DRINKING WATER MANAGEMENT: AN EXPERIMENTAL INVESTIGATION INTO SITE SELECTION FOR RIVER BANK FILTRATION

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Abstract

The drinking water problem occurring during lean periods (March to July) in a city like Varanasi, India, situated on the crescent shaped concave bank of the Ganga can be minimized with a suitably placed river-bank filtration system. A laboratory model was constructed which showed that the phreatic line slope and the free seepage height played a vital role and the suitable site for the withdrawal of filtered water occurred in the outer half portion of the river. Its location should be situated away from the meandering zone with the least possibility of choking of soil pores. The diverging and converging pattern of streamlines of the city side bank defined the zone of stability, erosion and meandering.

KEYWORDS: River bank; filtration; seepage height; phreatic line; streamlines

INTRODUCTION

River bank filtration (RBF) is the process under which the soil column between river and intake bore well provides the filtering media. By this process we can get filtered river water having better quality than direct intake water from river and having more dissolved oxygen (DO) than the ground water. If the river contribution to water supply is higher than ground water we get more water with sufficiently high DO content with less consumption of power. Due to the sinuosity of river course, interaction between surface water and ground water equipotential lines (Φ), and stream lines ψ continuously change. These changes should factor in the selection of the location of bore wells. Since erosion and deposition on concave and convex banks, respectively, continuously change, the site selection of RBF should be based on the suitability of aquifers and stability of banks. These criteria were applied to the laboratory experiments conducted in this study. It is assumed that the aquifer is found throughout the length of concave bank.

For quantitative and qualitative management of a river bank filtration system, catchment zone, mixing proportions in the pumped raw water, flow paths, and flow velocities of the bank filtrate need to be known. Wett et al. (2002) showed that a stable filter layer is of major importance for the quality of delivered water; the zone of location of intake wells should be free from a meandering zone. Hiscock and Grischek (2002) investigated river bank filtration which can occur under natural conditions or be induced by lowering the groundwater table below the surface water level by extracting water from adjacent boreholes. An important factor is the formation of a layer within the riverbed that has a reduced hydraulic conductivity due to clogging from sediment particles, micro-organisms and colloids. The hydraulic conductivity of river bed is therefore a principal factor in determining the volume of bank filtrate.

Sheets et al. (2002) measured field parameters, such as water level, specific conductance, and water temperature, at a stream flow gauging station and at five monitoring wells at two separate sites corresponding to two nearby production wells. The differences in levels between the monitoring wells and the river during pumping and non pumping periods show whether the river is contributing to intake well or not. During non pumping periods when ground water movement is towards river, choked soil pores may be cleared.

Chen (2003) presented analytical solutions that can be used to evaluate stream filtration and base flow reduction induced by groundwater pumping. Bakker and Hemker (2004) derived analytical solutions for flow through an elongated box-shaped aquifer but did not discuss the selection of bore well sites, depending upon the quality of withdrawal and its stability condition. Also the numerical simulations are insufficient on account of the following reasons: (i) Stream lines and equipotential lines are frequently changing along the bank in the longitudinal direction. (ii) The coefficient of permeability also changes as the bank region from concave to convex changes. (iii) The phreatic line slope is also a function of withdrawal from the bank which is difficult to be accommodated mathematically.

The objective of this study therefore is to experimentally determine the location of bore wells such that maximum volume of good quality water can be extracted from river side, the soil of the site is stable and choking of soil pores due to pollutants logging near the bank is minimized.

Material & Methods

Experiments were conducted in the Hydraulic Laboratory, Civil Engineering Department of Institute of Technology, Banaras Hindu University, Varanasi, India. A rectangular tank 90 cm × 89.5 cm × 35 cm, as shown in Figure 1, was used for experimentation. The tank was divided into three compartments. The first compartment was supported by a fine mesh (100 no. gauge) welded over a hard iron net. This part is a 10 cm wide and 90 cm long straight flume, and serves as a straight canal (model). A small pipe 1.25 cm in diameter was welded on the front side of the canal to serve as an inlet pipe. A sluice valve, 12 cm above the bottom of canal, was fitted to this pipe in order to regulate the supply of water into the canal. On the exit side, a small pipe 1.25 cm in diameter was attached to serve as exit pipe and it was fitted with a sluice valve 8 cm above the bottom of canal to control the flow. A 7 cm × 7 cm weir at 20 cm above the bottom of canal was provided at the tail end to measure discharge. The outlet valve and the weir allowed to maintain the desired water level in the canal during experimentation.

The second compartment of the tank served as a porous medium, and was filled with the Ganga sand which has a specific gravity of 2.59, coefficient of permeability of 1.01 x 10^{-3} cm/s, coefficient of uniformity (Cu) of 1.53, and coefficient of curvature (Cc) of 0.838. This compartment was divided into three sections: 1-1 (downstream), 2-2 (central), and 3-3 (upstream). Fourteen wells, each a perforated pipe 1.9 cm (3/4") in diameter and 35 cm long, were placed in rows in the three sections between canal and river. The wells were designated as W_{11}, W_{12}, W_{13}, W_{14}, and W_{15} in section 1-1; $W_{21,\ldots,}W_{24}$ in section 2-2; and $W_{31,\ldots,}$ W_{35} in section 3-3. In this nomenclature, W_{11}=1st well in section 1-1 at the constant distance from the canal; W_{12}=2nd well in section 1-1 from the canal; W_{21}=1st well in section 2-2 from the canal; and similarly for other wells.

The location of each observation well was measured from the canal side. Each well was covered by 100 no. gauge fine mess to prevent the fine sand from entering into the well. There were five wells 9 cm apart in the upstream row, four wells 11 cm part in the central row and five wells 9 cm apart in the downstream row. These rows also served to divide the canal and the curved channel into three sections or reaches. A rubber pipe of small diameter was connected through each well (acting as a peizometer) to the vertical piezometer for the respective wells.

The third compartment of the tank was a 15 cm wide curved flume with a radius of curvature (r_c) of 82.5 cm and arc length (l_r) of 95.6 cm. This compartment served as the model of the curved river with a uniform cross-sectional area. The seepage face of the river model (the face through which canal water seeped) was made of a fine mess fixed over coarser iron net which was welded. The fine mess allowed only the movement of water. A small pipe 2.5 cm in diameter was fitted with a sluice valve 5 cm above the river bottom on the front side, as an inlet pipe, to supply water to the river.

The inlet pipe was connected to a pump (1.5 HP) to draw water from a steel drum which served as a reservoir; water coming into the drum was recirculated to the inlet of river model by the electrical pump. An outlet pipe 1.25 cm in diameter was fitted at the end of the river model 5 cm above the river bottom in order to discharge the river water and control the head in the river. A 13.3 cm x 10 cm rectangular weir was fitted 10 cm above the river bottom. The weir and the exit pipe served as the outlet. On the upstream and downstream sides of the observation tank perpex sheets were fixed to visualize the flow, and streamlines were visualized by injecting potassium permanganate at different points in the porous medium. This model (not to the scale) explains quantitatively the ground water flow pattern towards the river under the hydraulic gradient between channel and river model. The seepage from channel to river model is through the sand bed working as an aquifer.

Experiments were conducted to investigate seepage profiles as a result of the interaction between ground water and surface water through river bank filtration (RBF). This experimentation is the replication of real situation existing in the field. This permits to determine how the interaction between river water and ground water leads to criteria for choosing the location of intake bore well by taking into consideration the choking of soil pores and meandering. The curved flume simulated the river and the straight flume simulated groundwater. A head difference, resembling the real situation, was created between the straight canal and the curved flume. This head difference was the cause of groundwater flow and the groundwater-surface water interaction and the consequent development of seepage profiles. When the pump was started, the inlet valve on the riverside opened, and the water flowed in the river model. Three sets of experiments were conducted for different head differences. The head differences between straight flume and curved flume for different sets of experiment are shown in Table 1.

The water level in each well was observed using the vertical piezometer and the head at a well was calculated by deducting the piezometer reading from the height of model (or say canal height =35 cm). Then the water was withdrawn through each well and observations were made. In all experiments, variations in the water level and hence in the phreatic line were observed for each head difference. Steps for making observations were similar for the three sets of experiments.

Water levels in wells W_{11} to W_{15}, W_{21} to W_{24} and W_{31} to W_{35} were measured using the piezometer for three sets of head differences between the canal and river model. In each set of wells, the water was extracted individually from each well and the water levels in different wells were recorded. These measurements permitted to give a set of phreatic lines and their variation. These measurements depict the change in phreatic line characteristics when ground water is withdrawn individually from different wells. Also obtained was free seepage height under various scenarios. These parameters relate the quantity of drinking water withdrawn to river meandering.

Discharge was measured with the use of a rectangular notch (attached at the downstream end of the river model) when the flow (in river) stabilized. After the stabilization of flow, the inlet valve on the canal side was opened and the tap water was allowed to flow in the canal to the desired level. At the steady state condition (i.e., when rate of seepage attained a constant value of head (h) over the notch on the river side), the rate of seepage was obtained. Then water levels in all the wells were simultaneously recorded with the help of a vertical piezometer connected to the respective wells.

The velocity in the river model was calculated using a Pitot tube in 3 different sections. In every section the velocity was recorded at longitudinally three equi-distant locations from outer walls of the river model. The water from the outlet of canal model was collected in a drum 47 cm in diameter to have the coefficient of discharge of the rectangular notch at the desired head.

The discharge in the river was calculated using the weir formula as

$$Q_r = \frac{2}{3} C_d B \sqrt{2g} H^{\frac{3}{2}} \quad (1)$$

where Q_r = the discharge in the river, C_d= the coefficient of the rectangular weir (assumed as 0.6); B=the length of the weir; and H = the head of water above the weir. The average velocity (V_r) of flow in the river was computed as $V_r = Q_r / A$, where A= cross-sectional area of flow in the river (Ackers et al. 1978).

For the first set of experiments, the depth of water in the canal was maintained up to a constant height of 14 cm by controlling the flow using outlet and inlet valves. The discharge in the canal was measured by observing the time of collection of water in a container of known volume. The total discharge through the river model (including the seepage discharge) was obtained by recording the head over the notch, and the rate of seepage discharge was obtained by deducting the river discharge from the total discharge.

Then, water was withdrawn from the intermediate wells one by one with the use of a small diameter rubber pipe through the suction of water from the wells (siphon action). The discharge of the extracted water was calculated by recording the time it took to fill the measuring flask of 500ml. The level in each well was observed using the vertical piezometer when water was withdrawn from a corresponding well and the variation in phreatic line was then drawn. This process was repeated for all 14 wells. The mean variation in water level due to the extraction of water was recorded from all 14 wells one by one.

The boundary conditions for streamlines and equipotential lines of ground water flow at the interface of ground water and river vary with space and time. Characteristics of curvilinear flow of river also vary with space and time. The simple method of experimentation and qualitative model employed in this study can be effectively used to determine the location of bore wells for efficient extraction of safe drinking water through river bank filtration.

ANALYSIS AND DISCUSSION OF RESULTS

All experimental results are presented in Table 2. From amongst all possible scenarios investigated experimentally, seven of them are discussed here. Important wells from the river bank filtration (RBF) point of view can be easily observed from Table 2. The wells farther from the river model are not included in the discussion, except W_{11}, to show that there is no contribution of river in the farther wells. Also, only one well W_{15} is selected from the nearest three wells to avoid repetition of discussion.

These six scenarios are: (1) No water was withdrawn from the wells to show how phreatic line characteristics varied due to the introduction of curvilinear flow in the downstream section. (2) The water was withdrawn from well W11 of the downstream section corresponding to the three different head differences. (3) The water was withdrawn from well W14 corresponding to the three different head differences. (4) The water was withdrawn from well W15 corresponding to the three different head differences. (5) The intake well W23 was in the central section for the three different head differences. (6) The intake well W34 was in the upstream section, which had the same mirror image location as W14 of the downstream section.

By plotting non-dimensional head values as Δh/ΔH versus X/L seepage profiles were constructed in dimensionless form for each scenario; here Δh = head in the well with respect to the water level in the curved flume (river model) at a distance X from the canal, ΔH = average head difference between the canal water level and curved flume water level, X=location of the intake well from the canal, and L = distance between the canal and curved flume. In this way, the phreatic line variation was depicted under the influence of curvilinear flow in the downstream section and under various conditions of withdrawal of water through different wells. The following results were obtained after experimentation under different scenarios.

Scenario 1: Corresponding to the average head difference of 9.8 cm between straight and curved flumes, seepage flow was computed. The free seepage heights (the level difference between surface water and groundwater at the interface, shown in Figure 2 as Hfsd, Hfsc, Hfsu) of the corresponding sections were 4.297 cm (0.443×9.7) for the downstream section, 4.098 cm (0.414×9.9) for the central section, and 3.9cm (0.39×10) for the upstream section. Seepage profiles are presented in non-dimensional term (Δh/ΔH), as shown in Figure 3.1(a). The value of Δh/ΔH in the straight flume was 1 and in the curved flume was 0.443. The seepage profile varied from 1 to 0.443 for the downstream wells; whereas for central wells it varied from 1 to 0.414 and for the upstream well it varied from 1 to 0.39. It was observed that the free seepage height was greater along the downstream well section than along central well and upstream well sections.

From a meandering point of view, the downstream section was comparatively unstable. This can be explained by noting that the volume of water flowing in the river was increasing in the downstream wells (as seepage discharge was also added). More and more water accumulated on the concave side and the extra water was forced to remain on the side on account of centrifugal action (which developed due to the curvilinear nature of the river). Therefore, the rising head of water on the outer bank side enhanced the free seepage height with the increase in distance of the downstream river section. For the average head difference of 9.1 cm and 6.3 cm, it can be seen that the variation of the seepage profile from the canal to river in Table 2 and also in Figures 3.1(b) and 3.1(c). The same trend was observed for the head difference of 9.8cm in that the free seepage height was greater along the downstream well section.

Scenario 2: In this case, water was withdrawn at a rate of 2.86 cm^3/s through the intermediate well W_{11} of the downstream section. For the head difference of ΔH=9.8 cm between the straight flume and the curve flume water level, one can see the variation of the seepage profile from the canal to river in Table 2 and Figure 3.2(a). In this scenario, the free seepage height fell more in the downstream section and was lowest amongst the three sections. This reduction in the free seepage height caused the gradient of seepage line steeper and the seepage velocity to increase. Due to the increase in seepage velocity, meandering would be enhanced and river bank would be unstable. The water level in the intake well decreased from 20.8 cm ($\Delta h/\Delta H$=0.948) to 15.9 cm ($\Delta h/\Delta H$=0.4433) due to the withdrawal of water, and the water level fell in each well. The water level fell more in the longitudinal well (W_{12}, non-dimensional head =0.67, well along the seepage line) than in the transverse well (W_{21}, non-dimensional head =0.808, well perpendicular to the direction of seepage line).

Since the velocity of water was along the seepage line, more variation in the water level was observed in the longitudinal well. When the head difference was ΔH=9.1 cm and the water was withdrawn at a rate of 2.85 cm^3/s from well W11, the variation in seepage profile from the canal to river can be seen in Table 2 and Figure 3.2(b). Again the seepage height decreased more in well W_{11}; indeed it fell down in each section. The water level in the intake well decreased from 19.6 cm ($\Delta h/\Delta H$=0.922) to 15.1 cm ($\Delta h/\Delta H$=0.422), and the radius of influence was created surrounding this well. Similar results were observed for a head difference of 6.3 cm, as shown in Figure 3.2(c). It was thus concluded that the free seepage height was directly proportional to the head difference (ΔH) between the straight channel and curved flume. As the value of the head difference (ΔH) decreased from 9.8cm to 6.3 cm the value of free seepage height decreased for each well section and therefore the gradient of seepage lines also decreased. The water level in the intake well was also proportional to the head difference (ΔH).

Scenario 3: In this case, the water was withdrawn at a rate of 2.834 cm^3/s from well W14 for three different head differences, and in all three cases seepage profiles were similar. However, the water level in the intake well went below the river for all the three hydraulic head differences. For a head difference of ΔH=9.8 cm, one can see the variation in the seepage profile from canal to river in Table 2. The water level in the intake well decreased from 17.2 cm ($\Delta h/\Delta H$=0.577) for no water withdrawal to 11.4 cm ($\Delta h/\Delta H$=-0.0206) due to the withdrawal of water [Figure 3.4(a)]. Similarly the water level in the intake well for a head difference of 9.1 cm [Figure 3.4(b)] and 6.3cm [Figure 3.4(c)] fell below the river water level.

Scenario 4: In this case the intake well was located in the downstream section at a distance of 45 cm from straight flume and the same three different head differences were considered. This is the nearest well to the curved flume. The water was withdrawn at a rate of 2.90 cm^3/s from well W_{15} for ΔH=9.8 cm. One can see the variation in seepage profile from canal to river in Table 2 and Figure 3.5(a). The water level in the intake well decreased from 15.9 cm ($\Delta h/\Delta H$=0.433) for no water withdrawal to 8.9 cm ($\Delta h/\Delta H$= -0.278) due to the withdrawal of water. When the head difference was ΔH=9.1 cm and the rate of withdrawal was 2.63 cm^3/s, variations in seepage profiles are shown in Table 2 and Figure 3.5(b). The water level in the intake well decreased from 15.2 cm ($\Delta h/\Delta H$=0.433) for no water withdrawal to 7.1 cm ($\Delta h/\Delta H$= -0.4667) due to the water withdrawal. For ΔH=6.3 cm and the rate of withdrawal of 2.65 cm^3/s, variations in seepage profiles are shown in Table 2 and Figure 3.5(c). The water level in the intake well decreased from 14.5 cm ($\Delta h/\Delta H$=0.46) for no water withdrawal to 5.3 cm ($\Delta h/\Delta H$= -1.0) due to the withdrawal. The water level in the intake well depended on the head difference (ΔH). As ΔH decreased from 9.8cm to 6.3cm, water level in the intake well also decreased from 8.9 cm ($\Delta h/\Delta H$ = – 0.278) to 5.3 cm ($\Delta h/\Delta H$= -1.0). At this location of intake well, the water level was always below the river water level at all three different head differences. Thus, the water coming in the intake well (through seepage) was predominantly from the river and the contribution of groundwater was little.

Scenario 5: The intake well W_{23} was located in the central well section at a distance of 33 cm from the straight flume. For ΔH=9.8 cm and rate of withdrawal of 2.73 cm^3/s from the intake well, variations in seepage profiles are shown in Table 2 and Figure 3.6(a). The water level in the intake well decreased from 17.2 cm ($\Delta h/\Delta H$=0.586) for the no water withdrawal case to 13.0 cm ($\Delta h/\Delta H$=0.16), when water was withdrawn. For ΔH=9.1 cm and rate of withdrawal of 2.80 cm^3/s variations in the seepage profile are shown in Table 2 and Figure 3.6(b). The water level in the intake well decreased from 16.6 cm ($\Delta h/\Delta H$=0.593) for the no water withdrawal case to 12.9 cm ($\Delta h/\Delta H$=0.189) when water was withdrawn. For ΔH=6.3 cm and the rate of withdrawal of 2.86 cm^3/s, variations in seepage profile are shown in Table 2 and Figure 3.6(c). The water level in the intake well decreased from 15.4 cm ($\Delta h/\Delta H$=0.603), for the no water withdrawal case to 11.1 cm ($\Delta h/\Delta H$=-0.0794) when water was withdrawn. The negative value indicates that the water level in the intake well fell below the river water level.

From the above discussion it is seen that as the head difference (ΔH) decreased the free seepage height decreased. It was minimum for withdrawal central section, but it varied with ΔH (the non-dimensional head of the last well was 0.333 at ΔH=9.8 cm, 0.341 at 9.1 cm and 0.238 at 6.3 cm). Also the water level in the intake well was dependent upon the head difference (ΔH). The lower the value of ΔH (=6.3 cm), the lower was the water level in the withdrawal well (= -0.0794), whereas for higher head differences, higher was the level: the level was 0.162 for ΔH=9.8 cm, and 0.187 for ΔH =9.1 cm.

Scenario 6: The intake well, W_{34}, was located in the upstream section at a distance of 36 cm from the straight flume. For ΔH=9.8 cm and rate of withdrawal of 2.99 cm^3/s, the variation of seepage profile is shown in Table 2 and Figure 3.10(a). The water level in the intake well decreased from 16.3cm ($\Delta h/\Delta H$=0.5) for no water withdrawal case to 11.8cm ($\Delta h/\Delta H$=0.05) when water was withdrawn. For ΔH=9.1cm and the rate of withdrawal of 2.86 cm^3/s, the variation in seepage profile is shown in Table 2 and Figure 3.10(b). The water level in the intake well decreased from 15.7cm ($\Delta h/\Delta H$=0.494) for no water withdrawal to 11.1cm ($\Delta h/\Delta H$= -0.011) when water was withdrawn. The negative head showed that the water level in the intake well fell below the river water level. For ΔH=6.3 cm and the rate of withdrawal of 2.78 cm^3/s, the variation of seepage profile is shown in Table 2 and Figure 3.10(c). The water level in the intake well decreased from 14.8cm ($\Delta h/\Delta H$=0.515) for no water withdrawal case to 9.6 cm ($\Delta h/\Delta H$= -0.297) due to the withdrawal of water. Again, the negative head showed that the water level in the intake well fell below the river water level, meaning the contribution of river water to the seepage discharge in the intake well. From the above discussion it is seen that as the head difference (ΔH) decreased the free seepage height also decreased. It was minimum for the withdrawing upstream section and was dependent upon ΔH (non-dimensional head of the last well was 0.25 at ΔH=9.8 cm, 0.219 at 9.1cm and 0.187 at 6.3cm). The water level in the intake well was dependent upon the head difference (ΔH). The lower the value of ΔH (=6.3 cm), the lower was the water level in the withdrawing well (= -0.297), whereas at a higher head difference ΔH=9.8 cm, level was 0.05 and it was -0.011 at ΔH= 9.1 cm. This suggests that the intake well should be located such that there is major participation of river water instead of ground water with the improved quality (quality is related with the distance of intake well from the river; greater the distance better will be the quality).

CONCLUSIONS

The following conclusions are drawn from this study:

(i) Phreatic line slopes of three different sections (d/s, central and u/s) are different and therefore discharge from each well is different.

(ii) In general (except for two wells of the central section) with the fall in head difference (ΔH) between the ground water level (canal model) and river water level the rate of withdrawal from intake well decreases.

(iii) In any particular section, the water level in the intake well is dependent upon the head difference (ΔH) between the ground water level (canal model) and river water level. The smaller the value of ΔH, the lower is the water level in the intake well and greater is the chance of contribution of river to the intake water.

(iv) In the case of intake wells W_{13}, W_{14}, W_{15}, W_{23}, W_{24}, W_{33}, W_{34} & W_{35} the dimensionless head ($\Delta h/\Delta H$) falls below the water level in the river. This refers to the situation where the contribution of river water is dominant in the seepage discharge in the intake well. If river contribution is higher than ground water we get more water with sufficiently high D.O. content with less consumption of power. Hence, for river bank filtration the location of the intake well should be such that there is a major contribution of filtered river water instead of ground water.

(v) The free seepage height decreases more in the section of wells through which water is withdrawn for a particular head difference.

(vi) For a constant head difference of 9.8 cm, there are four cases when it is seen that the dimensionless head ($\Delta h/\Delta H$) falls below the water level in the river model. These four cases are when water is withdrawn through W_{14}, W_{15}, W_{24}, &W_{35}. Wells W_{15} and W_{24} are nearest to the river and very likely to be affected by the meandering of river. Well W_{35} is not suitable, because it is susceptible to pollutant logging and consequently choking of soil pores. Therefore, well W_{14} is important from an RBF point of view.

(vii) As the head difference (ΔH) is decreasing the contribution from river side is pronounced in more number of intake wells. For a head difference of 9.1 cm, the useful wells are W14 and W34. Between these two wells, W14 is having a larger dimensionless head of –0.021. But for ΔH=6.3 cm, the useful wells are W13, W14, W22, W23, W32, W33 and W34, where the dimensionless head ($\Delta h/\Delta H$) falls below the water level in the river model. When head falls, many wells can be considered where the river water influence can be seen. The lowest value obtained in these wells is –0.762 for well W14. So well W14 is supposed to be the best well because from this intake well we get more water with sufficiently high D.O. content and less consumption of power.

APPENDIX I: REFERENCES

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APPENDIX I: NOTATION

A	= Cross-sectional area of water in the river model
B	= length of weir
Cc	= Coefficient of curvature
Cd	= Coefficient of discharge
Cu	= Uniformity Coefficient
D	= Diameter
D10	= Effective Size, 10 percent finer size
D30	= Effective Size, 30 percent finer size
D60	= Effective Size, 60 percent finer size
Dw	= Drawdown in the intermediate well
Δh	= Head in the well with respect to the water level in curved fume at distance X from the canal
ΔH	= Average head difference between the canal water level and curved flume water level
K	= Coefficient of permeability
L	= Distance between the straight channel and curved Channel
Nr	= Reynolds number
Q	= Flow rate through sand bad
r	= Distance from the pumping well
Ss	= Specific elastic storage
s	= Draw down
t	= Time
tmin	= the minimum travel time
Vr	= Average velocity of flow in river model
X	= Distance of intermediate well from the canal side
ρ	= Density of Water
μ	= Dynamic Viscosity of Water

Table 1. Head Differences between Straight Flume and Curved Flume for Different Sets of Experiments.

Experimental Set	Head at the Straight Flume (cm)	Head at the Curved Flume (cm)	Average Head Difference Between Straight and Curved Flume (ΔH) (cm)
1.	21.3	D/S Section= 11.6	9.8
		Central Section= 11.4	
		U/S Section= 11.3	
2.	20.3	D/S Section= 11.3	9.1
		Central Section= 11.2	
		U/S Section= 11.2	
3.	17.9	D/S Section= 11.6	6.3
		Central Section= 11.6	
		U/S Section= 11.5	

Table 2: Variation in seepage profile and head responsible for river contribution in each scenario.

Well no.	Average Head difference between canal & river (ΔH) cm	Withdrawal rate (cm^3/s)	Variation in seepage profile in dimensionless terms	Free seepage height (cm)	Dimensionless head responsible for river contribution
W_{11}	9.8	2.86	d/s- 1 to 0.3917	3.799	Nil
			central- 1 to 0.404	3.999	Nil
			u/s- 1 to 0.38	3.80	Nil
	9.1	2.85	d/s- 1 to 0.433	3.897	Nil
			central- 1 to 0.428	3.895	Nil
			u/s - 1 to 0.417	3.919	Nil
	6.3	2.82	d/s - 1 to 0.381	2.399	Nil
			central- 1 to 0.397	2.499	Nil
			u/s- 1 to 0.391	2.499	Nil
W_{14}	9.8	2.834	d/s- 1 to 0.278	2.696	0.0206
			central- 1 to 0.373	3.693	Nil
			u/s- 1 to 0.37	3..70	Nil
	9.1	2.80	d/s- 1 to 0.244	2.196	0.211
			central- 1 to 0.4065	3.699	Nil
			u/s- 1 to 0.4065	3.699	Nil
	6.3	2.88	d/s- 1 to 0.333	2.09	Nil
			central- 1 to 0.38	2.394	Nil
			u/s- 1 to 0.375	2.4	Nil
W_{15}	9.8	2.90	d/s - 1 to -0.278	-2.70	0.278
			Central- 1 to 0.363	3.6	Nil
			u/s- 1 to 0.37	3.70	Nil
	9.1	2.63	d/s - 1 to -0.467	-4.2	0.4467
			central- 1 to 0.406	3.7	Nil
			u/s- 1 to 0.417	3.8	Nil
	6.3	2.65	d/s - 1 to -1	-6.3	1.0
			central- 1 to 0.381	2.4	Nil
			u/s - 1 to 0.391	2.5	Nil
W_{23}	9.8	2.73	d/s- 1 to 0.391	3.8	Nil
			central- 1 to 0.333	3.3	Nil
			u/s- 1 to 0.36	3.6	Nil
	9.1	2.80	d/s- 1 to 0.4	3.6	Nil
			central- 1 to 0.341	3.1	Nil
			u/s- 1 to 0.374	3.4	Nil
	6.3	2.86	d/s- 1 to 0.381	2.4	Nil
			central- 1 to 0.238	1.5	0.0794
			u/s- 1 to 0.312	2.0	Nil
W_{34}	9.8	2.99	d/s- 1 to 0.412	4	Nil
			central- 1 to 0.374	3.7	Nil
			u/s- 1 to 0.25	2.5	Nil

	9.1	2.86	d/s- 1 to 0.433	3.9	Nil
			central- 1 to 0.406	3.7	Nil
			u/s- 1 to 0.219	2.0	0.011
	6.3	2.78	d/s- 1 to 0.397	2.5	Nil
			central- 1 to 0.349	2.2	Nil
			u/s- 1 to 0.187	1.2	0.297

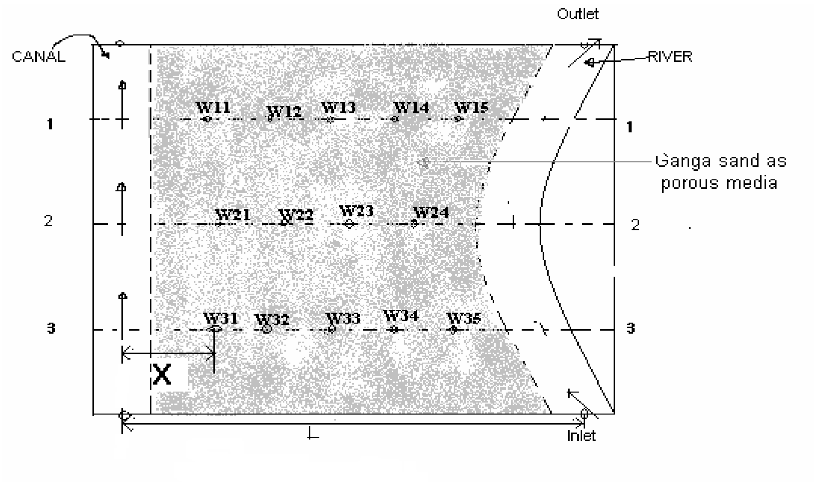


Figure 1. Plan of the experimental model

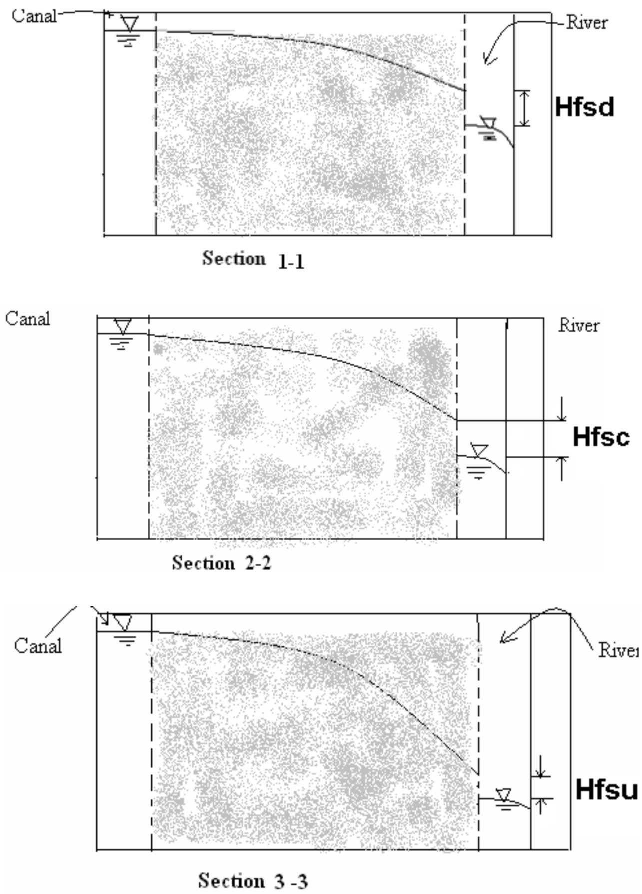


Figure 2. Sections of the experimental model.

No water is withdrawn

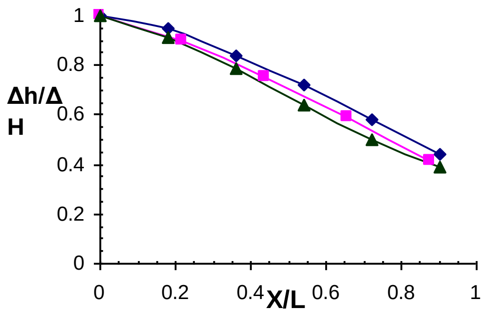


Fig 3.1.a. Seepage profile of wells at ΔH=9.8 cm

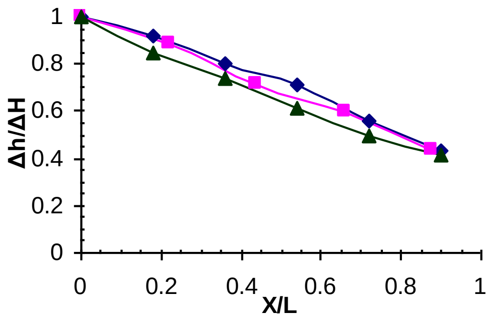


Fig 3.1.b. Seepage profile of wells at ΔH=9.1 cm,

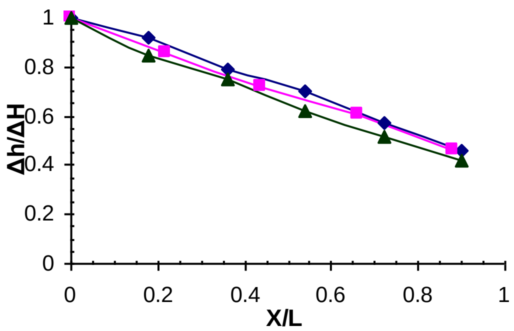
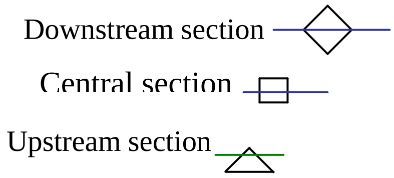


Fig 3.1.c. Seepage profile of wells at ΔH=6.3 cm

Water withdrawn from well W11



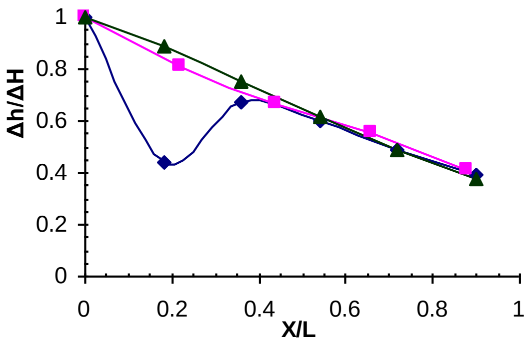


Fig 3.2.a. At ΔH=9.8 cm @2.86 cm3/s

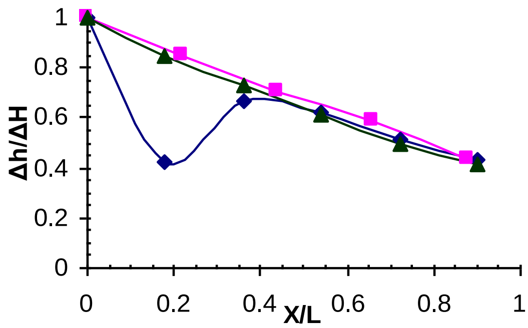


Fig 3.2.b. At ΔH=9.1 cm @2.85cm3/s

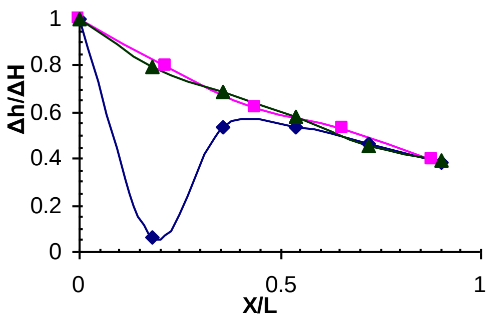


Fig 3.2.c. At ΔH=6.3 cm @2.82 cm3/s

Water withdrawn from well W14

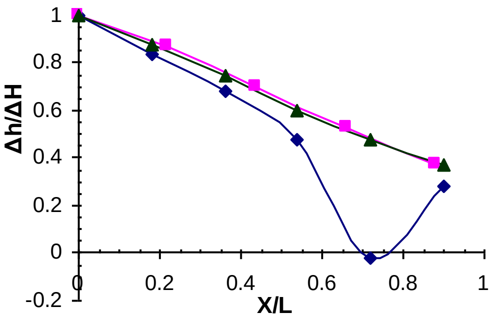


Fig 3.4.a. At ΔH=9.8 cm @2.834 cm3/s

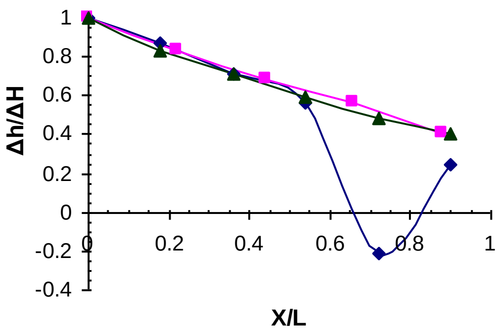


Fig 3.4.b. At ΔH=9.1 cm @2.80 cm3/s

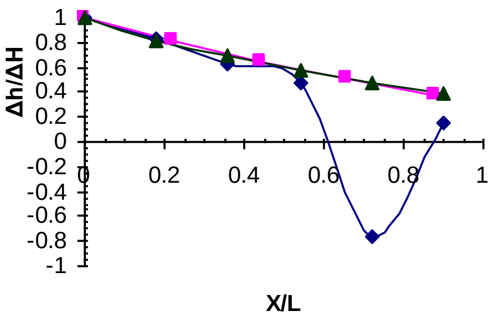


Fig 3.4.c. At ΔH=6.3 cm @2.73 cm3/s

Water withdrawn from Well 15

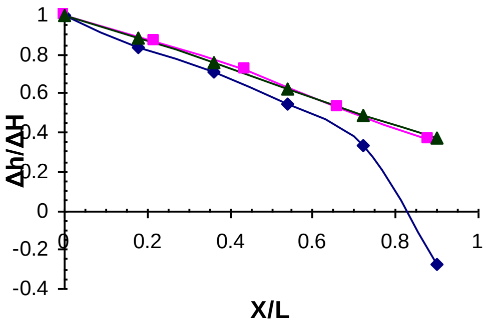


Fig 3.5.a. At ΔH=9.8 cm @2.90 cm3/s

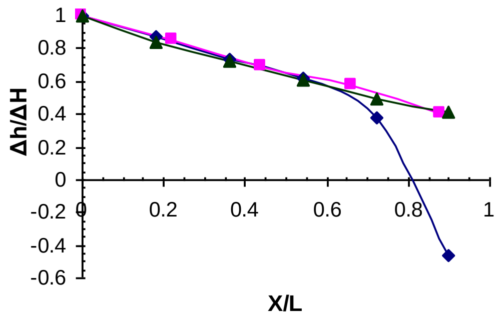


Fig 3.5.b. At ΔH=9.1cm @2.63 cm3/s

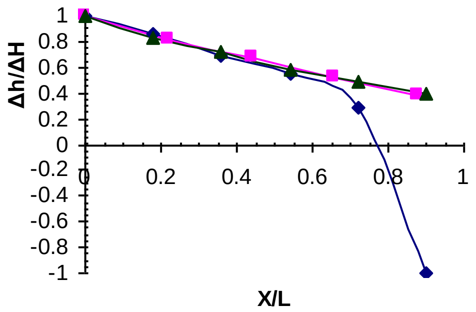


Fig 3.5.c. At ΔH=6.3 cm @2.65 cm3/s

Water withdrawn from Well 23

Water withdrawn from well W34

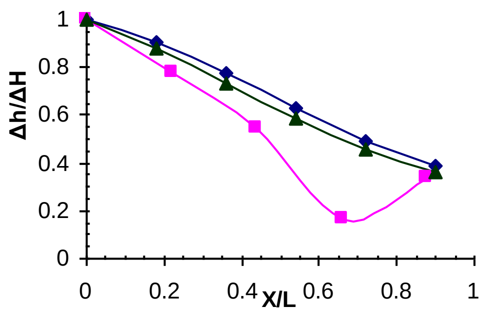


Fig 3.6.a. At ΔH=9.8 cm @2.73 cm3/s

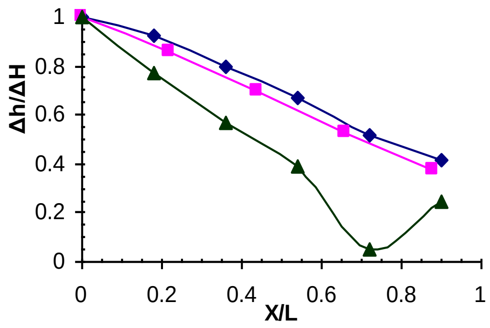


Fig 3.8.a. At ΔH=9.8 cm @2.99 cm3/s

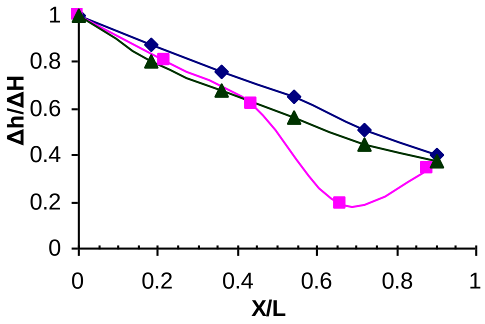


Fig 3.6.b. At ΔH=9.1 cm @2.80 cm3/s

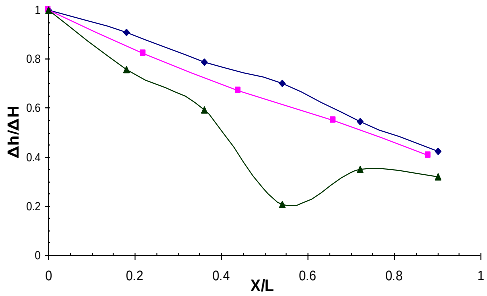


Fig 3.8.b. At ΔH=9.1 cm @2.86 cm3/s

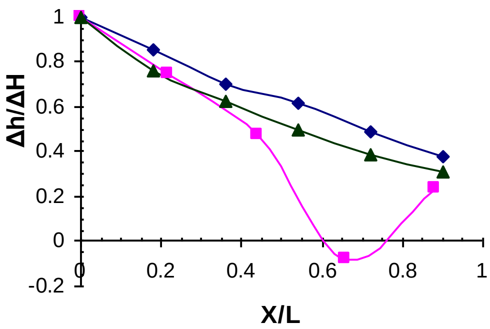


Fig 3.6.c. At ΔH=6.3 cm @2.86 cm3/s

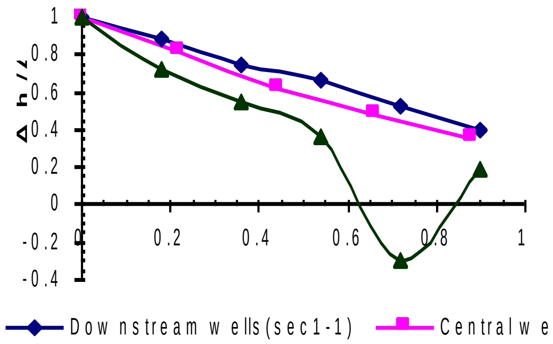


Fig 3.8.c. At ΔH=6.3 cm @2.78 cm3/s