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Seepage loss from a canal in Sri Lanka measured using tritium tracer

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Abstract. Using single borehole dilution technique, and tritiated water as tracer, lateral flow rate of water near the phreatic line has been measured at 24 points along the banks of an unlined canal. Permeability values range from 0.5×10^{-6} m sec^{-1} to 34×10^{-6} m sec^{-1}; the median value being 5×10^{-6} m sec^{-1}. The seepage loss is estimated to be 5×10^{-5} m^3 sec^{-1} m^{-1}. For the entire canal the value is 0.74 m^3 sec^{-1} which is 12% of the total flow. Limitations and merits of the technique are discussed.

Keywords. Canal seepage; tritium tracer; seepage loss; borehole dilution; nuclear hydrology.

1. Introduction

Usefulness of radioactive tracers to measure seepage loss from canals has been clearly demonstrated (Kaufman and Todd 1962; Krishnamurthy and Rao 1969). However, the applications have not been extensive enough, and the technique still seems to remain in the hands of nuclear experts rather than the field engineers. In this paper we present results on extensive measurements of seepage velocity along an unlined canal using tritium tracer. These results demonstrate not only that the method is simple enough to be adopted by persons not necessarily experts in nuclear techniques, but also that it gives reasonable values of *in-situ* field permeability and seepage loss.

2. Geohydrological considerations

Experiments were carried out on a canal in Sri Lanka which was commissioned in 1966. A reservoir impounds the water of river Kala Oya by means of a barrage

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on the river course. From the resorvoir, water is taken for irrigation down slope by means of two canals each on either side of the river. Seepage experiments were done on the canal flowing to the right side of the river (R B Canal). The canal passes through the soil overburden which primarily consists of sandy clay mixed with pebbles. Marked variation in soil texture is seen. The overburden, which lies on fractured basement rocks, is about 4 m thick. At several positions, ponds of water have been formed along the banks of the canal in regions where the canal crosses old *tanks*. (A *tank* is a formation to hold rain water, and is constructed by raising an earth embarkment on a sloping terrain at the lower perimeter of the catchment area).

3. Experimental

A typical cross section of the canal is shown in figure 1. A convenient segment of the canal, about 4 km long, was selected. Holes were drilled on both the banks by a 10 cm diameter hand auger. These went below the phreatic line generating a water column of about half a metre (10 cm to 110 cm). The holes were kept uncased. Occasionally large pebbles blocked the passage of the auger and forced the abandonment of such holes. Tritiated water (10 ml; 1 micro-curie) was introduced into each hole by means of a transfer tube. The tracer was well mixed by gently blowing air bubbles through the water column. A 10 ml capacity lead bucket was used to collect samples of water at 6 hourly intervals during the day time and 10 hr intervals during nights for four days.

In the laboratory, 1 ml fraction of a decanted sample was mixed with INSTA-GEL and its activity was counted with a Tri-Carb (Model 3300, Packard) liquid scintillation counter. In most cases an exponential decrease of activity in the hole was clearly seen. Some sites gave erratic results and had to be discarded during data processing.

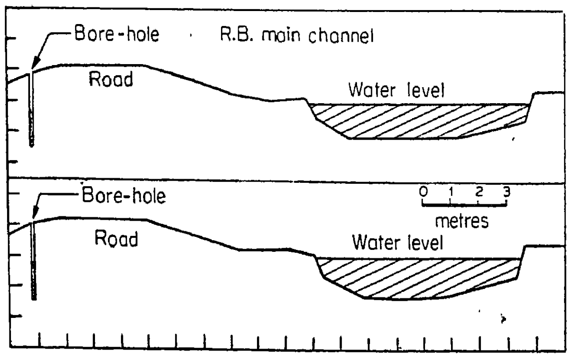

Figure 1. Cross section of the canal at two typical bore-hole positions. Each division on the graph is one metre.

4. Results and Discussion

The rate of flow of water, Q is calculated from the following relation (Halevy *et al* 1967)

$$Q = \frac{U}{t} \ln \frac{c_0}{c}$$

Where U = dilution volume of the borehole, and
t = time during which the initial activity c_0 becomes c.

Since this water flows through a cross sectional area hd where h is the height of the water column in the borehole whose diameter is d, we can calculate the linear flow velocity v from Q using the relation :

$$v = \frac{Q}{hd} = \frac{U}{t} \ln \frac{c_0}{c} \cdot \frac{1}{hd} = d \left[\frac{\pi}{4} \cdot \frac{1}{t} \cdot \ln \frac{c_0}{c} \right] \quad (1)$$

It is only the diameter of the borehole that enters into the final expression. If the hydraulic gradient at a borehole is 'g' the velocity of seepage is related to permeability k by ($v = kg$).

Equation (1) gives only an apparent velocity which has to be corrected for diffusional dilution and for geometry effects, the latter arising due to the flow in a borehole being larger than in the undisturbed soil. The corrected velocity $v_c = (v - v_d)/2$ where v_d is velocity due to dilution by diffusion effects and the factor of 2 in the denominator is a correction for the geometry factor (Halevy *et al* 1967). Assuming a constant value for the diffusivity ($D = 2 \times 10^{-5}$ cm^2 sec^{-1}) of water in soil (Goel *et al* 1977; Dharmasiri *et al* 1982) v_d is calculated as $\sim 10^{-5}$ cm sec^{-1}. In clayey soils v_d may be smaller, and since the value of v is already low for such soils, the correction is very large. In figure 2 we show two histograms for k, one with corrections for diffusional effects and the other without corrections. When an average value of k is taken only a marginal correction is needed for diffusion effects, which has been made.

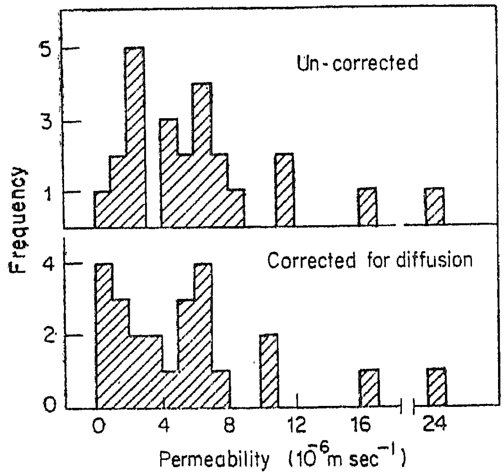

Figure 2. Histogram showing permeability values for different bore-holes. The lower graph is obtained after making a correction for dilution due to diffusion. Note that the correction is significant only for low permeabilities.

The values of permeability obtained are well within the ranges measured for loams and silts (Halevy *et al* 1967). Average value of k, after making all the corrections, is 6×10^{-6} m sec^{-1}. The median value would be more like 5×10^{-6} m sec^{-1}, though one would need larger sets of data to ascertain whether the two values are really different. For the present purpose 5×10^{-6} m sec^{-1} is taken as a representative value for k.

Some bias is introduced in rejection of those sets of data where the activity did not decrease exponentially. Possible reasons for such a behaviour are the presence of layered flow through a borehole or a change in the size of the borehole during a run due to lateral collapse of its walls. Clearly such boreholes would correspond to more permeable (sandy) soils and their rejection tantamounts to pushing the average to the lower side.

5. Seepage loss

The seepage loss for the canal depends on its dimensions, the permeability of the soil, distance of the governing drainage and the head difference. The tracer method gives us a reasonably good value of *in situ*, k the permeability. Except when steady condition can be obtained, the discharge, as calculated from a theoretical formula might only give an approximate value.

For a canal of trapezoidal cross-section flowing in a permeable medium of infinite depth with deep water table, the amount of water seeping out per unit length is given by Vedernikov (1934) as :

$$q = k\,(B + AH).$$

Here, q = seepage loss per unit time per unit length, B = width of the canal at the water level, H = height of water in the canal, and A is a function depending on B/H and cot α where α is the inclination of the bank with the horizon. Within ranges of B, H, and α that are admissible for the present case, the value of A would vary from 2.8 to 3·2 (Harr 1962). We can take an average value of 3. The value of q/k comes out to be 11 m.

On the other hand if we assume that the basement fractured rock (below about 4m) is impermeable, we have to use a different expression for the calculation of q/k. The solution is given by Muskat (1937) and Vedernikov (1934) and is available in graphical form in standard books (Harr 1962). We now have

$$q/k = \cot \alpha \; H.f.\,(B/H,\; T/H)$$

For admissible values of B/H, T/H, and α, (T = 4m) the value of q/k is 9 to 10 m. It may however, be pointed out that a basement of fractured rocks, as these are in the present case, may not be completely impermeable. Still a reasonable value for q/k is obtained.

Another limiting case that may be considered is the influence of the horizontal drainage layer. Pavlovsky solved this problem in 1936 (Aravin and Numerov 1953). The seepage loss is

$$q/k = (B + 2H) = 10 \text{ m}$$

All the three limiting cases give a value of about 10 m for q/k. Unfortunately we did not measure phreatic lines for the boreholes. This would have probably given us some further clues for using a more appropriate expression for q/k. The seepage loss per unit length is

$$q = 10k = 5 \times 10^{-5} \text{ m}^3 \text{ (m sec)}^{-1}$$

The length of the canal is 14.76 km and the flow discharge is 210 cusecs. The seepage loss for the entire canal is calculated as 0.74 m^3 sec^{-1} (26 cusecs) which is about 12% of the inflow. The value is not too unreasonable for a fresh canal flowing through sandy clay mixed with pebbles. Accuracy could be improved by measuring the canal parameters and k throughout the reach of the canal and also by recording the phreatic lines at the borehole sites.

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