickness below drawdown level of drain
- K : permeability
- L : distance to constant head boundary

For convenience the solutions to this equation are plotted on Figure 8 for different drain depths. The distance L is difficult to estimate for any particular case and hence it is a major drawback to use this method to calculate accurately seepage flows in the general case. It has been taken here as 100m roughly the distance at which induced flow becomes negligible.

Flow divergence factors must be applied to the above relationship to account for canal proximity and different drain canal aquifer geometries and properties. For the model configuration, these factors would be 1.2 and 1.8 for distances of 30m and 15m

Anisotropy

Strongly anisotropic soils are ideal for interceptor drains because the original seepage interception ratio is improved and the quantities of induced flow diminish. The drain becomes a more efficient seepage retrieval device producing a higher percentage of recovered original seepage water in the effluent. The effects of anisotroppy are:

1. Approximately 20% improvement in proportion of original seepage water in drain effluent composition will occur as the anisotropic ratio changes between 1 and 8

2. Up to 15% reduction (for 2 and 3m deep drains) in effluent discharge as a proportion of original canal seepage can be expected for the same anisotropic change. The percentage reduction rises from nearly zero at distance in the region of parallel flow, ranging between 8% and 15% from 30m to 18m from the Canal centreline

Conclusions from the studies

The main points to be taken from the results are:

1. The presence of a drain causes additional seepage to flow from the canal, all of which is collected by the drain.

2. For drains installed along or near the toe of the embankment (about 30m from branch canal centrelines) the maximum amount of original seepage recovered will be about 12% and 27% for 2m and 3m drains respectively in isotropic soils and 13% and 31% for the same drains in anisotropic soils $\left(\frac{KH}{Kv} = 8\right)$. Drains located actually in the embankment would recover more of the original seepage but disproportionately increase the induced quantities.

ELECTRIC ANALOG MODEL STUDIES OF INTERCEPTOR DRAINS FORDWAH-SADIQIA

In order to test the feasibility of proposed interceptor drain along Hakra branch of Fordwah-Sadiqia SCARP, Electric Analog, Model Studies were conducted by WAPDA with the following main objectives:

- How much water is lost from the canal by seepage under present day conditions (K_H of aquifer = 2×10^4 feet per year).
- If the horizontal drains were constructed parallel to the canal how much water would it collect if it were kept pumped down to a level of 6 feet below NSL.

The physical parameters such as head of water in the canal, evapotranspiration and interceptor drain were converted to electrical system with the help of scale/conversion factors. Electrical potentials representing the physical parameters were simulated on the resistor network of model by means of filtered D.C. Power Supplies, Several detecting and measuring devices were used to determine the potential distribution at the internal nodes of the potential head dissipative resistance network of the model as well as potentials and currents at canal, water table and drain boundaries. Several trial experiments have been conducted with different boundary conditions and modifications. The conclusions drawn from the results of the electric analog model experiments are as below:

- The preliminary finding is that locating the drain 300 feet from the canal and keeping the drain pumped down to level 6 feet below natural land surface would substantially increase future canal seepage which is not desirable. The drain would, however, intercept present day seepage, that causes waterlogging, which is desirable
- The model does not reproduce the water table gradient which is rather steep near the canal as observed in the field. It may be attributed to the high permeabilities used to construct the model, whereas the canal may actually be silted.
- The anisotropy ratio ($K_V : K_H$) at such a shallower depth may not be as high as 1 : 10 which has been built into the model. This can evidently make the lateral component of flow about ten times bigger than that of vertical. If permeabilities are as high as modeled, the drain should be farther than 300 feet from the canal to avoid nearly doubling of future canal losses. However, the canal losses becoming permanent ground-water recharge presently appear to be small.

It was recommended that site specific data on permeabilities and head distribution should be collected and fed to the model.

CONCLUSIONS

- Subsurface interceptor drains installed parallel to the source of recharge can alleviate the problem of waterlogging by intercepting seepage.
- The rate of interception depends upon the head in the canal, depth of water table, permeability of soil and the position of interceptor drain.

- The interceptor drains installed at the toe or at a certain distance from the canal and kept pumped down to 2 to 3 metres would induce seepage. However, this additional seepage would be entirely attracted by the drain.
- Analog model studies have shown the feasibility of interceptor drains in intercepting considerable part of the seepage from the canals.
- The most important design consideration would be the applicability of model results to the field situation.
- Economic analysis of interceptor drains shows Economic Internal Rate of Return (EIRR) greater than 15%.

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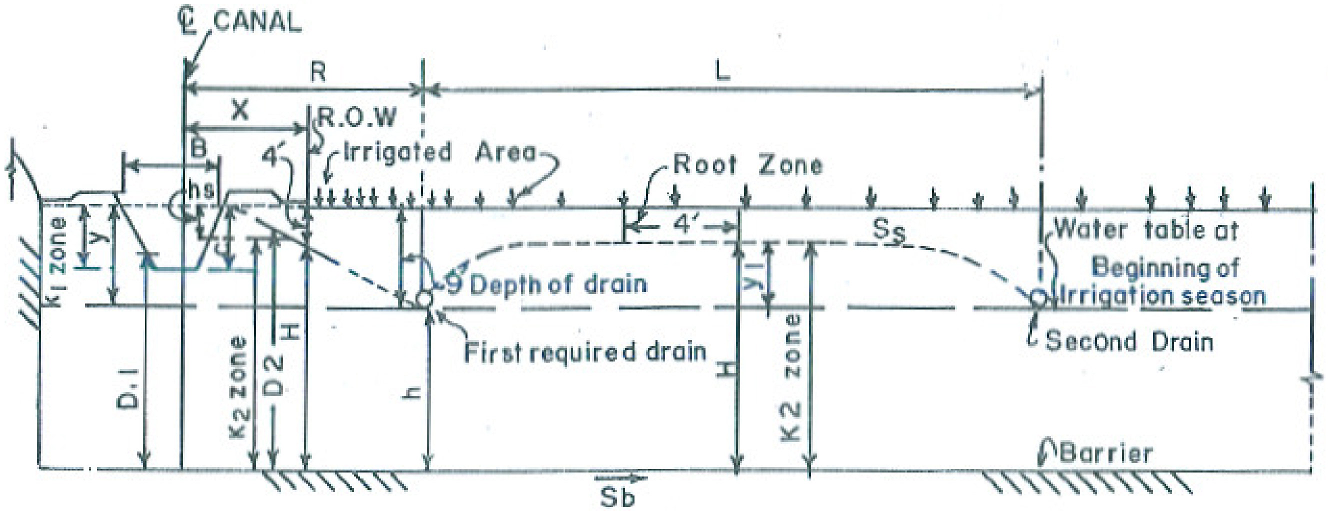

Fig: I LOCATION OF FIRST DRAIN BELOW AN UNLINED CANAL

INFLOWS TO DRAIN PIPES (cfs per 1,000 ft. run)

PIPE NO.	D3	D2	D1	L5	L4	L3	L2	L1	R1	R2	R3
FROM CANALS	.026	.035	.087	.092	.092	.092	.092	.552	.410-	.060	.060
FROM FIELD WATER	.030	.030	.000	.035	.035	.035	.035	.000	.000	.010	.010
TOTAL	.056	.065	.087	.127	.127	.127	.127	.552	.410	.070	.070

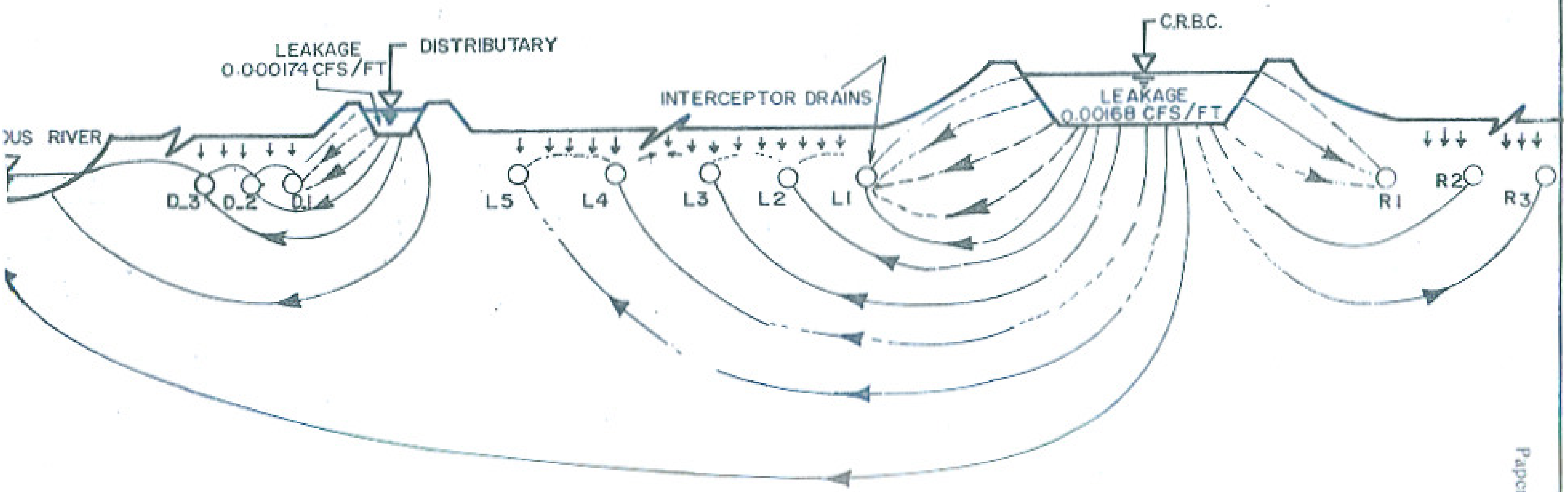

Fig: 2 DISTRIBUTION OF CANAL AND FIELD SEEPAGE ASSUMED FOR DESIGN

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Fig: 3 INTERCEPTOR DRAIN AND SPACED DRAINS
TYPICAL LAYOUT
PLAN

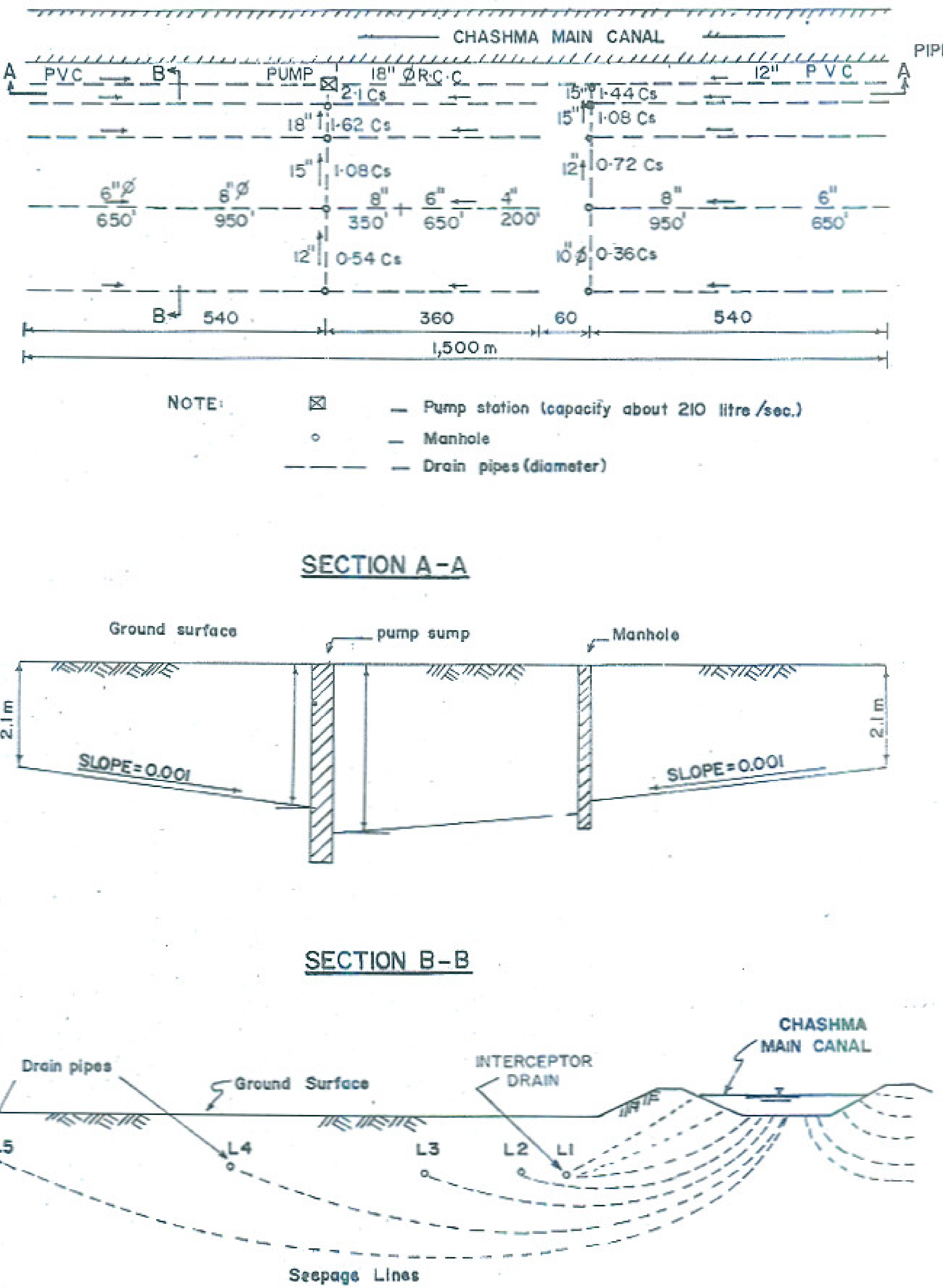

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Fig: 4 TYPICAL PUMPING STATION FOR INTERCEPTOR DRAINS

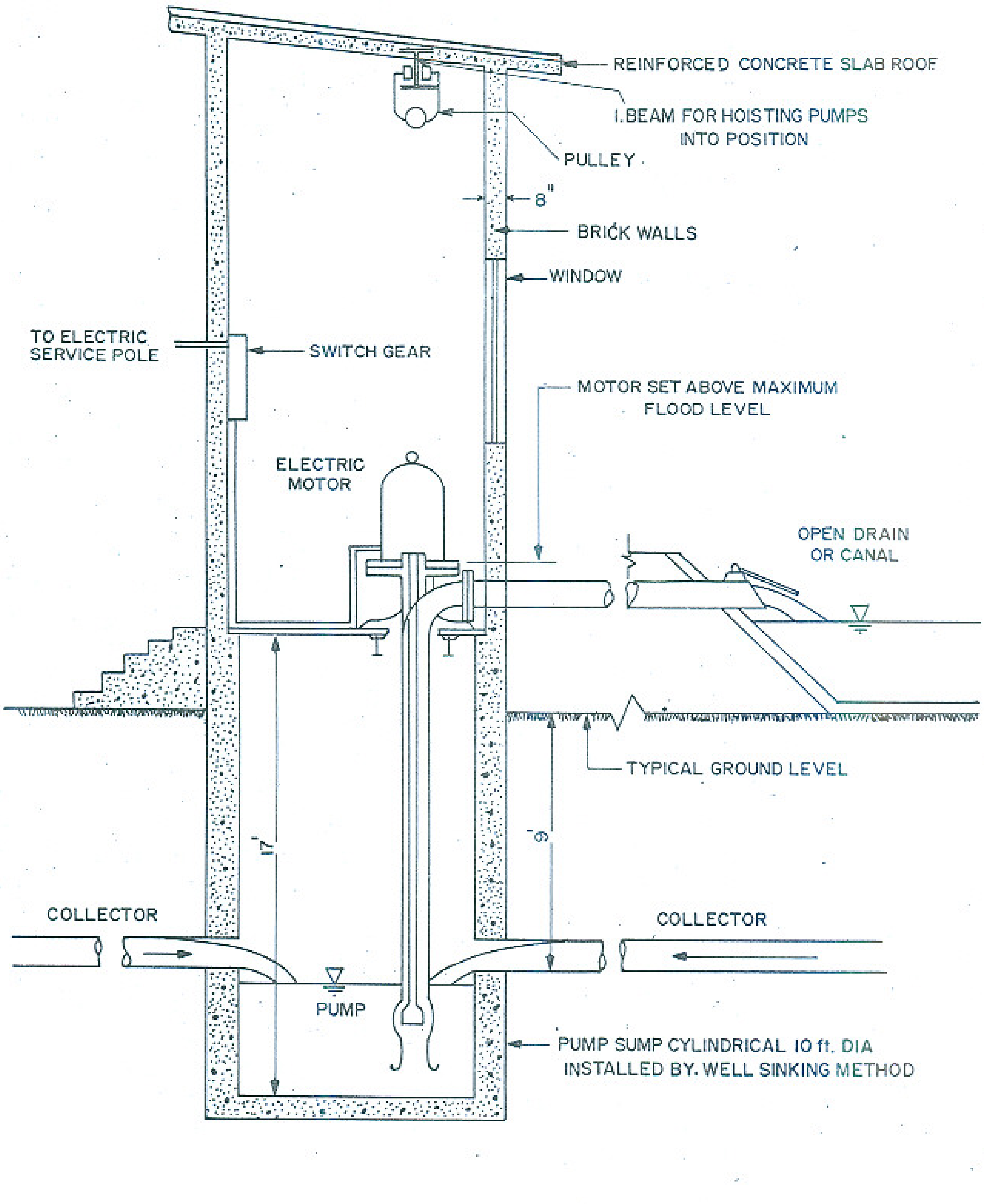

Fig: 5. POTENTIAL DISTRIBUTION USING FINITE ELEMENT SOLUTION

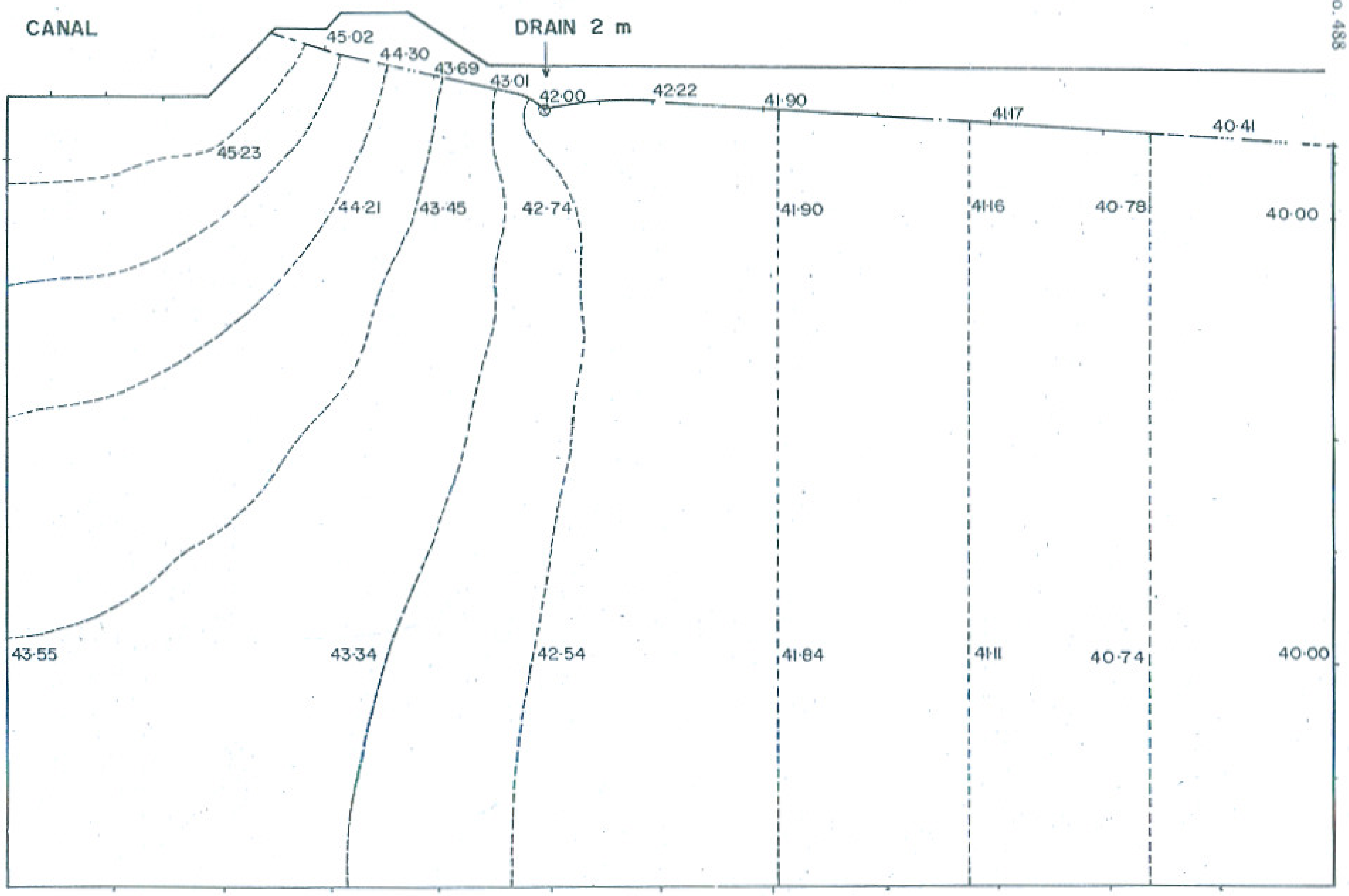

Fig.6 RESULTS OF MODEL STUDIES FOR CANALS OF HIGH ELEVATION IN ISOTROPIC SOIL

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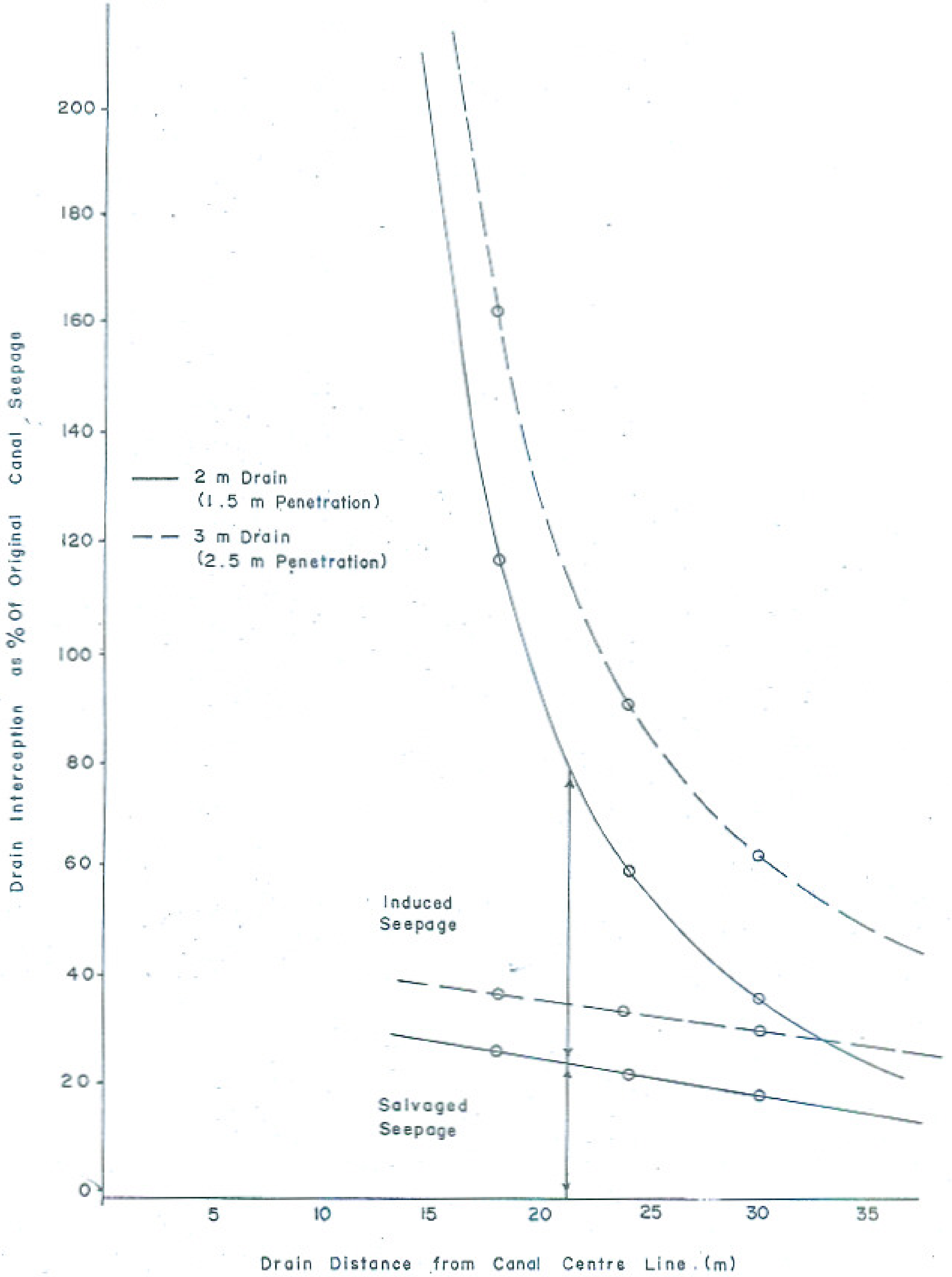


Fig. 7 RELATIONSHIP BETWEEN RECOVERED ORIGINAL SEEPAGE IN DRAIN EFFLUENT AND DRAIN DISTANCE FROM CANAL CENTRE LINE

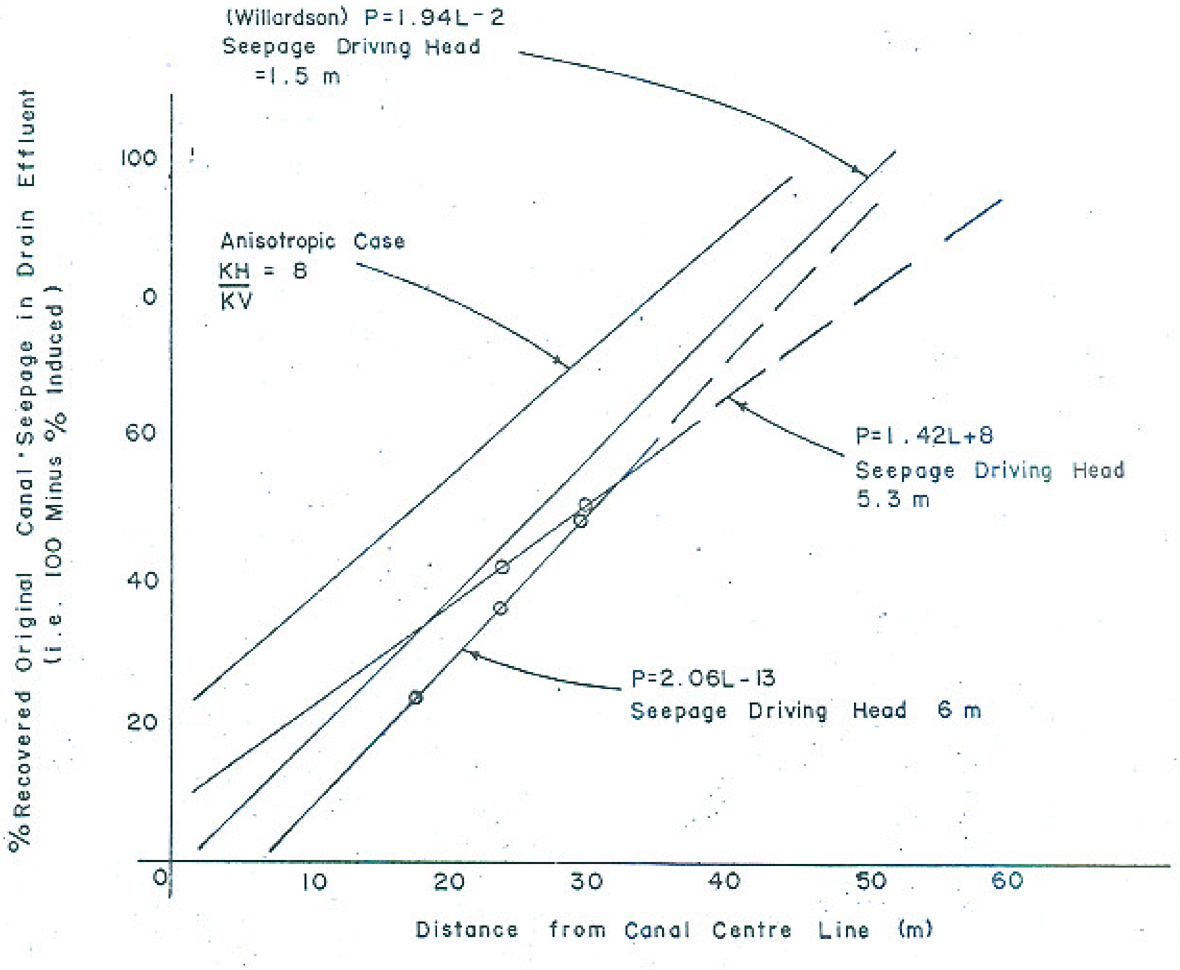
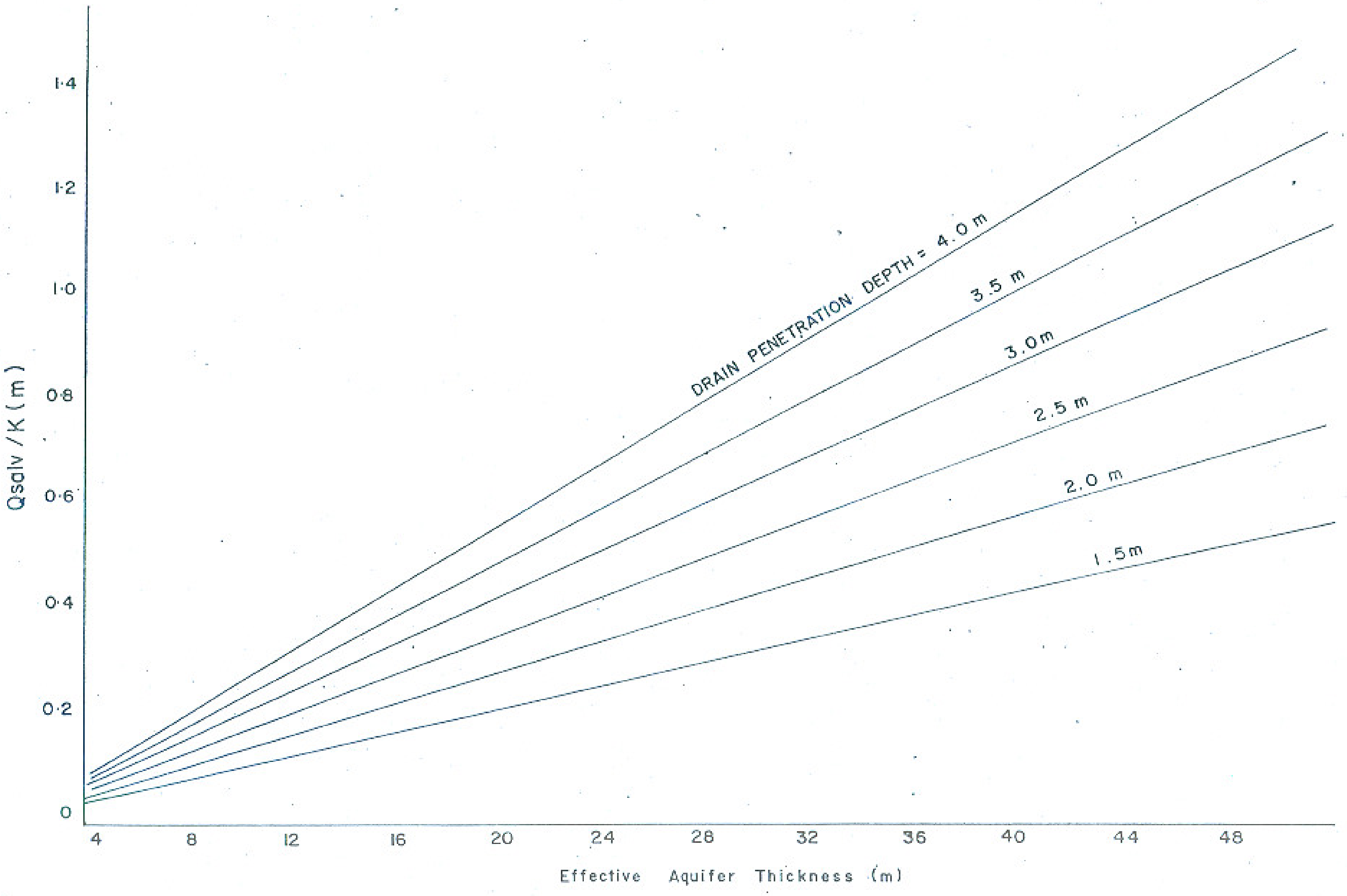


Fig:8.QSALVAGED/K FOR DRAINS AT LARGE DISTANCE FROM CANALS



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