PREDICTING INFILTRATION AND GROUND-WATER MOUNDS FOR ARTIFICIAL RECHARGE

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ABSTRACT: Planning systems for the artificial recharge of ground water via surface infiltration requires site investigations to predict infiltration rates and land requirements. Also, the ability of the vadose zone to transmit water to the underlying aquifer must be assessed, and aquifer conditions must be evaluated to predict the rise of ground-water mounds and to determine where ground water must eventually be pumped up again to prevent ground-water mounds below recharge areas from rising too high. A simple technique is presented to convert data from short-duration single-cylinder infiltrometer tests to final infiltration rates for large areas. Also, simple equations are developed to estimate heights of perched ground-water mounds above restricting layers in the vadose zone, to estimate how far from the recharge area ground water must be pumped, and to what depth to create a steady-state system with constant height of the ground-water mound in the recharge area. The procedures are useful for feasibility studies and the preliminary design of recharge projects, including long-term underground storage or water banking.

INTRODUCTION

There are several reasons why there is such great interest in the artificial recharge of ground water and why so many projects are considered or planned in numerous countries. One reason is that more storage is needed in times of water surplus for use in times of water shortage to meet the water demands of increasing populations, also because of possible climatic changes and the likelihood of more extremes in weather with longer droughts and heavier precipitation events. Because dams are losing popularity for various reasons such as ecological, environmental, social, economic, etc. (Pearce 1992; Devine 1995; Knoppers and van Hulst 1995), and because favorable dam sites are getting scarcer, building additional surface storage may be difficult. This leaves underground storage via artificial recharge as the main alternative. Artificial recharge may also have a vital role to play with the use of municipal wastewater (Bouwer 1993). Potable water reuse has recently been endorsed by the American Water Works Association and Water Environment Federation (McEwen and Richardson 1996), provided that such use is indirect [meaning that direct recycling via pipe-to-pipe systems is not permissible and that the water must first go through surface water (streams, lakes) or ground water (aquifers) before it can be used for municipal water supply]. The ground-water route has several advantages over the surface water route because once the water is in the aquifer there is no algae growth, evaporation, or chance of secondary pollution by animals or recreationists. Recharge also gives additional purification of the water by filtration through the underground materials [soil-aquifer treatment; Bouwer (1993)], and it provides seasonal or longer-term storage of the water to absorb differences between supply and demand. More importantly, it also enhances the aesthetics and public acceptance of potable water reuse because the water comes out of wells, not treatment plants. Because of these advantages, many water districts, municipalities, and other entities are considering the artificial recharge of ground water, even where the sands and gravels that are normally preferred for infiltration systems are not available and finer soils must be used, which may present "challenging" conditions to basin design and management (Bouwer 1998). Also, aquifers may have low transmissivities that could cause excessive ground-water mounding. Proper planning and feasibility studies, site investigations (soils, vadose zones, and aquifers), and pilot testing are of the utmost importance. Three fundamental questions need to be answered:

1. How fast will the water infiltrate into the ground? What is the capacity of the system, how much land will be required, what are the evaporation losses from the basins or other infiltration system?
2. How will the water move down to the ground water, are there any slowly permeable layers in the vadose zone that will restrict downward flow, and will the resulting perched ground-water mound reduce infiltration rates or produce other undesirable effects such as waterlogging or flooding basements?
3. How high will the ground-water mound in the recharged aquifer eventually rise, how much water can be stored or "banked" underground, and where and to what depth must ground water eventually be pumped to keep the mound below undesirable levels?

While these three questions are the main focus of this paper, additional comments and remarks are made to include some of the real-world issues associated with artificial recharge.

Surface infiltration systems for the artificial recharge of ground water may consist of weirs, dams, and T- or L-dikes in streambeds (in-channel systems), or of basins or similar facilities on the land itself (off-channel systems). Predicting and managing infiltration rates are among the most important aspects in planning, designing, and managing recharge systems, because they determine how much land is needed to put a certain flow of water underground, or how much ground-water recharge can be achieved with the available land. For surface infiltration systems in uniform soils without surface clogging, infiltration rates will be approximately equal to the vertical hydraulic conductivity of the soil (Bouwer 1978). Orders of magnitude for hydraulic conductivities of different soils can be found in Bouwer (1978) and other ground-water hydrology textbooks. Predicting long-term rises of perched ground water and aquifer ground-water mounds is necessary to ensure that these mounds will not reduce infiltration rates, cause water-logging of the recharge areas, or rise into landfills, basements, cemeteries, etc. Mound rises can be reduced by reducing recharge rates, by arranging the basins in long, nar-

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row recharge "strips" instead of in compact round or square areas, by dispersing the basins over larger areas, and/or by increased ground-water pumping from nearby wells.

PREDICTING INFILTRATION RATES

Traditionally, infiltration rates have been determined with cylinder infiltrometers consisting of metal cylinders about 20–40 cm in diameter and 8–20 cm high, which are pushed or driven about 3–8 cm into the ground (Bouwer 1986). The cylinders are filled with water. Constant water depth is maintained manually or with a Mariotte siphon or float valve while measuring the flow of water into the cylinder. Water also can be applied intermittently, allowing the water level in the cylinder to drop some distance and measuring the rate of fall. The cylinder is then refilled and kept full until the next rate or fall of the water level is measured. The test is continued long enough so that "final" infiltration rates are reached or approached, which may take more than a day if soils are relatively fine.

The biggest source of error in cylinder infiltration rates is divergence of the flow in the soil due to lateral unsaturated flow, as the water in the soil below the cylinder is "pulled" sideways and outward by capillary suction. This divergence causes the infiltration rates inside the cylinder to be significantly higher than the rates for a larger field or basin size area where edge or divergence effects are negligible. The finer the soil, the larger the capillary suction and lateral flow, and the more the cylinder will overestimate the large-area infiltration rate. Water depth inside the cylinder will also cause divergence, but this can be minimized by keeping this depth as small as possible.

Quantitative aspects of this overestimation of "true" large-area infiltration rates are shown in Fig. 1, which was obtained by electrical resistance network analog modeling (Bouwer 1960, 1986). The graph shows the relation between the final infiltration rate i of a single cylinder with diameter $2r$ and small water depth ($0.16r$), and the water-entry value h_{we} of the soil that reflects the capillary suction at the wetting front. Values of i are expressed as the ratio i/K or hydraulic gradient, where K is the hydraulic conductivity of the wetted zone. For final and truly vertical infiltration flow, $i/K = 1$. The abscissa shows the ratio of the water-entry value h_{we} of the soil to the diameter $2r$ of the cylinder. The water-entry value of the soil is the soil-water pressure head (negative value and expressed in centimeters of water or other length unit), whereby the soil-water content, and hence, the hydraulic conductivity, show a sharp increase when a dry soil is wetted (Bouwer 1978, p. 27). This is the point where the water phase becomes continuous and occupies most of the pore space. It also is the pressure head that can be expected at the advancing wet front below an infiltration system. Because of hysteresis, the water-entry value is greater (less negative) than the air-entry value or bubbling pressure of the soil, where water leaves the pores upon draining a soil and soil air becomes continuous. The water-entry value typically is about one-half of the air-entry value (Bouwer 1978). Water or air-entry values have been used to assess the effects of unsaturated or "negative-pressure" flow on water movement through soils by replacing the S-shaped curves of hydraulic conductivity versus pressure head by step functions, where K drops from saturated or near saturated (rewet) values to a very small value at the air-entry value for draining, and increases from a very small value to a much larger value at near-saturated or rewet conditions at the water-entry value for wetting (Bouwer 1964). For infiltration systems, the flow system is then treated as one of piston flow (Green and Ampt 1911; Bouwer 1978).

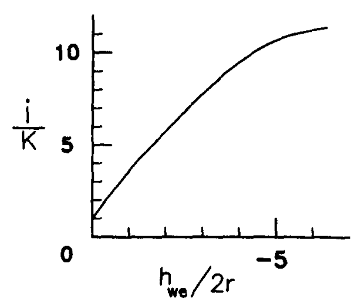

FIG. 1. Relation between Final Cylinder Infiltration Rate (Expressed as *i/K*) and Ratio of Water-Entry Value of Soil to Cylinder Diameter, as Determined by Electrical Resistance Network Analog (Bouwer 1960, 1986)

Divergence of the flow below a cylinder infiltrometer is governed by the water-entry value. The finer the soil, the smaller (more negative) this value is, the more divergence there will be, and the more the cylinder data will overestimate the true vertical infiltration of a large area. Approximate magnitudes of water-entry values are as follows (in centimeters of water):

- Coarse sands, −5
- Medium sands, −10
- Fine sands, −15
- Loamy sands, −20
- Sandy loams, −25
- Loams, −35
- Structured clay soils, −30
- Nonstructured clay soils, −100 or less (more negative)

These are general data based on measurements in laboratory and field settings by the senior writer and others (and recently verified by F. J. Leij of the U.S. Salinity Laboratory of Riverside, Calif., personal communication, 1997). {For more soil-specific information, see Leij et al. (1997) and references therein [e.g., Mualem (1976)].} For a loamy sand soil with a water-entry value $h_{we} = -20$ cm and a 40-cm-diameter infiltrometer $h_{we}/2r = -0.5$, Fig. 1 shows that $i/K = 2.2$. This means that the infiltrometer overestimates the final infiltration rate of a large inundated area by a factor of 2.2. For finer textured soils, and/or smaller cylinders, the overestimation is larger.

In an effort to eliminate the effects of divergence on measured infiltration, the double-ring or buffered infiltrometer was introduced (Bouwer 1986, 1994). With this device, two cylinders with diameters of, for example, 20 and 30 cm, are driven concentrically into the ground and flooded to maintain equal water levels in both cylinders to prevent flow components between the two bodies of water (Bouwer 1963). The thought, or rather, the hope, was that the infiltration in the annular area between the two rings would then absorb all of the divergence effects, so that the inner ring would give a true value of "the" vertical infiltration rate (gravity flow only) of a large area. However, the lateral gradients not only affect the outer part of the flow system, but also the inner part. Thus, the entire flow system is affected by divergence effects, including infiltration from the inner ring as evidenced by streamlines and equipotentials (Bouwer 1960). This effect on final infiltration illustrated in Fig. 2, is also obtained by the electrical resistance network analog model (Bouwer 1960, 1986), which shows the distribution of local infiltration rates i (expressed as a ratio to K to give vertical dimensionless hydraulic gradients) within a single cylinder (indicated by the solid vertical lines in Fig. 2) for $h_{we}/2r = -3.12$, which, for example, would apply to a 30-cm cylinder in a clay soil with $h_{we} = -93$ cm water. The curve shows that the final infiltration rates inside the entire cylinder are much greater than the true large-area infiltration flow where $i/K = 1$, as indicated by the $h_w/2r$

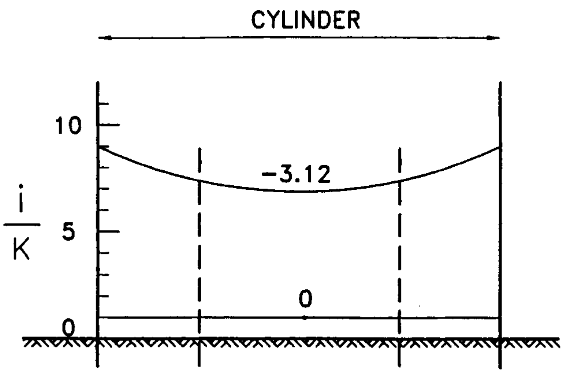

FIG. 2. Distribution of Final Infiltration Rates Expressed as i/K Inside Cylinder for $h_{we}/2r = -3.12$ and $h_{we}/2r = 0$ (Indicated on Curves), as Determined by Electrical Resistance Network Analog (Bouwer 1960, 1986)

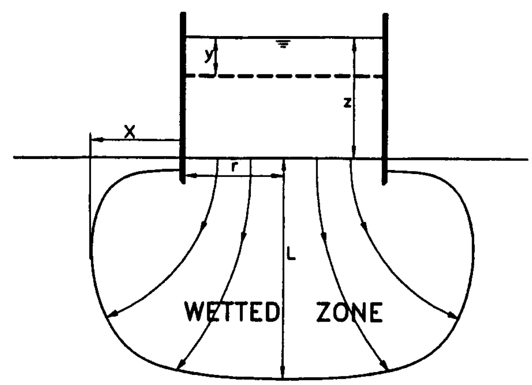

FIG. 3. Geometry and Symbols for Single-Ring Infiltrometer

= 0 line in Fig. 2. Thus, placing a smaller cylinder (dashed lines in Fig. 2) inside the outer cylinder to obtain a buffered infiltrometer does not produce true gravity-controlled vertical infiltration rates in the inner cylinder, but significantly overestimates the true infiltration rate. For the case of Fig. 2, this overestimation is by a factor of about 7. Nevertheless, buffered cylinder infiltrometers are still used and recommended (Bouwer 1994), despite the known effects of divergence on infiltration shown by electric analog studies (Bouwer 1960, 1986) and sand models (Swartzendruber and Olson 1961a,b). Buffered infiltrometers can be expected to give reasonable results only where soils are crusted or clogged at the surface and infiltration rates are controlled by the crust. However, double-ring infiltrometers theoretically would then not be necessary, except that installation of the cylinder may disturb the crust on the soil so that an equal-water-level buffer around the inner cylinder may improve the accuracy of the measurement.

There are two ways to get more accurate infiltrometer measurements: (1) use much larger infiltrometers; and (2) use smaller, more convenient cylinder infiltrometers for which the results are corrected for divergence. For example, if a large infiltrometer consisting of a 2 × 2 m bermed or boarded area is flooded for infiltration measurement on a loamy sand with $h_{we} = -20$ cm, $h_{we}/2r$ is about -0.1, for which Fig. 1 shows that i/K is about 1.3, which is much closer to the target value of 1 than the results of a small cylinder infiltrometer. The tests still need to be continued for a long time (hours to days) to approach final infiltration rates. Also, because of heterogeneity, tests must be performed at several locations within the study area. Thus, characterization of the infiltration rate for a proposed recharge project with large infiltrometers can be expensive. To reduce the costs and efforts, a simplified procedure has been developed to convert short-term infiltration rates measured with conventional single-cylinder infiltrometers to long-term or final infiltration rates for large areas. Basically, the single-cylinder infiltrometer is then used to determine the hydraulic conductivity of the wetted zone, which theoretically is equal to the final infiltration rate of a large area (with, of course, the usual assumptions of uniform soil and no other effects such as soil clogging, biological activity, entrapped air and other gases getting into or out of solution, etc.).

For this purpose, a suitable cylinder size may be 60 cm in diameter and 30 cm high with a beveled edge (Fig. 3). The steel cylinder is driven straight down to a depth of about 2–5 cm in the ground and the soil is packed against the inside and outside of the cylinder using a 2 × 5 cm stick and light hammer to get good soil-cylinder contact. Water is poured on a plate placed on the soil in the cylinder to avoid soil erosion. When the cylinder is filled to the top, pouring is stopped and clock time is recorded. The water level is then allowed to drop about 5–10 cm, the actual drop y is measured with a meter stick, the clock time recorded, and the cylinder is filled again to the top. Field experience indicates that this should be repeated for about 6 h or when the accumulated infiltration has reached about 50 cm, whichever comes first. The last drop of the water level y_n is measured and the clock time is recorded to obtain the time increment Δt_n for y_n. A shovel is used to dig outside the cylinder to determine the distance x of lateral wetting (divergence; Fig. 3).

The infiltration rate i_n during the last water level drop is calculated as $y_n/\Delta t_n$. Because most of the flow in the wetted zone is downward, vertical flow can be assumed in the entire wetted zone, so that the corresponding downward flow rate i_w in the wetted zone is calculated as

$$i_w = \frac{i_n \pi r^2}{\pi (r + x)^2} \quad (1)$$

The depth L of the wet front at the end of the test is calculated from the accumulated infiltration y_t (sum of water level drops in test) as

$$L = \frac{y_t \pi r^2}{n \pi (r + x)^2} \quad (2)$$

where n = fillable porosity of the soil. The value of n is estimated from soil texture and initial water content. For example, n may be about 0.3 for dry uniform soils, 0.2 for moderately moist soils, and 0.1 for relatively wet soils. Well-graded soils have lower values of n than uniform soils. The value of L can also be determined by digging down with a shovel immediately after the test to see how deep the soil has been wetted. This works best if the soil initially is fairly dry and there is good contrast between wet and dry soil. Applying Darcy's equation to the downward flow in the wetted zone during the last water level drop in the cylinder and again assuming vertical flow in the entire wetted zone then yields

$$i_w = K \frac{z + L - h_{we}}{L} \quad (3)$$

where z = average depth of water in the cylinder during the last water level drop y_n. The term h_{we} is the water-entry value of the soil and it is used to estimate the suction at the wet front as it moves downward. The value of h_{we} is estimated for the particular soil from the values listed earlier in this section.

Because K now is the only unknown, (3) can be solved for K as

$$K = i_w L/(z + L - h_{we}) \quad (4)$$

This calculated value of K can be used as an estimate of long-term infiltration rates in large and shallow inundated areas, without clogging of the surface and without restricting layers deeper down. Because of entrapped air, K of the wetted zone is less than K_{sat} at saturation; for example, about $0.5K_{sat}$ for sandy soils and about $0.25K_{sat}$ for finer soils (Bouwer 1978).

The above procedure is by no means exact. However, in view of spatial variability (vertical as well as horizontal) of soil properties and uncertainties about entrapped air effects, exact procedures and measuring water level drops with vernier equipped hook gauges are not necessary. The main goal is to account for divergence and limited depth of wetting, which is preferable to applying a flat reduction percentage to convert cylinder infiltration rates to large-area infiltration rates, as occasionally done. Because of spatial variability, cylinder infiltration tests should be carried out at a number of locations within a given site. Finally, the resulting infiltration rates should never be expressed in more than two significant figures.

To test the procedure in the field, infiltration rates were determined with single cylinders for an agricultural soil (sandy loam to loam) in an area west of Phoenix considered for ground-water recharge, and compared with infiltration rates in two test basins in the same area. Cylinder infiltrometers with a diameter of 60 cm and a height of 30 cm were installed (one each) at six randomly selected sites about 50 m apart in a field. Infiltration tests were carried out for about 6 h per test during which about 30 cm of water infiltrated into the soil. Final, large area infiltration rates, as calculated with (4) from the cylinder data, were 0.2, 0.3, 0.3, 0.4, 0.5, and 0.7 m/day for an average of 0.4 m/day. These rates were about 20–50% (average 30%) of the final rate of fall of the water level in the cylinders. These percentages can be expected to be lower for finer textured soils, and higher for coarser textured soils. Two test basins of about 40 m × 75 m each were then constructed in the same area and flooded with tap water to a relatively constant depth of 20 cm. For the next month or so, final infiltration rates were 0.3 m/day (fluctuating between 0.27 and 0.33 m/day) in one basin and 0.35 m/day (fluctuating between 0.33 and 0.37 m/day) in the other basin. This agreed very well with the average rate of 0.4 m/day obtained with the infiltrometers.

To go directly from infiltrometer data to the design and construction of a complete recharge system is often too big a step. Thus, if the infiltrometer data look promising, one or more test basins of at least about 30 × 30 m should be constructed to check the validity of the infiltrometer data in view of lateral and vertical spatial variability of the soil, and to study long-term infiltration behavior (for at least 1 year if possible) to measure the effects of clogging, particularly if the recharge water has suspended solids or if sewage effluent is used. The test basins also can be used to see how clogging should be managed by drying and cleaning, and to determine the best water depth (Bouwer and Rice 1989). For fine textured soils, infiltration rates also can be affected by fine-particle migration in the upper few millimeters of the soil and accumulation of the fine particles a short distance below soil surface. This process, which is called "wash-out-wash-in," also occurs in natural soils due to rain (Sumner and Stewart 1992) and requires special management of recharge basins to maintain maximum infiltration rates (Bouwer 1997a, 1998). Frequent scraping or shaving to remove surface clogging layers and disking or other tillage operations can cause deeper compaction of sensitive soils, which will reduce infiltration rates. When the soil contains cobbles or other large rocks, it can only be broken up by ripping. Frequent ripping, however, eventually will move the rocks to the surface where they seriously reduce hydraulic conductivity and infiltration rates (Bouwer and Rice 1984).

PREDICTING HEIGHTS OF PERCHED GROUND-WATER MOUNDS

When infiltrometers and test basins give surface infiltration rates that are acceptable for artificial recharge, the next step is to investigate the vadose zone to make sure that the infiltrated water can move unimpeded to the underlying ground water. Trenches or pits dug by a backhoe allow evaluation and inspection of the soil profile to a depth of about 7 m. If fine textured or cemented (caliche) materials are found at a certain depth, their hydraulic conductivity can be determined with infiltrometer tests or other above-the-water-table methods (Bouwer 1978) on the bottom of a trench or pit excavated to that depth. Doing these measurements in a sloping trench bottom will give a profile of hydraulic conductivity. Soil borings are necessary to evaluate deeper layers and aquifers. Large diameter (about 1 m) holes drilled with a bucket auger enable the assessment of infiltration rates and hydraulic conductivities at greater depths by removing loose material from the bottom, maintaining a few centimeters of water on the bottom, and measuring the infiltration rate. This can be managed from the top. Bucket augers can drill holes to a depth of about 50 m.

If a restricting layer is detected, the height of the perching mound that will be created by infiltration for ground-water recharge can be calculated by applying Darcy's equation to the vertically downward flow in the perching mound and the flow through the restricting layer (Fig. 4). This gives two equations with two unknowns: (1) The height L_p of the perched mound; and (2) the pressure head h_p at the top of the restricting layer. If the material below the restricting layer is relatively coarse, the pressure head of the water at the bottom of the restricting layer can be taken as zero. For finer materials below the restricting layer, the water-entry values listed in the section entitled, Predicting Infiltration Rates, can be taken as a first estimate. The resulting equation for the perched mound, assuming zero pressure head for the water at the bottom of the restricting layer, is then

$$L_p = L_r \frac{\dfrac{i}{K_r} - 1}{1 - \dfrac{i}{K_s}} \qquad (5)$$

where L_p = height of perching mound about restricting layer; L_r = thickness of restricting layer; i = infiltration rate; K_r = hydraulic conductivity of restricting layer; and K_s = hydraulic conductivity of soil above restricting layer.

Often, i will be much smaller than K_s because surface soils are finer textured than deeper soils, or there is a clogging layer on the surface soil that reduces infiltration. Also, i will often be much larger than K_r. For these conditions, (5) can be simplified to

$$L_p = i\frac{L_r}{K_r} \qquad (6)$$

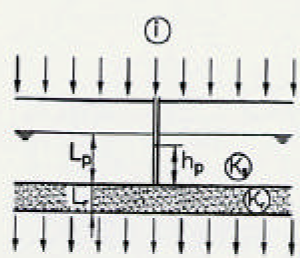

FIG. 4. Geometry and Symbols for Perched Mound above Restricting Layer in Vadose Zones

Rising ground-water levels (perched or aquifer mounds) can reduce infiltration rates. Where ground-water tables are well below the basin bottom, and the infiltration is restricted by a clogging layer on the basin wetted perimeters, the soil below the clogging layer will be unsaturated. For that case, the perched mound or actual ground-water mound should remain deep enough below the soil surface (bottom of basin) to keep the top of the capillary fringe above the water table well below the basin bottom, so that infiltration rates are unencumbered by water table height. Because the capillary fringe in permeable materials usually is less than 0.3 m high, depth to ground water thus should be at least about 0.5–1 m below the clogging layer on the basin bottom (Bouwer 1990) to maintain unimpeded infiltration rates. If there are no clogging layers, there will be hydraulic continuity between the infiltration basin and the aquifer. In that case, the depth to ground water outside the recharge area should be more than twice the width of the recharge project to have gravity-controlled infiltration rates that are unencumbered by depth to ground water and slope of the water table away from the recharge area, as indicated by electrical resistance network analog models (Bouwer 1962, 1990).

Whether or not a finer textured layer in the vadose zone is a perching layer depends on the recharge rate or infiltration rate i. If the surface soils are sandy loams with clogging layers on the basin bottoms that reduce infiltration rates, a loam layer deeper in the vadose zone may not perch water because its hydraulic conductivity may be higher than the infiltration rate. If, on the other hand, that same loam layer is in a sand and gravel profile where basin infiltration rates are much higher, it will cause the formation of perched ground water. Thus, every finer textured layer in the vadose zone does not automatically need a piezometer above it to see if perched ground water will form.

For relatively small basins or narrow recharge strips, perched mounds can spread laterally above the perching layer so that L_p is less than calculated with (5) or (6), which apply to vertical flow only. The same is true where perching layers are noncontinuous like lenses, or where there are breaches or other discontinuities in the perching layers. Thus, where perched mounds could be too high, better recharge may be achieved with long, narrow basins than with large and more square or round basins. Another possibility is to use deep infiltration basins excavated through the restricting layer. If the restricting layer is too deep for excavated basins, deeper recharge trenches or recharge shafts (large diameter holes in the vadose zone, also called dry wells or vadose zone wells) can be used (Bouwer 1997b, 1999).

PREDICTING HEIGHTS OF GROUND-WATER MOUNDS AND MOUND CONTROL BY GROUND-WATER PUMPING

Numerous papers have been published on the rise of ground-water mounds in response to infiltration and also on the fall of the mound after infiltration has stopped (Glover 1964; Hantush 1967; Mariño 1975a,b; Warner et al. 1989). One of the difficulties in obtaining meaningful results from the equations is getting a representative value of aquifer transmissivity. The most reliable transmissivity data come from existing recharge systems and calibrated aquifer models. Next come Theis-type pumping tests, step-drawdown and other pumped well tests, and slug tests, in decreasing order of "sampling" size. Slug tests, while simple to carry out, always have the problem of how to get representative areal values from essentially point measurements. Averages from various tests often seriously underestimate more regional values (Bouwer 1996, and references therein). Piezometers at two different depths in the aquifer below the recharge system enable the determination of both vertical and horizontal hydraulic conductivity with model simulation (Bouwer et al. 1974).

The analytical procedures for calculating mound heights are based on a constant transmissivity, which is acceptable if the mound height is small compared with the thickness of the original aquifer. However, if the analytical procedures are used to predict long-term mound rises, which for initially deep water tables can be on the order of 100 m, transmissivities then increase significantly as ground-water levels rise, so that the procedures overestimate long-term mound rises. The calculation then can be refined by using larger transmissivities that are more representative of the average transmissivity during the entire period for which the mound rise is calculated.

In thick unconfined aquifers, streamlines of recharge flow systems near the mounds are concentrated in the upper or "active" portion of the aquifer, with much less flow and almost stagnant water in the deeper or "passive" portion of the aquifer. Also, the streamlines in and near the mound are more vertical. Use of transmissivities of the entire aquifer for mound calculations can then seriously underestimate the rise of the mound. Previous work (Bouwer 1962) with resistance network analog modeling showed that for rectangular recharge areas, the thickness of the active, upper portion of an isotropic aquifer is about equal to the width of the recharge area. If vertical-to-lateral anisotropy exists, this effective thickness will even be less than the width of the recharge area. The effective thickness should be multiplied by K in a horizontal direction to obtain an "effective" transmissivity for mounding predictions. A good method for determining representative transmissivity values for artificial recharge is to have a large enough infiltration area that produces a significant ground-water mound, and to calculate transmissivities from the rise of the ground-water mound using, for example, the Hantush equation (Hantush 1967; Bouwer 1978). The fillable porosity to be used in the equations for mound rise usually is larger than the specific yield of the aquifer, because vadose zones often are relatively dry, particularly in dry climates. The fillable porosity should be taken as the difference between the water content in the vadose zone before and after wetting. Entrapped air causes the water content after wetting to be less than the saturated water content.

Operators and managers are frequently interested in the long-term effect of recharge on ground water, such as where will the ground-water mound be 20, 50, or 100 years from now; will the whole area become waterlogged; will the mound rise so high that it will threaten landfills, underground pipelines, basements, cemeteries, or produce other undesirable effects; how much water can be stored or banked underground; and how must the water be recovered from the aquifer to prevent undue mound rises? Computer models can be used to simulate regional recharge inputs and pumped well outputs for the aquifer, and to predict ground-water mounding for various scenarios of recharge and pumping. However, a quick concept concerning ultimate or quasi-equilibrium mound heights can be obtained from a steady-state analysis, where the mound is considered to be in equilibrium with a constant water table at some distance from the infiltration system. The constant distant water table is then established by ground-water pumping, discharge into surface water such as rivers or lakes, or some other control. Simple, steady-state equations were derived to determine how deep the distant water table should be and how close it should be to the infiltration system to prevent ground-water mounds from rising too high in the recharge area. Usually, recharge systems consist of a number of basins or other infiltration facilities grouped together in various layouts. Two cases were considered:

1. The basins form a long strip with a length at least 10 times the width, so that after a long time it still performs as an infinitely long strip (Glover 1964).

2. The basins are in a circular area or in a square or irregular area that can be handled as an equivalent circular area.

For the long strip (Fig. 5), the ground-water flow away from the strip was simply taken as linear horizontal flow (Dupuit-Forchheimer flow). Below the infiltration area, the flow in the aquifer was also assumed to be horizontal, so that the flow q at distance x from the center (for geometry and symbols see Fig. 5) per unit length of system can be described as

$$q = ix \tag{7}$$

where i = gross infiltration rate of the area (total infiltration or recharge flow divided by total infiltration area, including berms and other dry areas between flooded basins). The flow q at x can also be described as

$$q = Kh\frac{(-dh)}{dx} \tag{8}$$

Combining (7) and (8); integrating between $x = 0$, $h = H_c$, and $x = W/2$, $h = h_e$ (for meaning of symbols; see Fig. 5) and solving for the drop in height of the water table from the center to the edge of the infiltration strip then yields

$$H_c - h_e = \frac{iW^2}{8T} \tag{9}$$

where T = transmissivity of the aquifer (effects of water table height on T are ignored); and W = width of the recharge system (Fig. 5). The flow from $W/2$ to L_n can be described with Darcy's equation as

$$\frac{Wi}{2} = T\frac{h_e - H_n}{L_n} \tag{10}$$

where H_n = height of the control water table at distance L_n from the edge of the recharge area (Fig. 5). Combining (9) and (10) then yields

$$H_c - H_n = \frac{iW}{2T}\left(\frac{W}{4} + L_n\right) \tag{11}$$

for the ultimate rise of the ground-water mound below the center of the recharge strip above the constant water table at distance L_n from the edge of the recharge area (Fig. 5).

For a circular recharge area (Fig. 6), the ground-water flow will be radially away from the area. The flow below the recharge rate at distance r from the center can then be described as

$$q = i\pi r^2 \tag{12}$$

and as

$$q = 2\pi rKh\frac{(-dh)}{dr} \tag{13}$$

Combining (12) and (13) and integrating between $r = 0$, $h = H_c$ and $r = R$, $h = h_e$ (for meaning of symbols see Fig. 6) then yields

$$H_c - h_e = \frac{iR^2}{4T} \tag{14}$$

The radial flow from the recharge area to the control or natural water table at R_n from the center can be expressed with the Thiem equation as

$$\pi R^2 i = \frac{2\pi T(h_e - H_n)}{\ln R_n/R} \tag{15}$$

Solving (15) for $h_e - H_n$ and adding to (14) then yields

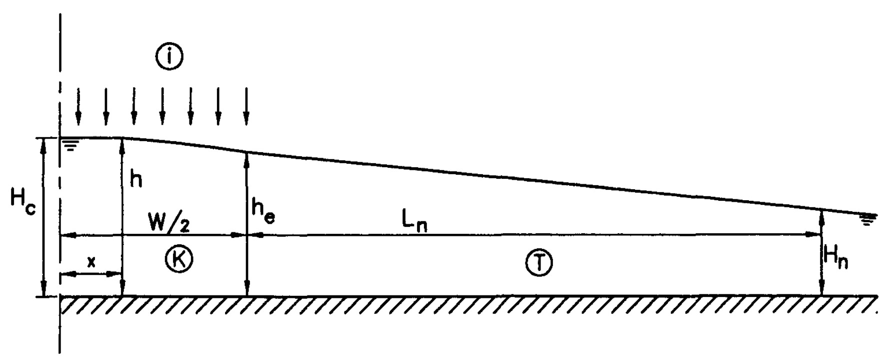

FIG. 5. Geometry and Symbols for Ground-Water Mound below Long Infiltration Area (Strip)

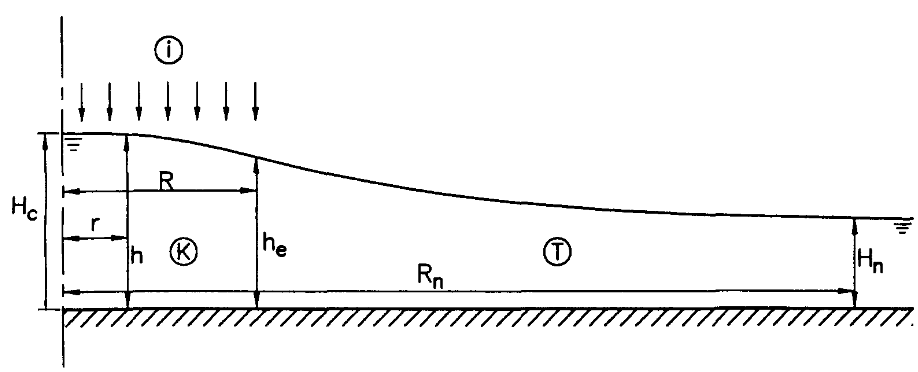

FIG. 6. Geometry and Symbols for Ground-Water Mound below Circular Infiltration Area

$$H_c - H_n = \frac{iR^2}{4T}\left(1 + 2\ln\frac{R_n}{R}\right) \qquad (16)$$

Eqs. (11) and (16) thus predict the final mound height below a long narrow and a round recharge area, respectively, for a given elevation of the control water table at distance L_n or R_n from the recharge area. If the ultimate mound height is too high, the constant, distant water table must be lowered farther, and/or R_n or L_n must be reduced by ground-water pumping from wells closer to the recharge area. Eqs. (11) and (16) can then indicate where ground water should be recovered and to what depth ground-water levels should be maintained to prevent ground-water mounds below the recharge areas from rising too high. Ultimate mound heights can also be reduced by making the recharge area longer and narrower, by making it larger by dispersing the basins, or by reducing recharge rates.

If h had been taken as a variable in the derivation of (11) and (16), T would have been shown as the product of K and the average thickness of the aquifer, as in the Thiem equation for flow to a well in unconfined aquifers (Bouwer 1978). Because of stratifications, heterogeneities, and anisotropy of aquifers, the average or effective K is difficult to determine. Also, T values are difficult to project from one pumping test for one aquifer thickness or one partially penetrating well to a much larger area and effective aquifer thickness for recharge flow. Considerable judgment is required to extrapolate T for aquifers with rising water levels. Well logs and lithologies should be used to see if T might increase linearly with rising water tables (vadose zone materials about the same as in aquifer), faster (vadose zone coarser than aquifer), or slower (vadose zone finer than aquifer). Because vertical flow was ignored in the derivation of (11) and (16), these equations underpredict mound rise in anisotropic soils where K in the vertical direction can be much less than K in the horizontal direction. Accurate predictions of T for rising ground-water levels are difficult to make and require considerable judgment. Early estimates of mound heights based on (11) or (16) can be refined by modeling, and ultimately, can be verified by monitoring when the project is operational.

EXAMPLE

A recharge system is proposed in a new area with no wells or other ground-water discharges. The question is, how much water can be stored underground before the mound rises too high and where should wells eventually be installed to establish a constant, far-away control ground-water level that will create an equilibrium condition between recharge and pumping to prevent further mound rises? The transmissivity is 930 m^2/day, the ground-water depth is 91 m, the maximum permissible rise of the mound is 82 m, and the fillable porosity is 0.2. The recharge rate is 50,000,000 m^3 per year, and the annual accumulated infiltration is 60 m or 0.17 m/day. Taking a square recharge area with essentially zero dry areas between the basins (close-basin layout), the recharge area then will be 83 ha or 912 × 912 m. Using the Hantush equation, the mound is calculated to rise to its maximum level (82 m) in 30 years, which will then have stored 1,500,000,000 m^3 of water underground. To keep the mound 9 m below the ground surface, a control water table at some distance from the recharge area must be established at the original ground-water depth of 82 m below the top of the mound. How far should this control water table be from the recharge system? Replacing the square recharge area by an equivalent circular area gives a radius R of the circular area of 515 m. Substituting these values into (16) then gives R_n = 10 km. Thus, wells must be installed and pumped in the vicinity of a circle about 10 km from the center of the recharge system or closer to keep the ground water at 10 km from the center at a depth of at least 91 m, so that the top of the ground-water mound in the recharge area remains at least 9 m below ground surface.

CONCLUSIONS

Planning and feasibility studies for systems for artificial recharge of ground water must include estimates of infiltration rates, potential for perched ground-water mounds, buildup of ground-water mounds, and control of ground-water mounds by ground-water pumping. Simple techniques and equations are presented to address these issues. Because of spatial variability and other uncertainties that influence the performance of artificial recharge systems, infiltration rates and ground-water level responses are difficult to predict and big projects should always be constructed in phases, so that performance data obtained in the first phase can be used to develop better design and management criteria for the next phase, etc. Thus, because it almost takes a recharge system to design a recharge system, the preferred approach is to start small and simple, learn as you go, and expand as needed.

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