Instituto Tecnológico
GeoMinero de España

# APLICACION DE TECNICAS HIDROGEOLOGICAS A LOS PROCESOS DE REGENERACION Y RECUPERACION DE AGUAS RESIDUALES DE ORIGEN URBANO.

## INFORME FINAL

## ANEXO Nº 5

MINISTERIO DE INDUSTRIA, COMERCIO Y TURISMO

35738

# ANEXO - 5

# BIBLIOGRAFIA RECOPILADA

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# GLOSARIO DE TERMINOS TECNICOS MAS USADOS

* ACTIVATED SLUDGE = LODOS ACTIVADOS
* CLOGGED = COLMATADO
* FLOODING/DRYING CYCLE = CICLO DE INUNDACION/SECADO
* HIDROLOGICAL BASIN = CUENCA HIDROGEOLOGICA
* HIDRAULIC LOADING = CARGA HIDRAULICA
* LANDFILL = VERTEDERO
* LEACHING = LIXIVIACION
* MONITORING WELL = POZO DE CONTROL
* OVERFLOW = DESBORDAMIENTO, AVENIDA
* RAW WATER = AGUA BRUTA
* RAPID-INFILTRATION LAND TREATMENT SYSTEMS = SISTEMAS DE TRATAMIENTO POR INFILTRACION RAPIDA EN EL TERRENO
* REMOVE = ELIMINAR, QUITAR, TRASLADAR
* REPLENISH = RELLENAR, RECARGAR
* SEWAGE = AGUAS RESIDUALES
* SOIL-AQUIFER TREATMENT (SAT) = TRATAMIENTO SUELO-ACUIFERO
* SPRINKLE = ROCIAR
* SUPPLY = SUMINISTRO, ABASTECIMIENTO
* TRICKLING FILTER = FILTRO DE AGUAS RESIDUALES
* UNSATURATED ZONE = ZONA NO SATURADA
* VADOSE ZONE = ZONA DE AIREACION, ZONA VADOSA

# Theory of Ground-Water Recharge for a Strip Basin

by Abnish C. Amar[a]

## ABSTRACT

Formulation and solution of one-dimensional analytical and computational models, linear as well as nonlinear, pertaining to a typical ground-water recharge pattern are presented. The objective of this investigation is to direct attention toward an evaluation and correlation of the various theoretical models based on the Dupuit-Forchheimer (D-F) approximations for the unsteady hydrodynamic behavior of the proposed recharge system under saturated flow conditions. Comparisons with the pertinent experimental data, presented in the form of dimensionless graphs, indicate that the linear D-F theory is valid for relatively small water-table rises but for large rises it predicts substantially greater rises than actually occur, particularly beneath and in the vicinity of the recharge basin. The range of validity is considerably improved between theory and experiments for relatively large rises using the nonlinear theory.

## INTRODUCTION

There has been an increasing use of artificial ground-water recharge in water management schemes in recent years. Among the various examples, particular mention may be made of the use of ground-water recharge as an effective means of reclaiming low quality storm and wastewater as well as recharging treated sewage as a means of disposal. As a prerequisite to an optimum design of such a recharge system, the practicing engineer must be able to predict with reasonable accuracy, the motion of the water table and the flow field of the proposed system for a given geometry, soil properties, and recharge rate. The movement of the water after it has infiltrated into the ground and especially after it has percolated to a depth where it encounters an existing water table or a stratum of relatively low permeability is of primary interest from a hydrodynamic standpoint. Specifically, this will enable the designer to: (1) avoid high water-table mounds beneath the recharge area, if desired; (2) allow sufficint time of travel from recharge area to the collection facility so that the recharged water attains a desired quality; and (3) minimize the spread of recharged water into areas outside the region. For a comprehensive literature survey of the general problem of artificial ground-water recharge, including the works of several investigators such as Bianchi and Haskell (1968), Glover (1961), Hantush (1967) and Polubarinova-Kochina (1962), reference may be made to Amar (1973) who has also presented solutions of a variety of theoretical models.

The purpose of this paper is to mathematically formulate the problem of recharging an unconfined aquifer from a strip basin to an initially horizontal water table overlying an impervious base using the

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Dupuit-Forchheimer (D-F) approximations. Formulations pertaining to both linear and nonlinear theory will be considered. Linearized versions of the governing partial differential equations will be handled using analytical techniques such as eigenfunction expansions and Green's function. The corresponding nonlinear equations will be computationally solved using the implicit finite difference formulation. The dimensionless solution curves derived from the theoretical formulations, exhibiting the effect of significant parameters influencing the hydrodynamic behavior, will be compared with the pertinent experimental data obtained from sand-model studies. It will be assumed that the flow is unsteady and the aquifer medium is homogeneous and isotropic.

## PROBLEM FORMULATION

As derived by Brock and Amar (1974), according to D-F theory the physical problem for the determination of a rising free surface of a transient ground-water mound in a region subjected to artificial recharge is described by the nonlinear parabolic partial differential equation

$$(HH_x)_x = \frac{n_e}{k} H_t - \frac{p}{K} \qquad (1)$$

where p is the volumetric recharge rate $[L^3/L^2T]$, K is the permeability [L/T], and $n_e$ is the effective porosity which may be somewhat less beneath the basin ($x \leqslant L$) than that outside the basin ($x \geqslant L$). Equation 1 may readily be obtained by combining the continuity equation with Darcy's law or by integrating the Laplace equation over the depth of flow, H. Let $n_i$ and $n_o$ be the effective porosities in these regions. Then

$$p = p_o,\ n_e = n_i \quad x \leqslant L \qquad (2a)$$

$$p = o,\ n_e = n_o \quad x \geqslant L \qquad (2b)$$

where $p_o$ is the assumed constant recharge rate from the basin. As schematically illustrated in Figure 1, the above equation should be solved subject to the following initial and boundary conditions for a finite domain

$$H(x, t = 0) = a \qquad (3a)$$

$$H(x = B, t) = a \qquad (3b)$$

$$H(x = -B, t) = a \qquad (3c)$$

The above formulation given by equations 1-3 will now be solved in subsequent sections and results compared with pertinent laboratory experiments.

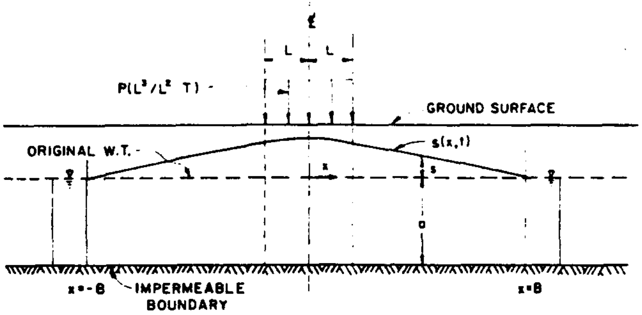

Fig. 1. Definition sketch—ground-water mound in a finite domain beneath a strip recharge basin.

## SIMPLIFIED LINEAR MODELS – SOLUTIONS BY ANALYTICAL TECHNIQUES

Equation 1 is nonlinear and no general analytical solution is available. However, letting H = a + s and assuming that the height, s of the ground-water mound is small compared to the initial thickness, a, of the zone of saturation (i.e., $s/a << 1$), a linearized version of this equation may be obtained as

$$s_t = \alpha\, s_{xx} + \frac{p}{n_e} \qquad (4)$$

where diffusivity, $\alpha = Ka/n_e$. For the case of uniform porosity (i.e., $\lambda_o = n_i/n_o = 1$), equations 2 become

$$p = p_o \quad x \leqslant L \qquad (5a)$$

$$p = 0 \quad x \geqslant L \qquad (5b)$$

Also the counterpart of equations 3 is given by

$$s(x, t = 0) = 0 \qquad (6a)$$

$$s(x = B, t) = 0 \qquad (6b)$$

$$s(x = -B, t) = 0 \qquad (6c)$$

or equivalently taking advantage of symmetry about the center line, equation 6c may be replaced by

$$s_x \Big|_{x = 0} = 0 \qquad (6d)$$

### Infinite Domain

When $B \to \infty$, the corresponding boundary conditions for the infinite domain are obtained. This infinite domain case has previously been investigated by Polubarinova-Kochina (1962); its solution in closed form, when expressed in dimensionless variables (defined below) and corrected for a few minor errors, becomes (Polubarinova-Kochina, 1962) for $|x'| < 1$:

$$s''(x',t') = \tfrac{1}{2}\Big(t'\phi\big(\frac{1-x'}{2\sqrt{t}}\big) + \frac{(1-x')\sqrt{t'}}{\sqrt{\pi}}\, e^{-(1-x')^2/4t'} - \frac{(1-x')^2}{2}\,\big[1-\phi\big(\frac{1-x'}{2\sqrt{t'}}\big)\big] + t'\phi\big(\frac{1+x'}{2\sqrt{t'}}\big) + \frac{(1+x')\sqrt{t'}}{\sqrt{\pi}}\, e^{-(1+x')^2/4t'} - \frac{(1+x')^2}{2}\,\big[1-\phi\big(\frac{1+x'}{2\sqrt{t'}}\big)\big]\Big) \qquad (7)$$

for $|x'| > 1$:

$$s''(x',t') = \tfrac{1}{2}\Big(t'\phi\big(\frac{1+x'}{2\sqrt{t'}}\big) + \frac{(1+x')\sqrt{t'}}{\sqrt{\pi}}\, e^{-(1+x')^2/4t'} - \frac{(1+x')^2}{2}\,\big[1-\phi\big(\frac{1+x'}{2\sqrt{t'}}\big)\big] - t'\phi\big(\frac{x'-1}{2\sqrt{t'}}\big) - \frac{(x'-1)\sqrt{t'}}{\sqrt{\pi}}\, e^{-(1-x')^2/4t'} + \frac{(x'-1)^2}{2}\,\big[1-\phi\big(\frac{x'-1}{2\sqrt{t'}}\big)\big]\Big) \qquad (8)$$

where, $\phi(\omega) = \mathrm{erf}(\omega) \triangleq \frac{2}{\sqrt{\pi}} \int_0^{\omega} e^{-\eta^2}\, d\eta$, and

$$x' = x/L,\ s'' = s'/p_o' = ska/p_oL^2,\ p_o' = p_oL^2/ka^2, \qquad (9)$$
$$t' = tka/n_oL^2 = t\alpha_o/L^2$$

Using an approximation for the error function, erf($\omega$)—formula 7.1.26 of Abramowitz and Stegun (1970)—the values of s″ were numerically evaluated. A graphical representation of some of the results obtained from computer calculations is shown by solid lines in Figure 2 (curves corresponding to $\lambda_o = 1.0$).

**Finite Domain**

When the aquifer is bounded by vertical equipotentials at $x = \pm B(B < \infty)$ the case of finite lateral controls is obtained and will hereafter be referred to as finite domain. A comprehensive literature survey (Amar, 1973) indicates that no analytical solution in closed form is presently available for this case. The solution of the physical problem, using the method of eigen-function expansions, is presented in Appendix A.

The formal solution of the unsteady problem is described by equations

for $|x'| \leqslant 1$:

$$s''(x',t') = \underbrace{-\frac{x'^2}{2} + (B' - \tfrac{1}{2})}_{\text{steady state solution.}} \underbrace{- \frac{16B'^2}{\pi^3} \sum_{n=1}^{\infty} \frac{1}{(2n-1)^3}\, e^{-(2n-1)^2 \frac{\pi^2 t'}{4B'^2}} \sin\big(\frac{2n-1}{2B'}\pi\big)\cos\big(\frac{2n-1}{2B'}\pi x'\big)}_{\text{transient part}} \qquad (10)$$

for $|x'| \geqslant 1$:

$$s''(x',t') = \underbrace{-x' + B'}_{\text{steady state solution.}} \underbrace{- \frac{16B'^2}{\pi^3} \sum_{n=1}^{\infty} \frac{1}{(2n-1)^3}\, e^{-(2n-1)^2 \frac{\pi^2 t'}{4B'^2}} \sin\big(\frac{2n-1}{2B'}\pi\big)\cos\big(\frac{2n-1}{2B'}\pi x'\big)}_{\text{transient part}} \qquad (11)$$

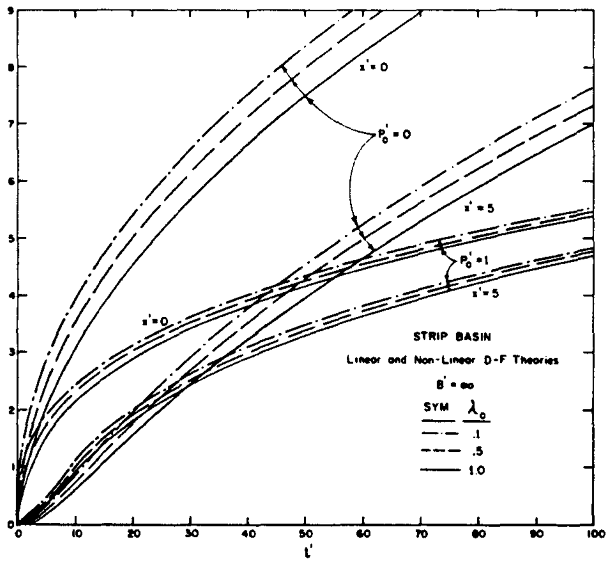

Fig. 2. Theoretical curves s″ vs. t′ corresponding to one-dimensional linear and nonlinear D-F theory for B′ = ∞.

The transient part of this solution which is in the form of an infinite series was found to be rapidly convergent except for small time periods, $(t'/B'^2) \leq 0.25$. Free surface profiles were numerically evaluated from equations 10 and 11 for a few values of B′. The results are graphically presented in Figure 3 in the form of s″ vs. t′ curves at selected values of x′. This Figure also shows the corresponding results from the analytical solution of the infinite domain case, B′ = ∞. An examination of this Figure clearly shows that the values of s″ increase as B′ increases for fixed values of x′ and t′, except for small values of x′ and t′.

## GENERALIZED NONLINEAR MODEL

The generalized nonlinear equation according to D-F theory governing one-dimensional, transient hydrodynamic behavior of the recharge problem under consideration for saturated flow conditions was derived by Brock and Amar (1974) in a previous paper. The equation referred to is simply the non-dimensionally transformed version of equation 1 and may be rewritten as

for $|x'| \leq 1$:

$$\lambda_o s''_{t'} = (1 + s''p'_o) s''_{x'x'} + p'_o (s''_{x'})^2 + 1 \qquad (12)$$

for $|x'| \geq 1$:

$$s''_{t'} = (1 + s''p'_o) s''_{x'x'} + p'_o (s''_{x'})^2 \qquad (13)$$

Equations 12 and 13 should be solved subject to the dimensionless counterparts of initial and boundary conditions, equations 6a, 6b and 6d which are

$$s''(x', t' = 0) = 0 \qquad (14a)$$

$$s''_{x'}(x' = 0, t') = 0 \qquad (14b)$$

$$s''(x' = B', t') = 0 \qquad (14c)$$

According to this formulation, the dependent variable s″ is a function of x′, t′, $\lambda_o$, B′ and $p'_o$. Based on the comparisons between theoretical linear models and experimental data presented elsewhere by Amar (1973) and Brock and Amar (1974), the use of these simplified models has raised two questions: (a) To what extent do the analytical solutions represent the true behavior of the physical system? and (b) What is the true behavior of such a system? Our ability to answer these questions seems to depend on the development of numerical techniques which are well adapted to modern digital computers and require a minimum number of restrictions on the parameters of the system. It will be demonstrated that such an analysis will help to explain the observed deviations between the known linearized mathematical solutions and the experimental results in certain cases. With these considerations in mind, the present boundary value problem was computationally solved using the implicit finite difference formulation. The details of this computational algorithm are presented in Appendix B.

A computer program was developed utilizing this algorithm which predicts the position of the moving free surface; some of the solution curves are shown in Figure 4 for $\lambda_o = 1.0$ and for various values of $p'_o$. This Figure also shows the corresponding solution curve pertaining to the analytical model presented earlier ($p'_o = 0$, $\lambda_o = 1.0$). An examination of Figure 4 indicates that the compu-

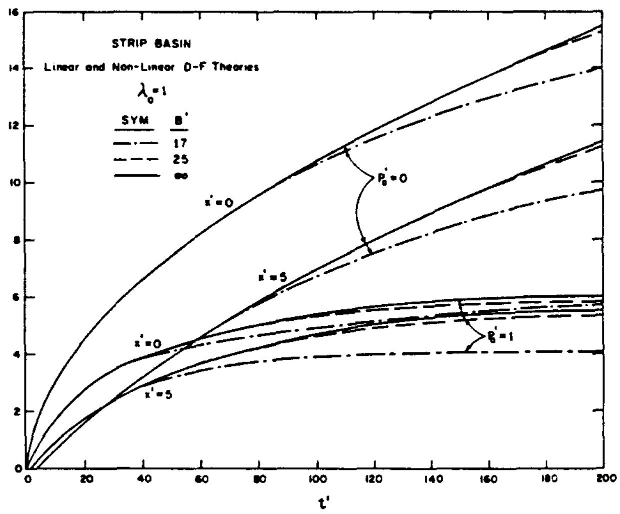

Fig. 3. Theoretical curves s″ vs. t′ corresponding to one-dimensional linear and nonlinear D-F theory for $\lambda_o$ = 1 and selected values of B′.

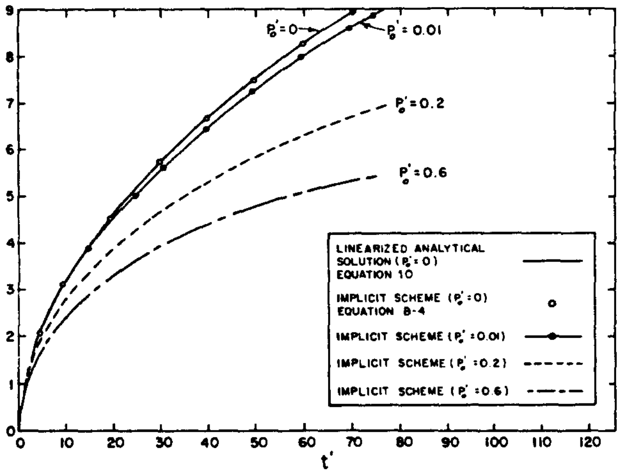

**Fig. 4. Theoretical curves s" vs. t' corresponding to one-dimensional linear and nonlinear D-F theory for $\lambda_o = 1.0$, finite domain ($B' = 17.0$) at $x' = 0$.**

tational scheme for $p_o' = 0$ shows excellent agreement with the analytical solution of the linearized model. Relative accuracy of the theoretical models developed herein will be discussed in a later section.

## DEVELOPMENT OF SOLUTION CURVES SHOWING EFFECT OF SIGNIFICANT PARAMETERS

The implicit formulation was used in developing the theory curves so as to exhibit separately the effect of significant parameters B', $p_o'$ and $\lambda_o$. These are presented in Figures 2, 3 and 5 and are discussed here.

The concept of variable porosity, illustrated in Figure 2, has probably for the first time been treated in a quantitative framework. It was noticed that s" increases as $\lambda_o$ decreases for all fixed values of x', t', $p_o'(> 0)$ and $B'(< \infty)$. This is attributable to the reduction in the available pore space beneath the recharge basin that results with the decreasing values of $\lambda_o$; consequently, s" increases.

The effect of B' for the linear theory ($p_o' = 0$) as presented in Figure 3 has been discussed earlier. The nonlinear counterparts for the same values of x' and $\lambda_o$ are also graphically superimposed in Figure 3 for $p_o' = 1.0$. An examination of this Figure indicates that the nonlinear hydrodynamic behavior is similar to that of the linear theory except that the nonlinear curves have a sharper curvature than their linear counterparts and furthermore that the effect of the boundary is felt sooner (smaller t' as $p_o'$ increases).

A further illustration of the effect of $p_o'$ for $B' = \infty$ and $\lambda_o = 1$ is graphically presented in Figure 5. An examination of curves in Figure 5 ($x' = 0$) indicates that the values of s" progressively decrease as $p_o'$ increases. This is typical of the behavior beneath the recharge basin. However, an opposite effect was noticed outside the recharge basin (results not reported here) for values of t' up to approximately 50, depending upon the values of $p_o' > 0$. A plausible explanation of this behavior may be found solely in terms of time-lag for the full exhibition of the nonlinear behavior outside the recharge basin. The behavior at higher values of t' is analogous to that noticed beneath the recharge basin.

Based on the information presented in this section, the theoretical results of the computational nonlinear model, encompassing the effects of various parameters B', $p_o'$, and $\lambda_o$, are considered significant in that they are expected to considerably improve agreement with the experimental observations for the physical problem under consideration. A brief description of the laboratory model along with comparisons of experimental and theoretical curves are presented in the following sections.

## LABORATORY SAND MODEL

The model (Figure 6) consists of a watertight box constructed of 12.7 mm (0.5 in.) thick plexiglass sheets. The inside dimensions are 750 mm (29.5 in.) in height, 5460 mm (215 in.) in length and an average width of 153 mm (6.04 in.). The box was filled with clean silica sand of a uniformly compacted density to a depth of 630 mm (24.8 in.) extending over a length of 5000 mm (197 in.). The size of the sand was selected so as to minimize the thickness of the unsaturated zone above the water table while still maintaining a Reynold's number

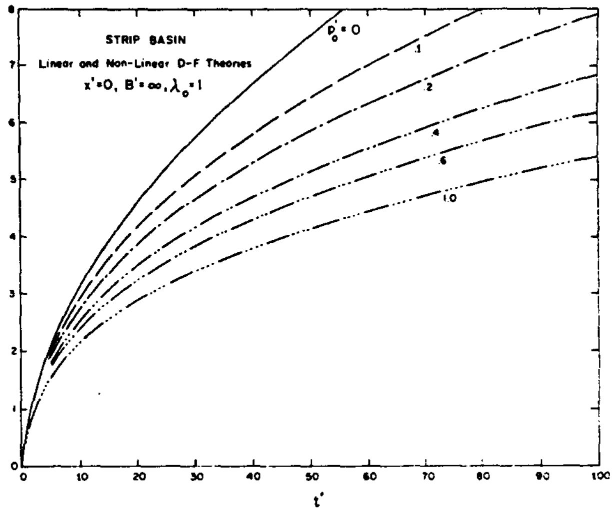

**Fig. 5. Theoretical curves s" vs. t' for various values of $p_o'$ for an infinite domain ($B' = \infty$) and $\lambda_o = 1.0$.**

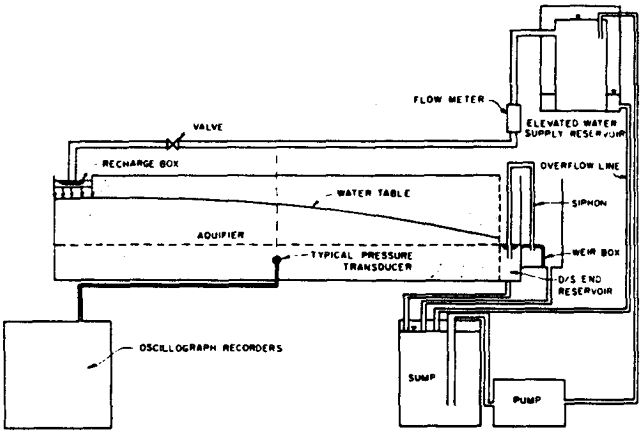

**Fig. 6. Elevation view of sand model with major appurtenant features.**

within the Darcy range. The sand was separated from a 230 mm (9 in.) long reservoir at each end of the box by a fine mesh stainless steel screen supported by a 6.3 mm (0.25 in.) thick plexiglass sheet with many holes drilled through it. The upstream reservoir which was sealed off from the sand facilitated the creation of an impervious boundary at this end and served as the center line of the recharge basin at x = 0. The water level in the downstream reservoir was maintained at a fixed level by a vertically adjustable weir.

Water was supplied to the recharge box, placed adjacent to the upstream boundary, from a constant head tank. Experimental determination of aquifer parameters such as effective fillable porosity and permeability corresponding to a wetted moisture profile were made. To develop solution curves, a series of experiments was conducted simulating the condition of recharging from a long strip basin onto a shallow as well as a deep unconfined aquifer with a horizontal base. The rise of the water table at different locations was automatically recorded on an oscillograph recorder in the form of a continuous (time dependent) curve at each pressure tap station.

## COMPARISON OF THEORETICAL CURVES WITH EXPERIMENTAL DATA

Detailed experimental results from sand-model studies were presented by Brock and Amar (1974) in the form of dimensionless graphs in a previous paper. The comparisons and correlations between theory and experiments are confined herein to those representative curves which are considered significant in determining the relative accuracy of the various D-F models over a wide range of parameters relating to the hydrodynamics of the recharge phenomenon.

An examination of Figures 7 and 8 indicates that the linear D-F theory shows the greatest discrepancy with respect to the experimental data except for small values of t′ because the nonlinear effect is not exhibited at small t′. The discrepancy is generally greatest beneath the recharge basin due to the increased effect of nonlinearity in this region. As expected, this discrepancy progressively increases with increasing values of t′ and $p'_0$. It was also found that in general this discrepancy

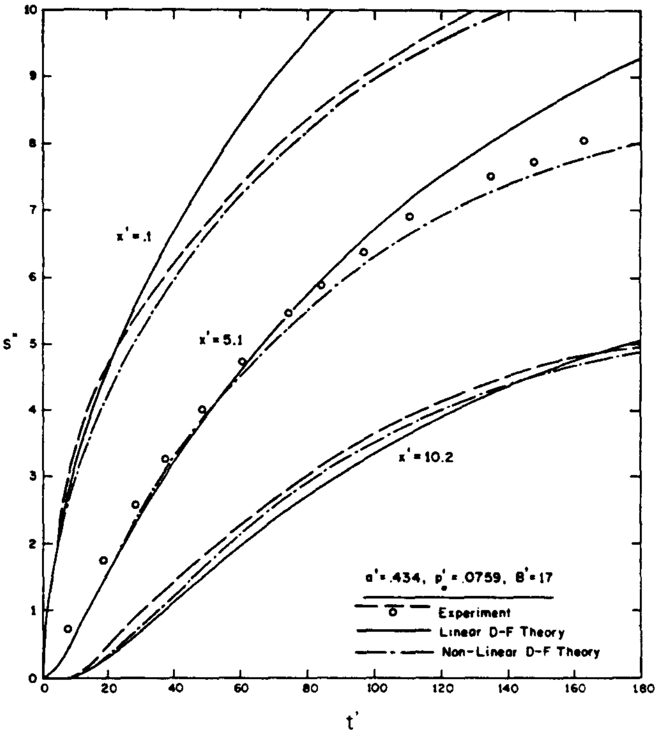

**Fig. 7. Comparison of theory curves with experimental data for a′ = .434, $p'_0$ = .0759, B′ = 17.0.**

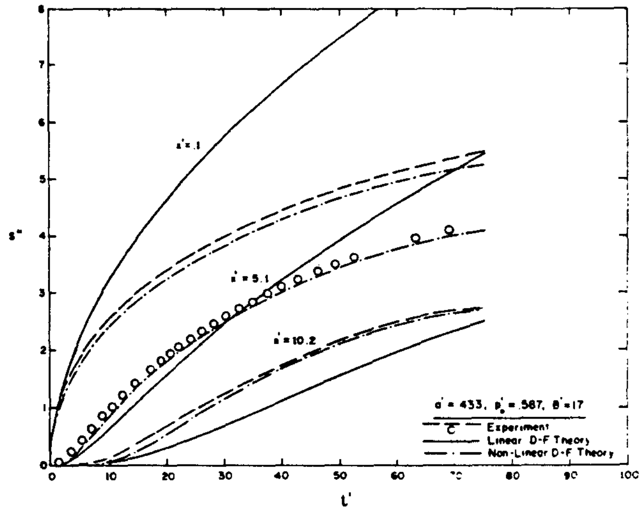

**Fig. 8. Comparison of theory curves with experimental data for a′ = .433, $p'_0$ = .587, B′ = 17.0.**

increases with increasing values of a', particularly for smaller x', other parameters being kept constant.

A further inspection of Figures 7 and 8 ($p'_o$ ranges from 0.0759 to 0.587) reveals that the discrepancy between experimental data and D-F theory is considerably reduced when one considers the nonlinear formulation both beneath and outside the recharge basin. As an example, reference may be made to the experimental and theoretical curves depicted in Figure 8. For $a' = 0.433$, $B' = 17$, the linear theory curve is approximately 68% above the experimental curve for $p'_o = .587$ at $x' = 0.1$, $t' = 70$. The difference is reduced to less than 3% when comparison of the same experimental curve is made with the corresponding nonlinear curve for the same conditions. This may be attributed to the fact that $s'$ ($= s/a$) is assumed to be small in the linearized version whereas no such restriction exists in the nonlinear formulation. It may be noticed that the difference is relatively smaller at lower values of $p'_o$ and $t'$ and for larger values of $x'$. An inspection of Figures 7 and 9 (with approximately the same value of $p'_o$) further reveals that the discrepancy between theory and experiments tends to increase with increasing values of a'. This discrepancy is expected to be all the more conspicuous for relatively higher values of $p'_o$ due to the variation of the horizontal velocity component over the depth of flow, H (Amar, 1974). This variation of horizontal velocity component is ignored in the D-F theoretical formulation and may very well help to explain the small discrepancy between the D-F theory and experimental curves indicated above.

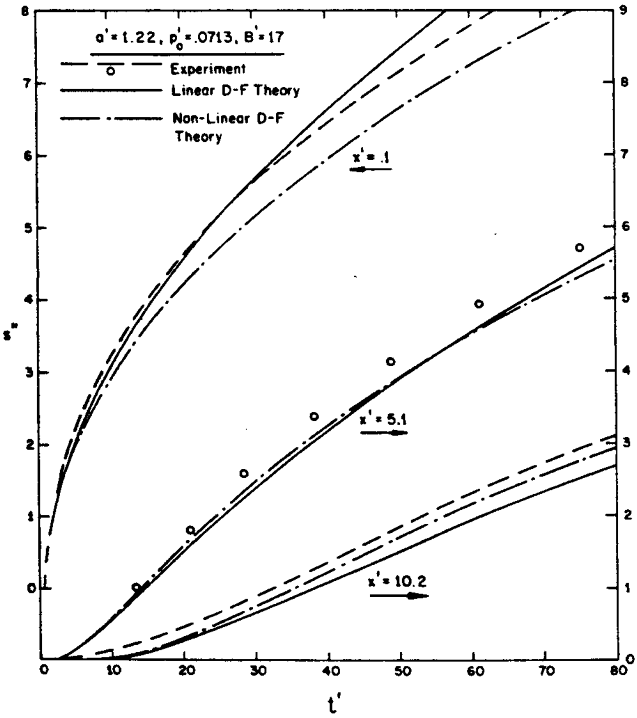

**Fig. 9. Comparison of theory curves with experimental data for $a' = 1.22$, $p'_o = .0713$, $B' = 17.0$.**

## SUMMARY AND CONCLUSIONS

One-dimensional theoretical models pertaining to both linear and nonlinear D-F theory have been formulated and their analytical and/or computational solutions presented herein. The latter were based on the method of finite differences. The solution curves obtained corresponding to these formulations have been graphically presented, showing separately the effect of significant parameters such as B', $p'_o$ and $\lambda_o$. It was found that for a finite as well as an infinite domain, the nonlinear effect is directly a function of $p'_o$ and variations of the effective porosity in the regions beneath and outside the recharge basin are exhibited by decreasing values of $\lambda_o$ ($\lambda_o = 1.0$ indicates the uniform porosity case).

Based on the correlations between theory and experimental data presented in the preceding section, it may be seen that the linearized D-F models appear to be of limited use for most practical purposes and that the nonlinear version is a noticeable improvement over its linearized counterpart. The present analysis substantially increases the accuracy and generality of conclusions reached inasmuch as it helps to explain the observed deviations between the known linearized mathematical solutions and experimental results in many cases. The small discrepancy between the nonlinear D-F theory and experimental data, particularly for high values of a' may be attributed to the neglect of the variation of horizontal velocity component over the depth of flow, H, and will be considered in a subsequent paper dealing with potential theory.

## ACKNOWLEDGMENTS

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## NOTATION

The following symbols are used in this paper:

$a$ = initial depth of water

$a'$ = $a/L$ = dimensionless depth

$B$ = distance from center line of basin to reservoir

$B'$ = $B/L$ = dimensionless distance

$H$ = depth of flow

$i, j$ = index coordinates in t and x directions, respectively

$L$ = effective length of recharge basin

$n$ = an integer, 1,2,3, . . .

$n_e$ = effective porosity

$n_f$ = effective fillable porosity

$n_i$ = effective porosity beneath recharge basin

$n_o$ = effective porosity outside recharge basin

$p$ = recharge rate

$p_o$ = recharge rate from basin

$p'_o$ = $p_o L^2/ka^2$ = dimensionless recharge rate from basin.

$s$ = water table rise above initial depth

$s'$ = $s/a$ = dimensionless water table rise

$s''$ = $s'/p'_o = ska/p_o L^2$ = dimensionless water table rise

$t$ = time

$t'$ = $tka/n_o L^2$ = dimensionless time

$x$ = horizontal coordinate

$x'$ = $x/L$ = dimensionless horizontal coordinate

$\lambda_n$ = eigen values of the problem

$\lambda$ = $n_e/n_o$ = ratio of effective porosities

$\lambda_o$ = $n_i/n_o$ = ratio of effective porosities

$\alpha$ = $ak/n_e$ = diffusivity

$\alpha_i$ = $ak/n_i$ = diffusivity beneath the recharge basin

$\alpha_o$ = $ak/n_o$ = diffusivity outisde the recharge basin

$\phi_n$ = eigen-functions of the problem

$\xi\ \eta\ \lambda$ = dummy integration variables

$\Delta x$ = elemental distance in x-direction

$\Delta x'$ = dimensionless value of $\Delta x$

$\Delta t$ = elemental time increment

$\Delta t'$ = dimensionless value of $\Delta t$

## APPENDIX A

## ANALYTICAL SOLUTION OF EQUATION 4

Assuming that the solutions for the dependent variable, $s(x, t)$ exist in the form of products of functions of the respective independent variables, we express the solutions as

$$s(x, t) = \sum_{n=1}^{\infty} s_n(t)\ \phi_n(x) \qquad \text{(A-1)}$$

where $\phi_n(x)$ are the eigen-functions of the problem.

Defining a self-adjoint operator

$$L \triangleq \alpha \frac{d^2}{dx^2} \tag{A-2}$$

and determining eigen-functions of the problem from the equation

$$L\phi_n(x) = \lambda_n \phi_n(x) \tag{A-3}$$

where the eigen values $\lambda_n$ comprise a countable number of infinite values. From equations A-2 and A-3, we have

$$\alpha \frac{d^2[\phi_n(x)]}{dx^2} - \lambda_n \phi_n(x) = 0 \tag{A-4}$$

which must be solved, subject to the boundary conditions:

$$\phi_n(x = B) = \phi_n'(x = 0) = 0 \tag{A-5}$$

It may readily be shown that no nontrivial solutions exist for the systems of equations A-4 and A-5, for $\lambda_n \geqslant 0$. For $\lambda_n < 0$, the solution of equation A-4 is given by

$$\phi_n(x) = A_n \sin(\sqrt{\lambda_n/\alpha}\; x) + B_n \cos(\sqrt{\lambda_n/\alpha}\; x) \tag{A-6}$$

using the boundary condition $\phi_n' (x = 0) = 0$, the value of $A_n$ is found to be zero. For a nontrivial solution, the eigen values are given by

$$\lambda_n = \alpha \frac{(2n-1)\pi}{2B}, \; n = 1,2,3, \ldots \tag{A-7}$$

∴ the eigen vectors (or eigen functions) become

$$\phi_n(x) = B_n \cos \left[ \frac{(2n-1)\pi}{2B} x \right] \tag{A-8}$$

The value of the normalization constant, $B_n$ determined from the orthogonality condition, is found to be

$$B_n = \sqrt{2/B}, \; n = 1,2,3, \ldots \tag{A-9}$$

$$\therefore \phi_n(x) = \sqrt{2/B} \cos\left[\frac{(2n-1)\pi}{2B}\right] x, n = 1,2,3,.. \tag{A-10}$$

Next, the driving function $f(x) = [p(x)]/n_e$ may also be expanded in the form of an infinite series as

$$f(x) = \sum_{n=1}^{\infty} f_n \phi_n(x) \text{ where } f_n = \int_0^B d\xi\, \phi_n^*(\xi) f(\xi), \tag{A-11}$$

$\phi_n^*(\xi)$ being the complex conjugate of $\phi_n(\xi)$.

Thus equation 4 becomes

$$\dot{s}_n(t) - \lambda_n s_n(t) = f_n; n = 1,2,3, \ldots. \tag{A-12}$$

whose solution, using the known initial condition is

$$s_n(t) = \int_0^t d\tau\, e^{\lambda_n(t-\tau)} f_n; n = 1,2,3, \ldots. \tag{A-13}$$

Now using equation A-1, the complete solution can be written in the form,

$$s(x,t) = \sum_{n=1}^{\infty} \left[\int_0^t d\tau\, e^{\lambda_n(t-\tau)} f_n\right] \phi_n(x) \tag{A-14}$$

but $\quad f_n = \int_0^B d\xi\, \phi_n^*(\xi) f(\xi)$

$$= \int_0^L d\xi \sqrt{2/B} \cos\left(\frac{2n-1}{2B}\pi\xi\right) \frac{p}{n_e}$$

$$= \frac{2\sqrt{2B}\, p}{n_e(2n-1)\pi} \sin\left(\frac{2n-1}{2B}\pi L\right) \tag{A-15}$$

From equations A-14 and A-15, we get

$$s(x,t) = \sum_{n=1}^{\infty} \left[\int_0^t d\tau\, e^{\lambda_n(t-\tau)} \frac{2\sqrt{2B}\, p}{n_e(2n-1)\pi} \sin\left(\frac{2n-1}{2B}\pi L\right)\right] \phi_n(x)$$

$$= \frac{2\sqrt{2B}\, p}{n_e \pi} \sum_{n=1}^{\infty} \left[\frac{1}{(2n-1)} \sin\left(\frac{2n-1}{2B} L\pi\right) \int_0^t d\tau\, e^{\lambda_n(t-\tau)}\right] \phi_n(x) \tag{A-16}$$

The value of the integral

$$\int_0^t d\tau\, e^{\lambda_n(t-\tau)} \text{ is found to be } \frac{1 - e^{\lambda_n t}}{-\lambda_n} \tag{A-17}$$

From equations A-7, A-10, A-16, and A-17 after simplifying, we arrive at our formal solution:

$$s(x,t) = \frac{16B^2 p}{n_e \alpha \pi^3} \sum_{n=1}^{\infty} \frac{1}{(2n-1)^3} \left(1 - e^{-\alpha(2n-1)^2 \frac{\pi^2}{4B^2} t}\right) \cdot \sin\left(\frac{2n-1}{2B}\pi L\right) \cos\left(\frac{2n-1}{2B}\pi x\right) \tag{A-18}$$

When rendered dimensionless, this becomes

$$\left.\begin{aligned} s''(x',t') = {} & \frac{16B'^2}{\pi^3} \sum_{n=1}^{\infty} \frac{1}{(2n-1)^3} \cdot \\ & \sin\left(\frac{2n-1}{2B'}\pi\right) \cos\left(\frac{2n-1}{2B'}\pi x'\right) \end{aligned}\right\} \text{steady state solution}$$

$$\left.\begin{aligned} & -\frac{16B'^2}{\pi^3} \sum_{n=1}^{\infty} \frac{1}{(2n-1)^3} e^{-(2n-1)^2 \frac{\pi^2}{4B'^2} t'} \\ & \sin\left(\frac{2n-1}{2B'}\pi\right) \cos\left(\frac{2n-1}{2B'}\pi x'\right) \\ & \qquad n = 1,2,3,.. \end{aligned}\right\} \text{Transient part} \tag{A-19}$$

where $B' = B/L$.

Numerical evaluation of this expression presents considerable difficulty in that the steady state part in the present form is not rapidly convergent. However, this difficulty can be circumvented by replacing the above steady state part by the one obtained from the solution of the steady state dimensionless versions of equation 4 — viz. —

$$\frac{d^2s''}{dx'^2} + 1 = 0 \qquad |x'| \leqslant 1 \qquad \text{(A-20)}$$

$$\frac{d^2s''}{dx'^2} = 0 \qquad |x'| \geqslant 1 \qquad \text{(A-21)}$$

subject to boundary conditions

$$s''(x' = B') = s''(x' = -B') = 0 \qquad \text{(A-22)}$$

The system of equations A-20 through A-22 was solved by Green's function method (Amar, 1973), which yielded the following solution

$$\text{for } |x'| \leqslant 1: \qquad s''(x') = -\frac{x'^2}{2} + B' - \tfrac{1}{2} \qquad \text{(A-23)}$$

$$\text{for } |x'| \geqslant 1: \qquad s''(x') = -x' + B' \qquad \text{(A-24)}$$

Now replacing the steady state part of equation A-19 by expressions A-23 and A-24 the formal solution of the unsteady problem is given by equations 10 and 11 of text.

## APPENDIX B

### IMPLICIT FINITE DIFFERENCE FORMULATION AND COMPUTATIONAL ALGORITHM FOR THE SOLUTION OF SYSTEM OF EQUATIONS 12 THROUGH 14

Since the explicit difference scheme, despite its computational simplicity (Ames, 1969), suffers from the unfortunate consequence that if $\Delta x$ is chosen rather small in the interests of accuracy, the permissible $\Delta t$ [note that the limit imposed on $\Delta t$ is proportional to $(\Delta x)^2$] turns out to be so very small that an enormous number of cycles is required to complete a problem. Consequently, the author turns to a three-time level implicit scheme so as to avoid severely restrictive stability criterion of the explicit scheme as well as the computational complexities of the corresponding two-level implicit scheme, e.g., the iterative solution technique known as the Crank-Nicolson Method (Mitchell, 1969). We proceed with the implicit formulation for the nonlinear problem as follows.

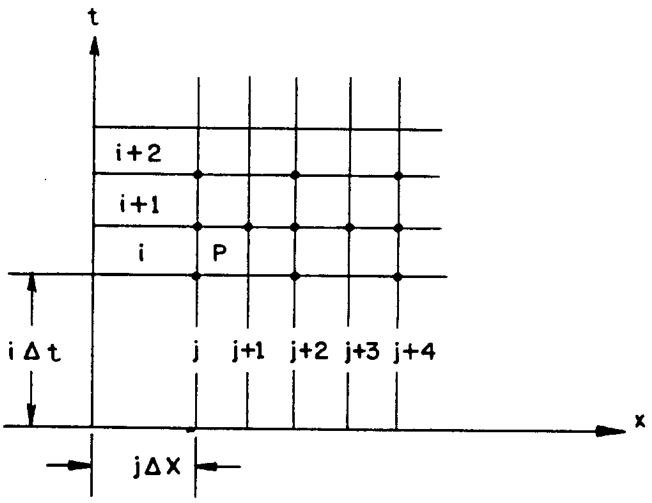

**Fig. B-1. Diagram of a computational grid pertaining to an implicit scheme representing equation B-4. Point P is a typical node in the grid system with coordinates ($t_i$, $x_j$).**

The difference scheme is built on the computational grid shown in Figure B-1. Dropping primes and rewriting equation 12, we get

$$\lambda_o s_t = \frac{\partial}{\partial x}\left[(1 + sp_o)\, s_x\right] + 1 \qquad \text{(B-1)}$$

Using $\Delta x$ and $\Delta t$ as the dimensionless mesh spacings in the x and t directions, respectively, the values of the various continuous derivatives may be approximated in the discretized finite difference plane ($x = j\Delta x$, $t = i\Delta t$). Then equation B-1 becomes

$$\begin{aligned}\frac{\lambda_o}{2\Delta t}(s_{i+1,j} - s_{i-1,j}) &= \frac{1}{(\Delta x)^2}\,[(1 + p_o s_{i,j+\frac{1}{2}})(s_{i,j+1} - s_{i,j}) \\ &\quad - (1 + p_o s_{i,j-\frac{1}{2}})(s_{i,j} - s_{i,j-1})] + 1 \\ &= \frac{1}{(\Delta x)^2}(s_{i,j+1} - 2s_{i,j} + s_{i,j-1}) \\ &\quad + \frac{p_o}{(\Delta x)^2}\,[(s_{i,j+\frac{1}{2}})(s_{i,j-1} - s_{i,j}) \\ &\quad - (s_{i,j-\frac{1}{2}})(s_{i,j} - s_{i,j-1})] + 1 \qquad \text{(B-2)}\end{aligned}$$

where the difference operators on the right-hand side are the usual centered difference approximations (Forsythe and Wasow, 1960. Isaacson and Keller, 1966). Since this scheme is known to be unconditionally unstable, we transform the above scheme to a three-level implicit-scheme of high accuracy and unconditional stability along the lines devised by Lees (1966). Also see Mitchell (1969). This requires the following substitutions in the right-hand side of equation B-2:

$$\left.\begin{aligned}
s_{i,j+1} &= \tfrac{1}{3}(s_{i+1,j+1} + s_{i,j+1} + s_{i-1,j+1}) \\
s_{i,j} &= \tfrac{1}{3}(s_{i+1,j} + s_{i,j} + s_{i-1,j}) \\
s_{i,j-1} &= \tfrac{1}{3}(s_{i+1,j-1} + s_{i,j-1} + s_{i-1,j-1}) \\
s_{i,j+1/2} &= \tfrac{1}{2}(s_{i,j+1} + s_{i,j}) \\
s_{i,j-1/2} &= \tfrac{1}{2}(s_{1,j} + s_{i,j-1})
\end{aligned}\right\} \quad \text{(B-3)}$$

Rearranging terms in the resulting equation so that the unknowns (terms involving i + 1) are on the left-hand side and all the knowns are on the right-hand side, as well as adding 1 to each of the indices i and j to eliminate zeros for these indices, and treating j as an integer, yields (for $|x| \leqslant 1$) an equation suitable for computer application (see Figure B-1) with $r = \Delta t/\Delta x^2$:

$$\begin{aligned}
&-\tfrac{1}{3}(2r + 2p_o r s_{i+1,j+1})s_{i+2,j} \\
&\quad + [1 + \frac{4r}{3} + \frac{4p_o r}{3}(s_{i+1,j+3} + s_{i+1,j+1})]\, s_{i+2,j+2} \\
&- \tfrac{1}{3}(2r + 2p_o r s_{i+1,j+3})s_{i+2,j+4} = \tfrac{1}{3}(2r + 2p_o r s_{i+1,j+1})s_{i,j} \\
&\quad + [1 - \frac{4r}{3} - \frac{4p_o r}{3}(s_{i+1,j+3} + s_{i+1,j+1})]\, s_{i+1,j+2} \\
&+ \tfrac{1}{3}(2r + 2p_o r s_{i+1,j+3})s_{i,j+4} + \tfrac{1}{3}(2r + 2p_o r s_{i+1,j+1})s_{i+1,j} \\
&\quad - \tfrac{1}{3}[4r + 4p_o r(s_{i+1,j+3} + s_{i+1,j+1})]\, s_{i+1,j+2} \\
&+ \tfrac{1}{3}(2r + 2p_o r s_{i+1,j+3})s_{i+1,j+4} + 2\,\Delta t/\lambda_o \qquad \text{(B-4)}
\end{aligned}$$

It should be noted that the order of accuracy of formula B-2 remains intact and the system of equations to be solved at $t = (i + 2)\Delta t$ is linear, thus avoiding the complication of solving a set of nonlinear equations at each time step. For a mathematical proof of the convergence of equation B-4, reference may be made to Lees (1966). The finite difference approximation for equation 13 may be obtained in an identical manner. The difference form for the centerline boundary condition $s_x(x = 0, t) = 0$ (after dropping primes) is

$$(s_{i,j+1} - s_{i,j-1})/2\Delta x = 0 (i \geqslant 1) \qquad \text{(B-5)}$$

where j refers to the center line coordinate in the x-direction.

As indicated by Remson, *et al.* (1971), the above system of equations generated in approximating the governing partial differential equations is tridiagonal. The method used to solve this system of equations is the so-called "tridiagonal algorithm," generally attributed to Thomas and Wormleaton (1971) and is one of the most efficient methods for solving such a system. Douglas (1959) has shown that the scheme is extremely stable with respect to round-off errors.

The basis of the solution technique for system of equation B-4 is the decomposition of the coefficient matrix into the lower triangular matrix and an upper triangular matrix, and obtaining a solution of the system of equations corresponding to the lower triangular matrix by forward substitution and finally that of the upper triangular matrix by backward substitution. Further details of numerical procedure may be found in Remson *et al.* (1971).

# 18. Wastewater Renovation in Rapid Infiltration Systems

H. Bouwer

## 18.1 Introduction

The ability of vadose zones and aquifers to remove pollutants from low quality water flowing through such media is utilized in rapid-infiltration or soil-aquifer treatment systems. With these systems, partially treated sewage effluent or similar low quality water (processing plant effluent, urban runoff, etc.) is infiltrated into the soil (normally with basins or furrow systems), from where it percolates down to the groundwater. It then moves laterally through the aquifer for some distance before it leaves the aquifer or is collected from the aquifer as "renovated" water. The renovated water can be collected with wells or subsurface drains for reuse or disposal or it can seep naturally into streams or other surface water (Fig. 1). The latter systems are used to minimize pollution of streams or other receiving waters by giving the wastewater soil-aquifer treatment rather than discharging it directly into the surface water.

## 18.2 Basin Management

The infiltration basins or furrows must be regularly dried to allow atmospheric oxygen to enter the soil for decomposition of organic matter, nitrification of ammonium, and other aerobic processes. Drying also helps restore infiltration rates, which normally decrease during infiltration due to accumulation of solids, algae, etc. on the soil surface. Infiltration schedules may range from 8 h wet, 16 h dry each day to flooding and drying periods of several weeks each. The optimum schedule depends on the local conditions of soil, climate, and wastewater characteristics (suspended solids, biochemical oxygen demand) and whether the system is to be operated for maximum hydraulic capacity or for maximum removal of nitrogen from the water. Hydraulic loading rates of rapid-infiltration systems typically are in the range of 50–500 m $\mathrm{yr}^{-1}$ (0.5–5 million $\mathrm{m}^3\,\mathrm{yr}^{-1}\,\mathrm{ha}^{-1}$ of basin area). Mechanical removal of accumulated solids from the soil surface may be periodically required to maintain high infiltration rates.

Because the bottom of infiltration basins is usually covered by a clogging layer of accumulated solids, infiltration rates are reduced, soil-water pressure heads beneath the clogged layers are negative, and the soil below the clogged layer is unsaturated. Infiltration rates for soils flooded with secondary sewage effluent

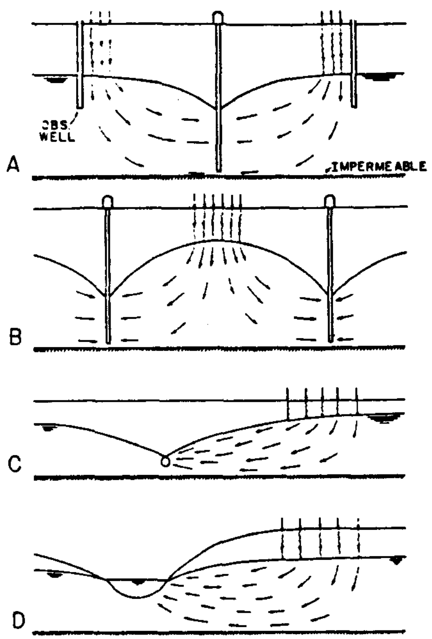

Fig. 1 A–D. Soil-aquifer treatment systems with infiltration areas in two parallel rows and line of wells midway in between (A), infiltration areas in center surrounded by a circle of wells (B), collection of renovated water by subsurface drain (C), and natural drainage into surface water (D)

typically are about one-half of the rates for clear water, which in turn are about equal to the hydraulic conductivity of the upper soil layers. In warm, arid climates, drying periods are about as long as flooding periods, so that the long-term infiltration rate or hydraulic loading rate, will be about one-fourth the hydraulic conductivity of the top soil expressed for the same time period. For primary effluent and/or more humid, colder climates, the ratio between hydraulic loading rate and saturated hydraulic conductivity of the soil can be much less, for example 0.1.

To allow flexibility in scheduling flooding and drying in a rapid-infiltration system, a relatively large number of basins is better than only a few basins. Since overloading of the infiltration system must be avoided, a few reserve basins should always be included to take the wastewater during times of low infiltration rates (rainy or cold periods, for example) or when part of the system is undergoing repair. Draining the basins by gravity at the end of a flooding period hastens the drying and gives greater hydraulic loading rates than when the basins are dried by stopping the inflow and letting all the water go into the ground by infiltration.

Suspended algae (for example, *Carteria klebsii*) in the wastewater should be minimized. Such organisms can clog the soil and cause calcium carbonate to precipitate due to the pH increase of the wastewater as the algae absorb $CO_2$ from the water for photosynthesis. Growth of filamentous algae (for example, *Oscillatoria* sp.) on the bottom of the infiltration basins is not as serious and can actually increase infiltration rates as oxygen bubbles form in the algal mat. This causes portions of the algal mat to float to the surface, carrying suspended materials up with them and exposing fresh, "rejuvenated" basin soil.

## 18.3 Quality Improvement

As the wastewater (partially treated sewage effluent, for example) moves through the soil and down to the groundwater, suspended solids, bacteria, viruses, and biodegradable carbon will be essentially completely removed. Some residual organic carbon, including synthetic materials, can be expected in the renovated water. Heavy metals and phosphate concentrations will be greatly reduced during passage through the unsaturated zone and aquifer, especially in calcareous materials. For secondary sewage effluent where nitrogen is mostly in the ammonium form, the nitrogen will be essentially completely converted to nitrate if short, frequent infiltration periods are held. It will stay as ammonium if infiltration periods are long and the cation exchange complex in the soil has become saturated with ammonium and it will be partially converted to nitrogen gas and nitrogen oxides if infiltration periods are intermediate, so that ammonium is adsorbed in the soil during flooding and nitrified and denitrified during drying (Bouwer and Chaney 1974, Bouwer et al. 1974b, 1980, Bouwer 1982). Most of these quality improvements take place in the top meter of the unsaturated zone. However, for additional polishing treatment and peace of mind, the travel distance of the water through the vadose zone and aquifer should always be taken as large as possible.

## 18.4 Flushing Meadows Project

Since hydraulic capacity of a rapid infiltration system and quality improvement of the wastewater by soil-aquifer treatment are site specific, local experimentation is usually desirable to see what results can be expected and how the rapid infiltration system should be best designed and managed. An example of such a pilot project is the Flushing Meadows Project in the Salt River floodplain west of Phoenix, Arizona. This project, installed in 1967, consisted of six parallel infiltration basins 6.1 × 213 m each that were intermittently flooded with secondary sewage effluent (activated sludge process, nonchlorinated). The soil was fine loamy sand for the top 1 m underlain by sand and gravel in various layers to a depth of about 75 m where a clay layer began. The water table was at a depth of 3 m and renovated water was sampled from depths of 6–9 m beneath the basins and at various distances from the basins to study the effect of additional lateral movement through the aquifer on quality of renovated water. The monitoring wells consisted of 15 cm diameter cased wells open at the bottom. The Flushing Meadows Project was operated from 1967 until 1978. In the first 5 years, the basins were flooded and managed to maximize hydraulic loading rates. In the second 5 years, the basins were managed to maximize denitrification. Main results obtained from the Flushing Meadows Project were as follows (Bouwer et a. 1974a, b, 1980):

1. The largest hydraulic capacity of the system was obtained with flooding periods of 2–3 weeks alternated with drying periods of 10–20 days. At this schedule and using a water depth of 0.3 m in the basins, the hydraulic loading rate was 122 m yr$^{-1}$. Vegetation or gravel layers in the basins offered no particular advantages. Thus, the best bottom condition was bare soil. Volunteer vegetation seemed to have no undesirable effects, so weed control was not necessary. Sludge-like accumulation on the basin bottoms was removed every 1 or 2 years. After 10 years of operating the project and a total infiltration of 754 m, there were no signs of reductions in hydraulic loading or in hydraulic conductivity of the underlying aquifer.
2. Nitrogen removal from the effluent water as it seeped through the ground to become renovated water was about 30% at maximum hydraulic loading, but 65% if the loading rate was reduced to about 70 m yr$^{-1}$ by using 9 day flooding and 12 day drying cycles and by reducing the water depth in the basins to 0.15 m. The form and concentration of nitrogen in the renovated water sampled from the aquifer below the basins were slow to respond to a reduction in hydraulic loading (Bouwer et al. 1980). In the tenth year of operation (1977), the renovated water contained 2.8 mg l$^{-1}$ ammonium nitrogen, 6.25 mg l$^{-1}$ nitrate nitrogen, and 0.58 mg l$^{-1}$ organic nitrogen, for a total nitrogen content of 9.6 mg l$^{-1}$. This was 65% less than the total nitrogen content of the secondary sewage effluent, which averaged 27.4 mg l$^{-1}$ (mostly as ammonium nitrogen) in that year.
3. Phosphate removal increased with increasing distance of underground movement of the sewage water. After 9 m of downward movement to and in the underlying aquifer, removal was about 40% at high-hydraulic loading and 80% at reduced hydraulic loading. Additional lateral movement of 61 m through the aquifer increased the removal to 95% in 1977 (i.e., a concentration of 0.51 mg l$^{-1}$ phosphate phosphorus in the renovated water versus 7.9 mg l$^{-1}$ in the effluent). After 10 years and a total infiltration of 754 m, there were no signs of a decrease in phosphate removal.
4. Fluoride removal paralleled phosphate removal. Fluoride concentrations in 1977 were 2.08 mg l$^{-1}$ for the effluent, 1.66 mg l$^{-1}$ for the renovated water sampled from wells between the basins, and 0.95 mg l$^{-1}$ for renovated water sampled from a well 30 m away from the basins.
5. Boron was not removed and was present at concentrations of 0.5 to 0.7 mg l$^{-1}$ (0.59 mg l$^{-1}$ in 1977) in both effluent and renovated water.
6. Concentrations of zinc, copper, cadmium, and mercury in effluent and renovated water were below the maximum limits for drinking and permanent irrigation. The concentration of lead in the renovated water was 0.066 mg l$^{-1}$, which exceeded the 0.05 mg l$^{-1}$ limit for drinking water, but was well below the 5 mg l$^{-1}$ limit for permanent irrigation (Nat. Acad. of Sci., Nat. Acad. of Eng., 1972).

7. The biochemical oxygen demand (BOD) was reduced from a range of 10–20 mg $l^{-1}$ for the effluent to essentially zero for the renovated water. Corresponding ranges for chemical oxygen demand (dichromate technique) were 30–60 mg $l^{-1}$ in the effluent and 10–20 mg $l^{-1}$ in the renovated water. Not all organic carbon was removed from the effluent water as it seeped through the soil and aquifer. The total organic carbon (TOC) contents of the renovated water averaged 5.2 mg $l^{-1}$ for the wells between the basins and 3.3 mg $l^{-1}$ for the wells 30 m away from the basins. The average TOC content of the secondary sewage effluent was 19.2 mg $l^{-1}$.

8. The secondary sewage effluent contained 21 virus units (PFU's) per liter, but viruses could not be detected in renovated water sampled below the basins (Gilbert et al. 1976). Fecal coliforms were present at concentrations of up to several hundred per 100 ml in the renovated water sampled below the basins, but could not be detected in renovated water sampled 91 m away from the basins. Hence, a lateral movement of about 100 m through the aquifer apparently produced renovated water free from fecal coliforms. Laboratory studies showed that distilled water (rain!) could remobilize previously adsorbed viruses in the soil only during the first 24 h of a drying period, but that addition of calcium chloride to the water prevented most of this desorption (Lance et al. 1976). This finding is important for managing rapid infiltration systems in humid climates.

## 18.5 23rd Avenue Project

Following the successful results of the Flushing Meadows Project, the City of Phoenix became interested in wastewater renovation by soil-aquifer treatment on a commercial scale. This led to the construction in 1975 of the 23rd Avenue Project, where the lower, 16-ha pond of a 2-ponds-in-series stabilization lagoon system was converted into four parallel rapid infiltration basins of 4 ha each (Fig. 2). The renovated water, which is to be pumped from three wells on the center dike of the basin area, could be delivered to an irrigation district for direct sale of the renovated water or for exchange for high quality groundwater pumped from district wells. This groundwater would then be used to augment the city's water supply. For a complete report on this project, reference is made to Bouwer and Rice (1984).

The soils at the 23rd Avenue Project are sandier and more gravelly with more boulders than at the Flushing Meadows Project. The groundwater is also deeper, i.e., between 15 and 25 m before 1978, and between 5 and 15 m after 1978. The groundwater table rise after 1978 was due to a series of wet years, resulting in releases of water through the Salt River, which flows south of the infiltration basins and which recharged the groundwater. The vadose zone below the 23rd Avenue Project consists of an irregular succession of sand, gravel, and boulder layers with occasional silty strata (Fig. 3).

For the first 2 years of operation of the project, the secondary effluent (activated sludge, nonchlorinated) from the 23rd Avenue sewage treatment plant flowed through a 32-ha lagoon before it entered the infiltration basins (Fig. 2).

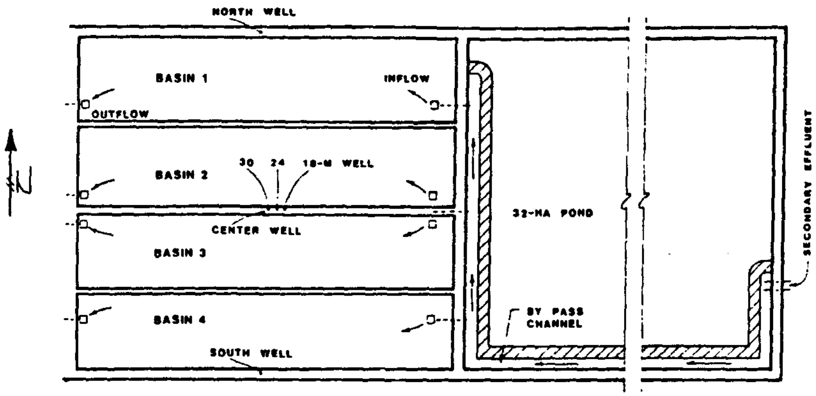

Fig. 2. 23rd Avenue Rapid-Infiltration Project showing the four infiltration basins, monitoring wells, production well (*center well*), and 32 ha lagoon with bypass channel

Fig. 3. Wall of gravel pit near 23rd Avenue Project as indicator of vadose zone below infiltration basins

This resulted in a heavy growth of suspended algae (*Carteria klebsii*) in the effluent water before it entered the infiltration basins. Consequently, hydraulic loading rates were only 21 m yr$^{-1}$. To reduce the growth of algae in the secondary effluent, a bypass channel was constructed around the 32-ha pond so that the effluent could flow directly to the infiltration basins. In addition, the water depth in the infiltration basins was reduced from about 1 m to 0.2 m to increase the rate of turnover of the water in the infiltration basins, thus reducing the time that a given body of water was exposed to sunshine and, hence, the opportunity for growth of suspended algae. After the bypass channel was put into operation and the surface soil in the infiltration basins was ripped to break up possible crusts of calcium carbonate formed by the previous high algal activity, hydraulic loading rates were about 100 m yr$^{-1}$. At this rate, the 16 ha infiltration system has a capacity of about 44,000 m$^3$ d$^{-1}$.

The aquifer below the infiltration basins is unconfined, very permeable (sands and gravels, similar to Fig. 3), and at least 70 m thick (from project well logs). The actual thickness probably is several hundred meters more as indicated by other well logs and the general geology of the area.

The project has several observation wells 18–30 m deep and one production well which is perforated from 30–54 m and has a capacity of about 13,000 m$^3$ d$^{-1}$ (center well in Fig. 2). Thus, two more wells symmetrically located on the center dike would have to be drilled to pump renovated water out of the aquifer at the same rate as it arrives at the aquifer from the infiltration basins. The new wells would be drilled somewhat deeper than the center well to obtain the needed additional capacity. Pumping the renovated water out at the same rate as the infiltration rate should establish an equilibrium system in the aquifer and should prevent movement of renovated water into the aquifer outside the infiltration system.

The sewage water used for the 23rd Avenue infiltration project was secondary effluent, activated sludge process. Initially, this effluent was not chlorinated. However, in November 1980, a chlorination facility was put into operation at the treatment plant that dosed the effluent with chlorine at the rate of 1.5 mg l$^{-1}$. This offered an opportunity to study how the performance of the soil-aquifer treatment system was affected by chlorination of the effluent. Therefore, in discussing some of the water quality aspects, distinction is made between the pre- and post-chlorination periods. Quality parameters of the secondary effluent prior to infiltration and of the renovated water from the center well (unless mentioned otherwise) were as follows [see Bouwer and Rice (1984) for complete report]:

1. The total dissolved salts content of the secondary effluent entering the ground in the last few years of the project averaged about 750 mg l$^{-1}$ and that of the renovated water about 790 mg l$^{-1}$. The increase of 40 mg l$^{-1}$ can only be partly explained by evaporation from the basins and the soil. Mobilization of calcium carbonate in the soil and aquifer as the pH of the effluent dropped from slightly alkaline to neutral due to biological reactions could also have contributed to the increase in dissolved salts.

2. The suspended solids content of the secondary effluent going into the basins before the bypass channel was constructed was often in the 50–100 mg l$^{-1}$ range in the summer and about 15 mg l$^{-1}$ in the winter. The high summer values were due to algae growth in the 32 ha lagoon. After the bypass channel was put into operation, the suspended solids content of the secondary effluent entering the infiltration basins averaged about 11 mg l$^{-1}$ with a range of 5–20 mg l$^{-1}$. The suspended solids content of the renovated water was about 1 mg l$^{-1}$, mostly fines from the aquifer.

Table 1. Fecal coliform concentrations of secondary effluent and renovated water in three different periods (colonies/100 ml)

| | | Prechlorination pre-bypass | Prechlorination post-bypass | Postchlorination post-bypass |
|---|---|---|---|---|
| Secondary effluent entering basin | | $10^4$ | $1.8 \times 10^6$ | $3.5 \times 10^3$ |
| Renovated water from center well | Average | 2.3 | 22 | 0.27 |
| | range | 0–40 | 0–160 | 0–3 |

3. Nitrogen was mostly in the form of ammonium in the secondary effluent (average about 16 mg l$^{-1}$ $NH_4$-N). There were also about 2 mg l$^{-1}$ organic nitrogen and traces of nitrite and nitrate. The 2 week flooding and drying cycles must have been conducive to denitrification in the soil, because the total N content of the renovated water averaged about 5.6 mg l$^{-1}$ of which 5.3 was as nitrate, 0.1 as ammonium, 0.1 as organic, and 0.02 as nitrite. The nitrogen removal percentage thus was about 70%. This removal was the same before and after the secondary effluent was chlorinated, indicating that the low residual chlorine of the effluent by the time it infiltrated into the ground apparently had no effect on the nitrogen transformations in the soil.

4. Phosphate-phosphorus concentrations in the last few years averaged 5.5 mg l$^{-1}$ for the secondary effluent and 0.37 mg l$^{-1}$ for the renovated water pumped from the center well. The shallower wells showed a higher phosphate content, indicating that precipitation of calcium phosphate continued in the aquifer.

5. From a microbiological standpoint, there were three periods, (a) the prechlorination, prebypass channel period when nonchlorinated secondary effluent flowed through the 32 ha lagoon before entering the infiltration basins; (b) the prechlorination, postbypass channel period when nonchlorinated secondary effluent flowed around the lagoon and directly into the infiltration basins; and (c) the postchlorination, postbypass channel period when chlorinated secondary effluent flowed directly into the infiltration basins. The fecal coliform densities in the secondary sewage effluent entering the basins and in the renovated water from the center well are shown in Table 1. The high fecal coliform concentrations in the renovated water generally occurred when water that had infiltrated at the start of a new flooding period arrived at the intake of the well. Apparently, the absence of a clogged soil surface in the basins when flooding was resumed and the decreased bacterial activity in the soil during drying allowed fecal coliforms to move through the unsaturated zone to the groundwater. After a few days of continued flooding, however, fecal coliforms were again completely removed by the clogged surface and in the unsaturated zone and did not show up in the renovated water from the wells. The data in Table 1 show that the higher the fecal coliform concentration in the secondary effluent going into the ground, the more fecal coliforms show up in the renovated water. Virus concentrations in the renovated wa-

ter, determined on samples of 1,000 to 2,000 l, were 1.3 pfu/100 l before the secondary effluent was chlorinated and zero after it was chlorinated. Chlorination thus had a beneficial effect on fecal coliform and virus concentrations in the renovated water.

6. The total organic carbon (TOC) concentration of the secondary effluent averaged 12 mg $l^{-1}$ where it entered the infiltration basins and 14 mg $l^{-1}$ at the opposite ends. This increase was probably due to biological activity in the water as it moved through the basins. The renovated water had a TOC content of 2 mg $l^{-1}$. The TOC removal was the same before and after chlorination of the secondary effluent, indicating that chlorination had no effect on the microbiological processes in the soil. The concentration of organic carbon in the renovated water of 2 mg $l^{-1}$ was higher than the 0.7 mg $l^{-1}$ typically found in unpolluted groundwaters and which is mostly due to humic substances like fulvic acid (Thurman 1979). Thus, the renovated sewage water from the soil-aquifer treatment process probably contained a number of synthetic organics, some of which could be carcinogenic or otherwise toxic.

## 18.6 Identification of Trace Organics

The nature and concentration of trace organics in the secondary sewage effluent and renovated water were determined by Stanford University's Environmental Engineering and Science Section, using gas chromatography and mass spectrometry. The studies were carried out for 2 months with nonchlorinated effluent and then for 2 months with chlorinated effluent, taking weekly and biweekly samples. As could be expected, the results showed a wide variety of all sorts of organics (including priority pollutants), many in concentrations on the order of μg $l^{-1}$. For a full report on this work, reference is made to Bouwer et al. (1983). The main results will be summarized in the following paragraphs.

A comparison between halogenated aliphatic hydrocarbons and chlorinated aromatics in the secondary effluent as it entered the infiltration basins before and after chlorination showed that chlorination resulted in higher chloroform concentrations and produced three additional brominated trihalomethanes (Table 2). Otherwise, chlorination had no significant effect on the chlorinated organics in the secondary effluent. This was also true for the nonhalogenated aliphatic and aromatic compounds where differences between the pre- and postchlorination periods were small and most likely due to normal concentration fluctuations in the effluent. Nonhalogenated compounds identified in the secondary effluent included the aliphatic hydrocarbons 5-(2-methylpropyl) nonane, 2,2,5-trimethylhexane, 6-methyl-5-nonene-4-one, 2,2,3-trimethylnonane, and 2,3,7-trimethyloctane; and the aromatic hydrocarbons o-xylene, m-xylene, p-xylene, $C_3$ benzene isomer, $C_3$ benzene isomer, styrene, 1,2,4-trimethyl benzene, ethylbenzene, naphthalene, phenanthrene, and diethylphthalate. The concentrations of these materials were less than 1.5 μg $l^{-1}$ and mostly below 1 μg $l^{-1}$ except for the diethylphthalate which had a concentration of 20 μg $l^{-1}$ in the prechlorination period and 15 μg $l^{-1}$ in the postchlorination period.

**Table 2.** Halogenated organic compounds in secondary effluent during pre- and postchlorination period (μg $l^{-1}$)

| | Secondary effluent (basin inflow) | |
|---|---|---|
| Constituent[a] | Prechlorination | Postchlorination |
| *Halogenated aliphatic hydrocarbons* | | |
| Chloroform | 2.88 | 4.79 |
| 1,1,1-Trichloroethane | 2.45 | 1.79 |
| Carbon tetrachloride | 0.13 | 0.15 |
| Bromodichloromethane | –[b] | 0.51 |
| Trichloroethylene | 0.91 | 0.53 |
| Dibromochloromethane | – | 0.46 |
| Tetrachloroethylene | 2.21 | 1.82 |
| Bromoform | – | 0.13 |
| *Chlorinated aromatics* | | |
| o-Dichlorobenzene | 4.11 | 3.18 |
| m-Dichlorobenzene | 1.15 | 0.53 |
| p-Dichlorobenzene | 2.70 | 2.82 |
| 1,2,4-Trichlorobenzene | 0.33 | 0.44 |
| Trichlorophenol | 0.01 | 0.02 |
| Pentachlorophenol | 0.02 | 0.04 |
| Pentachloroanisole[c] | 0.63 | 0.26 |

[a] Identification confirmed by comparison with standards

[b] – Not detected

[c] Only compound that is not a priority pollutant

Of the volatile organics, about 30%–70% volatilized from the effluent as it flowed through the infiltration basins. These compounds included the chlorinated aliphatic hydrocarbons chloroform, 1,1,1-trichloroethane, trichloroethylene, and tetrachloroethylene; the chlorinated aromatic hydrocarbons o-dichlorobenzene, m-dichlorobenzene, p-dichlorobenzene, 1,2,4-trichlorobenzene, and (chloromethyl)-benzene; the aliphatic hydrocarbons 2,2,5-trimethylhexane, 5-(2-methylpropyl) nonane, and 2,2,3-trimethylnonane; and the aromatic hydrocarbons o-xylene, m-xylene, 1,2,4-trimethyl benzene, $C_3$-benzene isomer, and naphthalene.

## 18.7 Removal of Trace Organics in the Unsaturated Zone

To study the removal of trace organics from the effluent water in the unsaturated zone, renovated water samples were obtained from an 18 m deep well in the center of the project (Fig. 2). This well, which was cased with steel pipe open at the bottom, yielded renovated water from the top of the aquifer, so that the results would give a good indication of the removal of trace organics in the unsaturated zone. Table 3 shows the concentrations of nonhalogenated organics in the secondary sewage effluent as sampled in the basins and the percentage reduc-

Table 3. Percentage decrease in concentration of nonhalogenated hydrocarbons during passage through unsaturated zone

| | Prechlorination period | | Postchlorination period | |
|---|---|---|---|---|
| | Geometric mean concentration of secondary effluent (27 samples) | Average decrease in renovated water from 18 m well (6 samples) | Geometric mean concentration of secondary effluent (27 samples) | Average decrease in renovated water from 18 m well (6 samples) |
| Constituent | (μg $l^{-1}$) | (%) | (μg $l^{-1}$) | (%) |
| *Aliphatic hydrocarbons* | | | | |
| 5-(2-Methylpropyl) nonane | 0.35 | >94 | 0.57 | >96 |
| 2.2.5-Trimethylhexane | 0.11 | >82 | 0.18 | >89 |
| 6-Methyl-5-nonene-4-one | 0.41 | 93[a] | 0.94 | 98[a] |
| 2.2.3-Trimethylnonane | 0.21 | 76[a] | 0.25 | >92 |
| 2.3.7-Trimethyloctane | 0.12 | 50[a] | 0.27 | >93 |
| *Aromatic hydrocarbons* | | | | |
| o-Xylene | 0.45 | 67[a] | 0.50 | 88[a] |
| m-Xylene | 0.76 | 78[a] | 1.00 | 98[a] |
| p-Xylene | 0.17 | 53[a] | 0.12 | 92[a] |
| $C_3$-Benzene isomer | 0.56 | 84[a] | 0.34 | >94 |
| $C_3$-Benzene isomer | 0.48 | 85[a] | 0.53 | 96[a] |
| Styrene | 0.26 | >92 | 0.58 | 98[a] |
| 1.2.4-Trimethyl benzene | 0.80 | 78[a] | 1.04 | 96[a] |
| Ethylbenzene | 0.19 | 53[a] | 0.15 | 67 |
| Naphthalene | 0.22 | 68[a] | 0.63 | 91[a] |
| Phenanthrene | 0.10 | 80 | 0.10 | 90 |
| Diethylphthalate | 19 | 20 | 10 | 90 |

[a] Level of significance for the difference between basin and well concentrations based on a t-test comparison is less than or equal to 0.1

tion in these concentrations after the water had passed through the unsaturated zone and was sampled as renovated water from the 18 m well. Table 4 shows the same data for the halogenated organics.

The results show that nonhalogenated hydrocarbons (Table 3) decreased 50%–99% during percolation through the soil with concentrations in the renovated water being near or below the detection limit. However, most of the compounds could still be detected in the renovated water. Reduction percentages were generally higher during the postchlorination period as a result of higher concentrations in the effluent water observed for many of the nonhalogenated compounds. These compounds are subject to microbial decomposition and, presumably, were removed during soil percolation by this process. Decreases in the concentrations of the nonhalogenated priority pollutants were comparable to those for the other nonhalogenated aliphatic and aromatic hydrocarbons. Concentration variations in the renovated water were less than those in the sewage effluent in the basins. Thus, percolation through the unsaturated zone had the effect of damping fluctuations in concentrations and eliminating extreme values.

Table 4. Percentage decrease in concentration of halogenated organic substances during passage through unsaturated zone

| | Prechlorination period | | Postchlorination period | |
|---|---|---|---|---|
| | Geometric mean concentration of secondary effluent (27 samples) | Average decrease in renovated water from 18 m well (6 samples) | Geometric mean concentration of secondary effluent (27 samples) | Average decrease in renovated water from 18 m well (6 samples) |
| Constituent | (μg $l^{-1}$) | (%) | (μg $l^{-1}$) | (%) |
| *Chlorinated aliphatic hydrocarbons* | | | | |
| Chloroform | 2.72 | 61[b] | 3.46 | 88[b] |
| 1.1.1-Trichloroethane | 2.94 | 34 | 1.41 | 84[b] |
| Carbon tetrachloride | 0.12 | 0 | 0.12 | 42[b] |
| Bromodichloromethane | –[a] | – | 0.26 | > 62 |
| Trichloroethylene | 0.91 | –180[b] | 0.39 | –267[b] |
| Dibromochloromethane | – | – | 0.23 | > 57 |
| Tetrachloroethylene | 2.63 | – 97[b] | 1.69 | 31[b] |
| Bromoform | – | – | 0.08 | > 10 |
| *Chlorinated aromatics* | | | | |
| o-Dichlorobenzene | 3.52 | 25 | 2.40 | 10 |
| m-Dichlorobenzene | 0.79 | 58[b] | 0.38 | 5 |
| p-Dichlorobenzene | 2.25 | 33[b] | 1.81 | 10 |
| 1.2.4-Trichlorobenzene | 0.19 | 42[b] | 0.38 | 71[b] |
| Trichlorophenol | 0.01 | 0 | 0.02 | 0 |
| Pentachlorophenole | 0.02 | 0 | 0.04 | 0 |
| Pentachloroanisole | 0.43 | –150 | 0.18 | –120 |

[a] – Not detected

[b] Level of significance for the difference between basin and well concentrations based on a t-test comparison is less than or equal to 0.1

The halogenated organic compounds (Table 4) generally decreased to a lesser extent with passage through the unsaturated zone than the nonhalogenated compounds (Table 3). Of the halogenated aliphatic hydrocarbons, the renovated water concentrations of chloroform and 1.1.1-trichloroethane were lower than those in the basin water during both periods. The brominated trihalomethanes present in the secondary effluent with chlorination were not detected in the renovated water samples. This may have been the result of slow transport due to sorption or to chemical or biological transformation. The concentrations of trichloroethylene and pentachloroanisole were significantly higher in the renovated water than in the basin during both sampling periods. Tetrachloroethylene exhibited a similar concentration increase in the prechlorination period, but not in the postchlorination period.

The chlorinated aromatics appeared to be relatively refractory and mobile in the ground because they showed much less concentration decrease than the nonchlorinated aromatic hydrocarbons. Less decrease in the dichlorobenzenes

Table 5. Average TOX concentrations and TOX/TOC ratios for secondary effluent in basins and renovated water from 18 m well

| | Prechlorination | | Postchlorination | |
|---|---|---|---|---|
| | TOX $\mu g\ l^{-1}$ | TOX/TOC mol Cl/mol C | TOX $\mu g\ l^{-1}$ | TOX/TOC ml Cl/mol C |
| Secondary effluent (average for basin) | 84 | 0.0031 | 142 | 0.0050 |
| 18 m Well | 65 | 0.0069 | 55 | 0.0059 |

was observed after chlorination than before. Complete breakthrough appeared to occur for the chlorophenols, but concentrations were near detection limits so that positive conclusions could not be made. A combination of biodegradation and sorption processes might have been responsible for the decreases observed.

Total organic halogen (TOX) concentrations and ratios of TOX to total organic carbon (TOX/TOC) for the secondary sewage effluent and renovated water are shown in Table 5. TOX concentrations of the secondary effluent were significantly higher with chlorination than without. However, the renovated water TOX concentrations were similar for both periods. The ratio of TOX to TOC was higher in the renovated water than in the secondary effluent samples, implying that the halogenated organic compounds comprise the more refractory and mobile portion of the TOC.

## 18.8 Other Organic Micropollutants

In addition to the aliphatics and aromatics mentioned, other compounds tentatively identified in organic extracts of the samples of secondary sewage effluent and renovated water using gas chromatography/mass spectrometry were: fatty acids, resin acids, clofibric acid, alkylphenol carboxylic acids (APEC's), trimethylbenzene sulfonic acid, steroids, n-alkanes, caffeine, Diazinon, alkylphenol polyethoxylates (APE's), and trialkylphosphates. Several of the compounds were detected only in the secondary effluent and not in the renovated water. A few others, Diazinon, clofibric acid, and tributylphosphate, decreased in concentration with soil passage, but were detected in the renovated water. The APE's appeared to undergo rather complex transformations during ground filtration. They appeared to be completely removed with soil percolation during the prechlorination period, but after chlorination two isomers were found following soil passage, while others were removed.

## 18.9 Conclusions

The results of the Phoenix studies and of similar work on other projects (McKim 1978, Idelovitch and Michail 1982) show that the unsaturated zone can serve as an effective treatment system for partially treated wastewater. The renovated water from the Phoenix projects, for example, meets the health, agronomic, and aesthetic requirements for unrestricted irrigation, including vegetable crops that are consumed raw (Bouwer 1982). The water also meets the standards for lakes with primary contact recreation (Matters 1981). Potable use of the renovated water will require additional treatment, for example, activated carbon adsorption, reverse osmosis, and disinfection. Such treatment, however, will be more effective and economical for renovated water from a soil-aquifer treatment system than for effluent from a conventional sewage treatment plant.

In the Phoenix studies, secondary effluent was used because that was what the treatment plants provided. The secondary (biological) treatment step, however, is not necessary because the soil-aquifer treatment system can handle relatively large amounts of organic carbon. Thus, where sewage effluent is to be used for a rapid-infiltration system, primary treatment may suffice (Rice and Gilbert 1978, Lance et al. 1980, Leach et al. 1980, Carlson et al. 1982, Rice and Bouwer 1983). Some additional clarification or lime precipitation of the primary effluent may be desirable, however, to reduce suspended solids and improve the quality of the primary effluent.

Where posttreatment of the renovated water is needed, the soil-aquifer treatment system should be designed as in Fig. 1A because the wells discharge essentially 100% renovated water. In contrast, the wells in the system of Fig. 1B discharge a mixture of renovated wastewater and native groundwater. This provides the benefit of blending, but it also increases the capacity and, hence, the cost of the posttreatment plant. Encroachment of renovated sewage water into the aquifer outside the soil-aquifer treatment system of Fig. 1A can be avoided by monitoring groundwater levels below the outer edges of the infiltration areas and by managing infiltration and pumping rates so that the water levels in the monitoring wells do not rise above the general groundwater level outside the system. By maintaining water levels in the monitoring wells at the same elevation as the water table outside the system, very little if any native groundwater will be drawn into the soil-aquifer treatment system. This is important where there are legal or other restrictions on pumping indigenous groundwater. Finally, the cost of putting partially treated wastewater underground and pumping it from wells as renovated water is low and soil-aquifer treatment systems do not require highly trained operators.

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# 19. Use of Soil-Aquifer System for Effluent Purification and Reuse

E. Idelovitch

## 19.1 Introduction

In areas where groundwater basins have been depleted by overpumping and where there is danger of groundwater salinization by seawater intrusion, sewage effluents are used for groundwater replenishment. In such cases, the recharge of the effluent to a potable aquifer is the ultimate use of the reclaimed water (Fig. 1). A very high degree of treatment (virtually to drinking water quality) is provided prior to recharge. The recharge can be carried out either by spreading basins or by injection wells. The recharged effluent loses its identity and its movement in the aquifer can hardly be traced. Production wells located in the vicinity of the recharge zone pump an admixture of recharged effluent and native groundwater, which is supplied to unrestricted uses, including drinking. The main concerns connected with such a scheme are: the high cost, because of the high level of treatment required prior to recharge, and the possible long-term health hazards associated with the ingestion of effluent, even though of very high quality.

A different approach to groundwater recharge with effluent was adopted in the Dan Region Project – the largest and most advanced wastewater reuse scheme in Israel (Idelovitch et al. 1980). This approach, which is referred to as soil-aquifer treatment (SAT), consists of controlled passage of the effluent through the un-

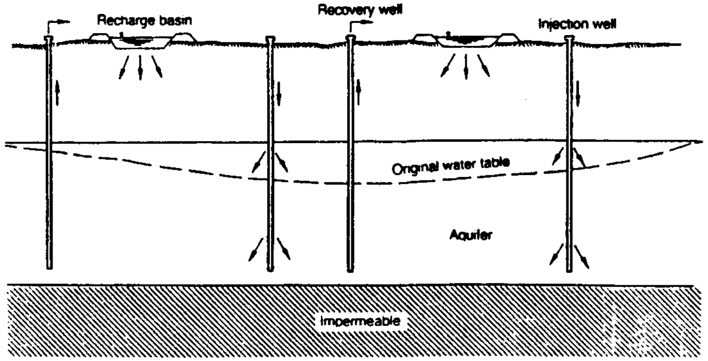

Fig. 1. Groundwater recharge for aquiver replenishment

# RECLAIMING SEWAGE EFFLUENT BY SOIL-AQUIFER TREATMENT[1]

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Sewage effluent is an important water resource, but it should be treated so that it meets the quality requirements for its intended use. Irrigation is a likely user for sewage effluent because it does not require water of top quality, and some of the pollutants in sewage effluent are nutrients for the crop. Any large-scale use of sewage effluent for irrigation typically requires that the effluent be treated for unrestricted irrigation so that farmers can grow what they want. The water can then also be safely used for park, golf course, and landscape irrigation.

## IN-PLANT TREATMENT

Unrestricted irrigation typically requires sewage treatment to remove all pathogenic organisms (for public health reasons, Matters 1981), to remove some of the nitrogen (for agronomic reasons; Baier and Fryer 1973; Ayers and Westcot 1985), and to remove as much of the suspended solids and biodegradable organic material as possible (for aesthetic reasons). The need for nitrogen removal is dependent on the nitrogen and irrigation water requirements of the crop and on the nitrogen concentration of the effluent. In mild climates where irrigation requirements are low and/or where forage or other crops that benefit from high nitrogen applications are grown, nitrogen removal may be less critical or may not even be necessary at all. Where there is considerable industrial input in the effluent, source control may be required to keep minor elements at permissible concentrations (Nat. Acad. of Sci. and Nat. Acad. of Eng. 1972).

> **Sewage effluent is an important water resource, but it should be treated so that it meets the quality requirements for its intended use.**

In-plant treatment to meet these requirements typically consists of primary and secondary treatment, followed by coagulation, sedimentation, filtration, and chlorination or other disinfection. This treatment, however, removes very little nitrogen. Additional nitrogen removal is very expensive. Thus, when nitrogen concentrations in the sewage effluent are too high, the best solution for the in-plant treatment option would be blending the effluent with other water.

## SOIL-AQUIFER TREATMENT

Another option would be to treat the sewage by means of a ground water recharge system. This requires geohydrological conditions favorable for ground water recharge with infiltration basins (permeable soils, unconfined aquifer with sufficient transmissivity, no seriously restricting layers in the vadose zone). The key to successful reclamation of sewage effluent by ground water recharge is control of the effluent water in the aquifer so that it can be taken out of the aquifer again instead of moving further and further and eventually contaminating wells used for drinking water. Thus, the systems really are not ground water recharge systems as such; rather, they are soil-aquifer treatment (SAT) systems.

> **The key to successful reclamation of sewage effluent by ground water recharge is control of the effluent water in the aquifer...**

With SAT systems, a portion of the underground environment is dedicated as a natural filter for advanced treatment of sewage effluent. This can be achieved with several systems, as shown in Figure 1. The system in Figure 1A typically is used to prevent direct discharge of sewage effluent or processing plant wastes into surface water. The system in Figure 1B is used where ground water tables are high. The system in Figure 1C is used where ground water is deep. The wells in this system pump essentially 100 percent renovated sewage water. Spread of sewage water outside the portion of the aquifer dedicated to sewage treatment is prevented by managing infiltration and pumping rates so that ground water levels below the periphery of the system (see observation wells in Figure 1C) will never rise higher than the ground water levels in the aquifer outside the SAT system. The SAT system of Figure 1C would also be suitable for small systems where there are only a few basins around a centrally located well (Figure 2). With systems as in Figure 1D, infiltration basins are grouped together, and the wells tend to pump a mixture of renovated sewage water and native ground water. Systems 1C and 1D can also be operated to provide seasonal storage in the aquifer, allowing the ground water mound to rise in the winter or wet season and pumping the mound down in the summer or dry season. In that case, additional wells may have to be installed to collect all the sewage water from the aquifer, especially where there are natural ground water gradients.

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[1] Contribution from the Agricultural Research Service, U. S. Department of Agriculture.

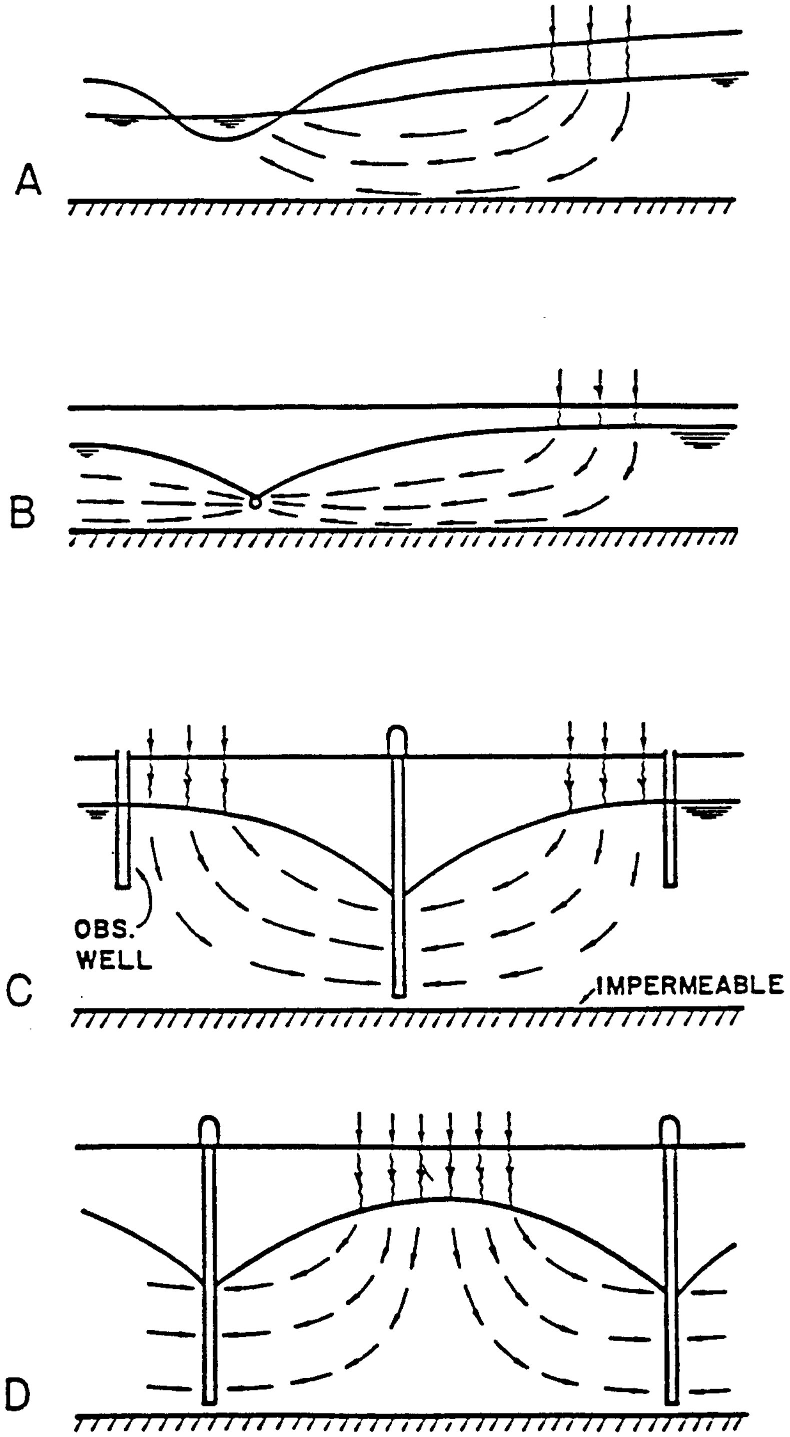

Figure 1. Schematic of soil-aquifer treatment systems with natural drainage of renovated water into stream, lake, or low area (A), collection of renovated water by subsurface drain (B), infiltration areas in two parallel rows and line of wells midway between (C), and infiltration areas in center surrounded by a circle of wells (D).

SAT systems typically remove all BOD, suspended solids, and pathogenic organisms from the water. Phosphate concentrations are greatly reduced, and nitrogen concentrations are significantly reduced if the infiltration system is managed to stimulate denitrification in the soil. Concentrations of heavy metals and other minor elements also can be appreciably reduced. Total dissolved salts (TDS) concentrations are slightly increased (a few percent, for example) because of evaporation from the infiltration basins. Thus, renovated sewage effluent from an SAT system tends to meet the public health, agronomic, and aesthetic requirements for unrestricted irrigation. The water would also meet the requirements for primary contact recreation. The BOD of the renovated water from an SAT system is essentially zero. However, there may still be a few mg/l organic carbon left. While this carbon may mostly be in the form of humic and fulvic acids, it also consists of a wide spectrum of aliphatic and aromatic, halogenated and nonhalogenated hydrocarbons, many at concentrations on the ppb level. These trace organic compounds include known or suspected carcinogens or other toxic materials. Because of their presence, renovated sewage water from SAT systems cannot be used for drinking without further treatment. This treatment would consist of activated carbon infiltration and possibly reverse osmosis. The water would also have to be disinfected. The residual organic compounds in the renovated water are also the main reason that this water should not be allowed to move beyond the aquifer portion dedicated to SAT, especially if the water can move toward drinking water wells.

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**With SAT systems, a portion of the underground environment is dedicated as a natural filter for advanced treatment of sewage effluent.**

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Infiltration and hydraulic loading rates for SAT systems depend on soil, effluent quality (suspended solids content), and climate. Desirable soils are those in the sandy loam to loamy sand range (fine enough to give good treatment, coarse enough to give good infiltration rates). For good secondary effluent, suitable soils, and reasonably dry and warm climates, hydraulic loading rates typically are on the order of 50 to 150 m per year, using flooding and drying periods of about 10 to 20 days each. This schedule can also stimulate denitrification in the soil.

## FIELD STUDIES

Hydraulic loading rates and water quality improvement obtained in SAT systems are site specific, and on-site experimentation normally is desirable to see how large projects should be designed and managed for best results. To illustrate what can be achieved with SAT systems, the results from two projects in Phoenix, Arizona, USA, will be summarized.

The city of Phoenix is interested in renovating part of its sewage effluent by SAT so that it can be used for unrestricted irrigation. The renovated water will then be exchanged with a nearby irrigation district for high quality ground water from that district. This high quality ground water will then be used to augment the city's municipal water supply via an exchange with another irrigation district for high quality surface water (Bouwer and Chase 1984). There are two major treatment plants in the Phoenix area: the 91st Avenue treatment plant (activated sludge, chlorination, capacity about 450,000 $m^3$/day) and the 23rd Avenue treatment plant (activated sludge, chlorination, capacity about 150,000 $m^3$/day). The SAT system for the Phoenix area would consist of a series of infiltration basins arranged in two parallel strips with wells on the line midway between the strips (Figure 1C).

The feasibility of SAT in the Phoenix area was studied with two experimental systems, a small test project installed in 1967 (Figure 3), and a larger, demonstration project installed in 1975 (Figure 4). The latter will be part of a future operational project (Figure 4) that will have a basin area of 48 ha and a projected capacity of about 50 million $m^3$/year. The results of these two test projects will be summarized in the remainder of this article.

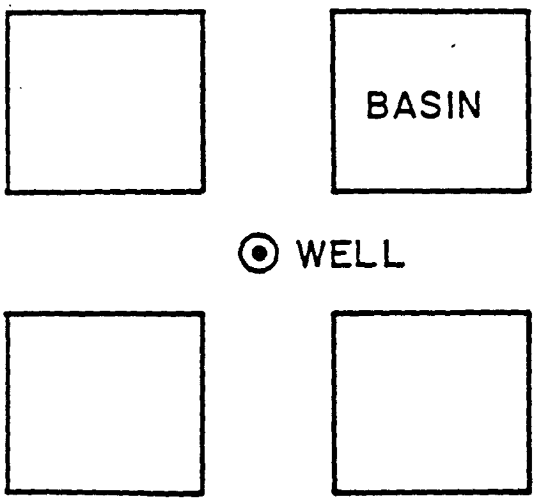

Figure 2. Schematic of four small infiltration basins with well in center for pumping renovated sewage water from aquifer.

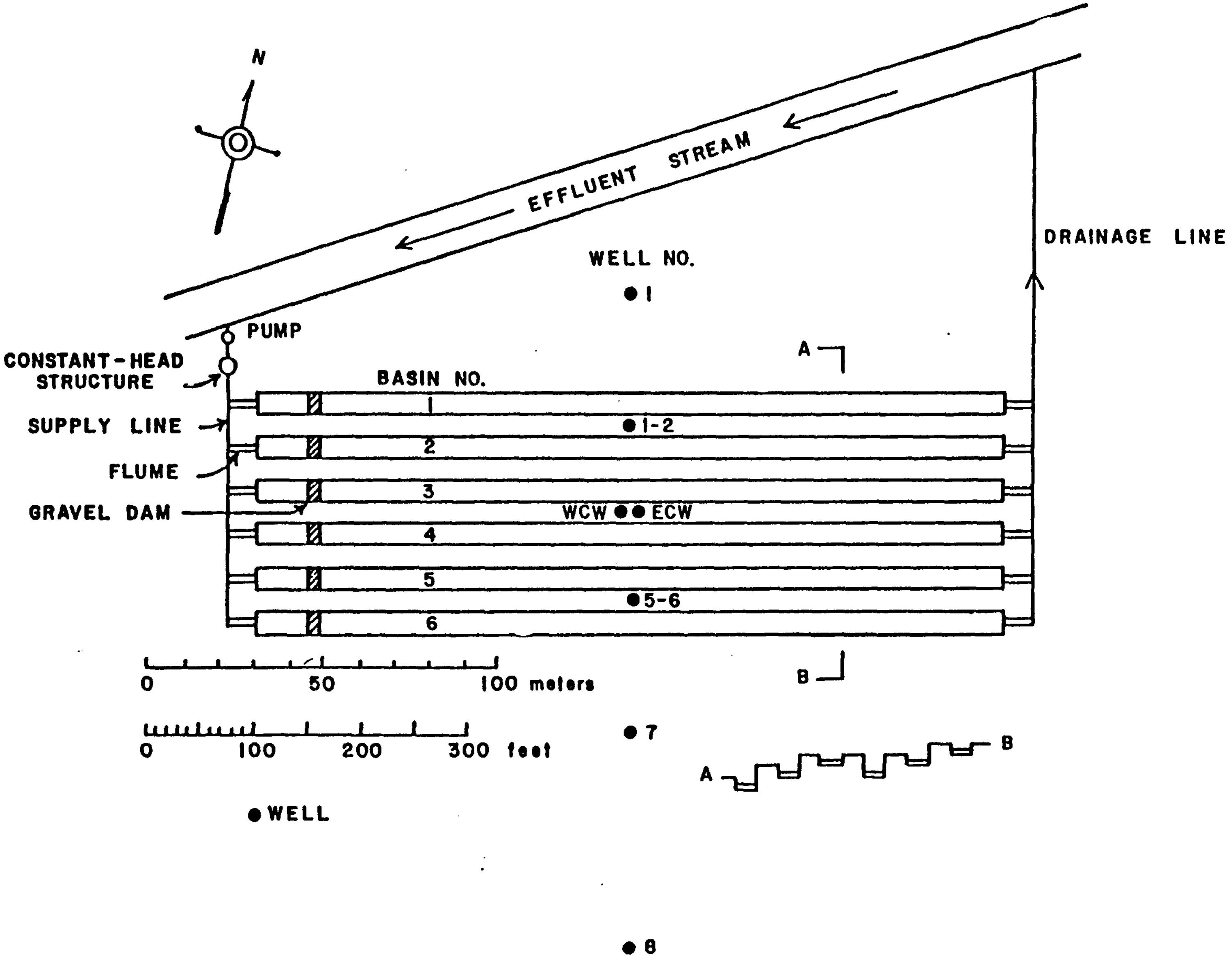

Figure 3. Schematic of Flushing Meadows Project showing infiltration basins and monitoring wells.

## PROJECT DESCRIPTIONS AND HYDRAULIC LOADING RATES

The first project was the Flushing Meadows project (Bouwer et al. 1974a and b, and 1980). This was an experimental project installed in the Salt River bed in 1967. It consisted of six parallel, long, narrow infiltration basins of about 0.13 ha each (Figure 3). The soil consisted of about 1 m of loamy sand underlain by sand and gravel layers. The ground water table was at a depth of around 3 m. Monitoring wells 6 to 9 m deep were installed at various points between the basins and away from the basins (Figure 3). This made it possible to sample renovated wastewater from the aquifer below the basins and after the water had moved laterally for some distance through the aquifer.

The second project was the 23rd Avenue project (Bouwer and Rice 1984). This is a demonstration and future operational project installed in 1975 on the north side of the Salt River bed. It consists of a 16-ha lagoon split lengthwise into four infiltration basins of 4 ha each (Figure 4). The soil lacks the loamy sand top layer of the Flushing Meadows project. Thus, the soil profile consists mostly of sand and gravel layers. However, the ground water table was much deeper and ranged between 5 and 25 m depth (mostly around 15 m) for the study period. Monitoring wells for sampling renovated wastewater were installed in the center of the project at depths of 18, 24, and 30 m, and on the north and south sides of the basin complex at depths of 22 m (Figure 4). In addition, a large production well (capacity about 10,000 $m^3$/day) was drilled in the center of the project with the casing perforated from 30 to 54 m depth.

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**At a hydraulic loading rate of 100 m per year, 1 ha of infiltration basin can handle 100 x 10,000 = $10^6$ $m^3$ of wastewater per year.**

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Flooding and drying schedules were mostly 9 days flooding-12 days drying at the Flushing Meadows project and 14 days flooding-14 days drying at the 23rd Avenue project. Water depths in the basins were about 15 to 20 cm. During flooding, infiltration rates typically were between 0.3 and 0.6 m/day, yielding a total infiltration or hydraulic loading of between 60 and 120 m/year for Flushing Meadows and about 100 m/year at the 23rd Avenue project.

For the 23rd Avenue project, the effluent from the treatment plant initially flowed through a 32-ha lagoon (Figure 4) before it entered the infiltration basins. This gave problems of soil clogging in the infiltration basins due to heavy growth of algae in the lagoon, especially in the summer. The unicellular algae Carteria klebsii were particularly troublesome. In addition to forming a "filter cake" on the bottom of the infiltration basins, the algae removed $CO_2$ from the wastewater for photosynthesis which raised the pH and, in turn, caused precipitation of $CaCO_3$, which further aggravated the soil clogging. Algae growth and resulting soil clogging were prevented by constructing a bypass canal around the 32-ha lagoon (Figure 4), reducing the detention time of the effluent from a few days in the lagoon to about one-half hour in the canal. After the bypass canal was put into operation, hydraulic loading rates for the infiltration basins were almost 100 m/year. This was about five times as much as the hydraulic rate of 21 m/year obtained before the bypass canal was put into operation.

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**For both projects, most of the quality improvement of the wastewater occurred in the vadose zone (i.e., the zone between soil surface and the ground water table).**

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At a hydraulic loading rate of 100 m per year, 1 ha of infiltration basin can handle 100 x 10,000 = $10^6$ $m^3$ of wastewater per year. Thus, the 150,000 $m^3$/day of effluent from the 23rd Avenue wastewater treatment plant would require 55 ha of infiltration basins. Almost all of this area can be obtained by also converting the 32-ha lagoon east of the present infiltration system into infiltration basins (Figure 4). This would give a total basin area of about 48 ha that can handle about 48 million $m^3$/year of effluent. The wells for pumping the renovated water from the aquifer can be located on the centerline through the project (Figure 4). At a capacity of 10,000 $m^3$/day per well, twelve wells would be needed to pump renovated water out of the aquifer at the same rate as it infiltrates as wastewater in the basins, thus creating an equilibrium situation (Bouwer and Chase 1984).

For both projects, most of the quality improvement of the wastewater occurred in the vadose zone (i.e., the zone between soil surface and the ground water table). The quality improvements for these projects will be summarized in the following paragraphs. For additional details, reference is made to H. Bouwer et al. (1974b, 1980, and 1984) and to E. J. Bouwer et al. (1984).

### Suspended solids

The suspended solids content of the renovated water at the Flushing Meadows project was less than 1 mg/l. For the 23rd Avenue project it averaged about 1 mg/l for the large production well. Most of these solids probably were fine aquifer particles that entered the well through the perforations in the casing. The suspended solids content of the secondary effluent at the 23rd Avenue project averaged about 11 mg/l.

### Total dissolved solids

The total salt content of the water increased slightly as it moved through the SAT system (from 750 to 790 mg/l at the 23rd Avenue project). Evaporation from the basins (including from the soil during drying) should increase the TDS content by about 2 percent. The rest of the increase was probably due to mobilization of calcium carbonate due to a pH drop from 8 to 7 as the effluent moved through the vadose zone.

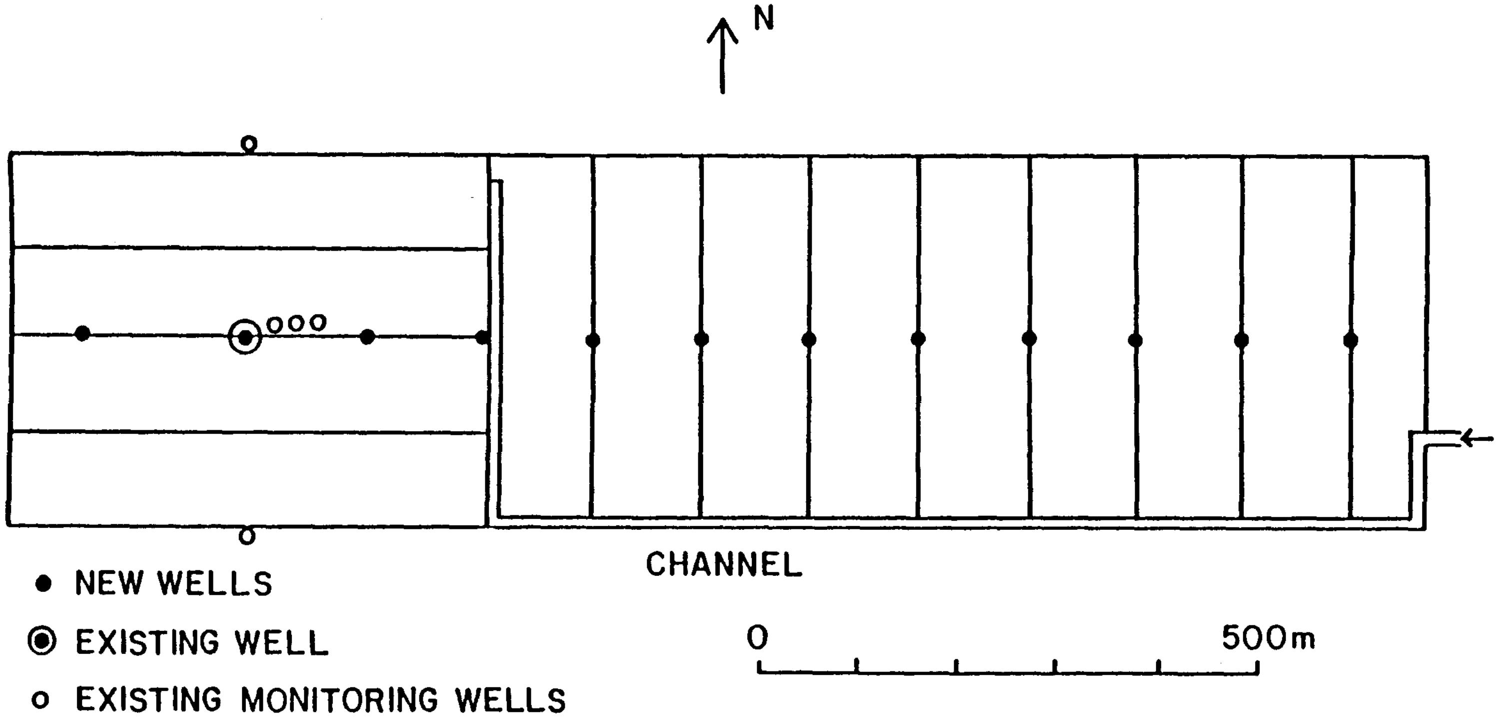

Figure 4. Sketch of 23rd Avenue project with 16-ha lagoon (left) that was split into four infiltration basins for demonstration project, and 32-ha lagoon (right) that can be split into nine infiltration basins to increase capacity of system to that of plant outflow. The twelve wells for pumping renovated water are shown as dots. Also shown is the bypass-supply channel.

### Nitrogen

At the Flushing Meadows project, nitrogen removal from the effluent water as it seeped through the vadose zone to become renovated water was about 30 percent at maximum hydraulic loading (100-120 m/year), but 65 percent if the loading rate was reduced to about 70 m/year by using 90-day flooding and 12-day drying cycles and by reducing the water depths in the basins from 0.3 to 0.15 m. The form and concentration of nitrogen in the renovated water sampled from the aquifer below the basins were slow to respond to the reduction in hydraulic loading (Bouwer et al. 1980). In the tenth year of operation (1977), the renovated water contained 2.8 mg/l of ammonium nitrogen, 6.25 mg/l nitrate nitrogen, and 0.58 mg/l organic nitrogen, for a total nitrogen content of 9.6 mg/l. This was 65 percent less than the total nitrogen of the secondary sewage effluent, which averaged 27.4 mg/l (mostly as ammonium) in that year.

> **...low residual chlorine content of the effluent by the time it infiltrated into the ground apparently had no effect on the nitrogen transformations in the soil.**

At the 23rd Avenue project, the total N content in the secondary sewage effluent averaged about 18 mg/l, of which 16 mg/l was as ammonium. The 2-week flooding and drying cycles must have been conducive to denitrification in the vadose zone because the total N content of the renovated water from the large center well averaged about 5.6 mg/l of which 5.3 mg/l was as nitrate, 0.1 mg/l as ammonium, 0.1 mg/l as organic, and 0.02 mg/l as nitrite. The nitrogen removal thus was about 70 percent. This removal was the same before and after the secondary effluent was chlorinated in the treatment plant, indicating that the low residual chlorine content of the effluent by the time it infiltrated into the ground apparently had no effect on the nitrogen transformations in the soil.

The flooding and drying sequence that maximizes denitrification in the vadose zone depends on various factors and must be evaluated for each particular system. Pertinent factors include the ammonium and carbon contents of the effluent entering the soil, infiltration rates, cation-exchange capacity of soil, exchangeable ammonium percentage, depth of oxygen penetration in the soil during drying, and temperature. The combined laboratory and field data from the Flushing Meadows experiments showed that to achieve high nitrogen removal percentages, the amount of ammonium nitrogen applied during flooding must be balanced against the amount of oxygen entering the soil during drying. Flooding periods must be long enough to develop anaerobic conditions in the soil. Infiltration rates must be controlled to the appropriate level for the particular effluent, soil, and climate at a given site. Most of the nitrogen transformations in the Flushing Meadows studies occurred in the upper 50 cm of the vadose zone.

### Phosphate

Phosphate removal increased with increasing distance of underground movement of the sewage water. After 3 m of downward movement through the vadose zone and 6 m through the aquifer, phosphate removal at the Flushing Meadows project was about 40 percent at high hydraulic loading and 80 percent at reduced hydraulic loading. Additional lateral movement of 60 m through the aquifer increased the removal to 95 percent (i.e., to a concentration of 0.51 mg/l phosphate phosphorus vs. 7.9 mg/l in the effluent). After ten years of operation and a total infiltration of 754 m of secondary effluent, there were no signs of a decrease in phosphate removal.

> **After ten years of operation ...there were no signs of a decrease in phosphate removal.**

At the 23rd Avenue project, phosphate phosphorus concentrations in the last few years of the research averaged 5.5 mg/l for the secondary effluent going into the ground and 0.37 mg/l for the renovated water pumped from the center well. The shallower wells showed a higher phosphate content, indicating that precipitation of phosphate again continued in the aquifer.

For example, renovated water sampled from the 22-m-deep north well showed phosphate phosphorus concentrations that averaged 1.5 mg/l. Most of the phosphate removal probably was due to precipitation of calcium phosphate.

### Fluoride

Fluoride removal paralleled phosphate removal, indicating precipitation as calcium fluoride. At the Flushing Meadows project, fluoride concentrations in 1977 were 2.08 mg/l for the effluent, 1.66 mg/l for the water after it had moved 3 m through the vadose zone and 3 to 6 m through the aquifer, and 0.95 mg/l after it had moved an additional 30 m through the aquifer. At the 23rd Avenue project, fluoride concentrations averaged 1.22 mg/l in the secondary effluent and 0.7 mg/l in the renovated water from the center well.

### Boron

Boron was not removed in the vadose zone and the aquifer of the Flushing Meadows and 23rd Avenue projects, and was present at concentrations of 0.5 to 0.7 mg/l in both effluent and renovated water. The lack of boron removal was due to absence of significant amounts of clay in the vadose zone and aquifer.

### Metals

At the Flushing Meadows project, movement of the secondary effluent through 3 m of vadose zone and 6 m of aquifer reduced zinc from 193 to 35 g/l, copper from 123 to 16 g/l, cadmium from 7.7 to 7.2 g/l, and lead from 82 to 66 g/l (Bouwer et al 1974b). Cadmium thus appeared to be the most mobile metal.

### Fecal coliforms

The secondary effluent at the Flushing Meadows project was not chlorinated and contained $10^5$ to $10^6$ fecal coliforms per 100 ml. Most of these were removed in the top meter of the vadose zone. Some penetrated to the aquifer, however, especially when a new flooding period was started. The deeper penetration of fecal coliforms at the beginning of a flooding period was attributed to less straining of bacteria at the soil surface because the clogged layer had not yet developed. Also, the activity of native soil bacteria at the end of a drying period was lower, producing a less antagonistic environment for the fecal coliforms in the soil when flooding was resumed. Fecal coliform concentrations in the water after 3 m of travel through the vadose zone and 6 m through the aquifer were 10 to 500 per 100 ml when the renovated water consisted of water that had infiltrated at the beginning of a flooding period, and between 0 and 1 per 100 ml after continued flooding. Additional lateral movement of about 100 m through the aquifer was necessary to produce renovated water that was completely free from fecal coliforms at all times.

**...chlorination...effluent...and additional movement of the water through the aquifer...produced renovated water that was essentially free from fecal coliforms.**

At the 23rd Avenue project, fecal coliform concentrations in the secondary sewage effluent entering the infiltration basins were 10,000/100 ml prior to November 1980 when the effluent was not yet chlorinated in the treatment plant and was first passed through a 32-ha lagoon. This concentration increased to 1.8 x $10^6$/100 ml when the unchlorinated effluent was bypassed around the lagoon and flowed directly into the infiltration basins. It then decreased to 3500/100 ml after the effluent was chlorinated and still bypassed around the lagoon. The corresponding fecal coliform concentrations for these periods in the water pumped from the large center well from a depth of 30 to 54 m averaged 2.3, 22, and 0.27/100 ml, respectively. The corresponding ranges were 0 to 40, 0 to 160, and 0 to 3 per 100 ml, respectively. Considerable fecal coliform concentrations were observed in the renovated water from the shallower wells, especially when the fecal coliform concentration of the infiltrating effluent was 1.8 x $10^6$/100 ml. At that time, water from the 18-m-deep well showed coliform peaks after a new flooding period was started that regularly exceeded 1000/100 ml and at one time even reached 17,000/100 ml. Thus, a considerable number of fecal coliforms passed through the vadose zone. However, chlorination of the effluent and resulting reduction of the fecal coliform concentration to 3500/100 ml prior to infiltration, and additional movement of the water through the aquifer to the center well produced renovated water that was essentially free from fecal coliforms.

### Viruses

At the Flushing Meadows project, the virus concentrations of nonchlorinated secondary effluent averaged 2118 plaque forming units (PFU)/100 liter (average of six bimonthly samples taken for one year). They included polio, echo, coxsackie, and reoviruses. No viruses could be detected in renovated water sampled after 3 m of movement through the vadose zone and 3 to 6 m movement through the aquifer. At the 23rd Avenue project, virus concentrations in the renovated water from the center well averaged 1.3 PFU/100 liter before chlorination of the secondary effluent, and 0 PFU/100 liter after chlorination of the secondary effluent. The combined effects of chlorination and SAT thus apparently resulted in complete removal of the viruses.

**The combined effects of chlorination and SAT thus apparently resulted in complete removal of the viruses.**

### Organic carbon

At the Flushing Meadows project, the biochemical oxygen demand (BOD) of the effluent water after moving 3 m through the vadose zone and 6 m through the aquifer was essentially zero, indicating that almost all biodegradable carbon was mineralized. However, the renovated water still contained about 5 mg/l total organic carbon (TOC), as compared to 10 to 20 mg/l of TOC in the secondary effluent.

At the 23rd Avenue project, the TOC concentration of the secondary effluent averaged 12 mg/l where it entered the infiltration basins, and 14 mg/l at the opposite ends of the basins. This increase probably was due to biological activity in the water as it moved through the basins. The renovated water from the 18-m well (intake about 5 m below the bottom of the vadose zone) had a TOC content of 3.2 mg/l and that from the center well (which pumped from 30 to 54 m depth) had a TOC content of 1.9 mg/l, indicating further removal of organic carbon as the water moved through the aquifer. The TOC removal in the SAT system was the same before and after chlorination of the secondary effluent, indicating that chlorination had no effect on the microbiological processes in the soil.

The concentration of organic carbon in the renovated water of 1.9 mg/l was higher than the 0.2 to 0.7 mg/l typically found in unpolluted ground water. The latter are mostly due to humic substances like fulvic and humic acids (Thurman 1979). The renovated sewage water from the SAT process thus could contain a number of synthetic organic compounds, some of which could be carcinogenic or otherwise toxic.

### Removal of trace organic compounds in the vadose zone

The nature and concentration of trace organics in the secondary sewage effluent and in the renovated water from the various wells of the 23rd Avenue project were determined by Stanford University's Environmental Engineering and Science Section, using gas chromatography and mass spectrometry. The studies were carried out for two months with nonchlorinated effluent, and then for three months with chlorinated effluent, taking weekly or biweekly samples. As could be expected, the results showed a wide variety of organic compounds, including priority pollutants (many in concentrations on the order of g/l, see E. J. Bouwer et al. 1984; and H. Bouwer and Rice 1984).

**Soil percolation removed 50 to 99 percent of the nonhalogenated organic compounds, probably mostly by microbial decomposition.**

The chlorination had only a minor effect on the type and concentration of organic compounds in the sewage effluent. Of the volatile organic compounds, 30 to 70 percent were lost by volatilization from the infiltration basins. Soil percolation removed 50 to 99 percent of the nonhalogenated organic compounds, probably mostly by microbial decomposition. Concentrations of halogenated organic compounds decreased to a lesser extent with passage through the soil and aquifer. Thus, halogenated organic compounds (including the aliphatic compounds chloroform, carbon tetrachloride, tri-chloroethylene, and 1,1,1-tri-chloroethane, and the aromatic di- and tri-chlorobenzenes and chlorophenols) were more mobile and refractory in the underground environment than the nonhalogenated compounds, which included the aliphatic nonanes, hexanes, and octanes, and the aromatic xylenes, $C_3$-benzenes, styrene, phenanthrene, and diethylphthalate.

Other organic micropollutants

In addition to the aliphatic and aromatic compounds mentioned, other compounds tentatively identified in organic extracts of the samples of secondary sewage effluent and renovated water using gas chromatography-mass spectrometry were fatty acids, resin acids, clofibric acid, alkylphenol polyethoxylate carboxylic acids (APECs), trimethylbenzene sulfonic acid, steroids, n-alkanes, caffeine, Diazinon, alkylphenol polyethoxylates (APEs) and trialkylphosphates. Several of the compounds were detected only in the secondary effluent and not in the renovated water. A few others, Diazinon, clofibric acid, and tributylphosphate, decreased in concentration with soil passage, but were still detected in the renovated water. The APEs appeared to undergo rather complex transformations during ground filtration. They appeared to be completely removed with soil percolation during the pre-chlorination period, but after chlorination two isomers were found following soil passage while others were removed.

The results of these studies showed that soil-aquifer treatment is effective in reducing concentrations of a number of synthetic organic compounds in the sewage water, but that the renovated water still contains a wide spectrum of organic compounds, albeit at very low concentrations. Thus, while the renovated water is suitable as such for unrestricted irrigation and recreation, recycling it for drinking would require additional treatment such as carbon filtration to remove the remaining organic compounds. The water would also have to be disinfected, and reverse osmosis may be desirable. Such treatment of renovated sewage water from an SAT system would, however, be much more effective and cheaper than for sewage effluent after conventional primary and secondary treatment (Semmens and Field 1980). Thus, SAT could also play an important role in recycling sewage effluent into potable water.

**The results of the Phoenix studies show that the renovated water from the SAT projects meets the public health, agronomic, and aesthetic requirements for unrestricted irrigation...**

## CONCLUSIONS

The results of the Phoenix studies show that the renovated water from the SAT projects meets the public health, agronomic, and aesthetic requirements for unrestricted irrigation, including vegetable crops that are consumed raw (Bouwer 1982). The water also meeds the standards for lakes with primary contact recreation (Matters, 1981). Potable use of the renovated water will require additional treatment, for example, activated carbon adsorption, reverse osmosis, and disinfection. Such treatment, however, will be more effective and economical for renovated water from a soil-aquifer treatment system than for effluent from a conventional sewage treatment plant (Semmens and Field 1980).

In the Phoenix studies, secondary effluent was used because that was what the treatment plants provided. In general, however, the secondary (biological) treatment step is not necessary because the SAT system can handle relatively large amounts of organic carbon. Thus, where sewage effluent is to be used for a rapid-infiltration system, primary treatment may suffice (Carlson et al. 1982, Lance et al. 1980, Leach et al. 1980, Rice and Gilbert 1978, Rice and Bouwer 1984). Some additional clarification or lime precipitation of the primary effluent may be desirable, however, to reduce suspended solids and to improve the quality of the primary effluent.

The cost of putting partially treated wastewater underground and pumping it from wells as renovated water is low, and SAT systems do not require highly trained operators. Thus, where land availability and hydrogeological conditions are favorable for ground water recharge by surface spreading of wastewater, soil-aquifer treatment can play an important role in the safe reuse and recycling of municipal wastewater.

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# CHAPTER 8

# Renovation of Wastewater with Rapid-Infiltration Land Treatment Systems

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With rapid-infiltration (RI) systems, sewage effluent or other wastewater is applied to relatively permeable soil at rates that are much higher than the evapotranspiration rates. Thus, most of the wastewater moves down to the groundwater and is renovated by filtration through the vadose zone and subsequent movement through the aquifer. This treatment aspect is becoming more and more important as the need for water reuse increases and protection of native groundwater becomes vital. For this reason, rapid-infiltration systems, increasingly called *soil-aquifer treatment systems*, can be a significant part of the complete treatment process of wastewater for reuse.

In most systems, the wastewater is applied to the soil with infiltration basins. On sloping land, contour terraces or furrows can be used. Where topographies are very irregular, sprinkler systems can be used, but these require additional energy for pumping and also create aerosol problems. Application or "hydraulic loading" rates of rapid-infiltration systems typically are of the order of 20 to 150 m/per year. Since evapotranspiration rates are of the order of 0.5 to 2.5 m/per year, about 90 to almost 100 percent of the applied wastewater will move down to the groundwater.

While some early rapid-infiltration systems undoubtedly were installed to get rid of the wastewater and to put it out of sight, more recent systems are installed primarily to reduce pollution of surface water (streams, lakes, oceans) or to obtain the benefits of soil-aquifer treatment in the direct reuse of the wastewater. To reduce pollution of the surface water that is to receive wastewater, the rapid-infiltration system is located on high ground that drains naturally into the surface water (Figure 8.1). While direct reuse of the waste-

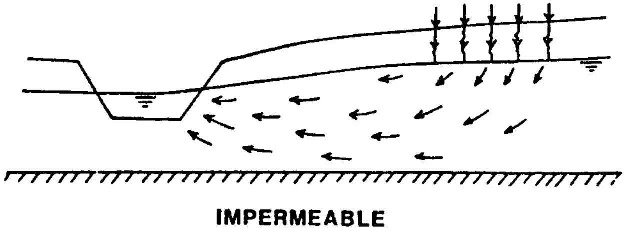

**Figure 8.1** RI system with renovated water draining into stream.

water normally is not the main objective of these systems, they do permit indirect reuse of the water. Examples of these systems, which are basically the reverse of bank-filtration systems, are the systems at Bielefeld, Germany; Lake George, New York; Fort Devens, Massachusetts; and Vineland, New Jersey. With these systems the portion of the aquifer between the rapid-infiltration basins and the surface water is dedicated to wastewater renovation. Wastewater enters the stream as renovated water, which pollutes the surface water much less than does direct discharge of the wastewater.

Where there is no surface water that the renovated water can drain into and where soil-aquifer treatment and direct reuse of the water are the main objectives, the renovated water can be collected by wells if the groundwater is deep, or by open or closed drains if it is shallow (Figure 8.2). Again, the portion of the aquifer between the rapid-infiltration basins and the collection facility for the renovated water serves as a wastewater treatment system. Examples of these systems include the Dan Region, Israel, project (see Chapter 15); the Boulder, Colorado, project; the Phoenix, Arizona, project; the Santee, California, project; and the Brookings, South Dakota, project. There are also various infiltration projects in western Europe (dune systems in the Netherlands, Dortmund system in Germany) where polluted surface water is first passed through a soil-aquifer treatment system with rapid-infiltration basins before it goes to the drinking water treatment plant (see Chapters 16 and 17).

Rapid-infiltration soil-aquifer treatment systems are capable of removing essentially all biodegradable organics, suspended solids, and bacteria and viruses from the wastewater. They can also remove almost all the phosphorus and significantly reduce concentrations of nitrogen and heavy metals. Properly designed and managed systems produce a clear, essentially odorless renovated water that can be used for unrestricted irrigation, primary contact recreation, and other purposes. Potable use would require additional treatment, including activated carbon filtration to remove refractory or trace organics, disinfection, and possibly also reverse osmosis or other desalination. The inclusion of a natural, underground, and invisible step

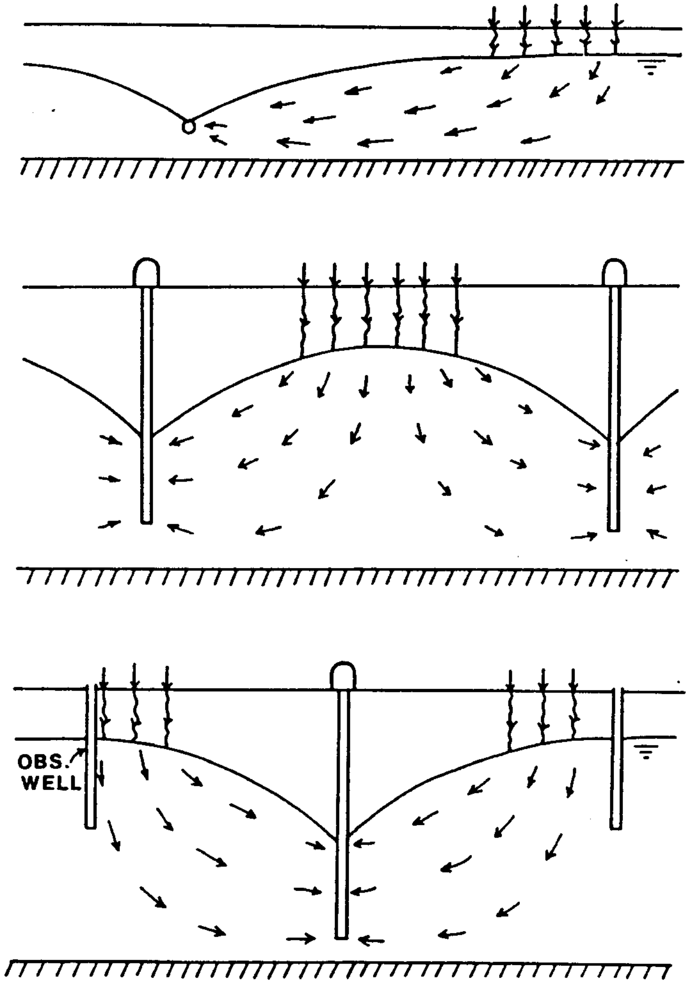

**Figure 8.2** Schematic of RI systems with recovery of renovated water by drains *(top)*, wells surrounding the basins *(center)*, and wells midway between two parallel strips of basins *(bottom)*.

such as soil-aquifer treatment in the recycling of municipal wastewater for potable use undoubtedly would enhance the aesthetics and public acceptance of complete wastewater reuse. Other advantages of rapid-infiltration

(RI) systems include simplicity of operation of the infiltration basins, low cost of putting wastewater underground and of getting the renovation treatment in the natural soil-aquifer system, reliability of the system, and low land requirements. For example, at a hydraulic loading rate of 70 m/per year, a city of 100,000 persons with a sewage flow of 300 L/person/day would require 16 hectares (ha) of infiltration basins.

## SITE SELECTION, DESIGN, AND MANAGEMENT OF RI SYSTEMS

### Soils and Hydraulic Loading

RI systems require soils that are permeable enough to give high hydraulic loading rates, but fine enough to give adequate treatment to the wastewater as it moves through the soil. Where nitrogen removal is important, the soil should also have some cation exchange capacity to adsorb ammonium during flooding that can be nitrified and denitrified during drying. This can normally be accomplished with a few percent clay in the soil. The best soils for RI systems are, therefore, in the texture range of sandy loam, loamy sand, and fine sand. Such soils must have a depth of at least 1 m before coarser material is encountered. To minimize movement of suspended material into the soil and to avoid clogging of deeper soil layers, profiles where the fine soil is on top and the coarser material is further down are better than the reverse. Profiles with fractured rock or very coarse sands or gravels at relatively shallow depth should be avoided, because such "macropores" can give the wastewater a shortcut to the aquifer before it is fully renovated. Profiles with restricting layers that form unacceptably high perched groundwater mounds and seriously reduce infiltration rates should also be avoided.

Hydraulic loading rates are best evaluated by on-site experimentation, using test ponds with a minimum size of 2 x 2 m, the same wastewater that will be used in the actual system, and different combinations of wetting and drying periods to cover the range that may be expected for the actual system. Preliminary estimates of hydraulic loading rates can be made from final infiltration rates for pure water and/or hydraulic conductivity measurements of the soil profile, using a conversion factor to allow for the lower quality of the wastewater and the periodic drying of the basins. Where the soil consists of various layers, the final infiltration rate can best be estimated as the harmonic mean of the hydraulic conductivities of the various layers determined *in situ* [1].

At the Flushing Meadows Project west of Phoenix, Arizona, the hydraulic conductivity of the surface soil was measured with the double-tube and air-entry permeameter methods as 1.2 m/per day or 438 m/per year. The maximum hydraulic loading rate for secondary sewage effluent was achieved at a schedule of 2 to 3 weeks flooding and 2 weeks drying and was about 100 m/per year, or 100/438 = 0.23 of the hydraulic conductivity of the soil beneath the basins. During flooding, infiltration rates were about 0.6 m/per day, or one half the hydraulic conductivity; the basins were dry about half the time, thus yielding the conversion factor of about one fourth. In colder or wetter climates, or where the sewage effluent contains more BOD or suspended solids (especially algae), lower conversion factors should be used (0.05 to 0.15, for example). Infiltration and hydraulic conductivity measurements should be determined *in situ* and with established techniques [1]. Where earthmoving or other grading is needed for construction of the RI system, the infiltration and hydraulic conductivity measurements should be taken on the soil as it will be after the construction, and not on its natural condition prior to construction (this can make a big difference!).

### Basin Management and Layout

Regular drying of RI basins is necessary to allow suspended solids and other material that have accumulated on the bottom of the basins to dry and partially decompose, restoring infiltration rates that tend to decrease during flooding. Drying is also necessary to allow atmospheric air to enter the soil for decomposition of organic material and nitrification of ammonium. Flooding and drying schedules commonly used in existing systems range from flooding periods of 1 day or less and drying periods of several days to flooding and drying periods of 2 weeks each. Different schedules give different hydraulic loading rates and different quality improvement of the wastewater as it moves through the soil, particularly with respect to nitrogen transformation and removal. The optimum sequence of flooding and drying periods for maximum hydraulic loading and quality of renovated water is best evaluated by on-site experimentation.

The water depth in the basin should be kept relatively shallow, for example, less than 0.3 m. Small depths promote fast turnover of the wastewater in the basins during flooding and minimize growth of suspended algae (*Carteria klebsii*, for example) that can clog the bottom soil directly by forming a filter cake on the surface and indirectly by causing precipitation of calcium carbonate due to pH increases as the algae remove carbon dioxide from the water during photosynthesis. Growth of filamentous algae (*Oscillatoria*, for example) on the bottom can be beneficial because oxygen bubbles formed during photosynthesis can remain entrapped in the algal mat. This causes algal flakes to break loose and float to the surface, carrying settled solids and other bottom accumulations with them. This process rejuvenates the bottom soil to essentially unclogged conditions. Where this phenomenon was observed (Flushing Meadows Project, for example), infiltration rates remained constant and even increased during flooding, as opposed to the decrease in infiltration usually observed. High water depths in the basins may be periodically employed to control vegetation. Vegetation in rapid-infiltration basins generally has no significant beneficial effect on infiltration rates or on quality improvement of the wastewater. However, vegetation can aggravate insect problems and for that reason weeds should not be allowed to grow out of hand.

A rapid-infiltration system with a relatively large number of small basins is preferable to one with only a few large basins, because the large number of basins offers more flexibility in scheduling flooding and drying periods and in performing maintenance work. Depending on the recovery system for, or the disposition of, the renovated water, the basins can be arranged in one cluster (see Figure 8.2, center) or in two parallel strips (see Figure 8.2, bottom). Where groundwater mounding can be a problem, the basins should form a long, narrow strip perpendicular to the prevailing direction of groundwater movement to minimize rise of groundwater mounds.

Where infiltration rates at the end of a flooding period are relatively low, time is saved if excess water in the basins is removed by surface drainage at the start of a drying period. If this is not done and all the water in the basin has to disappear by infiltration into the soil, it may take days before the basin is finally dry. For this reason, each basin should have its own inlet and outlet controls, and higher basins should be able to drain into lower basins so that the drainage water from one basin that is ready for drying can be put into another basin. A few reserve or "catchall" basins should be placed at the lower end of the project if there is to be no surface discharge of wastewater from the system. Infiltration basins should be properly graded so that all water can drain out of the basins and there are no low places where water can remain standing. Such low spots barely get dry during normal drying periods, and, hence, receive little or no infiltration recovery. The low areas thus become essentially ineffective for rapid infiltration when flooding is resumed and result in a loss of hydraulic capacity.

The most critical factor in the successful operation of a rapid-infiltration system unquestionably is to have adequate basin area to handle the design flow. If the basin area is not sufficient to handle the flow at all times (for example, when rainy or cold periods hamper drying of the basins and infiltration recoveries are not complete), basins are flooded longer and dried less, which causes a further decline in infiltration rates and a decrease in the hydraulic capacity of the system. Also, since the operator has to put the wastewater somewhere, it is likely that more and more wastewater will be stored in the basins and water depths increase.

This tendency reduces the turnover rate of water in the basins, exposes the water longer to sunlight, and allows more suspended algae to develop. These algae then further reduce infiltration rates by clogging the bottom soil and causing precipitation of calcium carbonate. Thus, once a rapid-infiltration system is overloaded, its hydraulic capacity decreases even more until eventually the system completely fails, the wastewater has to be discharged somewhere else, and the basins need a thorough drying and cleaning before flooding can be resumed. This undesirable procession can be avoided by designing the system with plenty of basin area and including some reserve basins to take up excess flow in cold or wet periods when infiltration rates are below average. In the successful operation of a rapid-infiltration system, there is no substitute for having an adequate number and total area of infiltration basins.

Rapid-infiltration basins require little maintenance. If the wastewater has a low suspended solids content (for example, less than 15 mg/L), solids accumulate slowly on the bottom and regular drying of the basins is adequate to restore infiltration rates to normal levels. Under those conditions, cleaning of the basins may be required only once every year or two. The usual maintenance practice is to disk or cultivate the basin soil so that organics and other solids that have accumulated on the surface are mixed with the soil and no longer form a continuous layer. With this practice, however, the organics content of the topsoil gradually increases and the surface layer eventually becomes completely clogged and has to be removed and possibly replaced. A better practice is to remove the solids that have accumulated on the basin soil by "shaving" the bottom with a front-end loader or by some other technique, rather than incorporating the organics into the soil. Where the basin area is barely adequate to handle the design flow, more frequent cleaning or cultivating is necessary. In some systems, the basin soil is cultivated or broken up in every drying period.

Rapid-infiltration systems have few odor problems. Odor is most noticeable in the first few days of a drying period, when decomposing algae and other solids on the soil produce a fishy, oily smell. Mosquito problems can occur and flooding and drying may have to be scheduled to avoid breeding of mosquitoes. In warm periods, mosquitoes can develop from egg to adult in as little as 3 to 4 days. To minimize mosquito problems, vegetation should be controlled mechanically or by periodic high water levels.

Rapid-infiltration systems attract a variety of birds and other wildlife. This characteristic offers a unique opportunity to combine the wastewater management function of the RI system with that of a wildlife sanctuary. Rather than having rectangular basins separated by straight and sterile berms, the basins could be constructed as free-form lagoons and the berms and other areas between the basins could be attractively planted. Nature trails could wind their way through the system. Public access should be controlled. With proper design and management, the area could become a haven for birdwatchers, scout troops, students, and other nature lovers.

### Underground Flow System

After infiltration, the wastewater moves down through the vadose zone to the underlying groundwater. Since solids accumulation and biological action on the bottom clog the surface, infiltration rates tend to be less than the saturated hydraulic conductivity of the natural soil. The soil beneath the infiltration basins will be unsaturated, soil-water pressure heads will be negative, and the flow will be essentially downward due to gravity alone and at unit hydraulic gradient. If there are finer-textured soil layers further down, the wastewater can be backed up and form a zone of water with positive pressure heads above the restricting layer (like perched groundwater). Knowing the infiltration rates and the saturated hydraulic conductivity of the restricting layers, the height of the perched water mounds can be estimated. As with the permanent water table, the height of the perched water table should not rise higher than 0.5 m below the bottom of the basins to avoid reductions in infiltration rates.

Eventually, the wastewater will reach the underlying groundwater. If the aquifer is unconfined, the wastewater will displace the native groundwater and will move mostly horizontally through the aquifer away from the infiltration system. If the aquifer is confined, the wastewater will form an unconfined aquifer above the upper confining layer.

Knowing the transmissivity of the aquifer, mound rises can be predicted by methods such as those developed by Hantush and Glover [1, and references therein; 2]. Where the water drains naturally into surface water (Figure 8.1), lateral flow through the aquifer can be calculated with Darcy's equation. The width of the infiltration strip is then selected so that the renovated wastewater can be transmitted laterally through the aquifer without causing undue water table rises beneath the infiltration basins [3]. Where the renovated water is to be collected with drains (Figure 8.2, top), drainage theory can be used to determine the most desirable combination of width of infiltration area and distance between the infiltration area and the drains [3]. Where the renovated water is to be collected by wells (Figure 8.2), the flow system needs to be analyzed to determine the optimum location, spacing, pumping rate, and pumping lift for each well [4].

Encroachment of renovated wastewater into native groundwater is unlikely for the systems in Figure 8.1 and Figure 8.2, top and center. For the bottom system in Figure 8.2, it can be avoided by monitoring groundwater levels at the periphery of the infiltration system (see observation wells in Figure 8.2) and managing pumping rates of renovated water from the wells between the basins so that the water levels in the observation wells do not rise higher than the water table in the aquifer adjacent to the system. There will then be no hydraulic gradients away from the system and, hence, no movement of renovated wastewater into the aquifer outside the system. To be on the safe side, the pumping rates for the renovated water in system Figure 8.2, bottom, could be increased slightly to produce somewhat lower groundwater levels at the observation wells than in the aquifer outside the system. This will create a slight gradient toward the rapid-infiltration system, so that some native groundwater will be drawn into the renovated water as it flows from beneath the infiltration basins to the wells. If there is a strong natural gradient of groundwater in the area of the rapid-infiltration system, some recovery wells for the renovated water may have to be located downgradient from the infiltration system to make sure that all the renovated water is intercepted.

Most of the quality improvement of the wastewater takes place in the top 1 m of the soil beneath the infiltration basins. Considerable additional movement in the vadose zone and aquifer is needed, however, to complete the renovation process (dieoff of bacteria and viruses, phosphate precipitation, decomposition of organics, taste and odor removal, etc.). Thus, the distance between the infiltration basins and the point where the renovated wastewater leaves the aquifer again should always be made as large as possible. A rule of thumb is to allow at least 100 m distance of underground travel and an underground detention time of at least 1 mo. The larger the underground flow distance and the longer the detention time, however, the better.

## WATER QUALITY IMPROVEMENT IN SOIL AND AQUIFER

### Dissolved and Suspended Solids

The salt concentration of the renovated water tends to be slightly higher than that of the original wastewater because of evaporation from the basins and possible dissolution of calcium carbonate and other minerals in the soil. Annual evaporation losses from RI systems should be somewhat less than from a free water surface, because the evaporation rate from the soil will become less than the potential rate toward the end of the drying periods [5]. Annual evaporation from a free water surface in a warm, arid climate, such as in the southwestern United States, is in the range of 1.5 to 2.5 m. Thus, at a hydraulic loading rate of 100 m/per year, evaporation from the basins would cause the salt content of the renovated water to be at most 1.5 to 2.5 percent higher than that of the wastewater. Dissolution of calcium carbonates can occur in calcareous soils. These soils are normally alkaline and can release calcium ions upon leaching by sewage effluent, which tends to become neutral to slightly acid in the soil environment as carbon dioxide and organic acids are formed by biological decomposition of organics.

At the Flushing Meadows Project west of Phoenix, Arizona, [6, 7], the TDS content of the renovated water was about 2 percent more (averaging about 1,100 mg/L) than that of the secondary sewage effluent going into the ground. At the 23rd Avenue Rapid Infiltration Project in Phoenix, the TDS content of the secondary effluent and renovated water was essentially the same (average 910 mg/L). Since TDS concentrations of sewage effluent tend to vary daily as well as seasonally, small changes in the TDS content of the water as it moves through the soil and aquifer are difficult to detect because of the time it takes for the water to move through the vadose zone and aquifer to the monitoring wells. Total salt concentrations of sewage effluent tend to be 300 to 400 mg/L higher than those of the drinking water going into the city.

Cation exchange reactions may cause the initial cationic composition of the renovated water to differ from that of the wastewater going into the ground. Ammonium, for example, may be adsorbed for the first few months of operation of the project and not show up in the renovated water. This could lead to erroneous conclusions regarding the long-term capability of the soil-aquifer system to remove nitrogen from the water. As the cation exchange complex in the soil and aquifer equilibrate with the wastewater moving through it, however, the ionic composition of the renovated water will approach that of the original wastewater.

The concentration of suspended solids in the wastewater can vary widely. For good secondary effluent, it may be on the order of 10 mg/L. Suspended solids are essentially completely removed by filtering, which starts with the trapping of the larger suspended particles at the surface of the soil or at some depth. Individual particles may be blocked in the pores, or several particles may interact to form a bridge in the pore that prevents further movement of these particles in the direction of flow. Once movement of the larger suspended particles has been blocked, these particles themselves begin to function as a filter and trap successively smaller suspended particles until eventually the filter becomes clogged and the hydraulic conductivity and flow rates are reduced to a fraction of their original values. Laboratory studies with uniform spherical materials indicated that bridging occurred when the diameter of suspended particles entering or moving through the medium was more than 0.2 times the diameter of the particles of the medium itself [8]. Depending on how the particles in the porous medium were packed, bridging also occurred if the diameter of the suspended material was more than 0.07 that of the particles in the medium. When the size of the suspended particles was less than 0.07 times the particle size of the medium, the suspended particles moved through the medium without bridging or blocking. Fine to medium-textured soils thus can be expected to remove essentially all suspended solids from wastewater by straining. The suspended solids content of the renovated water from the Flushing Meadows and 23rd Avenue Projects west of Phoenix generally was less than 1 mg/L.

### Nitrogen

Sewage treatment plant effluent in the United States typically contains 10 to 30 mg per liter of nitrogen. For activated sludge or trickling filter effluent, the nitrogen will be mostly in the form of ammonium. For lagoon effluents or effluents from oxidation ditch–type systems, it can be more in the form of nitrate. Nitrogen is removed in soil-aquifer treatment systems by denitrification, which is a biological process that occurs under anaerobic conditions, requires organic carbon as an energy source for the denitrifying bacteria, and produces free nitrogen gas and oxides of nitrogen that can return to the atmosphere. The conditions for denitrification in soil-aquifer treatment systems can be enhanced for effluents where the nitrogen is mostly

in the ammonium form by flooding long enough to saturate the cation-exchange complex in the top 0.5 to 1 m of the soil (the zone that becomes aerobic during drying) with adsorbed ammonium. Upon drying the basin, atmospheric oxygen will then move into the soil profile and allow nitrifying bacteria to convert the adsorbed ammonium to nitrate. This nitrate can then diffuse into microanaerobic sites that are still present in the small pores and interstices in the otherwise aerobic zone, where it can be denitrified. The remaining nitrate is leached out when flooding is resumed. This produces a high-nitrate wave in the renovated water as it moves downward. This wave can be detected as a nitrate peak in the renovated water sampled from the top of the groundwater beneath the basins (Figure 8.3). At greater depth or at greater horizontal distance from the basins, the peaks become dispersed. They also begin to overlap as renovated water arrives that has infiltrated at various points in the system and has taken various times to travel to the monitoring well. The nitrate-N concentration of the renovated water then becomes more uniform (Figure 8.4).

The length of the flooding and drying periods for maximizing denitrification in soil flooded with wastewater where nitrogen is mostly in the ammonium form primarily depends on the nitrogen loading, as determined by the infiltration rates and the nitrogen concentration of the effluent; on the cation exchange capacity of the soil; on the exchangeable ammonium

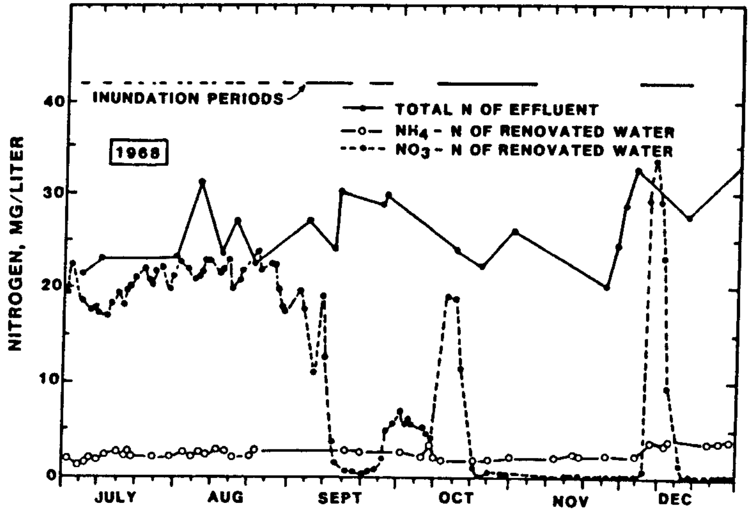

Figure 8.3 Nitrogen transformations at Flushing Meadows for short and long inundation periods.

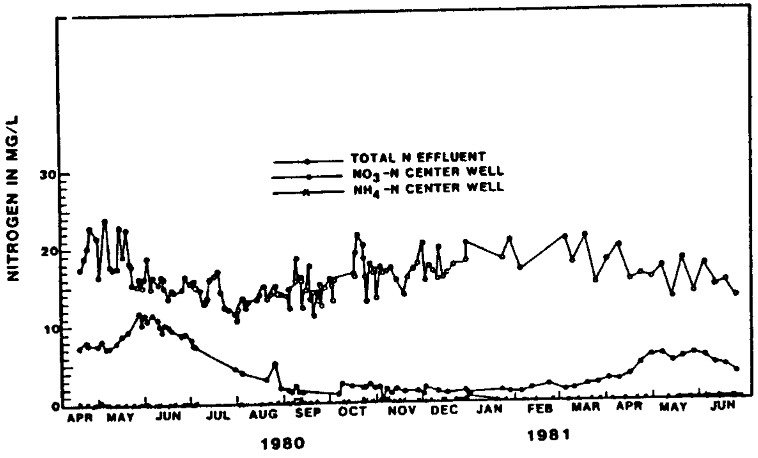

**Figure 8.4** Total nitrogen concentration of secondary effluent and nitrate and ammonium nitrogen concentrations in renovated water from the center well of the 23rd Avenue Project.

percentage, as determined by the cationic composition of the wastewater; on the depth to which the soil becomes aerobic during drying; and on the temperature. Most of these processes have been quantified and it is possible to make an estimate of the optimum flooding and drying schedule for denitrification [9]. Local experimentation may be required, however, to evaluate the optimum schedule for denitrification with more accuracy.

For the Flushing Meadows project, the optimum schedule for denitrification was 9 days flooding-12 days drying while maintaining a hydraulic loading rate of about 70 m per year by using a water depth of 0.15 m in the basins. At this schedule, the renovated water contained 65 percent less nitrogen than the secondary effluent [10]. At the 23rd Avenue Project, a sequence of 2 weeks flooding–2 weeks drying yielded a nitrogen removal of 74 percent. For both projects the residual nitrogen in the renovated water was mostly in the form of nitrate. Typical average concentrations in effluent and renovated water after about 9 years at Flushing Meadows and 4 years at 23rd Avenue are shown in Table 8.1. If the flooding periods were longer than the above values for maximum denitrification, more $NH_4$–N entered the soil during flooding than could be nitrified during drying. Thus, adsorbed $NH_4^+$ accumulated in the soil, reducing the amount of $NH_4^+$ that could be adsorbed during subsequent flooding, and increasing the $NH_4^+$ concentration of the renovated water [10]. On the other hand, if the flooding periods were shorter than those for maximum denitrification, the soil was not anaerobic enough for adsorption of $NH_4^+$, thus yielding more nitrification and less denitrification. Cycles of 2 days flooding and 5 days drying, for example, yielded essentially complete conversion of the total nitrogen in the effluent to nitrate nitrogen in the renovated water (see left side of Figure 8.3).

**Table 8.1** Nitrogen Concentrations (mg/L) in Secondary Effluent and Renovated Water from Flushing Meadows and 23rd Avenue Projects

| *Nitrogen Concentrations* | *Secondary Effluent* | *Renovated Water* |
|---|---|---|
| Flushing Meadows | | |
| Total N | 27 | 9.6 |
| $NH_4$-N | 21 | 2.8 |
| $NO_3$-N | 3 | 6.2 |
| Organic N | 3.7 | 0.6 |
| 23rd Avenue | | |
| Total N | 18 | 3.7 |
| $NH_4$-N | 15 | 0.1 |
| $NO_3$-N | 0.1 | 3.4 |
| Organic N | 2.4 | 0.2 |

Other processes may also stimulate denitrification. For example, an active algae layer on the bottom of the basins produces oxygen during the day, which could nitrify some of the ammonium in the infiltrating water. The resulting nitrate would then move into the soil, where it could be denitrified. Denitrification in the soil may also be enhanced by small applications of wastewater during drying periods with, for example, a sprinkler system. This would add organic carbon to the biologically active upper portion of the soil and create wet conditions that are favorable for denitrification. Denitrification of the nitrate that is leached out of the upper soil layers when flooding is resumed can be stimulated by using a minimum water depth in the basins for the first day or two of flooding. This action would reduce the initial infiltration rate, thereby giving the nitrate more time in the biologically active upper soil layers where it could be denitrified. Denitrification rates are also affected by temperature and become much reduced below 15° C. This variability may explain the increase in $NO_3$–N of the renovated water in winter and spring, as shown in Figure 8.4, taking the usual lags into account.

If the nitrogen in the effluent for a rapid infiltration system is already mostly in the nitrate form, denitrification can be enhanced by using long flooding periods to produce anaerobic conditions in the soil beneath the basins. Since nitrified effluents tend to have a low organic carbon content and denitrifiers require about 1 mg of organic carbon for each mg nitrate nitrogen to be denitrified, organic carbon may have to be added before the effluent enters the ground.

### Phosphorus

Phosphates in renovated water are removed by precipitation to amorphous and crystalline compounds with iron, aluminum, or calcium. In acid soils, iron and aluminum phosphates dominate; whereas in alkaline soils, calcium phosphates are more prevalent. Neutral soils have the slowest reactions and the greatest solubility for phosphate chemicals. The precipitation reactions are usually slow and the initial immobilization of the phosphate ion is primarily a fast sorption reaction. The adsorbed phosphate then slowly reverts to insoluble compounds. The kinetics of these reactions has been studied [11] and a model developed to predict phosphate removal from wastewater moving through soils. The reactions followed a first-order kinetic equation. Final or equilibrium concentrations of phosphate in renovated water from a soil-aquifer treatment system could be derived from the solubility products of the major phosphate compounds in the underground environment [11].

For the calcareous soils of the Flushing Meadows Project, phosphate removal in the vadose zone and aquifer was dependent on the phosphate loading rates as determined by hydraulic loading and phosphate concentration. This is illustrated in Figure 8.5, which shows that phosphate removal was less in the period 1969–1972 when hydraulic loading rates were of the order of about 100 m per year than in the period 1973–1977 when hydraulic loading rates were reduced to about 70 m per year to increase denitrification. Also, phosphate levels in the renovated water tended to go up and down with the phosphate concentrations of the input water. At the end of 1977, $PO_4$–P concentrations were 6.5 mg/L for the secondary effluent, 1.2 mg/L for the renovated water sampled from a depth of 9 m in the center of the basin area, and 0.3 mg/L in the renovated water sampled from a depth of 6 m at the distance of 30 m from the basin area. At the 23rd Avenue Project, $PO_4$–P concentrations after five years of operation were

| | | |
|---|---|---|
| Secondary effluent entering basins | 5.4 | mg/L |
| Secondary effluent in center of basins | 4.9 | mg/L |
| Secondary effluent at end of basins | 4.1 | mg/L |
| Renovated water from: | | |
| 18 m depth in center of basin area | 4.4 | mg/L |
| 24 m depth in center of basin area | 3.7 | mg/L |
| 30 m depth in center of basin area | 3.4 | mg/L |
| 21 m depth at north edge of basin area | 1.4 | mg/L |
| 21 m depth at south edge of basin area | 0.97 | mg/L |
| 30–54 m depth in center of basin area | 0.72 | mg/L |

These figures show the continuous removal of phosphate as the renovated water flows vertically and horizontally through the aquifer. Accumulation of phosphate precipitate per unit volume of soil and aquifer thus is very slow and significant reductions in porosity and hydraulic conductivity due to phosphate accumulation will take decades or centuries.

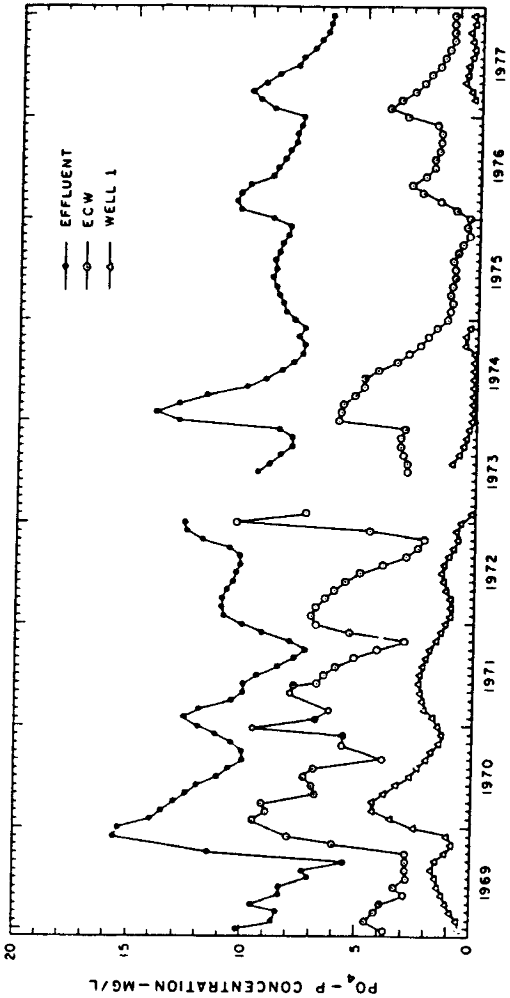

**Figure 8.5** Effect of soil-aquifer treatment on orthophosphate-P in renovated water of Flushing Meadows Project.

### Metals, Boron, Fluorine

The heavy-metal concentrations in sewage treatment plant effluents normally are low and below maximum limits for raw municipal water supplies and irrigation water [12, 13, and references therein]. Where the sewage contains discharges from metal-processing plants or other metal-containing wastewater, high and sometimes very high metal concentrations can be encountered. Where heavy-metal concentrations in the wastewater are unacceptably high, source control would be the logical approach to reducing them.

In the soil, heavy metals enter into general cation exchange reactions with clays and organic matter, specific sorption reactions with soil minerals, and also into chelation reactions with organic molecules. Heavy metals in sewage effluent probably occur largely as low molecular weight soluble chelates [12]. These chelating agents can increase the mobility of the metals in the soil-aquifer system. On the other hand, physical straining through fine-textured soils and sorption can remove high-molecular-weight metal complexes from the effluent. As the organic molecules are decomposed, the metals are free to react with iron and aluminum hydroxides, calcium, and other chemicals in the soil and become immobilized. These immobilization reactions are more pronounced at high pH and aerobic conditions than at low pH and anaerobic conditions.

The main problem with heavy metals is that they can accumulate in the soil to levels that may be undesirable for the growth and utilization of crops [12]. However, this is not critical for rapid-infiltration systems where the soil is not used for crop production. At the Flushing Meadows Project, concentrations of zinc, copper, and mercury in the renovated water were significantly below those in the sewage effluent [7]. Lead was reduced only slightly and cadmium not at all. Other studies also have indicated that cadmium is the most mobile of the heavy metals in the underground environment [14].

Boron is adsorbed to iron and aluminum hydroxide coatings on clay minerals, to iron and aluminum oxides, to micaceous clay minerals, and to magnesium hydroxy coatings that exist on weathering surfaces of ferromagnesian minerals [12, and references therein]. In sandy soil and aquifer materials consisting principally of quartz, boron is not significantly immobilized. This fact was noted at the Flushing Meadows and 23rd Avenue Projects, where boron concentrations in the renovated water were essentially the same as those in the secondary effluent, i.e., about 0.5 to 0.6 mg/L for most of the time. In 1971 and 1972, however, boron concentrations rose to 0.85 mg/L, possibly as a result of an intensive advertising campaign for a low phosphate laundry detergent that contained borate and perborate. If the renovated water is to be used for general irrigation, boron concentrations should not exceed 0.5 mg/L. Thus, trading phosphate for boron in such situations will indeed be like the proverbial tiger that was called in to chase away a dog.

Fluorine gets into wastewater through fluoridation of public water supplies (dose rate about 1 mg/L) and through discharge of certain industrial wastes (for example, from electronics industries). Fluoride is adsorbed by various soil components, especially hydrous aluminum oxides, and precipitated as fluorite ($CaF_2$) and fluorapatite. At the Flushing Meadows Project, fluoride concentrations in the secondary effluent gradually decreased from an average of 4.1 mg/L for the period 1967–1972 to 2.1 mg/L for 1972–1977. In 1977, the average fluoride concentration in the renovated water was 1.7 mg/L for wells within the basin area and 0.9 mg per liter for the wells 30 m outside the basin area. Thus, fluoride removal continued in the aquifer and somewhat paralleled the phosphate removal.

### Bacteria and Viruses

The diameter of small bacteria is of the order of 500 to 1,000 nm and that of viruses 30 to 300 nm. Thus, while small bacteria may still be adsorbed to soil particles, the larger bacteria and other microorganisms like protozoa and helminth eggs are mostly immobilized in soils by physical straining or filtering. Viruses usually are small enough to be adsorbed to clay particles and other reactive surfaces. Pathogenic and other organisms not native to the soil environment fail to reproduce and eventually die. The dieoff is accelerated by competition and other antagonistic effects of the native bacterial flora in the soil.

The isoelectric point of viruses depends on the particular species and ranges from a pH of about 3 to 7. Below this pH, the viruses are positively charged and can be adsorbed by the negatively charged surfaces of clay minerals and colloidal organic matter in the soil. Above the isoelectric point, viruses are negatively charged and can be immobilized via anion adsorption. However, cation adsorption seems to be the dominant process and virus immobilization in soil generally increases with decreasing pH. Virus immobilization also increases with increasing salt concentration of the soil water and with the concentration of divalent or trivalent cations as opposed to monovalent cations [15, see also Chapter 9].

Straining bacteria in rapid-infiltration systems is especially effective after the basins have been flooded for some time and a clogging layer of settled solids, algae, and microorganisms has developed on the bottom. At the Flushing Meadows Project, for example, fecal coliforms penetrated to the underlying groundwater (at 3 m depth) and to the intakes of monitoring wells at 6 and 9 m depths at the beginning of a flooding period, but were reduced to zero in the renovated water after about 1 week of flooding [7]. Fecal coliform concentrations in renovated water sampled below the basins sometimes peaked between 100 and 1,000 per 100 mL. For the well 30 m away from the edge of the basins, the additional travel through the aquifer reduced the peaks to less than 100 per 100 mL, while fecal coliform bacteria could no longer be detected in renovated water from a well 90 m away from the basins. Thus, the renovated water needed to move about 100 m laterally through the aquifer before fecal coliforms were completely removed. Most of the fecal coliforms (about 5 log cycles) were removed in the top half meter of the soil, however [7]. The fecal coliform concentration of the secondary effluent, which was not chlorinated, was on the order of $10^6$ per 100 mL.

The fecal coliform concentration of the secondary effluent at the 23rd Avenue Project also was about $10^6$ per 100 mL. Considerable dieoff occurred in the large infiltration basins (4 ha each), reducing the average fecal coliform density to about $10^4$ per 100 mL at the outlet end of the basins. In the first 2 years of the 23rd Avenue Project, there was a serious problem of algal clogging of the soil in the infiltration basins. While this reduced the hydraulic loading to only about 25 percent of full capacity, the "filter cake" of algae on the bottom effectively prevented fecal coliforms from reaching the groundwater. Thus, the renovated water was essentially free from fecal coliforms, except when the basins near the well had just been flooded and coliforms could penetrate deeper into the soil and aquifer system. This situation produced peaks in the fecal coliform concentrations of 10 to 40 per 100 mL for a day or so. Average fecal coliform concentrations for the wells at the north and south edges of the basin area were 1.25 and 2.3 per 100 mL, respectively.

In 1980, the hydraulic loading rates of the basins were increased fourfold, to about 90 m per year, by constructing a bypass channel around the 32-ha lagoon through which the effluent previously passed before it entered the infiltration basins (Figure 8.6). The bypass channel provided a shortcut to the secondary effluent, which greatly reduced the growth of algae in the effluent. The fecal coliform concentration of the secondary effluent entering the infiltration basins, however, increased to about $1.8 \times 10^6$ per 100 mL, while that of the renovated water of the production well in the center of the basin area reached an average of 22 per 100 mL with peaks of 140 and 160 per 100 mL. Thus, flowing the secondary effluent directly to the infiltration basins and increasing the hydraulic loading rate increased fecal coliform concentrations in the sewage effluent and in the renovated water. In the fall of 1980, however, the treatment plant started to chlorinate the effluent using a dose of 1.5 mg per liter of chlorine. This reduced the average fecal coliform concentration of the effluent to 3,500 per 100 mL and that of the renovated water from the production well in the center to 0.27 per 100 mL.

The relatively fine, loamy sands at the Flushing Meadows Project readily adsorbed viruses. Thus, while virus concentrations in the effluent at Flushing Meadows averaged 2,118 PFU per 100 L, no viruses could be detected in renovated water from any of the monitoring wells [16]. In this study, viruses were concentrated into small volumes suitable for assay from 200- to 400-L samples using established concentration techniques. At the 23rd Avenue Project, viruses were concentrated from 800- to 2,000-L samples. The results showed a range of 0.1 to 2.0 PFU per 100 L for the various

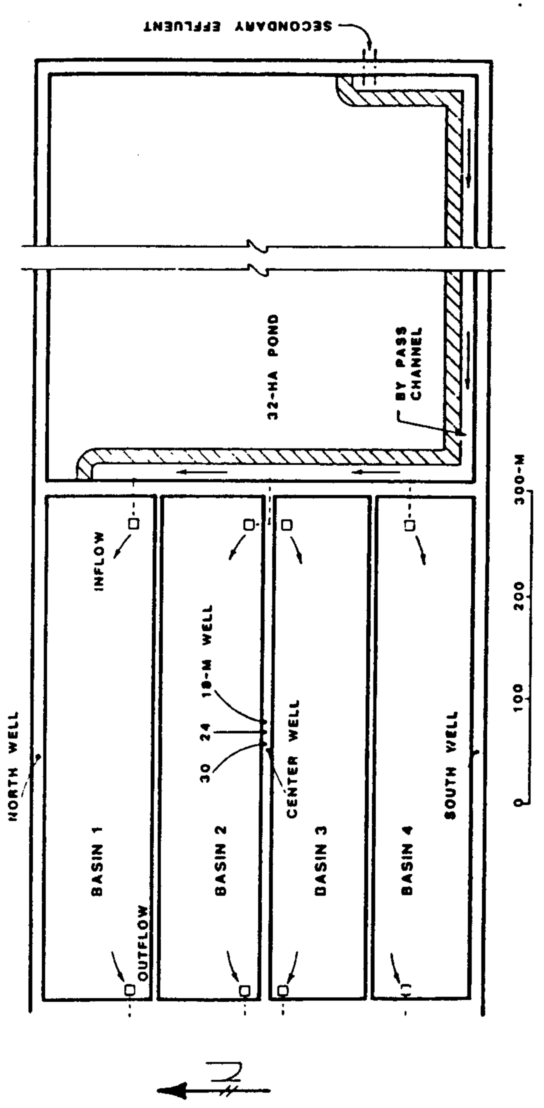

Figure 8.6 Schematic of 23rd Avenue Project in Phoenix, showing bypass channel, infiltration basins, production well in center, and monitoring wells.

wells, with an average of 1.0 PFU per 100 L for all wells. However, after the treatment plant started to chlorinate the effluent, virus concentrations in the renovated water from the center well were reduced to zero. In areas with more rainfall and/or coarser soil below the infiltration basins, deeper penetration of viruses and greater lateral movement in the aquifer itself can be expected [15].

## Organic Carbon

### *BOD, COD, and TOC*

The biochemical oxygen demand (BOD) of the renovated water at the Flushing Meadows Project was essentially zero (less than 1 mg/L), indicating that the soil-aquifer system removed basically all biodegradable material from the wastewater. The secondary (activated sludge) effluent used in this project had a BOD of 10 to 20 mg per liter. Both field and laboratory studies indicated that the BOD of the renovated water was not affected by the length of the flooding period (flooding periods ranged from 2 days to 6 weeks). Thus, even the anaerobic conditions and slower breakdown of organics during prolonged flooding produced essentially zero BOD levels in the renovated water. The chemical oxygen demand (COD, dichromate technique) of the renovated water of the Flushing Meadows Project ranged between 10 and 20 mg per liter and that of the secondary effluent between 30 and 60 mg per liter. Part of the COD of the renovated water probably was due to reduced iron and other inorganic compounds in the water.

The best parameter to characterize organics in wastewater and renovated water is the total organic carbon content, which for the renovated water averaged about 5 mg per liter for the Flushing Meadows Project and 2 mg per liter for that of the production well in the center of basin area at the 23rd Avenue Project. This residual organic carbon indicates that not all the organic compounds in municipal wastewater are biodegradable and that some refractory or trace organics persist in the renovated water. The presence of these organics is of concern in the potable use of the renovated water, because they could include compounds that are carcinogenic, mutagenic, or otherwise toxic. Some of the organic carbon probably is from humic and fulvic acids (unpolluted groundwater typically contains 0.7 mg per liter organic carbon, mostly as fulvic acid; [17]), but anthropogenic compounds could also be present.

A systematic study of the trace organics in the sewage effluent in the renovated water from various wells was carried out at the 23rd Avenue Project (see Figure 8.6). Samples were taken weekly for the effluent and biweekly for the renovated water, beginning in the period September–November 1980 (Period 1) when the treatment plant still discharged nonchlorinated effluent. At the end of November 1980, the chlorination facility of the plant was completed and the effluent was dosed at 1.5 mg Cl per liter. Samples were again taken in the period April–June 1981 (Period 2) to study the effect of chlorination on the trace organics in the secondary effluent and their fate in the soil and aquifer system below the infiltration basins. The samples were shipped to Stanford University's Water Quality Control Research Laboratory for analysis and interpretation of results.

### *Characterization of Organic Carbon by Gas Chromatography and Mass Spectrometry*

Two gas chromatographic procedures were used: volatile organic analysis (VOA) and closed-loop stripping analysis (CLSA). Three times during each of the two study periods, additional samples were collected for total organic halogen (TOX) determination, hexane-ether solvent extraction analysis (HEA), base-neutral solvent extraction analysis (BNSEA), and acid-phenol solvent extraction analysis (APSEA). These procedures allowed identification of organic priority pollutants through gas chromatography/mass spectrometry (GC/MS). Since organic concentrations in treated wastewater tend to follow log-normal rather than normal distributions, the results were interpreted with log-normal statistics, using geometric means and spread factors [18]. A detailed report on the analytical procedures and results can be found in Bouwer et al. [19]. The major findings are summarized in the following paragraphs.

Halogenated organic substances detected by VOA and CLSA in the basins are compared in Table 8.2. The compounds were similar to those found in other secondary municipal wastewater [20]. Trichlorophenol, pentachlorophenol, phenanthrene, and diethylphthalate were the only additional priority pollutants identified by the HEA, BNSEA, and APSEA procedures. The data indicate that chlorination resulted in a higher chloroform concentration and in the formation of the three brominated trihalomethanes. Otherwise, chlorination of the secondary effluent had little effect on the chlorinated organic concentrations measured by these procedures. Several of the chlorinated compounds actually had lower average concentrations during Period 2 than in Period 1. A similar comparison for the nonhalogenated aliphatic and aromatic hydrocarbons detected in the basin water (Table 8.3) indicates some compounds had concentrations that were higher in Period 2, others remained the same, and some had lower concentrations. This suggests that differences in concentrations between Periods 1 and 2 were the result of normal concentration fluctuations in the secondary effluent, rather than of the effects of chlorination.

### *Attenuation of Organics in Basins, Soils, and Aquifer*

A paired comparison of the basin inflow and outflow concentrations of the more volatile organic micropollutants (Table 8.4) indicates a concentration

**Table 8.2** Halogenated Organic Compounds Identified during Periods 1 (Nonchlorinated Effluent) and 2 (Chlorinated Effluent) in Basin Water (Concentrations in µg/L)

| | *Secondary Effluent (Basin Inflow)* | |
|---|---|---|
| *Constituent*[a] | *Period 1* | *Period 2* |
| Halogenated Aliphatic Hydrocarbons | | |
| chloroform | 2.88 | 4.79 |
| 1,1,1–trichloroethane | 2.45 | 1.79 |
| carbon tetrachloride | 0.13 | 0.15 |
| bromodichloromethane | —[b] | 0.51 |
| trichloroethylene | 0.91 | 0.53 |
| dibromochloromethane | — | 0.46 |
| tetrachloroethylene | 2.21 | 1.82 |
| bromoform | — | 0.13 |
| Chlorinated Aromatics | | |
| o-dichlorobenzene | 4.11 | 3.18 |
| m-dichlorobenzene | 1.15 | 0.53 |
| p-dichlorobenzene | 2.70 | 2.82 |
| 1,2,4–trichlorobenzene | 0.33 | 0.44 |
| trichlorophenol | 0.01 | 0.02 |
| pentachlorophenol | 0.02 | 0.04 |
| pentachloroanisole[c] | 0.63 | 0.26 |

[a] Identification confirmed by comparison with standards.
[b] Not detected.
[c] Only compound that is not a priority pollutant.

decrease between 30 and 70 percent as effluent moved through the basins for the combined data from Periods 1 and 2. Many of the organic compounds measured by the VOA and CLSA procedures have high Henry's law constants and are easily air stripped from wastewater. Hence, volatilization probably was responsible for the decreases in concentration. No difference in the percent decrease between the aromatic and aliphatic compounds shown was observed.

Results on the behavior of halogenated organic substances during soil and aquifer passage are shown in Table 8.5 for Periods 1 and 2. Similar data are shown for the nonhalogenated compounds in Table 8.6. The tables show the geometric mean basin concentration for each compound and the percentage reduction between the average basin concentrations and the 18-m well values. While data for the renovated water from the other wells were also presented in the complete report [19], only the data for the 18-m well were selected here because they refer to the renovated water after it had just reached the groundwater without benefit of significant movement through the aquifer itself. Thus, the data from the 18-m well in a sense express the "worst case" for the renovated water.

**Table 8.3** Nonhalogenated Organic Compounds Identified during Periods 1 and 2 in Basin Water (Concentrations in µg/L)

| | *Secondary Effluent (Basin Inflow)* | |
|---|---|---|
| *Constituent*[a] | *Period 1* | *Period 2* |
| Aliphatic Hydrocarbons | | |
| 5–(2–methylpropyl) nonane[b] | 0.49 | 1.10 |
| 2,2,5–trimethylhexane[b] | 0.15 | 0.38 |
| 6-methyl-5-nonene-4-one[b] | 0.34 | 1.51 |
| 2,2,3–trimethylnonane[b] | 0.31 | 0.64 |
| 2,3,7–trimethyloctane[b] | 0.18 | 0.40 |
| Aromatic Hydrocarbons | | |
| o-xylene[a] | 0.37 | 0.50 |
| m-xylene[a] | 0.73 | 1.33 |
| p-xylene[a] | 0.12 | 0.07 |
| $C_3$ benzene isomer[b] | 0.51 | 0.22 |
| $C_3$ benzene isomer[b] | 0.40 | 0.51 |
| styrene[a] | 0.15 | 0.77 |
| 1,2,4–trimethyl benzene[a] | 0.66 | 0.81 |
| ethylbenzene[a,c] | 0.20 | 0.09 |
| naphthalene[a,c] | 0.17 | 0.71 |
| phenanthrene[a,c] | — | 0.11 |
| diethylphthalate[a,c] | 20 | 15 |

[a] Identification confirmed by comparison with standards.
[b] Identification based on best mass spectrum fit with National Bureau of Standards Library of Mass Spectra. Concentrations shown are relative to the internal standard.
[c] Priority pollutant.

Nonhalogenated hydrocarbons (see Table 8.6) decreased (50 to 99 percent) during percolation through the soil with concentrations in the renovated water being near or below the detection limit. However, most of the compounds could still be detected in the renovated water. Reduction percentages were generally higher during Period 2 as a result of higher concentrations in the effluent water observed for many of the nonhalogenated compounds. These compounds are subject to microbial decomposition and, presumably, were removed during soil percolation by this process. Decreases in the concentrations of the nonhalogenated priority pollutants were comparable to those for the other nonhalogenated aliphatic and aromatic hydrocarbons (see Table 8.6). Concentration variations in the renovated water were less than those in the sewage effluent in the basins. Thus, percolation through the soil had the effect of damping fluctuations in concentrations and eliminating extreme values.

The halogenated organic compounds generally decreased to a lesser extent with soil passage than the nonhalogenated compounds. Of the hal-

**Table 8.4** Percentage Decrease in Organic Constituents across Infiltration Basins Using a Paired Comparison of Basin Inflow and Outflow Data

| *Constituent* | *Average Decrease across Basin (Difference between basin inflow and outflow) (%)* |
|---|---|
| Chlorinated Aliphatic Hydrocarbons | |
| Chloroform | 31 |
| 1,1,1-trichloroethane | 49 |
| trichloroethylene | 39 |
| tetrachloroethylene | 30 |
| Chlorinated Aromatic Hydrocarbons | |
| o-dichlorobenzene | 40 |
| m-dichlorobenzene | 65 |
| p-dichlorobenzene | 42 |
| 1,2,4-trichlorobenzene | 73 |
| (chloromethyl)-benzene | 65 |
| Aliphatic Hydrocarbons | |
| 2,2,5-trimethylhexane | 40 |
| 5-(2-methylpropyl) nonane | 51 |
| 2,2,3-trimethylnonane | 55 |
| Aromatic Hydrocarbons | |
| o-xylene | 35 |
| m-xylene | 35 |
| 1,2,4-trimethyl benzene | 52 |
| $C_3$-benzene isomer | 53 |
| naphthalene | 22 |

ogenated aliphatic hydrocarbons, the renovated water concentrations of chloroform and 1,1,1-trichloroethane were lower than those in the basin water during Periods 1 and 2. The brominated trihalomethanes present in the secondary wastewater with chlorination were not detected in the renovated water samples. This may have been the result of slow transport due to sorption or to chemical or biological transformation. The concentrations of trichloroethylene and pentachloroanisole were significantly higher in the renovated water than in the basin during both sampling periods. Tetrachloroethylene exhibited a similar concentration increase in Period 1 but not in Period 2.

The chlorinated aromatics appeared to be relatively refractory and mobile in the ground because they showed much less concentration decrease than the nonchlorinated aromatic hydrocarbons. Less decrease in the dichlorobenzenes was observed in Period 2 than in Period 1. Complete breakthrough appeared to occur for the chlorophenols, but concentrations were near detection limits so that positive conclusions could not be made. A combination of biodegradation and sorption processes might have been responsible for the decreases observed.

**Table 8.5** Percentage Concentration Decrease in Halogenated Organic Substances by Ground Filtration

| | *Period 1* | | *Period 2* | |
|---|---|---|---|---|
| *Constituent* | *Geometric Mean Basin Concentration (27 Samples) (μg/L)* | *Average 18-m Well Decrease (6 samples) (%)* | *Geometric Mean Basin Concentration (27 samples) (μg/L)* | *Average 18-m Well Decrease (6 samples) (%)* |
| Chlorinated Aliphatic Hydrocarbons | | | | |
| chloroform | 2.72 | 61[b] | 3.46 | 88[b] |
| 1,1,1-trichloroethane | 2.94 | 34 | 1.41 | 84[b] |
| carbon tetrachloride | 0.12 | 0 | 0.12 | 42[b] |
| bromodichloromethane | —[a] | — | 0.26 | >62 |
| trichloroethylene | 0.91 | −180[b] | 0.39 | −267[b] |
| dibromochloromethane | — | — | 0.23 | >57 |
| tetrachloroethylene | 2.63 | −97[b] | 1.69 | 31[b] |
| bromoform | — | — | 0.08 | >10 |
| Chlorinated Aromatics | | | | |
| o-dichlorobenzene | 3.52 | 25 | 2.40 | 10 |
| m-dichlorobenzene | 0.79 | 58[b] | 0.38 | 5 |
| p-dichlorobenzene | 2.25 | 33[b] | 1.81 | 10 |
| 1,2,4-trichlorobenzene | 0.19 | 42[b] | 0.38 | 71[b] |
| trichlorophenol | 0.01 | 0 | 0.02 | 0 |
| pentachlorophenol | 0.02 | 0 | 0.04 | 0 |
| pentachloroanisole | 0.43 | −150 | 0.18 | −120 |

[a] Not detected.

[b] Level of significance for the difference between basin and well concentrations based on a t-test comparison is less than or equal to 0.1.

**Table 8.6** Percentage Concentration Decrease in Nonhalogenated Hydrocarbons by Ground Filtration

| | Period 1 | | Period 2 | |
|---|---|---|---|---|
| *Constituent* | *Geometric Mean Basin Concentration (27 samples) (μg/L)* | *Average 18-m Well Decrease (6 samples) (%)* | *Geometric Mean Basin Concentration (36 samples) (μ/L)* | *Average 18-m Well Decrease (6 samples) (%)* |
| Aliphatic Hydrocarbons | | | | |
| 5–(2-methylpropyl) nonane | 0.35 | >94 | 0.57 | >96 |
| 2,2,5–trimethylhexane | 0.11 | >82 | 0.18 | >89 |
| 6-methyl-5-nonene-4-one | 0.41 | 93$^a$ | 0.94 | 98$^a$ |
| 2,2,3–trimethylnonane | 0.21 | 76$^a$ | 0.25 | >92 |
| 2,3,7–trimethyloctane | 0.12 | 50$^a$ | 0.27 | >93 |
| Aromatic Hydrocarbons | | | | |
| o-xylene | 0.45 | 67$^a$ | 0.50 | 88$^a$ |
| m-xylene | 0.76 | 78$^a$ | 1.00 | 98$^a$ |
| p-xylene | 0.17 | 53$^a$ | 0.12 | 92$^a$ |
| $C_3$–benzene isomer | 0.56 | 84$^a$ | 0.34 | >94 |
| $C_3$–benzene isomer | 0.48 | 85$^a$ | 0.53 | 96$^a$ |
| styrene | 0.26 | >92 | 0.58 | 98$^a$ |
| 1,2,4-trimethyl benzene | 0.80 | 78$^a$ | 1.04 | 96$^a$ |
| ethylbenzene | 0.19 | 53$^a$ | 0.15 | 67 |
| naphthalene | 0.22 | 68$^a$ | 0.63 | 91$^a$ |
| phenanthrene | 0.10 | 80 | 0.10 | 90 |
| diethylphthalate | 19 | 20 | 10 | 90 |

[a] Level of significance for the difference between basin and well concentrations based on a t-test comparison is less than or equal to 0.1.

Total organic halogen (TOX) concentrations and ratios of TOX to total organic carbon (TOX/TOC) are shown in Table 8.7. The basin water TOX was significantly higher with chlorination (Period 2) than without (Period 1). However, the renovated water TOX concentrations were similar for both periods. The ratio of TOX to TOC was higher in the renovated water than in the secondary effluent samples, implying that the halogenated organic compounds comprise the more refractory and mobile portion of the TOC.

### *Other Organic Micropollutants*

In addition to the halogenated aliphatics and aromatics mentioned, other compounds tentatively identified in organic extracts of the basin and renovated water samples using gas chromatography/mass spectrometry were fatty acids, resin acids, clofibric acid, alkylphenol carboxylic acids (APECs), trimethylbenzene sulfonic acid, steroids, n-alkanes, caffeine, Diazinon, alkylphenol polyethoxylates (APEs), and trialkylphosphates. Several of the compounds were detected only in the secondary effluent and not in the renovated water. A few others—Diazinon, clofibric acid, and tributylphosphate—decreased in concentration with soil passage, but were detected in the renovated water. The APEs appeared to undergo rather complex transformations during ground filtration. They appeared to be completely removed with soil percolation during Period 1, but during Period 2, two isomers were found during soil passage while others were removed.

The results of the organics studies thus show that a wide spectrum of trace organics occurred in the secondary effluent, that some of these organics were attenuated more in the basin-soil-aquifer system than others, that the renovated water also contained a wide spectrum of trace organics, and that a chlorine dose of 1.5 mg per liter did not have a significant effect on the concentrations of the trace organics detected in either the effluent or the renovated water. Since nitrogen removal and TOC reductions in the soil-aquifer system were essentially the same for the nonchlorinated and chlorinated effluent, chlorination had no adverse effects. The chlorination lowered the fecal coliform concentrations in both effluent and renovated water. However, if a zero fecal coliform count in the renovated water is the

**Table 8.7** Average TOX Concentrations and TOX/TOC Ratios for Secondary Effluent in Basins and Renovated Water from 18-m Well

| | Period 1 | | Period 2 | |
|---|---|---|---|---|
| | *TOX (μg/L)* | *TOX/TOC (mol Cl/mol C)* | *TOX (μg/L)* | *TOX/TOC (mol Cl/mol C)* |
| Secondary effluent (average for basin) | 84 | 0.0031 | 142 | 0.0050 |
| 18-m well | 65 | 0.0069 | 55 | 0.0059 |

main objective, it would be better to chlorinate the renovated water after it is pumped from the wells, because this water has a much lower suspended solids content, which increases the efficiency of the chlorination.

## PRE- AND POSTTREATMENT GROUNDWATER RECHARGE

The disposition and use of the renovated water are key elements in the design of a rapid-infiltration system and the required pre- and posttreatment of the water. Ideally, the most economical and effective total treatment system starts with a formulation of the quality requirements of the renovated water, whether for unrestricted irrigation, drinking, recreational lakes, or other purposes. Then the entire sequence of pretreatment, soil-aquifer treatment, and posttreatment is designed so that renovated water with the desired quality is produced at minimum cost and minimum adverse environmental impact. Unfortunately, rapid-infiltration systems often are added as an afterthought. The sewage treatment plant is already in existence (usually primary and secondary treatment), a problem develops with the effluent discharge or there is an interest in reuse that requires a better-quality effluent than that of the treatment plant, and a rapid-infiltration system is considered.

Since soil-aquifer systems can handle much more BOD than applied with secondary sewage effluent, there is no need to have biological treatment (trickling filter or activated sludge) prior to infiltration. Thus, if the entire system is designed and built from scratch, the sewage treatment plant only needs to apply primary treatment, which can save as much as 50 percent of the pretreatment costs. Primary effluent may even be more desirable than secondary effluent for rapid-infiltration systems because the higher organic carbon content of the effluent could enhance denitrification in the soil beneath the infiltration basins [9] and biodegradation of synthetic organics through cometabolism or secondary utilization [21, 22, 23, 24, 25]. Primary effluent, however, contains more suspended solids than secondary effluent, which will accelerate soil clogging in the infiltration basins. However, the solids tend to be quite biodegradable, so that hydraulic loading rates may not be significantly different [26, 27].

The suspended solids content of the primary effluent can be reduced by further settling or by flowing the effluent through a vegetated surface (overland-flow system) before it enters the infiltration basins. A presedimentation basin can also be used. However, such basins should be large enough to give the desired reduction in suspended solids content but small enough to avoid growth of algae in the water. Unicellular algae like *Carteria klebsii* in wastewater can severely reduce infiltration rates of rapid-infiltration basins by clogging the bottom and by causing precipitation of calcium carbonate due to the high pH resulting from uptake of carbon dioxide by photosynthesizing algae.

The best solution probably would be to give the primary effluent some chemical treatment before applying it to infiltration basins. Lime precipitation, as is done in the Dan Region Project south of Tel Aviv [28], not only gives clarification, but the high pH caused by the lime causes precipitation of phosphates, volatilization of ammonia, disinfection, and precipitation of heavy metals and other impurities. The resulting primary effluent thus has a high dissolved BOD, while nitrogen, phosphates, microorganisms, and metals are significantly reduced. It should, therefore, be quite suitable for rapid-infiltration (see Chapter 15).

Where renovated water from rapid-infiltration systems is recovered by wells or drains (see Figure 8.2), posttreatment of the renovated water is possible. Such systems can be designed to give the most economical combination of pretreatment of the raw wastewater, the quality improvement obtained by soil-aquifer treatment, and any posttreatment of the renovated water necessary to meet the quality criteria for the final product water. Where posttreatment is needed, the system in Figure 8.2, bottom (with drains if the water table is shallow) is preferable to the one in Figure 8.2, center because it yields renovated water only. The water yielded by systems as in Figure 8.2, center is diluted with native groundwater, which would needlessly increase the capacity of the posttreatment facility.

If the wastewater is to be reused for general, unrestricted irrigation, the final product water should meet the public health, agronomic, and aesthetic requirements for such use. The public health criteria normally are formulated by state health departments or other regulatory agencies. For unrestricted irrigation, they typically require essentially zero or low fecal coliform counts, low suspended solids content, and low BOD levels. California (California Administrative Code, Chapter 4, Title 22, Division 4, Environmental Health), for example, requires that the wastewater be adequately disinfected (7-day medium coliform concentration not in excess of 2.2/100 mL and 30-day maximum coliform concentration not in excess of 23/100 mL), well oxidized (organic matter stabilized), coagulated (colloidal and finely divided suspended matter removed), clarified (clarification of oxidized, coagulated effluents by further settling), and filtered (clarified wastewater that has passed through soils or a filter media). Proposed regulations in Arizona simply require that the geometric mean of the fecal coliform bacteria be less than 25 per 100 mL, that there be no turbidity, and that tape- and roundworm eggs and entamoeba cysts be nondetectable [29]. If the water is also to be used for irrigation of playgrounds, there is the additional requirement that virus concentrations be less than 125 PFU per 40 L.

Agronomic requirements for unrestricted irrigation can be taken from accepted irrigation water quality standards, such as those in Tables 8.8 [30] and 8.9 [31]. These standards show that most sewage effluents will meet chemical water quality standards for irrigation water, except for nitrogen, which for sewage typically will be well in the range of "increasing problems." These problems include crop lodging, delay in harvestability, reduced sugar

Table 8.8 Guidelines for Interpretation of Water Quality for Irrigation [30]

| *Problems and Quality Parameters* | *No Problems* | *Increasing Problems* | *Severe Problems* |
|---|---|---|---|
| Salinity effects on crop yield | | | |
| Total dissolved-solids concentration (mg/L) | <480 | 480–1920 | >1920 |
| Deflocculation of clay and reduction in infiltration rate | | | |
| Total dissolved-solids concentration (mg/L) | >320 | <320 | <128 |
| Adjusted sodium adsorption ratio (SAR) | <6 | 6–9 | >9 |
| Specific ion toxicity | | | |
| Boron (mg/L) | <0.5 | 0.5–2 | 2–10 |
| Sodium (as adjusted SAR) if water is absorbed by roots only | <3 | 3–9 | >9 |
| Sodium (mg/L) if water is also absorbed by leaves | <69 | >69 | |
| Chloride (mg/L) if water is absorbed by roots only | <142 | 142–355 | >355 |
| Chloride (mg/L) if water is also absorbed by leaves | <106 | >106 | |
| Quality effects | | | |
| Nitrogen in mg/L (excess N may delay harvest time and adversely affect yield or quality of sugar beets, grapes, citrus, avocados, apricots, etc.) | <5 | 5–30 | >30 |
| Bicarbonate as $HCO_3$ in mg/L (when water is applied with sprinklers, bicarbonates may cause white carbonate deposits on fruits and leaves) | <90 | 90–520 | >520 |

content in sugarbeets and sugar cane, inferior size or quality of fruits or vegetables, and increased susceptibility to disease [32]. Aesthetic aspects are important where the irrigation will be on a large scale in relatively densely populated areas. The aesthetic aspects primarily involve the appearance (clarity) and odor of the final product water. Renovated water from a soil-aquifer treatment system has the psychological advantage that it comes from a well after having passed through the soil and aquifer. Thus, it evokes less of an association with sewage than water from a complete treatment plant where raw sewage goes in at one end and the final product water comes out at the other.

Table 8.9 Recommended Maximum Limits in mg/L for Trace Elements in Irrigation Water [31]

| *Element* | *Permanent Irrigation of All Soils* | *Up to 20-yr Irrigation of Fine-Textured Neutral to Alkaline Soils (pH 6–8.5)* |
|---|---|---|
| Aluminum | 5 | 20 |
| Arsenic | 0.1 | 2 |
| Beryllium | 0.1 | 0.5 |
| Boron-sensitive crops | 0.75 | 2 |
| semitolerant crops | 1 | |
| tolerant crops | 2 | |
| Cadmium | 0.01 | 0.05 |
| Chromium | 0.1 | 1 |
| Cobalt | 0.05 | 5 |
| Copper | 0.2 | 5 |
| Fluoride | 1 | 15 |
| Iron | 5 | 20 |
| Lead | 5 | 10 |
| Lithium: citrus | 0.075 | 0.075 |
| other crops | 2.5 | 2.5 |
| Manganese | 0.2 | 10 |
| Molybdenum | 0.01 | 0.05[a] |
| Nickel | 0.2 | 2 |
| Selenium | 0.02 | 0.02 |
| Vanadium | 0.1 | 1 |
| Zinc | 2 | 10 |

[a] For acid soils only.

The most economical and effective treatment system to produce renovated sewage water that meets the public health, agronomic, and aesthetic requirements for unrestricted irrigation may be primary treatment, additional sedimentation or overland flow, soil-aquifer treatment with a rapid-infiltration system and wells or drains for collecting the renovated water, and chlorination. The chlorination is done at the end because it will have more effect on renovated water with its low suspended solids content than on primary effluent with its high suspended solids content. However, chlorination in the sewage treatment plant is acceptable because mild chlorination produces no significant increases in chlorinated and other trace organics and does not seem to unfavorably affect denitrification and decomposition of organics in the soil below the infiltration basins.

The soil-aquifer treatment system for producing renovated water for unrestricted irrigation can be operated with short, frequent flooding periods to achieve essentially complete conversion of the nitrogen in the effluent to nitrate in the renovated water and no nitrogen removal, or with longer flooding periods to maximize denitrification. The former will be best where nitrogen must be conserved as much as possible, or where nitrogen concentrations in the renovated water will be reduced by blending with other irrigation water. Operating the rapid-infiltration system to achieve maximum denitrification will be more appropriate where the renovated water will be used without dilution for irrigation of crops that are adversely affected by too much nitrogen. If the renovated water is to be used for recreational lakes with full body contact, the primary treatment should be followed by lime precipitation to reduce the phosphate level in the primary effluent. This action would also reduce the phosphate concentration in the renovated water and minimize growth of algae in the recreational lakes.

Where soil-aquifer treatment is to be used in conjunction with other technologies for producing water for potable purposes, the best treatment sequence could be primary treatment, lime precipitation, soil-aquifer treatment, disinfection, activated carbon adsorption, reverse osmosis if needed, and final disinfection if needed. The lime precipitation is added to obtain reductions in nitrogen, phosphorus, metals, and microorganisms before the effluent goes into the ground. The soil-aquifer treatment system should be operated for maximum denitrification to minimize the nitrogen concentration in the renovated water. Following soil-aquifer treatment, the water should be disinfected. Ozonation may be best because it would also break down organics and increase removal of trace organics by the subsequent activated carbon filtration. Reverse osmosis may be needed where the salt content of the water is too high for drinking or as a last step to remove final traces of nitrogen, organics, metals, and other impurities. The advantage of including the soil-aquifer treatment system in the total treatment train is that it removes essentially all biodegradable organic carbon, in addition to other constituents. Thus, the activated carbon filtration of the renovated water from the soil-aquifer treatment system with its TOC content of about 2 mg per liter can be done much more effectively and economically than that of secondary effluent from conventional sewage treatment plants with its TOC content of about 10 to 20 mg per liter [33].

Where the rapid-infiltration system is to be used for general groundwater recharge with treated sewage effluent or other wastewater and there are wells in the aquifer that pump water directly for drinking without further treatment, the pretreatment requirements of the wastewater prior to infiltration become much more stringent. In the extreme, the wastewater may have to be treated so that it meets drinking water quality standards before it goes into the rapid-infiltration basins. This means primary and secondary treatment, lime precipitation, ammonia stripping and/or selective ion adsorption for removing nitrogen, disinfection, activated carbon filtration, and possibly reverse osmosis [34]. The cost of this complete treatment probably will be much higher than the cost of a system where most of the organic carbon and many of the other impurities are removed by soil-aquifer treatment using a rapid-infiltration system, and where the movement of the renovated water in the aquifer is controlled with wells or drains to protect native groundwater resources outside the soil-aquifer treatment system.

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Separata de "Recarga Artificial de Acuíferos"
Boletín de "Informaciones y Estudios"
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## 21. RECUPERACION DE AGUAS RESIDUALES POR RECARGA ARTIFICIAL

Las aguas residuales, principalmente las domésticas, son una interesante fuente potencial —en ciertos casos real— de recursos de agua para usos diversos; las aplicaciones para riego son las más importantes, aunque incluso puede llegarse a usos domésticos y de bebida. Un detallado análisis ha sido publicado en 1977 por Shuval (408). Esta reutilización tiene un límite en la salinidad no eliminable que acumula el agua (150 a 300 mg/l por uso) en la posibilidad técnico-económica de depuración suficiente y garantizada y en la aceptación de ese agua, tal cual o mezclada con otras aguas, por parte de los usuarios. Aunque es posible aplicar técnicas de desalinización y éstas se prevén en ciertas áreas, en general el coste es excesivamente alto; sólo en condiciones extremas es aconsejable.

El agua residual tratada es en muchas ocasiones una alternativa que puede competir favorablemente en coste con otras fuentes de suministro de agua (183), si se resuelven los problemas de calidad inherentes.

La recarga artificial de aguas residuales presenta los siguientes campos de interés:

- Efectuar un tratamiento final del agua, previamente depurada.
- Mezclar el agua de reutilización con el agua natural subterránea.
- Romper la relación aparente visible entre el agua residual y el agua reextraida.
- Obtener tiempos de detención en el terreno que garanticen la buena calidad bacteriológica y virológica del agua.
- Acomodar la disponibilidad de agua residual a las variaciones de la demanda, en especial cuando se trata de usos de riego estacionales con aguas residuales producidas más o menos continuamente a lo largo de todo el año.
- Crear un almacén de agua dulce en un acuífero de agua salina o salada.
- Evacuar aguas residuales para evitar o reducir parte del tratamiento necesario para el vertido o tratar de eliminar ciertas substancias específicas, tales como el fósforo, que son fácilmente retenidas en el terreno.

La recarga se puede hacer tanto con obras de superficie como mediante pozos. Las obras de superficie pueden ser canales o balsas, o bien campos regados por aspersión (351), o consistir en terrenos agrícolas en uso, aplicando sobrerriego por inundación o por aspersión (99). Con utilizaciones prolongadas deben considerarse posibles cambios en el suelo (292). También se pueden recargar mediante pozos (460).

Los aspectos de colmatación se han tratado anteriormente (sección 13). Son tanto más graves cuanto menos tratamiento tenga el agua, más turbia sea, más crecimientos orgánicos origine y más fino el terreno. Es frecuente que se puedan presentar problemas de incompatibilidades químicas, en especial por precipitación de óxido férrico, sulfuros o carbonatos, según las circunstancias, o por producción de burbujas de gas (aire, gas carbónico, metano).

La mayor parte de experiencias se han realizado en regiones con cierto grado de aridez, o aisladas, donde la conservación de agua es más perentoria. Entre estas zonas, donde se han realizado más experiencias es en Estados Unidos, en muy diversos lugares de su geografía (California, Arizona, Florida, Long Island-New York, etcétera) y en Israel, aunque también existen experiencias en Inglaterra (areniscas y creta) y en Cataluña. En cierto modo, la recarga en Holanda y Alemania a partir de aguas contaminadas de los ríos Rhin y Ruhr participa en parte de aspectos de reutilización de aguas residuales.

En el momento actual los caudales de recarga de aguas residuales son aún más pequeños. Por ejemplo, unos

50.000 $m^3$/dia en Whittier Narrows (Los Angeles, California), 7.500 $m^3$/dia en Palo Alto (California) (380), 2.000 $m^3$/dia durante las experiencias de Bay Park (Long Island, New York) (169) (445), 8.000 $m^3$/dia en Stafforshire (Inglaterra) (119), 15 $hm^3$/año en la región de Dan (área metropolitana del Tel-Aviv, Israel) (241), con proyectos para llegar a 110 $hm^3$/año (407). El proyecto de Flushing Meadows (Arizona) ha sido objeto de detallados estudios y ensayos (90) (91) (fig. 46). En Santee (California) la recarga artificial en un aluvial es una de las etapas en un proceso progresivo para ensayar el posible uso del agua residual para usos recreativos cada vez con mayor contacto con el hombre.

Las condiciones para que la recarga se realice en buenas condiciones han sido resumidas por Bouwer (87) y con algunas modificaciones son:

— Que el nivel freático y el techo de la franja capilar no alcancen el nivel del suelo, con el fin de que exista un suficiente tránsito aerobio por el medio no saturado y para permitir además que el suelo se drene rápidamente en un proceso de secado.

— Que el agua tenga suficiente tiempo de residencia y recorrido en el terreno antes de su extracción, en general de varios meses y por lo menos de algunos metros.

— Que en lo posible el agua recargada no se extienda por el acuífero afectando a áreas con agua natural, para lo cual la zona de recarga debe estar en depresión o rodeada por los pozos o drenes de extracción.

Diversos trabajos analizan las condiciones y resultados desde el punto de vista químico y operativo (86) (171), que en su mayor parte ya han sido expuestos en apartados anteriores.

El movimiento del agua residual es aproximadamente predecible a partir del conocimiento de la forma del medio receptor, distribución de permeabilidades y coeficientes de almacenamiento y gradientes piezométricos, aunque en detalle las heterogeneidades del terreno juegan un papel que puede ser dominante, tanto para la circulación y extensión como para la recuperación. Con frecuencia la circulación se realiza por estratos o nive-

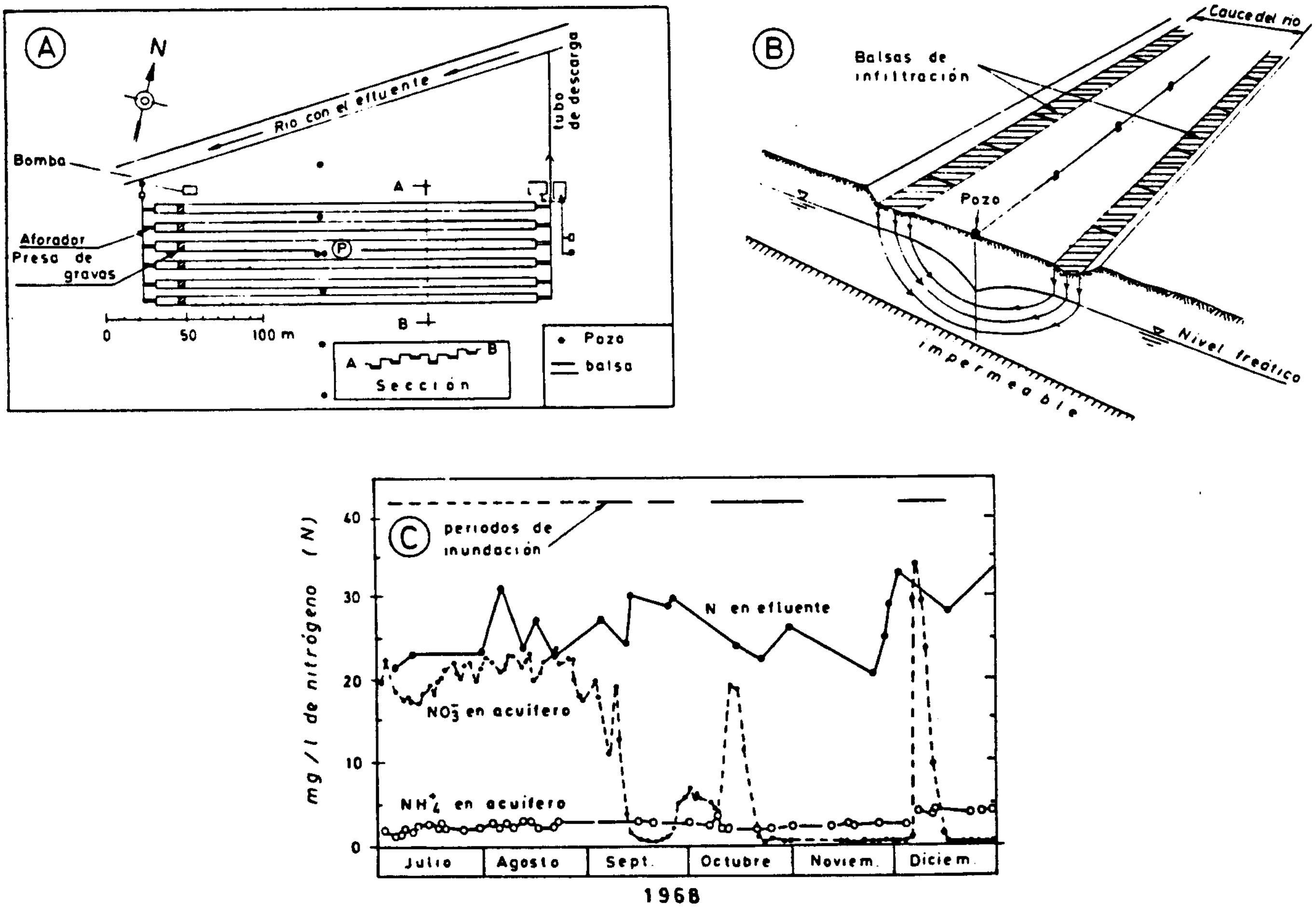

*Fig. 46.—Instalación de recarga de aguas residuales tratadas de Flushing Meadows (Phoenix, Arizona) en el cauce del Salt River (89). A es la disposición general, con balsas de 214 m. de largo y 6,1 de ancho, con 0,3 m. de agua. B señala una posible disposición para la explotación. C indica la evolución del nitrógeno combinado existente en el efluente al pasar al acuífero, tal como se observa en el pozo P, de 9,2 m. de profundidad; al establecerse un medio suficientemente anaerobio, el $NO_3$ se reduce en gran parte a $N_2$ no reactivo y $NH_4$, que queda retenido por cambio iónico; en los periodos en que no hay recarga el oxígeno penetra en el terreno y el siguiente frente de recarga aparece con una alta concentración de $NO_3$.*

les de mayor permeabilidad (139) (42) (65), mientras los menos permeables apenas reciben recarga y en roca consolidada dicha circulación puede reducirse a sólo las fisuras; las observaciones en creta señalan un movimiento preferente por planos de estratificación (34). El agua extraida puede estar notablemente diluida por dispersión y mezcla y porque el flujo del agua aleja el agua introducida del pozo de inyección; por ejemplo, se señala que en experiencias en el Condado de Orange, California, en los pozos duales de recarga sólo se recupera el 10 % del agua inyectada (25), y en una experiencia en el Besòs-Riera d'Horta, cerca de Barcelona, la recuperación fue prácticamente nula. Ello no quiere decir que ese agua se pierda, pues ha sido puesta en almacenamiento en el acuifero y, por tanto, está disponible como reserva si no se escapa por limites de control de potencial; no obstante, buena parte se mezcla con el agua nativa y, si ésta es de mala calidad, el agua puesta en almacenamiento no es utilizable regionalmente más que si el volumen total recargado es conmensurable con el almacenamiento en el acuifero.

Las técnicas de depuración de aguas residuales por tratamiento e infiltración en el terreno han sido objeto de numerosas publicaciones y ensayos, y posiblemente de muchas más numerosas actuaciones, a veces poco controladas, sobre las que hay pocos datos y conocimientos. Aunque una utilización indiscriminada puede originar graves problemas de contaminación, con una operación correcta se consigue una interesante y adecuada alternativa de evacuación de aguas residuales, que al mismo tiempo produce una recarga artificial (88) (89) (210) (279). Estos sistemas parecen también funcionar correctamente en paises frios, incluso en invierno (40). El método de surcos parece mejor que el de inundación y el de aspersión, en cuanto a la duración del dispositivo de infiltración con similares resultados en cuanto al tratamiento conseguido.

En instalaciones nucleares que tratan combustibles irradiados o de otro tipo, se inyectan en el terreno aguas con pequeños y moderados contenidos en ciertos radioisótopos, entre los que puede destacar el tritio, con la intención de un largo almacenamiento temporal y asi dar lugar a su descontaminación, recuperando el agua como en la NRTS de Idaho o en Handford (Washington). Los resultados obtenidos no son claros.

Muchas técnicas de inyección profunda de aguas residuales, en realidad son métodos de evacuación y almacenamiento en el terreno, en general de pequeños volúmenes, con la excepción de los pozos de inyección de aguas salinas y salmueras asociadas a la extracción de petróleo o a la mineria potásica. Muchos problemas técnicos de colmatación, pretratamiento, construcción y operación han sido tratados y resueltos desde hace tiempo por la industria extractiva del petróleo, que inyecta agua salina o salada para evacuarla y para elevar la presión de los yacimientos. Existen varias decenas de miles de pozos en operación, pero los resultados son a veces poco accesibles, mal conocidos u objeto de secreto industrial. No obstante, muchas aguas residuales, por su naturaleza fisico-quimica o incluso radioactiva, presentan problemas diferentes (125) (240) que no son fácilmente correlacionables con los de la inyección profunda de aguas salinas. Esta inyección profunda puede ser peligrosa (357) si no se realiza con precaución y siguiendo normas y recomendaciones bien estudiadas (456) (458). Muchas veces es poco recomendable.

La inyección profunda de aguas residuales domésticas tratadas o de aguas de escorrentia es propiamente recarga artificial, pero hay escasas experiencias. Se realiza en Florida, cerca de Miami, y en el área de la Bahia de Tampa, introduciendo el agua en niveles calcáreos profundos muy karstificados y que contienen agua salina o salada (187) (188) (189) (258) (454). Este mismo acuifero en otros lugares (Pensacola, Belle Glade) recibe agua residual industrial ácida con altos nitratos y DQO; la acidez ayuda a mantener y aun aumentar la permeabilidad local. Las reacciones anaerobias son notables y pueden formarse gases ($N_2$, $CH_4$, $CO_2$), y el agua se carga fuertemente en ión calcio (258). Sin embargo, las pruebas de extracción muestran que al pasar el tiempo el grado de interacción y la modificación del agua es menor, lo cual puede querer decir que al progresar el ataque el medio sólido y el ambiente se van convirtiendo en un medio cada vez menos reactivo.

# TECNOLOGIAS DE BAJO COSTE PARA LA DEPURACION Y RECICLADO.

## UNA EXPERIENCIA ESPAÑOLA

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Dentro de los distintos procesos empleados hoy en día para la depuración de las aguas residuales urbanas, existe una serie de técnicas aplicables que, por su sencillez de operación y reducido o nulo consumo energético, reciben la denominación genérica de «Tecnologías Blandas o de Bajo Coste».

Dada la rusticidad de algunos de estos sistemas podría en principio pensarse que se trata de métodos destinados a regiones subdesarrolladas, si bien la realidad es que su empleo se viene produciendo desde hace bastantes años en países como Estados Unidos, Alemania, Francia, Israel, etcétera.

En España estas tecnologías están empezando a tomar gran auge de aplicación en los últimos años, ya que, junto a su simplicidad y economía, presentan una elevada fiabilidad de funcionamiento. Además, producen beneficios derivados de eliminar de los cauces públicos unas notables fuentes de contaminación de los mismos, y presentan la ventaja adicional de que las aguas tratadas puedan ser recicladas para su uso en agricultura.

Su ámbito de aplicación se circunscribe generalmente a pequeñas poblaciones (menores de 25.000 habitantes), si bien en algunas ocasiones su utilidad se ha extendido a importantes núcleos urbanos. Así, por ejemplo, en Israel, una parte importante de las aguas residuales generadas por TelAviv-Jaffo (un millón de habitantes) son tratadas en un sistema de lagunado.

Los procedimientos más aplicados y en consecuencia de los que se tiene un mejor conocimiento, son el lagunado, el filtro verde, los filtros de turba y los contactores biológicos rotativos, si bien estos últimos se incluyen por su reducido consumo y su facilidad operativa, aunque en realidad pueden considerarse dentro del campo de la tecnología convencional.

### LA DEPURACION DE AGUAS MEDIANTE LAGUNAS

La depuración de las aguas residuales mediante lagunas se basa en la sedimentación y mineralización natural de la materia orgánica particulada, combinando este fenómeno físico-químico-biológico con la degradación de la fracción soluble, por medio de una reducida cantidad específica de microorganismos que actúan durante un dilatado período de tiempo.

En función de la clase de organismos presentes en las aguas de una laguna, ésta puede clasificarse en tres tipos: anaerobia, facultativa y aerobia, describiéndose a continuación con más detalle el sistema facultativo, por ser en realidad un proceso que de alguna manera engloba a los otros dos.

En una laguna facultativa podemos distinguir tres zonas en función de la profundidad.

En el fondo, como consecuencia

- Necesidad de grandes extensiones de terreno.
- Garantía de impermeabilidad para no contaminar acuíferos próximos.
- Condiciones climatológicas adecuadas.

## EL FILTRO VERDE

El filtro verde puede definirse como una instalación natural, en la que las aguas residuales son vertidas durante todo el año, consiguiéndose, por una parte, la depuración de los efluentes; y, por otra, el fomento del crecimiento de especies arbóreas maderables.

El proceso se basa en que el suelo es un soporte mineral de sales, aire, agua, microorganismos y las propias raíces de las plantas, cuya acción conjunta provoca de una manera natural la depuración de los efluentes contaminados, a través de acciones físicas, químicas y biológicas, aprovechándose a su vez el estado de semisaturación en que se encuentra el terreno para propiciar el desarrollo de plantas freatofitas, generalmente del género Populus (chopos).

Las aguas recibidas por el terreno sufren una serie de fenómenos físicos, químicos y biológicos que en esencia pueden sintetizarse en:

- Acciones físicas. La principal de estas acciones es la filtración, fenómeno que como es lógico depende de la granulometría y textura del suelo. Así en un terreno arcilloso, de granulometría fina, la filtración será muy efectiva pero excesivamente lenta, mientras que en otro altamente fisurado (karstificado) ocurrirá lo contrario: alta velocidad de paso pero escasa retención particular. Es pues conveniente disponer de terrenos de permeabilidad media, huyendo de suelos arcillosos y de los arenosos, con alto contenido en gravas o fisurados.
- Acciones químicas. Las aguas residuales urbanas contienen nitrógeno, fósforo, calcio, magnesio, sodio, potasio y diversas sales: como cloruros, sulfatos, etcétera.

Parte de estos elementos son asimilados por la vegetación para su propia nutrición, mientras que otros quedan libres y, junto con el exceso existente de los primeros, se incorporan al terreno. Según las reacciones producidas pueden ocurrir fenómenos de fijación al suelo o de percolación.

- Acciones biológicas. La materia orgánica contenida en los influentes sufre un contacto con las bacterias aeróbicas presentes en los primeros 20 centímetros de potencia del suelo.

Una parte de este componente orgánico es utilizado para la alimentación bacteriana, otra se oxida para producir energía de la masa biológica; y, por último, una tercera parte no es utilizada y percola.

Si las aguas recibidas por la chopera tienen un fuerte componente de tipo industrial deberá estudiarse cuidadosamente la implantación del sistema, ya que los efluentes de algunas instalaciones (químicas, papeleras, almazaras), pueden suponer un alto grado de toxicidad para plantación arbórea.

Para el cálculo de la superficie necesaria de filtro verde suele indicarse el ratio de una hectárea por cada 200 habitantes, proporción basada principalmente en experiencias centroeuropeas.

Si tenemos en cuenta el fuerte desequilibrio entre la relación precipitación/evapotranspiración existente entre Europa Central y nuestro país, es evidente que el citado ratio podrá ser mejorado, reduciéndose en consecuencia la superficie específica necesaria.

EL filtro verde presenta una serie de ventajas tales como:

- Rusticidad y sencillez operativa.
- No es necesario que los terrenos sean adquiridos, pudiendo estar en arrendamiento.
- Se produce un beneficio económico por la venta de la madera.
- Los terrenos son totalmente recuperables.
- No hay averías, olores ni mosquitos.

Entre los inconvenientes destacan fundamentalmente las grandes superficies de terreno que son necesarias.

## LOS FILTROS DE TURBA

En relación con los filtros de turba, puede decirse que el elemento base del proceso, la turba, constituye el primer paso en la formación de los carbones minerales, estando formada por masas vegetales que sufrieron una descomposición biológica en medio reductor y bajo un exceso de agua.

La depuración por medio de la filtración de las aguas residuales a través de turba se basa principalmente en aprovechar las propiedades de absorción y adsorción de de la acumulación de materia orgánica particulada originada por la decantación, se crea un estado de anoxia que da lugar al crecimiento de una fauna de tipo anaerobio. La fermentación que tiene lugar libera sustancias solubles y gases tales como metano, sulfhídrico, carbónico, etcétera, que ascienden por la masa de agua.

En la parte superior de la laguna existe una zona de agua con contenido de oxígeno suficiente para permitir la existencia de formas de vida de tipo aerobio. En esta zona se desarrollan algas que contribuyen a la oxigenación del agua.

Las bacterias presentes utilizan el oxígeno producido por las algas y desprenden $CO^2$ que es a su vez utilizado por éstas, cerrando así el ciclo simbiótico.

En esta zona oxigenada se sintetiza nueva materia celular, se oxidan los gases ascendentes y se nitrifican los elementos orgánicos disueltos.

Entre las dos capas descritas, anaerobia en el fondo, aerobia en superficie, existe otra intermedia, poblada por bacterias facultativas que actúan en función de las disponibilidades de oxígeno, siendo esta zona la que da nombre a este tipo de lagunas.

La profundidad normal de las lagunas de esta clase suele estar en torno a 1,30-1,50 metros.

Para el cálculo de la superficie de estanque facultativo necesario existen diversos métodos basados en la temperatura, latitud, radiación solar, etcétera, pudiendo citarse como más representativos el del World Bank, Israeli, Frances, Sudafricano, Mc Garry y Pescod, Indio, Oswald y Gotaas, etcétera.

Las lagunas anaerobias se caracterizan porque prácticamente toda la masa de agua (excepción hecha de una pequeña lámina superficial) se encuentra en estado de anaerobiosis como consecuencia de la alta carga que soportan, siendo su profundidad mayor de los tres metros.

Su principal cualidad es funcionar por una parte como decantador, y por otra como reactor anaerobio.

Ahora bien, si tenemos en cuenta que la actividad bacteriana anaerobia cesa a partir de un límite inferior de temperatura (15° C según Oswald; 10° C según Van Eck), a efectos de cálculo puede suponerse que la laguna actúa solamente como un decantador, asignándosele pues un rendimiento en reducción de DB05 del orden del 30-40 por 100.

No obstante, existen métodos de cálculo (World Bank, Israeli, etcétera), basados en la carga superficial y másica tolerables según la temperatura. El tiempo de retención varía entre dos y cinco días, si bien según en World Bank la permanencia óptima es de dos días únicamente.

Finalmente, las lagunas aerobias, también llamadas de maduración, mantienen toda la masa de agua en estado de oxigenación como consecuencia de su reducida profundidad (0,80-1,10 metros) y la baja carga a que son sometidas. Su principal misión es la de descontaminar las aguas de gérmenes patógenos gracias a la radiación ultravioleta solar, debiendo ser el tiempo de retención de cinco días como mínimo.

En una estación depuradora por lagunado generalmente se combinan los tres tipos de lagunas, estableciéndose un sistema en serie anaerobia-facultativa-maduración

Algunos procedimientos (método francés) prescinden del primer paso anaerobio y fraccionan la laguna facultativa en dos unidades en serie, mientras que otros métodos recurren a la recirculación cola cabecera de sistema, precindiendo igualmente del paso anaerobio

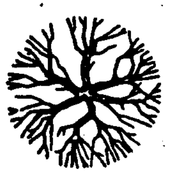
## Ventajas e inconvenientes del lagunado

El lagunado, como cualquier otro método de tratamiento de aguas residuales, presenta una serie de ventajas e inconvenientes que pueden aconsejar o no su utilización:

Entre las primeras caben destacarse:

- Consumo energético nulo, a no ser que sea necesario bombear hasta la planta.
- Reducido mantenimiento por personal no cualificado.
- Ausencia de averías.
- Carencia de equipos mecánicos.
- Gran inercia, que permite la adaptación a cambios de caudal contaminantes.
- Alto poder descontaminador de gérmenes patógenos.
- Facilidad constructiva.

En lo que respecta a los inconvenientes pueden reseñarse:



este carbón mineral, cualidades ya empleadas en diversos procesos industriales.

La materia en suspensión contenida en el agua residual es retenida en la turba, quedando el influente clarificado. Pero este carbón no sólo constituye un medio filtrante, sino que además desarrolla en su superficie una actividad bacteriana que degrada la materia orgánica. El rendimiento en reducción de materia orgánica ronda en torno al 75 por 100 o más, pudiendo soportar puntas de caudal de 1,5 veces el flujo normal.

El esquema del proceso es, en esencia, semejante al de un tratamiento convencional, constando de tratamiento primario, secundario y terciario (opcionalmente).

El tratamiento primario tiene como misión fundamental retirar del agua residual la mayor parte posible de sólidos en suspensión para retardar el fenómeno de la colmación de la turba, así como eliminar los aceites y grasas que puedan interferir en la vida biológica y crear películas y capas flotantes.

Consta esta fase del proceso de una reja gruesa de desbaste, tras la cual el agua es enviada a un filtro autolimpiable.

Este tipo de filtro, construido en acero inoxidable, consiste en un panel inclinado en el que existen multitud de finas ranuras por las que se filtra el agua. Los sólidos finos son retenidos y, debido a la pendiente del panel, van delizándose hasta caer a un cajón colector. Este filtro conviene que esté alojado en una caseta, que le proteja contra las heladas y otras inclemencias atmosféricas.

El agua, ya desprovista de gran parte de la materia en suspensión, es enviada a un separador de grasas donde es remansada, separándose los aceites por diferencia de densidad. Con esta fase puede decirse que concluye el tratamiento primario.

Las aguas residuales que ya han sido sometidas al tratamiento primario descrito, pasan seguidamente al tratamiento secundario, constituido por los filtros de turba.

Estos filtros están formados por tres capas que, de abajo a arriba, son: una de grava, otra de arena y una final de turba.

El terreno donde se asienten debe ser impermeable, para garantizar la no contaminación de las aguas subterráneas. Si no se cumple esta condición hay que recurrir a impermeabilizar con arcilla, láminas plásticas u hormigón.

La turba debe ser renovada con periocidad, ya que la colmatación a que se ve sometida produce su agotamiento. El período normal de funcionamiento suele ser de cinco años, si bien de experiencias francesas se deduce que, tras siete años, sigue funcionando perfectamente.

Los filtros se disponen en varias unidades, teniendo unas en funcionamiento y otras en conservación.

La alimentación se produce por medio de una tubería superior que distribuye el agua en el filtro. El influente, tras su paso por la turba, es recogido en el fondo por una tubería drenante o un canal que lo evacua o una arqueta de salida.

Los lechos de turba tienen una indudable ventaja sobre los sistemas tecnológicos convencionales: admiten sensibles variaciones de caudal sin afectar prácticamente al rendimiento, lo que les hace muy aconsejables en localidades con puntas de caudal temporales, por ejemplo por aumentos de población en fines de semana.

EL tratamiento terciario es opcional y consiste en someter al efluente de los lechos de turba a un lagunado aerobio, con el fin de eliminar la mayor parte de patógenos. Esta eliminación se produce fundamentalmente por la acción de la radiación ultravioleta, pudiendo también utilizarse la cloración como tratamiento terciario.

El cálculo de la superficie de filtro necesaria se lleva a cabo en función de la clase de turba empleada. Así, si se trata de turba negra un ratio razonable suele ser el de alimentar el filtro con un caudal de 30-40 l/m$^2$/hora, mientras que si la turba utilizada es la denominada «rubia» puede llegarse a los 60-70 l/m$^2$/hora.

El tratamiento de las aguas residuales mediante lechos de turba presenta las siguientes ventajas:

- Gastos de instalación razonable.
- Bajo coste de explotación y mantenimiento.
- Reducido consumo de energía. (Nulo si no se necesita impulsión inicial).
- Facilidad de operación. No es necesario personal cualificado.
- Fiabilidad de funcionamiento.
- Fácil adaptación estética al medio natural.
- Rápida adaptación a bruscas variaciones de carga de caudal sin variar sensiblemente los rendimientos.
- Ausencia de olores.
- Carencia de ruidos y mosquitos.
- Alta descontaminación bacteriana.

Con inconvenientes puede decirse que su campo de aplicación se circunscribe a localidades cuya población no sobrepase los 10.000 habitantes.

## CONTACTORES BIOLOGICOS ROTATIVOS

Los contactores biológicos rotativos (CBR) son una forma de depuración cuyo principio es similar al de los lechos bacterianos, aunque su funcionamiento sea sensiblemente diferente, englobando a los denominados biodiscos y a los biocilindros.

Los biodiscos constan en esencia de una batería de discos de PVC o polietileno, con diámetros que normalmente oscilan entre dos y tres metros, separados entre sí tres centímetros aproximadamente.

Los discos giran lentamente sobre un eje horizontal, que mantiene inmerso en el agua residual un 40 por 100 aproximadamente de la superficie de los mismos. En su fase de inmersión se va creando sobre las paredes de los discos una fina película biológica que llega a alcanzar unos dos milímetros de espesor. Esta película es oxigenada cuando, y debido al giro, la fase inmersa se encuentra en el exterior en contacto con el aire. Así pues, se establece un proceso bacteriano alimentación-respiración.

El sistema de biodiscos equivale en realidad a un reactor biológico en un tratamiento convencional, si bien ofrece algunas ventajas tales como:

- Menor espacio ocupado.
- Mejor homogeneización del agua residual.
- Menor consumo de energía.
- Condiciones de oxigenación garantizadas.
- No es necesaro recircular fangos del decantador secundario, ya que la masa bacteriana adherida a los discos está en constante crecimiento y cuando su espesor alcanza un punto límite, una parte de la misma se desprende, ayudada por la reacción anaerobia que tiene lugar en la zona más profunda de la lámina biológica.
- Mejor comportamiento ante la presencia de tóxicos, pues la flora bacteriana permanece inmersa en el agua un 47 por 100 del tiempo, mientras el 53 por 100 restante se encuentra en contacto con el aire y, por consiguiente, en condiciones de recuperarse.

A veces es conveniente, para mejorar el rendimiento depurador, establecer varios paquetes de biodiscos dispuestos en serie antes de la fase de decantación.

El sistema completo de una estación de tratamiento de este tipo es el tradicional en un sistema convencional; es decir, tratamiento primario-biodiscos-decantación, si bien algunas instalaciones adoptan como tratamiento previo uno de tipo anaerobio.

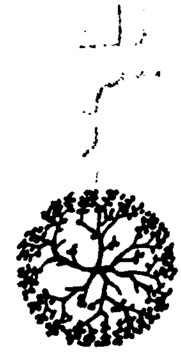
El tiempo de permanencia del agua en la cuba de los biodiscos es de dos o tres horas aproximadamente.

El cálculo de los biodiscos puede realizarse a través de varios métodos (Función de Monod, Hartmann, etcétera).

Una modificación de los biodiscos viene dada por los denominados biocilindros, formados por un cilindro en forma de jaula que alberga en su interior un relleno de elementos plásticos similares a los utilizados en los lechos bacterianos.

Su funcionamiento es idéntico al descrito para los biodiscos, si bien sus rendimientos son superiores.

Aparte de estas técnicas descritas existen otras que en la actualidad se encuentran en una fase más o menos desarrollada y entre las que pueden citarse la escorrentía superficial, el cultivo de plantas microfitas y macrofitas, el cultivo de peces, etcétera.



Ahora bien, hay que considerar que los métodos de cálculo y diseño de estos sistemas no son totalmente extrapolables de unos lugares a otros, pues factores tales como temperaturas, precipitación, vientos, radiación solar, etcétera, condicionan en gran manera el dimensionamiento de algunas de estas instalaciones.

## EXPERIENCIA EXPAÑOLA

Partiendo de esta premisa, y ante la necesidad de disponer de procedimientos de cálculo acordes con las condiciones españolas, con el fin de investigar y posteriormente aplicar a escala real estas tecnologías para la depuración de aguas residuales, la Consejería de Obras Públicas y Transportes de la Junta de Andalucía e INITEC, subscribieron un Convenio Marco de Cooperación a tal efecto, dado que en Andalucía se dan de forma acusada las condiciones necesarias para poder afirmar que la aplicación de estas técnicas puede suponer la solución real y económica de una gran parte de la contaminación de origen urbano, ya que de los 761 municipios andaluces, 726 son menores de 25.000 habitantes, situándose la mayoría en el medio rural.

Además, la utilización con fines agrícolas de las aguas tratadas puede representar un recurso actualmente desaprovechado y suponer, por otra parte, una sensible mejora de las aguas superficiales.

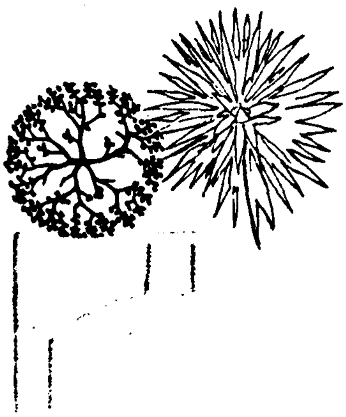
Una de las fases del Convenio incluye la construcción de una planta experimental de depuración de aguas residuales con tecnologías de bajo coste para, con los resultados en ella obtenidos y los suministrados por una serie de instalaciones a escala industrial, poder ajustar debidamente los parámetros de diseño para que las futuras depuradoras a implantar supongan una relación coste/rendimiento óptimo.

La planta experimental se encuentra situada en el término municipal de Carrión de los Céspedes junto a la Autovía A-49, en las proximidades de Sevilla y ocupa una superficie total de 19.000 m$^2$ aproximadamente.

En la planta se pretende analizar e investigar tanto las técnicas de depuración de bajo coste que han alcanzado un cierto grado de madurez en el mundo, como aquellas otras que en la actualidad se encuentran en una etapa más o menos desarrollada. Entre las primeras estarán presentes en principio instalaciones de lagunado (en sus versiones anaerobia, facultativa y maduración), filtros de turba, filtro verde y contactores biológicos rotativos, (gráfico 1).

Entre las técnicas menos conocidas se estudiarán los tanques anaerobios de lecho fijo, cultivos acuáticos, de algas, de peces y escorrentía sobre plantas emergentes, complementándose la instalación con unas pequeñas parcelas de investigaciones agrícolas.

No obstante lo expuesto, en la planta se dispone de suficiente terreno de reserva para, en su caso, poder llevar a cabo tanto posibles ampliaciones de las instalaciones existentes como la implantación de otras técnicas nuevas que pudieran considerarse de interés.



Las aguas residuales proceden de Carrión de los Céspedes y llegan a la planta a través del colector construido al efecto, incorporándose en primer lugar a una estación elevadora que es el comienzo de la línea de tratamiento.

Desde esta estación de impulsión, las aguas son enviadas a dos líneas distintas, que denominaremos I y II, diferenciadas en los procesos de tratamiento preliminares y subsiguientes.

En la línea I, los efluentes pasan por un sistema de tratamiento primario, consistente en un desarenado y un desengrasado estáticos, desde donde se incorporan a una cámara de regulación.

Esta cámara está concebida de tal modo que la carrera de agua en la misma sea muy reducida, para evitar fuertes variaciones de caudal como consecuencia de la carga hidráulica disponible. Desde aquí las aguas son enviadas por dos caminos diferentes, de los cuales el primero la transporta al sistema de lagunado y el segundo a una arqueta de reparto hacia los biodiscos, biocilindro, cultivos y escorrentía superficial.

La línea II, que alimenta a los filtros de turba y al filtro verde, no dispo-

## Biodiscos, biocilindros, cultivos

El funcionamiento de esta parte de la planta será el típico en esta clase de instalaciones, pasando el agua residual por la fases de decantación primaria, reacción biológica y decantación secundaria. Se cuenta con biodiscos y biocilindros de diseño propio, así como con un contactor biológico rotativo del tipo de rotores de espiral.

La fase de cultivos incluye los de plantas, algas y peces.

En el de plantas se realizarán experiencias con jacintos, lentejas de agua y plantas similares, cuyo cultivo a escala industrial puede tener gran importancia en la obtención de proteínas para alimentación animal.

En el sistema de colonización de algas se llevarán a cabo investigaciones sobre la producción masiva de las mismas para fines similares a los citados con los cultivos de plantas.

La experimentación con peces se efectuará fundamentalmente con aguas procedentes del lagunado, diluidas o no, y se tratará de cultivar especies típicas en estas instalaciones tales como carpas, gambusias, etcétera.

## Escorrentía superficial, filtros de turba y filtros verdes

La escorrentía superficial, hoy en día en fase experimental, se abordará mediante la aplicación de las aguas residuales sobre plantaciones de juncos y aneas, intentándose ajustar las cargas, longitudes de recorridos, etcétera, para la consecución del óptimo de depuración.

En los filtros de turba las aguas residuales brutas procedentes de la cámara de regulación II, tras pasar por un filtro autolimpiable y un desengrasador, se integran en los mismos. En éstos se ensayarán posibles mejoras y se ajustarán debidamente los condicionantes del diseño (carga hidráulica, potencia de turba, funcionamiento inundado o no, etcétera).

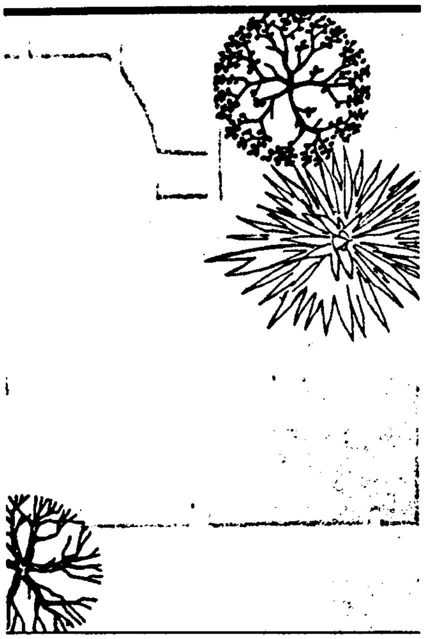
En un futuro próximo se prevé la experimentación con filtros concebidos de forma distinta, tanto geométricamente como en el sistema de alimentación.

En la parte de las instalaciones correspondiente a los filtros verdes se efectuarán ensayos diversos, encaminados tanto a comprobar la eficacia depuradora del sistema como al desarrollo de los árboles plantados para, en el futuro, poder disponer de datos reales de inversión-beneficio, calidad de agua infiltrada al freático, etcétera.

## Otras experiencias

Ya se ha indicado que en la parcela ocupada por las instalaciones se dispone de suficiente terreno de reserva para, en su día, ampliar las existentes o dar cabida a otras nuevas.

Igualmente puede ser motivo de ensayo la interconexión de los diversos tratamientos (por ejemplo, envío de aguas de tratamiento anaerobio a filtros de turba, uso del agua de escorrentía superficial para el cultivo de peces, obtención de biomasa, principalmente rotíferos, para alimentación de peces, etcétera), así como cambios en las técnicas iniciales planteadas (reemplazo de la turba por otro material, cambio de chopos por otro tipo de árboles, freatofitos o no, etcétera).



Por otra parte, si durante los cuatro años de duración de la experiencia se encontrase interesante la implantación de nuevas tecnologías, bien ensayadas en otros lugares, bien fruto de la propia investigación, se plantearán las pautas a seguir, tendentes a su ensayo y comprobación de su bondad de utilización en futuras plantas a nivel industrial.

Cabe destacarse la importancia que tendrá en el futuro el intercambio de información con organismos públicos y privados, españoles o extranjeros, que representará una ampliación de conocimientos a nivel general y específico, así como una proyección al exterior de una investigación que, tanto por su metodología como por sus conclusiones, pueda aplicarse a otros lugares de características similares e incluso exportarse a países de nuestro entorno.

Finalmente, debe considerarse que la planta experimental no sólo contribuirá a clarificar unos parámetros de diseño, sino que además servirá para formar a futuros técnicos y operadores y contribuirá de manera muy positiva en tareas didácticas de educación ambiental.

*GRAFICO 1*
ESQUEMA BASICO DE INTERCONEXION

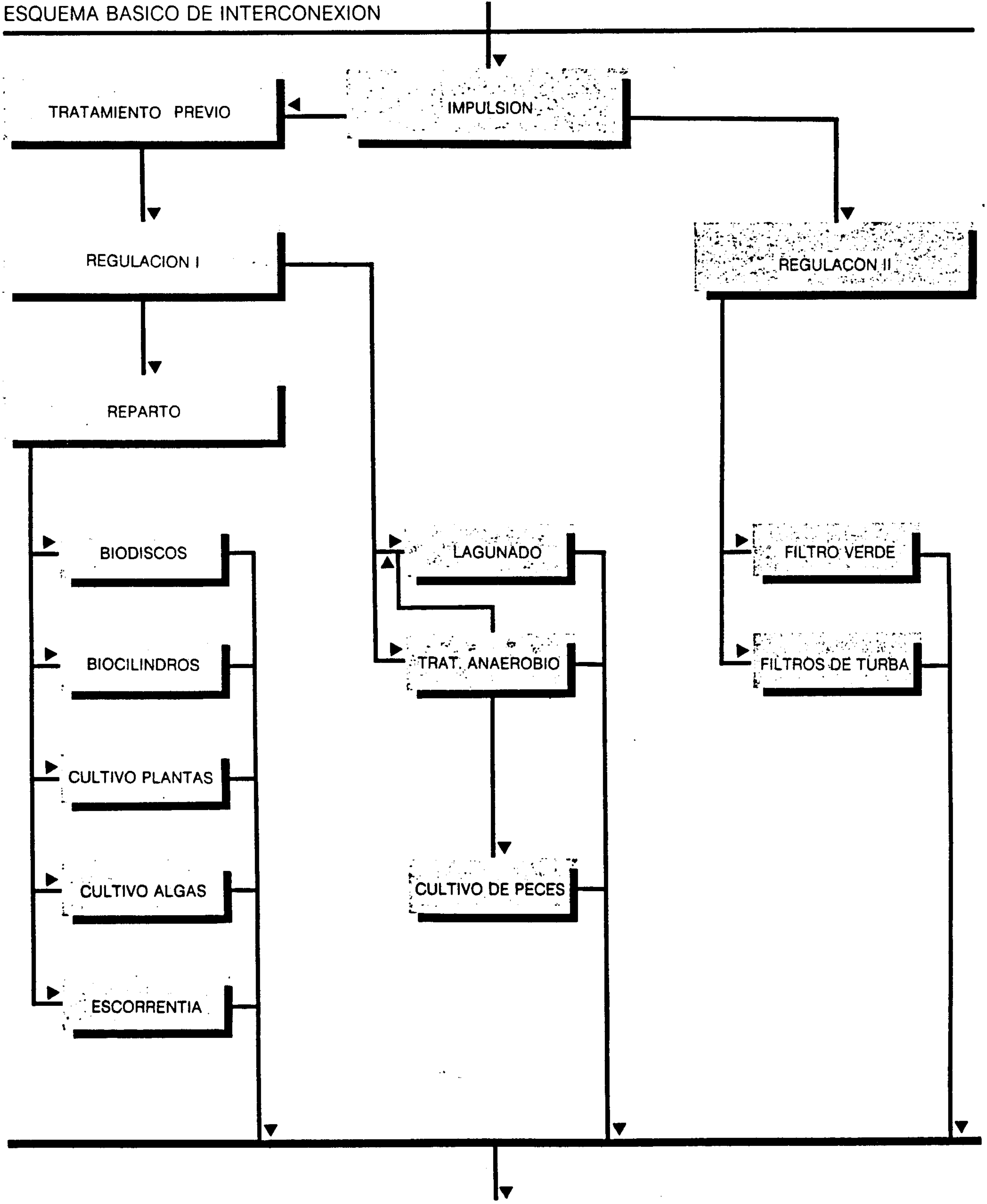
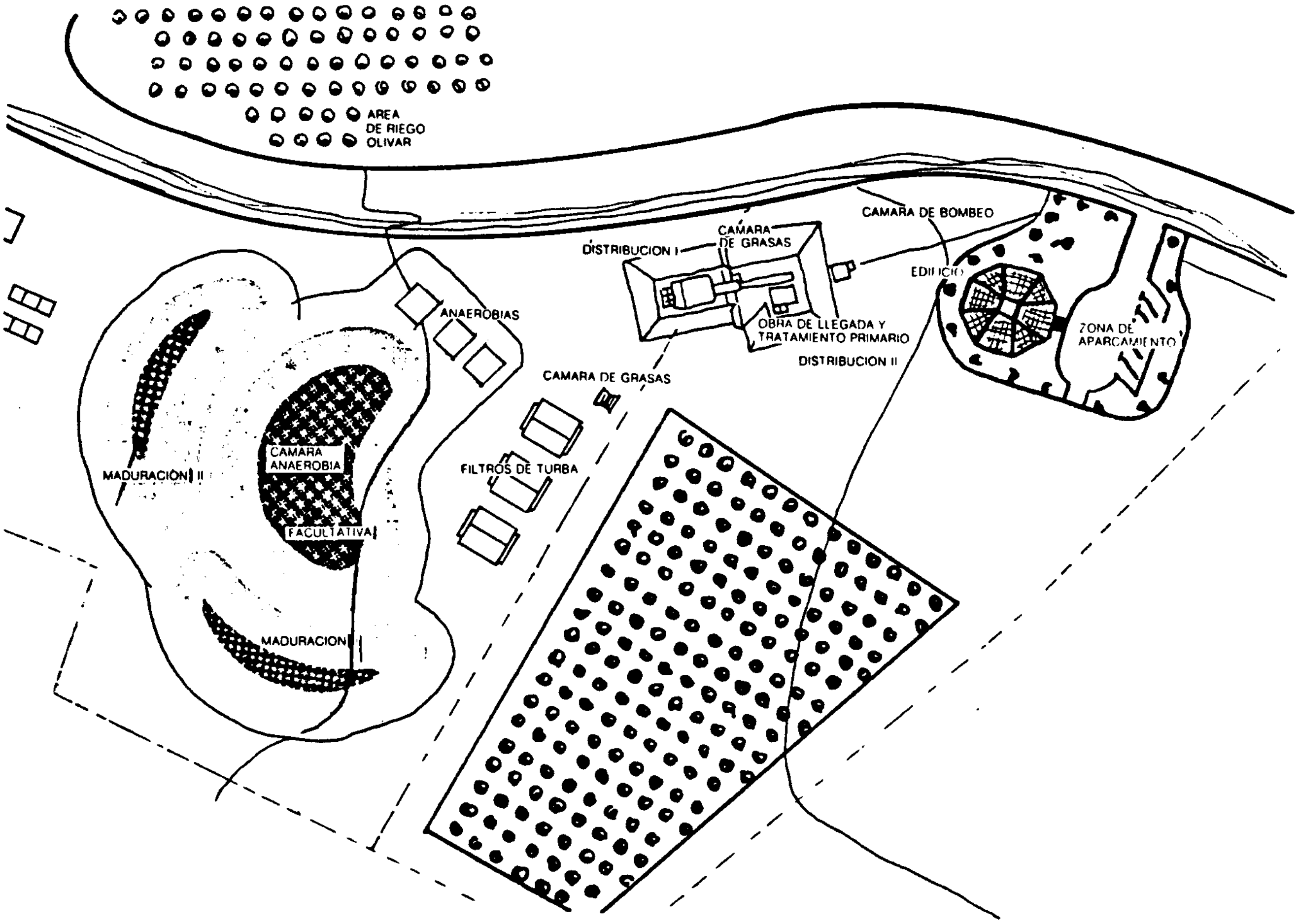


*GRAFICO 2*
PLANO Y DISTRIBUCION DE LA PLANTA PILOTO DE CARRION DE LOS CESPEDES (SEVILLA)



ne de tratamiento primario ya que los filtros de turba cuentan con su propio pretratamiento y en el filtro verde no es una obra necesaria. Así pues, el bombeo descarga directamente en la cámara de regulación II, concebida de forma similar a la anterior, (ver gráfico 2).

EL funcionamiento de cada una de las partes de la estación de tratamiento en el siguiente.

## Lagunas

El sistema consta de tres fases: anaerobia, facultativa y maduración, funcionando en serie.

El sistema facultativo-maduración está proyectado de tal forma que las alturas de lámina de agua puedan ser variadas con fines experimentales.

Junto a las lagunas anaerobias se dispone de una cámara de tratamiento anaerobio con lecho fijo. Desde ella, las aguas podrán ser integradas en el sistema facultativo-maduración o evacuarse directamente sin pasar por esta fase.

# Ground-Water Recharge and Its Effects on Nitrate Concentration Beneath a Manured Field Site in Pennsylvania

by James M. Gerhart[a]

## ABSTRACT

Ground-water recharge to a shallow, unconfined, fractured dolomite aquifer underlying agricultural land in Lancaster County, Pennsylvania occurs by two mechanisms. Direct recharge occurs through pathways such as near-surface bedrock fractures and sinkholes, and affects dissolved nitrate concentration of ground water within two to three days; its effects last only about one week. Gradual recharge occurs through small channels and pores in the unsaturated zone and affects dissolved nitrate concentration for several weeks or more after the effects of direct recharge have dissipated. Whether recharge causes an increase or decrease in dissolved nitrate concentration depends on the amount of nitrogen-rich manure spread on the site prior to the storm. Direct recharge from a storm in March 1984, a month in which 18 tons of manure were spread, resulted in a rapid decrease in dissolved nitrate concentration of about 2.5 milligrams per liter (mg/l) as nitrogen. Direct recharge from a storm in May 1984, after 384 tons of manure had been spread in April, resulted in a rapid increase in dissolved nitrate concentration of about 3 mg/l as nitrogen. Concentration changes caused by gradual recharge several weeks or more after the storms were of the same magnitude as those caused by direct recharge during the storm.

## INTRODUCTION

### Background

In October 1981, the U.S. Geological Survey and the Pennsylvania Department of Environmental Resources began evaluating stream and ground-water quality in the 188-square-mile Conestoga River headwaters area (Figure 1). This evaluation is part of the Rural Clean Water Program (RCWP) administered by the U.S. Department of Agriculture (U.S. Department of Agriculture, 1984). The goal of the Conestoga River RCWP is to reduce agricultural nonpoint-source contamination of streams and ground water

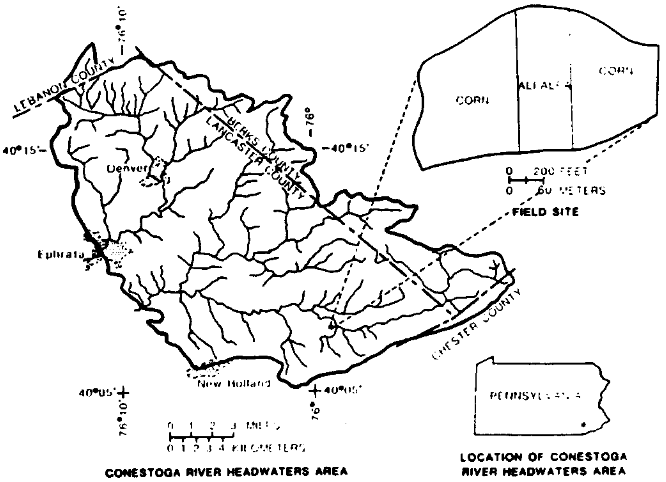


Fig. 1. Map showing location of Conestoga River headwaters area and field site.

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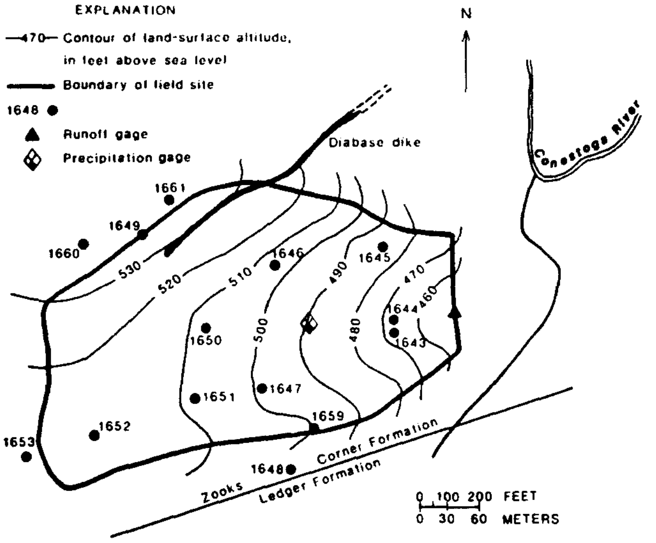

Fig. 2. Map showing instrumentation, topography, and geology at field site.

through the voluntary implementation of agricultural best management practices (BMP's). The specific objective of the evaluation effort is to quantify the effects of BMP's on water quality. The evaluation approach is to monitor water quality before, during, and after BMP implementation on farm field sites, small watersheds, and the entire Conestoga River headwaters area. This paper is based on pre-BMP water-quality monitoring results at a field site.

The field site is 21.7 acres of corn and alfalfa located on a dairy farm in northeastern Lancaster County, Pennsylvania (Figure 1). The purpose of water-quality monitoring at the site is to quantify the effects of terrace, manure storage, and nutrient management BMP's on ground-water and surface-runoff quality. In 1982 and 1983, the site was instrumented with a precipitation gage, 14 monitoring wells, and surface-runoff gage (Figure 2).

### Purpose and Scope

The purpose of this paper is to describe the ground-water recharge process at the site and to demonstrate the effect of recharge and manure spreading on the concentration of dissolved nitrate in ground water.

Geologic mapping, interpretation of drill cuttings, well-water levels, and specific-capacity data were used to define the hydrogeology of the site. Differences between precipitation and surface-runoff quantities, and rises in well-water levels were used to quantify recharge from September 1983 to August 1984. Dissolved nitrate analyses from a representative well (well 1643) were used in conjunction with data on manure applications to relate recharge to changes in dissolved nitrate concentration during the Spring and Summer of 1984.

### Methods

A continuous water-level recorder was used to measure water-table fluctuations in well 1643; precipitation on the field site was measured for five-minute intervals with a recording precipitation gage; and surface-runoff quantities were measured using a rated flume and a continuous stage recorder.

Water samples were collected from well 1643 with a one-liter, Kemmerer-type point sampler. Samples were collected adjacent to the major water-bearing fracture in the well, where ground water representative of the aquifer was entering the well. After collection, the samples were filtered through a 0.45-micron filter, preserved with mercuric chloride, and chilled to 4°C. They were analyzed for dissolved nitrate plus nitrite and dissolved nitrite at the Pennsylvania Department of Environmental Resources Laboratory in Harrisburg. The analyses were performed following procedures described by Skougstad and others (1979). Dissolved nitrite concentration was subtracted from dissolved nitrate plus nitrite concentration to obtain dissolved nitrate concentrations.

## DESCRIPTION OF FIELD SITE

### Topography and Soils

The site is a surface-drainage basin from which all surface runoff empties into a pasture adjacent to a tributary of the Conestoga River (Figure 2). The average east-west slope of the site is about 6 percent, but slopes on the steeper sections of the site exceed 15 percent.

Soils at the site are deep, well-drained silt loams developed on carbonate uplands (U.S. Department of Agriculture, 1956). Typically, they consist of 8 to 10 inches of brown silt loam underlain by yellowish-brown silty clay. However, due to differential erosion, the thickness of the silt loam layer at the site ranges from less than one inch to more than 1.5 feet. In the deeper runoff channels, the silt loam and silty clay layers have been removed by erosion.

### Geology

The site is underlain by dolomite of the Cambrian Zooks Corner Formation, which consists of interbedded finely crystalline dolomite, silty dolomite, and dolomitic siltstone (Meisler and Becher, 1971). South of the site, the Zooks Corner

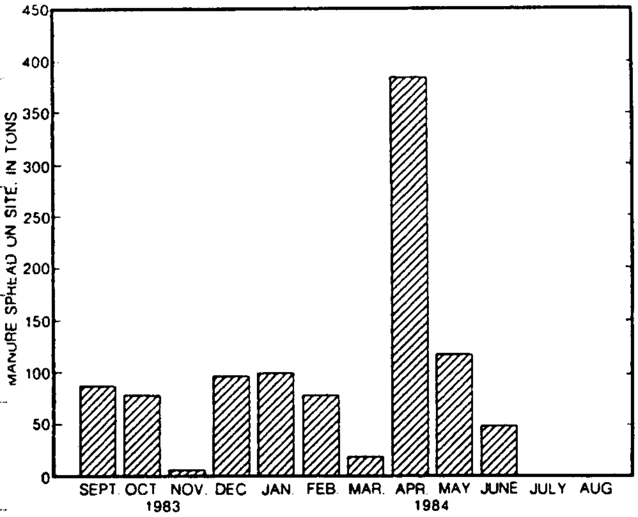

**Fig. 3. Graph of manure applications at field site, eptember 1983 through August 1984.**

ormation grades into the Cambrian Ledger Dolomite, which consists of massive, medium to oarsely crystalline dolomite (Figure 2). Based on xamination of outcrops of both formations, bedding strike ranges from about N60°E to N70°E nd bedding dip ranges from about 40°NW to 0°NW. In addition to bedding, another major structural feature is an intersecting set of joints hich trend northwest and northeast, and are early vertical. Cutting across the dolomite beds in the northwestern corner of the site is a diabase ike of Triassic age.

## Land Use

From 1980 to 1984, the 21.7-acre site was rmed as two corn fields separated by a 4-acre alfalfa field (Figure 1). Manure from about 65 airy cows and 35 heifers was spread on the corn elds, mainly during the nongrowing seasons. Manure applications from September 1983 to ugust 1984 are shown in Figure 3. The heaviest plication (384 tons) occurred in April just prior to plowing. Based on chemical analyses of the manure, there were 9.3 pounds of nitrogen in each n of manure. Thus, in April, more than 3,500 pounds of nitrogen were applied to the two corn fields—an average rate of application of about 200 ounds of nitrogen per acre. Application rates of his magnitude, combined with uneven spreading of manure, has resulted in highly variable soil itrogen content. In November 1983, following the orn harvest, the nitrogen content of 42 soil samples from the top 12 inches of the fields ranged om 6 to 342 pounds per acre (Dale Baker, Pennsylvania State University, written communication, 1984).

## Hydrogeology

The information obtained from the outcrops was combined with interpretations of the cuttings obtained during well drilling to determine the hydrogeologic framework of the site. The 14 wells were drilled by air-rotary methods and finished as open holes in the dolomite bedrock. They range from 75 to 142 feet deep and are cased to bedrock with 10 to 108 feet of 6-inch steel casing. The casings were grouted in place and a bentonite seal was placed around each well casing at land surface. The characteristics of six representative wells are shown in Figure 4.

The hydrogeologic framework consists of soil underlain by a zone of weathered and unconsolidated dolomite of variable thickness. This weathered zone ranges from about 10 to 100 feet in thickness and rests on relatively fresh dolomite bedrock. The weathered zone, thickest in the middle of the site, contains some fresh dolomite beds, some deeply weathered silty dolomite beds, and some clay layers. The dolomite bedrock contains secondary openings, some filled with silty weathered material and some with clay. Most of these openings probably are located along bedding contacts, but some may be located along joint surfaces. During the first two years of monitoring, several small sinkholes developed on the site.

The depth to the water table ranges from

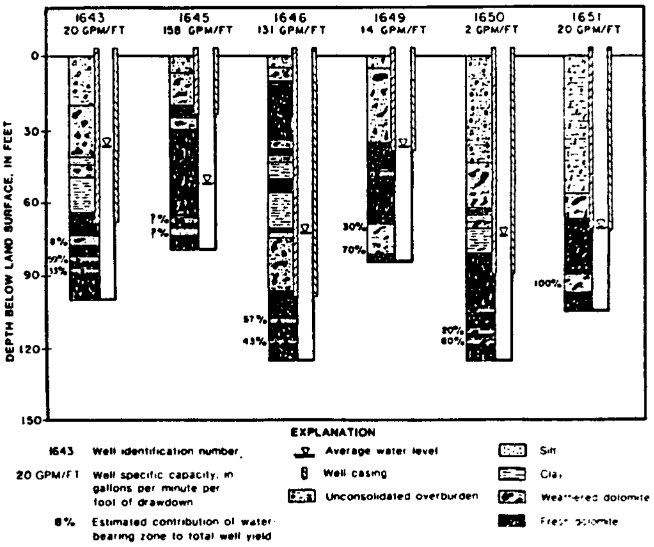

**Fig. 4. Diagram showing hydrogeologic logs of six representative wells at field site.**

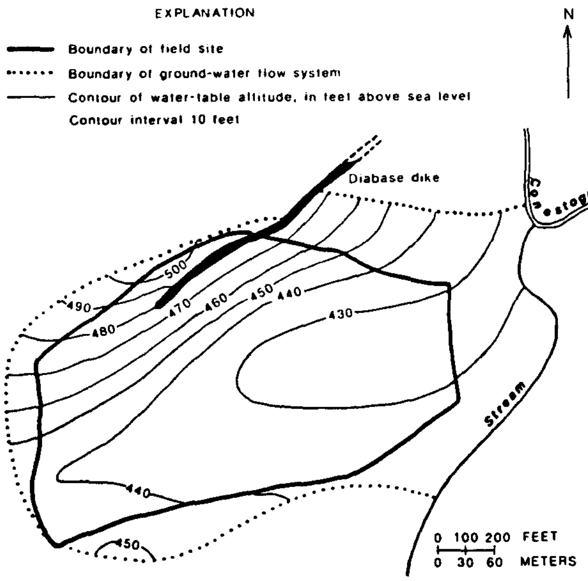

**Fig. 5. Map of water-table configuration at field site, November 1982.**

about 30 to 70 feet below land surface. Water levels suggest that the shallow, unconfined ground-water flow system is closed on three sides (Figure 5). The northern, western, and southern limits of the flow system are approximately the same as the surface-drainage divides. The eastern limit probably is the Conestoga River and its tributary. Given these limits to the shallow flow system, recharge to the water table beneath the site originates solely from precipitation on the site and discharge is to the streams to the east.

The water-table contours shown on Figure 5 indicate that the general ground-water flow direction is parallel to the land-surface slope and the bedding strike, or northeastward through the center of the site. The diabase dike, a virtually impermeable barrier, disrupts this northeasterly tendency and causes the ground water to the west of it to back up and flow southeastward. This results in a much steeper water-table gradient in the northwestern corner of the site due to lower permeability across bedding than along it.

Most of the 14 wells were pumped shortly after they were completed and water levels monitored in the pumped wells provided specific-capacity data. The 6 wells in Figure 4 are representative of the range of specific capacity of 13 of the 14 wells; the specific capacity of one well was negligible. For wells in which one or more major water-bearing fractures were encountered, specific capacity ranged from about 2 to 158 gallons per minute per foot of drawdown.

# GROUND-WATER RECHARGE

## Response Time

The relation between precipitation and water levels in well 1643 is shown in Figure 6 from September 1983 to August 1984. The magnitudes of the water-level responses differed because of differing soil-moisture and crop-cover conditions, but the response times always were short. The precipitation and water-level data for a storm on March 27 and 28, 1984 are shown on Figure 7. The water level started to rise about two hours after the storm began. In addition, more intense rain periods are reflected in the water-level curve by increases in slope that lag the more intense rain periods by several hours. This short response time, independent of soil-moisture and crop-cover con-

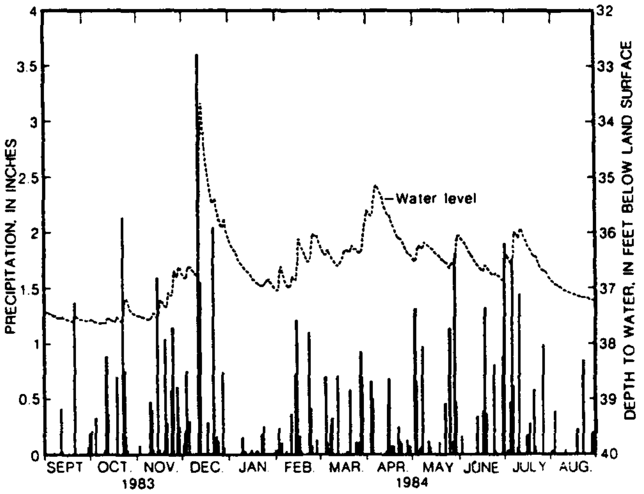

**Fig. 6. Graph of precipitation and water level in well 1643, September 1983 through August 1984.**

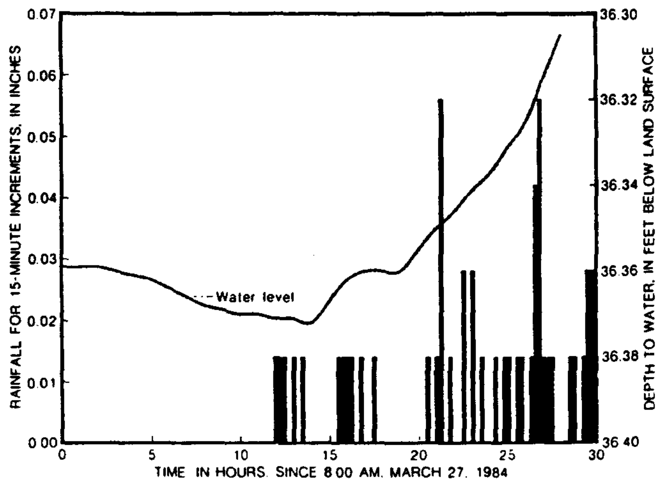

**Fig. 7. Graph of rainfall and water level in well 1643, March 27 and 28, 1984.**

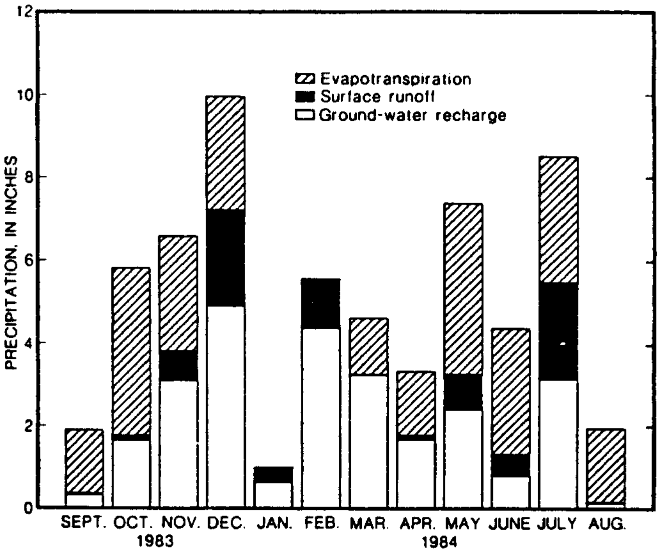

**Fig. 8. Relationship of evapotranspiration, surface runoff, and ground-water recharge at field site, September 1983 through August 1984.**

ditions, suggests that some of the precipitation was reaching the water table almost immediately. This recharge probably occurs through areas of thin soil cover, shallow fractured bedrock, or sinkholes.

### Recharge Amounts

The inches of recharge (R) were estimated from inches of precipitation (P) and inches of surface runoff (SR) for individual storms occurring under conditions of high soil moisture and minimal crop cover by the following equation:

$$R = P - SR \qquad (1)$$

It was assumed that under such soil-moisture and crop-cover conditions, there would be negligible evapotranspiration, and that the precipitation that did not run off the site would infiltrate the soil and become recharge. These calculated recharge amounts (in inches) were then used in conjunction with inches of water-level rise (WLR) measured during those storms to estimate the specific yield (SY) of the aquifer by the following equation:

$$SY = R/WLR \qquad (2)$$

These types of calculations were made for seven representative wells for several storms, and the resulting specific yields ranged from 0.08 to 0.19.

Applying an average specific yield of 0.13 to the water-level rises (in inches) in well 1643, the recharge amounts (in inches) for all storms were estimated using the following equation:

$$R = SY(WLR) \qquad (3)$$

Adding these calculated recharge amounts to measured surface-runoff amounts and then subtracting the sums from the measured precipitation amounts resulted in estimates of inches of evapotranspiration. Evapotranspiration estimated in this way also includes those amounts of precipitation in storage as ice or snow. Evapotranspiration, surface runoff, and recharge are shown from September 1983 to August 1984 in Figure 8. Recharge generally was highest in the winter and early spring. Even though more precipitation fell in the late spring and early summer, recharge was less because of increasing evapotranspiration. For the one-year period shown in Figure 8, evapotranspiration made up about 42 percent of total precipitation; surface runoff 14 percent; and recharge 44 percent.

## GROUND-WATER NITRATE CONCENTRATION

### Short-Term Response to Recharge

For two storms (March 27 and 28, 1984 and May 26 and 29, 1984), ground-water samples were collected before, during, and after the storms so that any rapid changes in dissolved nitrate concentration could be evaluated. The May storm consisted of two overlapping storms in terms of water levels. In both the March and May storms, the first post-storm samples were collected two to three days after the storms had begun, and each already had different concentrations of dissolved nitrate than the samples collected prior to the start of the storms (Figures 9 and 10). These rapid changes were attributed to the precipitation that recharged the water table almost immediately

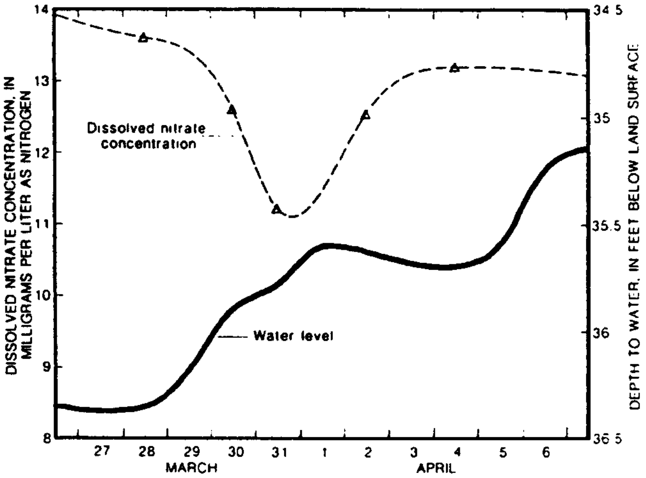

**Fig. 9. Graph of water level and dissolved nitrate concentration in well 1643 during and following storm of March 27 and 28, 1984.**

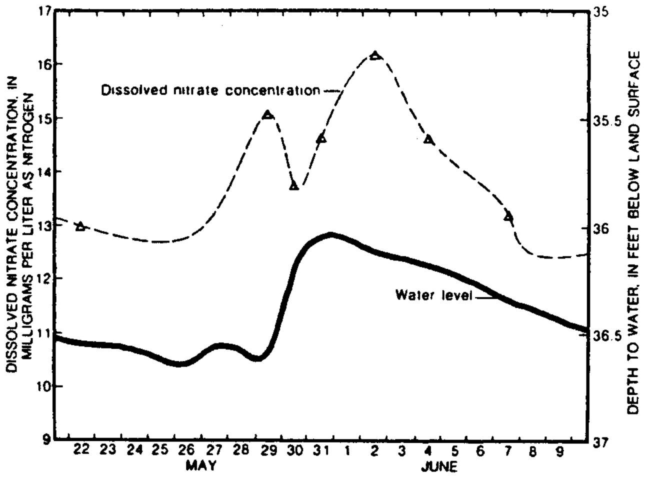

**Fig. 10. Graph of water level and dissolved nitrate concentration in well 1643 during and following storm of May 26 and 29, 1984.**

through relatively direct pathways. The maximum concentration change was approximately the same in both storms (about 2.5 and 3 mg/l as nitrogen) and in both cases, the dissolved nitrate concentrations returned to pre-storm levels about one week after the storms had begun. The May storm showed two rapid increases in dissolved nitrate concentration, reflecting the water-level peaks for the May 26 and May 29 storms.

The March storm caused a rapid decrease in dissolved nitrate concentration; the May storm caused a rapid increase. This difference was the result of differences in the nitrogen content of the recharge water between March and May. Less manure was spread in March than in April and May (Figure 3). As a result, less nitrogen was available on the land surface, and therefore less nitrogen was carried in recharge in March. The nitrogen content of the March recharge was relatively low; causing dissolved nitrate concentration to rapidly decrease from about 13.5 to about 11 mg/l as nitrogen. Conversely, the nitrogen content of the May recharge was relatively high, causing dissolved nitrate concentration to rapidly increase from about 13 to about 16 mg/l as nitrogen.

### Long-Term Response to Recharge

Some precipitation in each storm infiltrated the soil and began to move more slowly to the water table, probably through small channels and pore spaces in the unsaturated zone. To determine the effect of this gradual recharge on dissolved nitrate concentration, those samples collected at least one week after major storms were evaluated. As seen in Figure 11, this gradual recharge caused changes in dissolved nitrate concentration over a period of several weeks or longer. In March, when there was little manure spread on the site, low-nitrogen recharge caused a decline in the dissolved nitrate concentration. Following the first storm after the April applications of manure, concentrations began to rise slowly, reaching an August high of about 17.5 mg/l as nitrogen. The effects of this gradual recharge continued well after the storms that caused them, by steady downward flow through the unsaturated zone. Thus, for example, although there was no appreciable recharge from early April to early May, concentrations of dissolved nitrate continued to decrease. Similarly, the last major storm of the summer was in July, but concentrations continued to increase into August.

### Overall Effects of Recharge

When the rapid changes in dissolved nitrate concentration are superimposed on the gradual changes (Figure 12), the overall effects of recharge can be seen. Before the heavy applications of manure in April, all four samples collected during the March storm plot below or on the line reflecting the gradual changes in concentration. After the April manure spreading, seven samples were collected during three storms, and all plot above the line. This clearly demonstrates the diluting effects of low-nitrogen direct recharge before April and the concentrating effects of high-nitrogen direct recharge during and after April.

Because the quantities of direct and gradual recharge from each storm were exposed to the

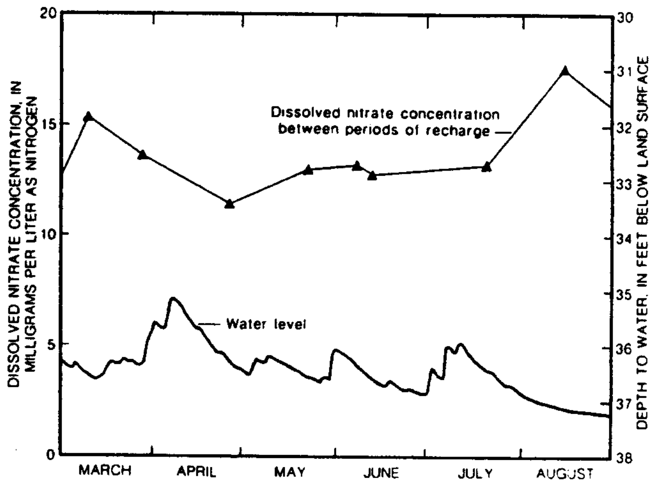

**Fig. 11. Graph of water level and dissolved nitrate concentration in samples collected from well 1643 at least one week after major storms, March through August 1984.**

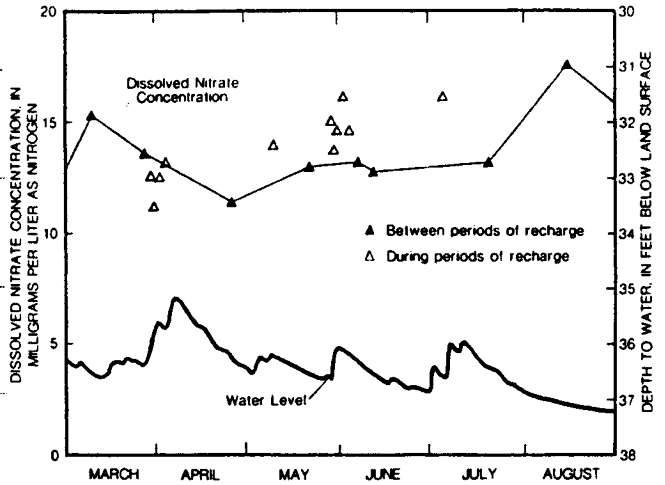

**Fig. 12. Graph of water level and dissolved nitrate concentration in all samples collected from well 1643, March through August 1984.**

same amounts of fresh manure, the nitrogen content of both types of recharge probably were similar. The effect of this is apparent in Figure 12. The concentration changes observed during the storms were indicative of the concentration changes that occurred several weeks or more after the storms. The lowest dissolved nitrate concentration during the March storm and the lowest concentration after the storm (in late April) were about 11 mg/l as nitrogen. Similarly, the higher concentration measured during the May and July storms was comparable to the highest concentration measured after the storms, in August.

## CONCLUSIONS

Ground-water levels respond to a major storm within several hours due to recharge through direct pathways to the water table such as near-surface fractures and sinkholes. This direct recharge affects concentrations of dissolved nitrate in ground water within two to three days after the storm begins; its effects dissipate in about one week. Gradual recharge through small channels and pore spaces in the unsaturated zone reaches the water table more slowly and affects dissolved nitrate concentrations for several weeks or more after the storm ends. The amount of fresh manure on the land surface at the time of the storm determines whether nitrate concentrations increase or decrease, as well as the magnitude of the increase or decrease. Concentration changes caused by direct recharge during a storm are comparable to concentration changes caused by gradual recharge several weeks or more after the storm has ended.

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# CHAPTER 15

## Groundwater Recharge for Wastewater Reuse in the Dan Region Project: Summary of Five-Year Experience, 1977–1981

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The first stage of the Dan Region Sewage Reclamation Project (Figure 15.1) consists of facilities for treatment and groundwater recharge of municipal wastewater discharged from the southern suburbs of the city of Tel Aviv–Jaffa and the neighboring municipalities of Holon, Bat Yam, and Rishon-Le-Zion, with a total connected population estimated at about 400,000.

The wastewater pumped to the treatment plant undergoes biological treatment in two parallel series of facultative oxidation ponds with effluent recirculation and chemical treatment by the high lime–magnesium process, followed by detention of the high pH effluent in polishing ponds, mainly for free ammonia stripping and natural recarbonation (Figure 15.2).

The treated effluent has been recharged to the regional groundwater aquifer since 1977 by means of spreading basins, located in the vicinity of the treatment plant (see Figure 15.1). Most of the recharged effluent, after additional treatment and prolonged detention in the soil-aquifer system, will be pumped by means of recovery wells for reuse in the south of the country (Figure 15.3).

In the final stage of the project, when the recovery wells will pump mostly recharged effluent, the reclaimed water will be supplied to nonpotable uses (mainly unrestricted irrigation of agricultural crops), by means of

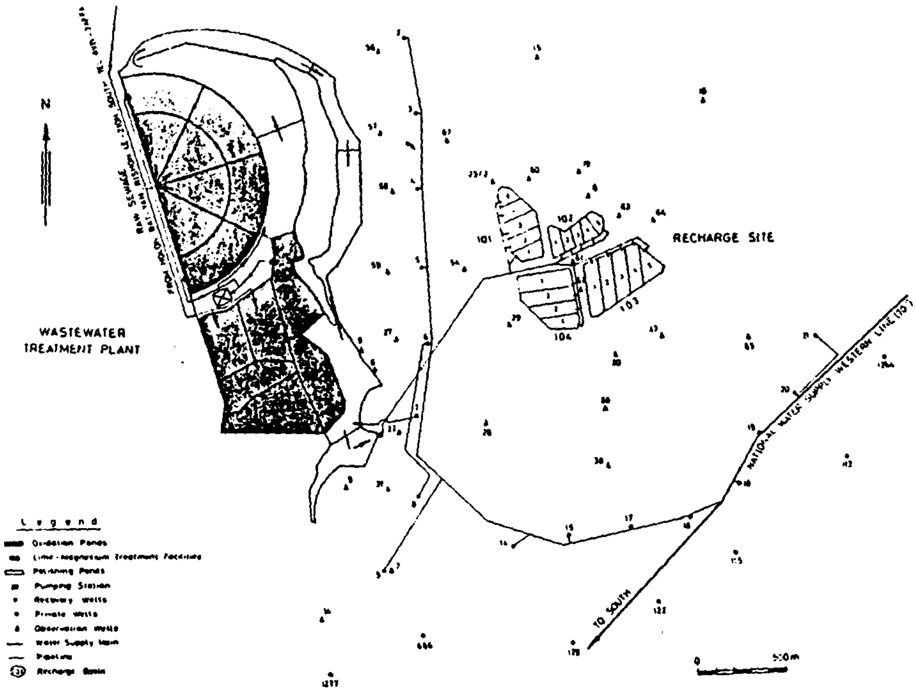

Figure 15.1 Layout of the Dan Region Sewage Reclamation Project.

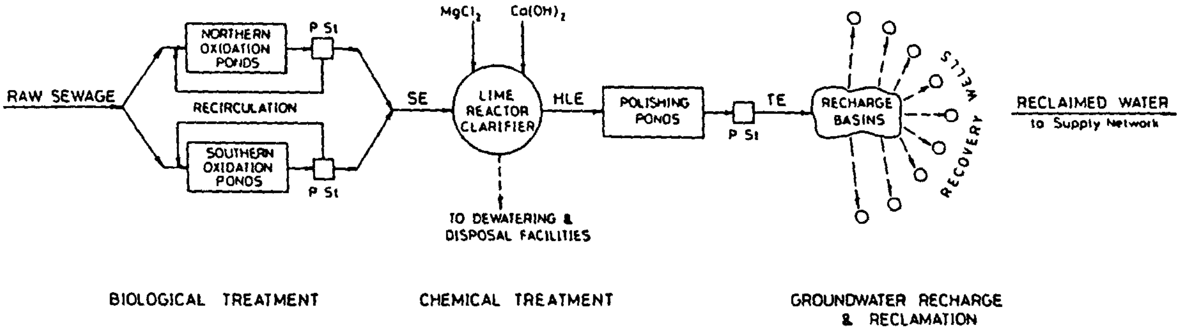

Figure 15.2 Flow diagram of Dan Region Sewage Reclamation Project.

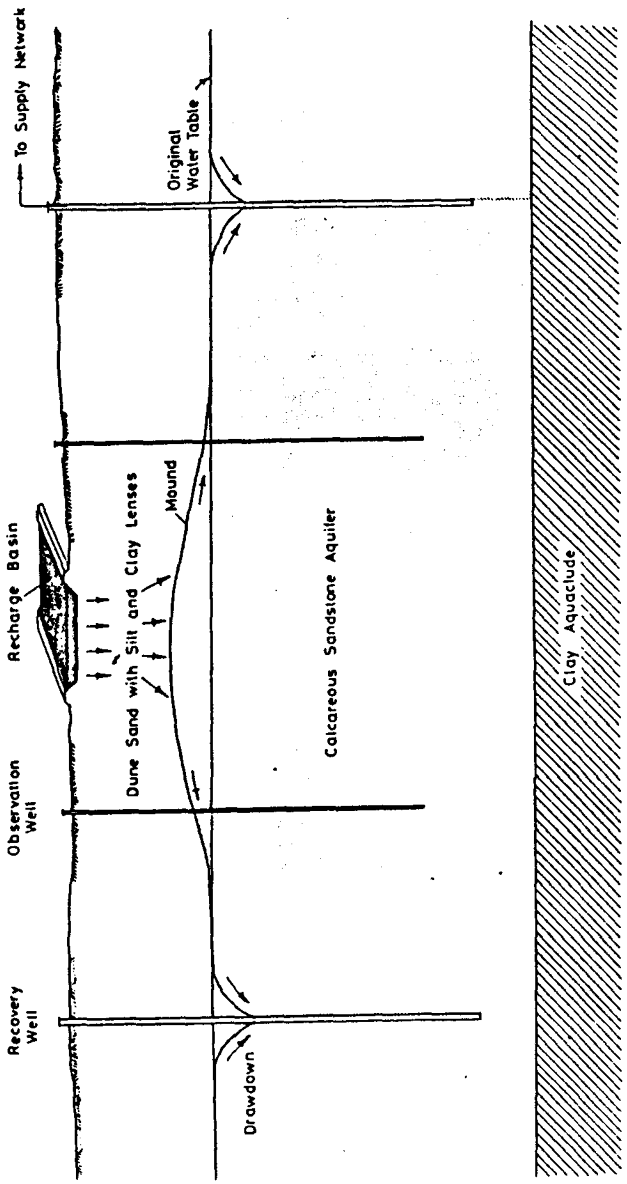

Figure 15.3 Basic diagram of recharge recovery system.

a dual system conveying separately potable and nonpotable water. At present most of the existing recovery wells pump native groundwater to the potable supply network; only a limited number of wells may pump an admixture of native groundwater and small amounts of recharged effluent.

The intentional recharge of municipal effluent to a potable groundwater aquifer, as adopted in the Dan Region Project, is a relatively new concept, referred to as *soil-aquifer treatment* (SAT). It consists of the recharge of partially treated effluent in such a way that a sub-basin is created, which can be hydrologically separated from the rest of the aquifer. The soil-aquifer system provides for additional effluent treatment by a variety of processes and for storage; the bulk of the recharged effluent is extracted by recovery wells and supplied to uses compatible with its quality. Accurate monitoring of the effluent in the aquifer is necessary for the proper operation of such a system; consequently, reliable tracers of the effluent in the groundwater are required.

## DESCRIPTION OF RECHARGE SITE

The recharge site is located in an area of rolling sand dunes near the Mediterranean coast, which lies above the central part of the coastal aquifer (Pleistocene). This aquifer is composed mainly of calcareous sandstone and is divided into subaquifers by silt and clay layers.

The climate of the area is typically Mediterranean. Summers are warm and dry and winters are mild, with rainy spells. The average annual precipitation is 500 to 550 mm. The average temperatures range between 20° to 30° C in summer and 10° to 20° C in winter.

### Recharge Facilities

The recharge facilities consist of four basins, referred to as 101, 102, 103, and 104 (see Figure 15.1) with a total area of about 30 ha. The upper soil layer of the recharge basins consists mostly of uniform fine sand (less than 0.3 mm).

Originally, only basin 102, which was to be used as a test basin, was divided into sub-basins in order to enable higher hydraulic loading per unit area and to permit greater operational flexibility, which was needed mainly for research purposes. All the sub-basins were provided with staff gauges for measuring water levels. Installations for the sampling of percolating effluent, which were specially designed for this purpose, were constructed in two of the five sub-basins of basin 102.

After some experience was gained, it was decided to subdivide the other basins, too, in order to ensure smooth operation and cleaning of the basins. In 1980, basin 101 was divided into five sub-basins, which were provided with staff gauges; three of the sub-basins were also provided with weirs for measuring the inflow of recharge water.

At the time this chapter was being written, basins 103 and 104 were also being divided into sub-basins.

### Recovery Facilities

A ring of recovery wells, spaced 300 to 400 m from one another, has been planned to surround the recharge area in order to ensure the pumping of recharged water to uses compatible with its quality and to minimize the influence of the recharge operation on private wells (see Figure 15.1). Of this ring seven wells located west of the recharge area have been in operation since 1974. Eight other wells are located south and southeast of the recharge area; two of these have been operated since 1979, while the other six just started operating. The western wells are located 400 to 1,200 m from the recharge site, whereas the southern wells are located 900 to 1,700 m from the recharge site (see Figure 15.1).

### Monitoring Program

A monitoring network of eighteen observation wells has been established in the vicinity of the recharge basins and recovery wells (see Figure 15.1). Observation well samples are taken after at least a half hour of pumping with compressed air, which is sufficient to exchange the water in the well several times.

Three types of groundwater quality analyses were established: indicative, basic, and comprehensive. The functions of these analyses are described below.

*Indicative*—to ascertain whether the recharged effluent has reached a certain well. This analysis includes chlorides, electrical conductivity, $KMnO_4$ consumption, $UV_{254}$ absorbance, and detergents.

*Basic*—to examine the groundwater quality (after effluent has reached the well) with respect to basic wastewater parameters, in order to determine the effect of the recharged effluent on groundwater quality and the purification capacity of the soil-aquifer system.

*Comprehensive*—to thoroughly determine groundwater quality with respect to water quality standards for irrigation and domestic uses.

The frequency of the indicative analysis is 1 to 4 months for the various observation wells, depending on their distance from the recharge basins, and 6 months for the production wells. The basic analyses are carried out every month in observation wells pumping effluent. The frequency of the comprehensive analysis is 6 months for selected observation wells and 12 months for selected production wells.

## RECHARGE OPERATION

The recharge operation carried out in the Dan Region project in the period 1977–1981 is briefly described below with respect to volumes recharged, recharge regime, infiltration rates, and groundwater hydrology [1–5].

### Volumes of Effluent Recharged

About 60 million $m^3$ of effluent were recharged to the groundwater aquifer during the 5-year period 1977–1981. The distribution of the recharge volumes by basins is shown in Table 15.1.

### Recharge Regime

The recharge basins were flooded intermittently to maintain high infiltration rates and to enhance effluent purification during percolation. Basin 102, which has been used for research purposes, was operated according to a cycle of 1 day of flooding and 2 to 3 days of drying. The recharge cycle in the other basins generally varied from 1 to 2 days of flooding and 5 to 6 days of drying to 3 days of flooding and 4 days of drying, depending on the hydraulic capacity of the various basins and the number of basins in operation.

**Table 15.1** Volumes of Effluent Recharged (million $m^3$)

| | *Basin* | | | | |
|---|---|---|---|---|---|
| *Year* | *101* | *102* | *103* | *104* | *Total* |
| 1977 | 3.5 | 1.7 | — | — | 5.2 |
| 1978 | 2.7 | 3.3 | 2.8 | 1.1 | 9.9 |
| 1979 | 2.5 | 2.0 | 3.4 | 2.7 | 10.6 |
| 1980 | 2.6 | 2.5 | 4.1 | 6.1 | 15.3 |
| 1981 | 0.8 | 4.8 | 6.0 | 7.0 | 18.6 |
| 1977–1981 TOTAL | 12.1 | 14.3 | 16.3 | 16.9 | 59.6 |

### Infiltration Rates

Infiltration rates, which were monitored especially in basin 102, were affected by effluent quality, climatic conditions, and the frequency of basin cleaning. An average infiltration rate of 1.5 to 2 m per day was maintained under a recharge regime of 1 day of flooding and 2 to 3 days of drying.

The yearly hydraulic loads in the various recharge basins are summarized in Table 15.2. The highest average loads (100–160 m/year in 1980–1981) were obtained for basins 101 and 102, which are divided into sub-basins and have been operated in the best manner. This figure, which does not take into account the need for cleaning periods, should be considered as a good estimate of the long-term recharge capacity that could be taken into account in the design of similar recharge operations. The other basins have been underloaded: Because of the availability of recharge basins in excess of present needs, basins 103 and 104, which were not subdivided, have not been fully utilized until now.

### Groundwater Hydrology

A radial groundwater mound was formed beneath the recharge area. After 5 years of recharge (1977–1981) the groundwater mound rose by about 8.5 m below the center of the recharge area and 3.5 m about 600 m distant. During the same period about 45 million m$^3$ were pumped from the Dan recovery wells; these wells pumped a total amount of almost 70 million m$^3$ considering also the pumpage that took place prior to the beginning of recharge (Table 15.3).

The depth of the unsaturated zone below the recharge basins was at the end of 1981 as follows: 28 to 30 m under basin 101; about 23 m under basin 102; 14 to 18 m under basin 103; and 20 to 22 m under basin 104.

**Table 15.2** Hydraulic Load in Spreading Basins (m/year)

| | | *Basin* | | | |
|---|---|---|---|---|---|
| *Year* | *Values* | *101* | *102* | *103* | *104* |
| | Max. | 155 | 267 | 62 | 81 |
| 1978 | Min. | 23 | 91 | 18 | 66 |
| | Average | 89 | 178 | 42 | 71 |
| | Max. | 109 | 192 | 50 | 99 |
| 1979 | Min. | 32 | 108 | 11 | 62 |
| | Average | 77 | 157 | 36 | 77 |
| | Max. | 161 | 148 | 67 | 85 |
| 1980 | Min. | 70 | 108 | 19 | 38 |
| | Average | 101 | 131 | 42 | 60 |
| | Max. | 166 | 222 | 120 | 132 |
| 1981 | Min. | 128 | 117 | 62 | 90 |
| | Average | 151 | 159 | 85 | 110 |

## QUALITY OF RECHARGE EFFLUENT

The recharge effluent is obtained by chemical treatment of the effluent from recirculated oxidation ponds. The chemical treatment is carried out in two steps: high lime–magnesium treatment in a reactor-clarifier, followed by detention of the high pH effluent in polishing ponds, mainly for free ammonia stripping and natural recarbonation. Consequently, the recharged effluent generally had a low alkalinity and a relatively low content of suspended solids, organic matter, nitrogen, and phosphorus.

However, it had a relatively high pH and Langelier saturation index, because the natural recarbonation, which takes place in the polishing ponds, is not complete and artificial recarbonation (by addition of carbon dioxide) is not carried out. In order to improve the chemical stability of the effluent prior to recharge, the tertiary effluent has been mixed since May 1978 with water from Dan recovery well 7 and, since April and August 1980, also with water from Dan recovery wells 8 and 6, respectively; the water of these wells contains a large proportion of effluent that seeped in the past from the treatment ponds. The mixing point is located in Pond F just before the entrance to Pond 18 (see Figure 15.1). Two out of these three wells are usually in operation. The amount of groundwater pumped by the wells represents some 15 to 25 percent of the recharged water. Occasionally this percentage was higher, e.g., in May and November 1979 and in May 1980, when the clarifier was not in operation or in October–November 1980, when all three wells were operated.

The effluent quality is usually higher in summer than in winter because of the greater efficiency of the lime clarification process as well as of the ammonia-stripping process.

The quality of the recharge effluent with respect to basic wastewater parameters is shown in Table 15.4 and Figure 15.4. Ammonia and total nitrogen are lower in summer than in winter because of the better ammonia removal at higher temperature: the concentration of ammonia was higher in 1980 and especially in 1981 (as compared to previous years), due to the shorter detention time of the high lime effluent in the polishing ponds, resulting from the larger volumes of effluent treated. Suspended solids and phosphorus concentrations were somewhat higher in winter because of the

Table 15.3 Quantities Pumped from Dan Project Recovery Wells (million $m^3$)

| Well No. | Total[a] 1974–1977 | Year 1977 | 1978 | 1979 | 1980 | 1981 | Total 1977–1981 | Total[a] 1974–1981 |
|---|---|---|---|---|---|---|---|---|
| 2 | 2.4 | 1.4 | 3.0 | 0.8 | 2.1 | 2.0 | 9.3 | 11.7 |
| 3 | 4.5 | 1.5 | 1.0 | 2.1 | 1.2 | 1.1 | 6.9 | 11.4 |
| 4 | 1.8 | 0.8 | 0.2 | 0.5 | 0.8 | 1.0 | 3.3 | 5.1 |
| 5 | 5.4 | 3.0 | 0.6 | 1.7 | 1.2 | 1.6 | 8.1 | 13.5 |
| 6 | 4.9 | — | — | — | 0.8 | 1.4 | 2.2 | 7.1 |
| 7 | 0.2 | — | 1.2 | 2.3 | 1.4 | 1.2 | 6.1 | 6.3 |
| 8 | 4.0 | ~0 | ~0 | ~0 | 1.5 | 0.9 | 2.4 | 6.4 |
| 19 | — | — | — | ~0 | 1.6 | 0.9 | 2.5 | 2.5 |
| 21 | — | — | — | 2.4 | 1.6 | 0.6 | 4.6 | 4.6 |
| TOTAL | 23.2 | 6.7 | 6.0 | 9.8 | 12.2 | 10.7 | 45.4 | 68.6 |

[a] Including December 1973

Table 15.4 Quality of Recharge Effluent

| | | Year[a] | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1977 | | 1978 | | 1979 | | 1980 | | 1981 | |
| Parameter | Units | Winter | Summer | Winter | Summer | Winter | Summer | Winter | Summer | Winter | Summer |
| Suspended solids | mg/L | 36 | 11 | 20 | 13 | 21 | 24 | 41 | 23 | 31 | 36 |
| BOD | mg/L | 9 | 8 | 6 | 8 | 13 | 14 | 16 | 14 | 13 | 20 |
| $BOD_f$ | mg/L | 2 | 3 | 3 | 4 | 5 | 5 | 4 | 5 | 4 | 7 |
| COD | mg/L | 80 | 75 | 59 | 68 | 61 | 65 | 76 | 69 | 77 | 95 |
| $COD_f$ | mg/L | 53 | 58 | 43 | 48 | 34 | 43 | 37 | 44 | 39 | 50 |
| $KMnO_4$ | mg/L | 14 | 15 | 12 | 11 | 12 | 15 | 15 | 14 | 9 | 15 |
| $KMnO_{4f}$ | mg/L | 10 | 14 | 10 | 10 | 8 | 12 | 8 | 9 | 4 | 9 |
| $UV_{254}$ absorbance | $m^{-1}$ | — | — | — | — | — | — | 230 | 208 | 195 | 251 |
| TOC | mg/L | 22 | 24 | 17 | 18 | 15 | 17 | 21 | 22 | 27 | 30 |
| Ammonia, as N | mg/L | 5 | 1 | 5 | 2 | 5 | 1 | 7 | 3 | 8 | 5 |
| Total nitrogen | mg/L | 11 | 7 | 11 | 10 | 11 | 10 | 14 | 9 | 16 | 14 |
| Phosphorus | mg/L | 2.6 | 0.9 | 1.0 | 0.6 | 1.5 | 1.1 | 2.6 | 1.8 | 3.8 | 2.9 |
| pH | — | 9.8 | 10 | 10.3 | 9.5 | 9.1 | 8.8 | 9.2 | 8.7 | 8.9 | 9.1 |
| Alkalinity, as $CaCO_3$ | mg/L | 180 | 100 | 153 | 104 | 158 | 136 | 167 | 147 | 176 | 137 |
| Chlorides | mg/L | 175 | 225 | 213 | 307[b] | 224 | 227 | 160 | 199 | 173 | 233 |
| Dissolved solids | mg/L | 610 | 640 | 620 | 760[b] | 645 | 684 | 518 | 600 | 539 | 646 |

[a] Winter—December through April; Summer—May through November.

[b] Affected by accidental intrusion of seawater into main sewer.

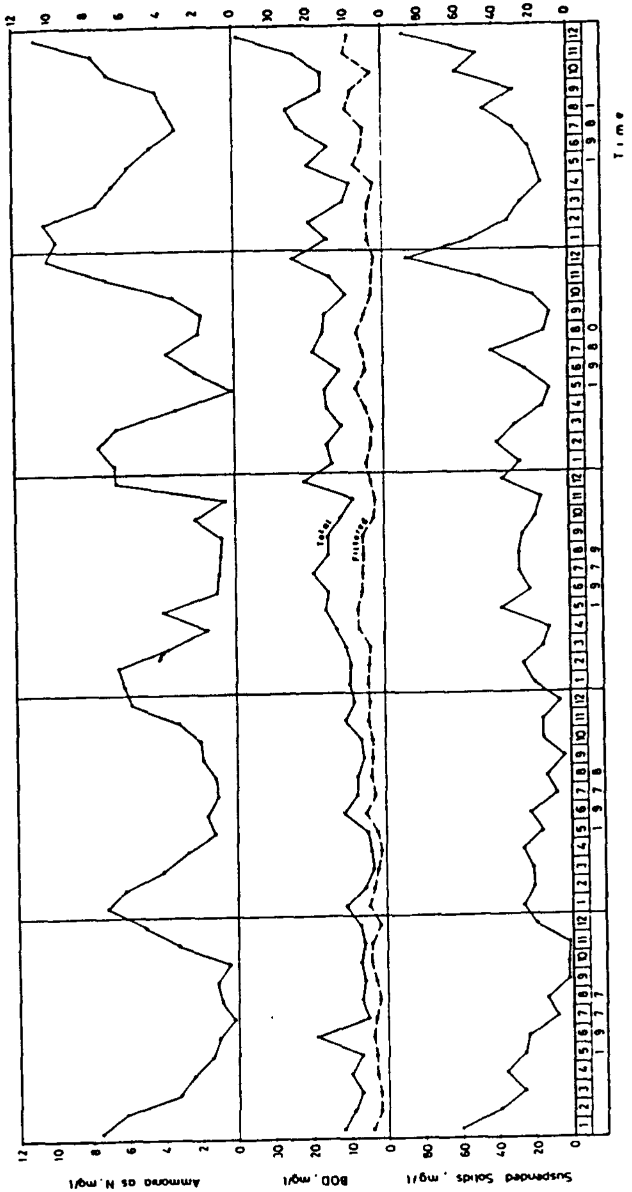

Figure 15.4 Quality of recharge effluent.

poorer performance of the clarification process, which is more sensitive to deviations of pH and magnesium dosage from optimum values.

Despite the high quality of the recharge effluent, its salinity—expressed as total dissolved solids or electrical conductivity—is much higher than that of the background water in the aquifer underlying the recharge area. With respect to major ions, a comparison of the two types of water represented by the Schoeller diagram (Figure 15.5) shows that chloride and sodium concentrations are much higher in the recharge water (about 6.5 mg/L) than in the groundwater (about 0.7 mg/L), whereas calcium, magnesium, and bicarbonate concentrations are approximately the same or sometimes even slightly lower in the recharge effluent, because of the softening effect of the high lime process.

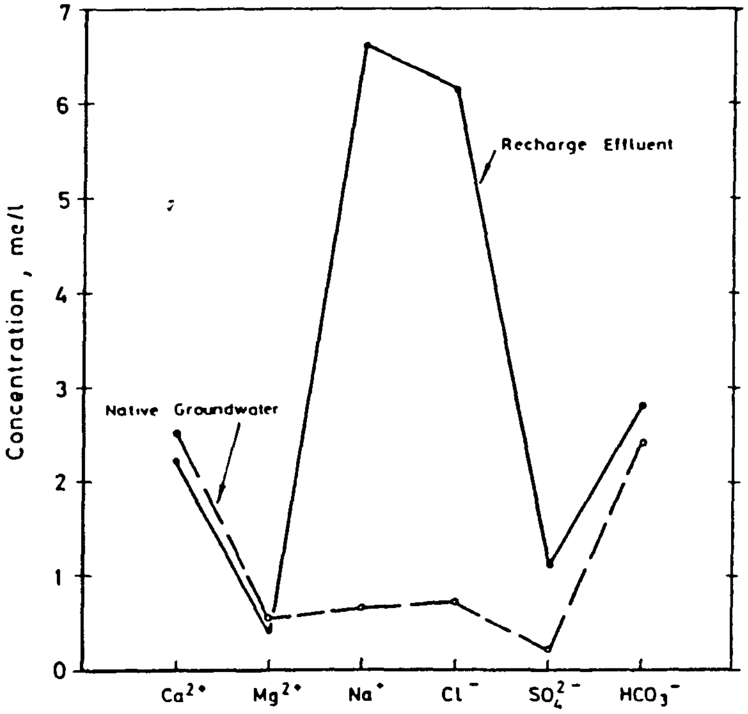

Figure 15.5 Ionic composition (Schoeller) of recharge effluent and native groundwater

## SPREAD OF THE RECHARGED EFFLUENT IN THE AQUIFER

In the Dan Region project, the chloride ion has served as a reliable tracer of the movement of recharged effluent in the aquifer, for two main reasons:

- It is virtually unaffected by any soil chemical, physical, or biologic process.
- There is a substantial difference (one order of magnitude) between chloride concentrations in the recharged effluent (150–250 mg/L) and in the native groundwater (20–25 mg/L); this permits detection of as little as 5 percent effluent in water pumped from a well.

The pattern of chloride concentration rise, which reflects the effluent percentage in the well water, is shown in Figure 15.6 for two representative wells located at different distances from the recharge basins.

The effluent percentage in a well at a given time is primarily a function of the distance of the well from the nearest recharge basin in full operation and of the recharge-pumping regime. In observation wells located close to the recharge basin, such as well 61 (60 m from the edge of basin 102, where recharge started in June 1977) a significant amount of effluent appeared after a relatively short time (a few months). In observation wells located further from the recharge basins, such as well 54 (about 300 m from the edge of basin 101, where recharge started in January 1977), a significant amount of effluent appeared after a longer time (more than a year). As the recovery wells are located at greater distances from the recharge basins, it can be assumed that pumping of significant amounts of effluent will occur only within a few years.

The pattern of lateral spread of the effluent in the aquifer in 1979, 1980, and 1981 (Figure 15.7) reflects essentially the distribution of the recharge volumes among various basins each year, as well as the pumping regime of the Dan wells. After 5 years of recharge (January 1977–December 1981) the recharge effluent had spread some 800 m east-west and some 600 m north-south from the center of the recharge area. The first signs of recharged effluent appeared in the nearest recovery well (Dan 5) in the second half of 1981. At present the recharged effluent is generally stored in the aquifer, where it displaces native groundwater toward the recovery wells.

## PURIFICATION OF THE RECHARGED EFFLUENT BY SOIL-AQUIFER TREATMENT (SAT)

Effluent percolating through the unsaturated zone and flowing in the aquifer is affected by numerous physical, chemical, and biological processes. The

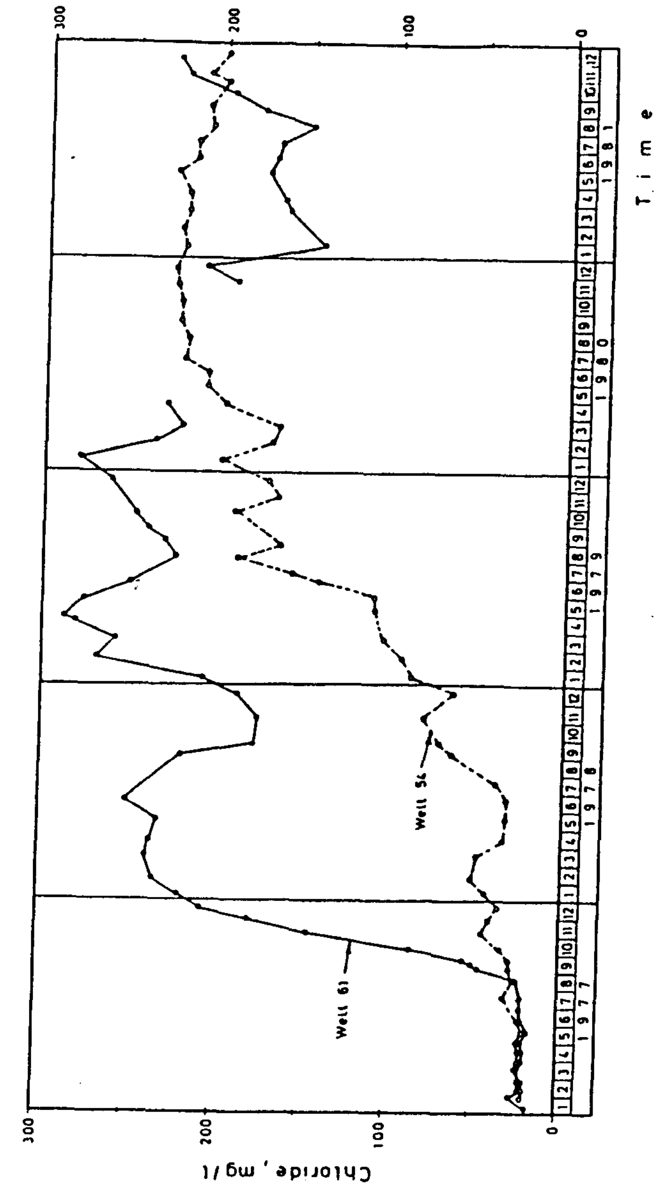

Figure 15.6 Chloride concentration as tracer of recharged water movement in the aquifer.

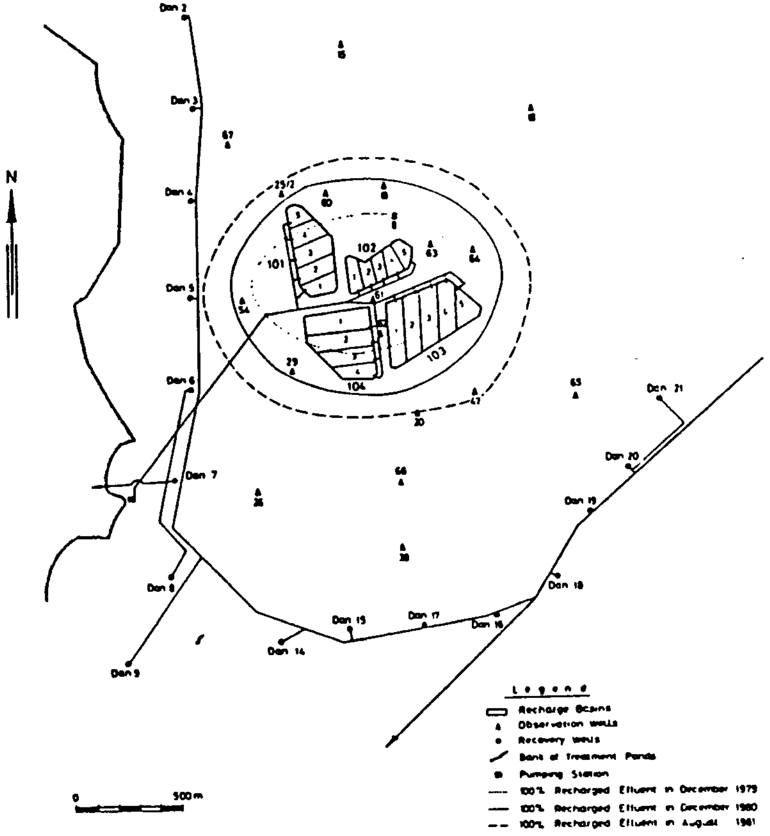

**Figure 15.7** Approximate spread of recharged water in the aquifer.

overall effect of these processes on effluent quality was evaluated by comparing the quality of the water sampled from various observation wells with the quality of the recharge effluent. During the period when the well water contains only a certain percentage of recharged effluent it was necessary to take into account the mixing of effluent with native groundwater. In such cases the measured concentration of a particular constituent in the well water was compared with the concentration expected in the well water (according to the effluent percentage calculated from chloride concentrations) as if it were not affected by passage through the soil-aquifer system.

Findings on water quality changes during groundwater recharge are presented with respect to general organics (expressed as COD, $KMnO_4$ consumption, and $UV_{254}$ absorbance); detergents; phosphorus; nitrogen; sodium, calcium and sodium adsorption ratio; trace elements; and toxic substances.

In order to better understand the purification process of the effluent during flow through the soil-aquifer system, the observation wells were grouped as follows:

*Group 1*—wells with a relatively short travel time from the nearest recharge basin (2–7 months) (Nos. 61, 62, 29, and 63); most of these wells have been pumping 100 percent recharged water for a long time (since 1978 or 1979), except well 29, which is close to basin 104, where recharge only started in 1979.

*Group 2*—wells with a longer travel time (a year or more), which started pumping recharged water more recently (in 1980 or 1981) (Nos. 19, 47, 54, and 60).

## Gross Organics

Organic compounds are presumably the main substances of concern in connection with wastewater reuse because of their great variety and the difficulties involved in their identification and measurement, on the one hand, and the lack of knowledge with regard to their health effects, on the other hand.

After biological and chemical treatments are carried out in the Dan Region Project (which does not include activated carbon adsorption), the concentration of gross organics expressed as COD, TOC, $KMnO_4$ consumption, or $UV_{254}$ absorbance is still relatively high, although the BOD concentration is low (Table 15.4). The role of the soil-aquifer system in further reducing the concentration of organics is, thus, particularly important in order to obtain high-quality reclaimed water. The removal of residual suspended solids (mostly organics) and of residual BOD was virtually complete in the soil-aquifer system.

## Permanganate Consumption

A considerable reduction in the organic content of the effluent—expressed as $KMnO_4$ consumption—occurred during percolation through the unsaturated zone and flow through the aquifer. Figure 15.8 reflects the fate of organics in wells 61 and 63, which are representative since they were the first to pump 100 percent recharged effluent.

The pattern of variation in the $KMnO_4$ consumption in these obser-

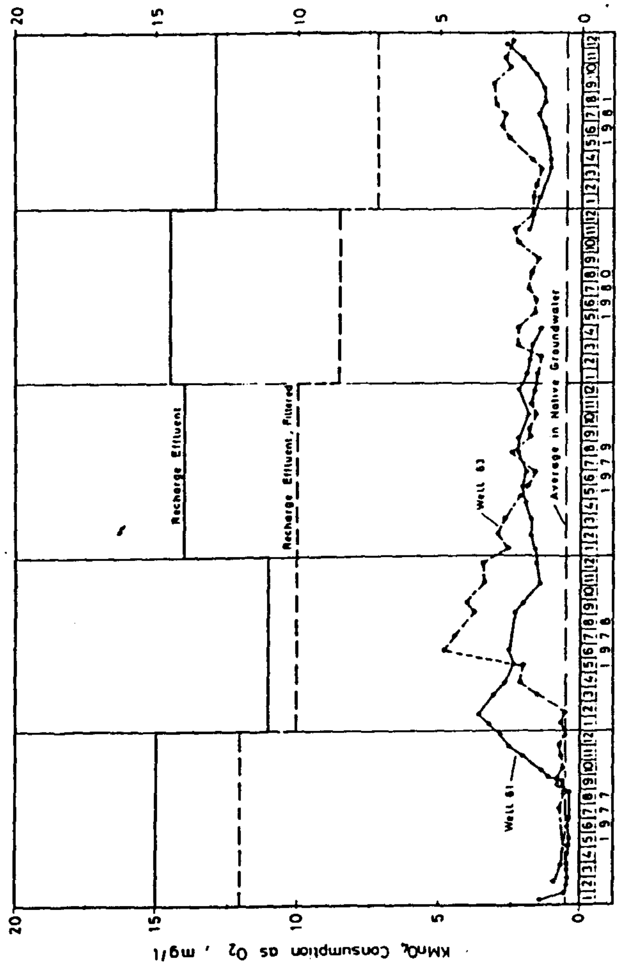

Figure 15.8 Organics expressed as $KMnO_4$ consumption before and after groundwater recharge.

vation wells consisted of two distinct periods. In the first period during which the effluent reached the well in increasing proportions, the $KMnO_4$ consumption gradually increased but only to levels that were less than those expected from the effluent percentage in the well. The removal efficiency of the soil-aquifer system was calculated from the effluent percentage, based on the chloride concentration in the well water and periodic average concentrations in the effluent according to the estimated travel time to the well. During this period the organics removal efficiency—expressed as percentage reduction of $KMnO_4$ consumption—was between 60 to 80 percent. At the end of this period the $KMnO_4$ consumption reached a peak of 4 to 5 mg per liter. In the second period, when the well was pumping 100 percent recharged effluent, the concentration of gross organics expressed as $KMnO_4$ consumption started decreasing and reached a low value (1–2 mg/L), which remained relatively stable in all the wells of Group 1. The gross organics removal efficiency of the soil-aquifer system, expressed as percentage reduction of $KMnO_4$ consumption, thus increased to 85 to 88 percent. The soluble organics removal in the soil-aquifer system (as measured by filtered permanganate consumption) was 75 to 78 percent; this figure is more representative for organics removal in the subsoil, since all the suspended organics are filtered out at the soil surface.

An exception to the above was observed in 1981 in wells 62/2 and 63, where higher values of $KMnO_4$ consumption (up to 3–4 mg/L) occurred, presumably because of development of anoxic-anaerobic conditions, which are less efficient in organics removal.

The wells of Group 2 presented slightly lower $KMnO_4$ consumption values—from 0.5 to 1.8 mg per liter.

### Chemical Oxygen Demand (COD)

Similar variation patterns during the time were noticed also for COD, which is another parameter measuring gross organics. In the fourth as well as in the fifth recharge years, the COD values in Group 1 wells ranged from 5 to 15 mg per liter; as above, higher values of 18 to 20 mg per liter were obtained in wells 62/2 and 63 in the last months of 1981. According to these COD results, the total and soluble organics decreased during recharge by 83 to 86 percent and by 71 to 74 percent, respectively. In Group 2 wells COD values were generally below 10 mg per liter.

### UV Absorbance

In recent years there has been increasing interest in the use of ultraviolet (UV) spectrophotometry for the evaluation of organic matter in water and wastewaters. A study carried out on nine types of water from the Dan

Region Sewage Reclamation Project indicated the feasibility of using $UV_{254}$ absorbance for routine monitoring of gross organics and its particular suitability for monitoring organic quality of reclaimed water and of effluent movement in the groundwater aquifer [6].

Thus $UV_{254}$ absorbance has been used since August 1979 as a routine parameter for organics monitoring along with $KMnO_4$ consumption and COD. Values of $UV_{254}$ absorbance in Group 1 wells ranged from 4 to 8 $m^{-1}$. In the last months of 1981 the values of $UV_{254}$ absorbance began to increase in wells 62/2 and 63 until it reached 10 $m^{-1}$ by the end of the year. According to these results, the gross organics concentration decreased by 60 to 80 percent in the soil-aquifer system.

The Group 2 wells had lower UV values, which ranged between 1 to 6 $m^{-1}$.

### Detergents

Detergents, which represent a specific class of organics, expressed a behavior similar to the gross organics parameters during this time. In the last two years (1980–1981), large fluctuation of detergents was observed in the Group 1 wells (50–400 μg/L); toward the end of 1981 detergent concentrations rose up to 600 μg per liter in wells 62/2 and 63. Based on the above concentrations, it appears that the removal of detergents in the soil-aquifer system fluctuated between 35 to 95 percent.

In the Group 2 wells detergent concentrations were generally below 200 μg per liter (78% removal), with few values up to 400 μg per liter in well 29.

The pattern of change in the organic content of the groundwater affected by the recharge effluent, namely the rise to a peak followed by descent to a stable value, indicates that the dominant process in the removal of organics is microbial degradation and not adsorption. The first period appears to correspond to the time needed for a new microbial population to develop, after adapting to the organic compounds found in the effluent. A similar behavior pattern was reported by Roberts et al. [7] for specific organic substances such as naphthalene, after injection of effluent into groundwater by wells.

This finding is of great importance in connection with the long-term behavior of the soil-aquifer as a treatment system, since it indicates that if the quality of the recharged effluent essentially remains the same and the recharge operation is properly managed to ensure that the bacterial population developed will continue to survive, a considerable reduction of the organic content of the effluent can be maintained for an indefinite time. In such a case, the capacity of the soil-aquifer system will never be exhausted with respect to organics removal.

### Phosphorus

Despite the considerable reduction of phosphorus concentration by tertiary chemical treatment, concentrations in the recharge effluent were still significant, especially in winter (about 1–3 mg/L). After five years of recharge, phosphorus concentrations in the observation wells, which have been pumping 100 percent recharged water for a long time or more recently, are still within the natural background range of 10 to 40 μg per liter. There are no indications of a decline in the soil capacity to fix phosphorus. The excellent phosphorus removal takes place presumably by adsorption in deep clay and silt layers as well as by calcium phosphate precipitation. Results from other research carried out in the Dan Region project in connection with the effluent seeped from the oxidation ponds showed that even when phosphorus concentrations in the percolating water were higher (10–12 mg/L), concentrations in groundwater containing a large percentage of effluent remained in the range of 10 to 50 μg per L.

The excellent removal of phosphorus by the soil-aquifer system seems to indicate that in groundwater recharge projects the removal of phosphorus in the biologic or chemical treatment step prior to recharge should not be of as great concern as it sometimes is.

### Nitrogen

The changes occurring during groundwater recharge in the concentration of nitrogenous compounds in sewage effluents have been extensively studied [8]. In the Dan Region Project the total nitrogen concentration in the native groundwater was about 2 mg per liter and consisted primarily of nitrates. The total nitrogen concentration in the recharge effluent was 7 to 16 mg per liter and consisted mainly of ammonia and organic nitrogen.

Nitrogen is found in the wells predominantly as nitrate ion. Concentrations in the Group 1 wells had the highest values (8–13 mg/L) during winter and spring; during summer and autumn the nitrate concentrations decreased to 1 to 5 mg per liter. In the Group 2 wells the nitrate concentrations fluctuated between 3 and 8 mg per liter. These data indicate that, while nitrification was complete and reliable, denitrification was partial and fluctuating [3]. This finding was to be expected, considering that the relatively short flooding cycles employed in the Dan Region Project favor the development of aerobic microbial populations in the soil-aquifer system.

In wells 62/2 and 63 a decrease in the nitrate concentration that started in June–July 1981 occurred in parallel with the appearance of small concentrations of ammonia: 0.16 mg per liter in August, which increased to 0.7 mg per liter at the end of the year, in well 62/2; 0.26 mg per liter in September 1981 and 0.4 mg per liter in October in well 63. In well 61 a similar decrease in the nitrate concentration occurred, but no ammonia appeared in this well. This phenomenon is very important, since it reflects a significant change in the nitrogen modification processes occurring in the soil-aquifer system; it is presumably a result of the availability of organic matter (as was mentioned above) and the development of anoxic conditions, which favor denitrification, nitrate decrease, and ammonia occurrence [9]. It should be remembered that during 1980 and 1981 the recharge operation took place virtually continuously (see Table 15.1) in one of the sub-basins of 102, and in basins 103 and 104, which are close to these wells. Also, the ammonia concentration in the recharge water was higher than in previous years, thus reducing the capacity of the soil to remove ammonia. This phenomenon will be followed up thoroughly in the future.

### Boron

Boron concentration is of particular importance in connection with the reuse of the effluent for unrestricted irrigation of agricultural crops. Removal of boron compounds by adsorption on magnesium hydroxide during high lime-magnesium treatment is presumably the main mechanism that reduces the boron concentration in the recharge effluent to acceptable limits of 0.3 to 0.5 mg per liter. The boron concentration in the native groundwater is only 0.02 to 0.03 mg per liter, or one order of magnitude lower than in the recharge effluent.

Boron was initially well removed during percolation through the entire unsaturated zone and flow through the aquifer (Figure 15.9), as indicated by boron concentrations measured in observation wells 61 and 63, which were much lower than those in the recharge effluent. However, after several months, boron concentrations (especially in well 63) gradually increased until they reached concentrations similar to those in the effluent.

The pattern of boron concentrations in the groundwater affected by the recharged effluent indicates a process of adsorption, which presumably occurs on clay layers as well as on the magnesium hydroxide usually present in the silt and sand fraction of arid zone soils [10].

After 5 recharge years, boron concentrations in the Group 1 wells fluctuated between 0.2 to 0.3 mg per liter, which is only slightly lower than the values in the recharge effluent. This means that, in the area close to the recharge basins, the adsorption capacity of the soil-aquifer system has been nearly exhausted. In some of the Group 2 wells, where boron concentrations are generally below 0.2 to 0.3 mg per liter, the boron removal process is still more pronounced.

### Sodium Adsorption Ratio (SAR)

The recharge effluent is somewhat softer than the native groundwater as a result of the high lime-softening process, but has a much higher sodium concentration (average 150 mg/L, as compared to 10 mg/L). Consequently, the SAR of the recharge effluent, which indicates the proportion between monovalent cations (basically Na) and divalent cations (Ca and Mg) was much higher (5–8) than that of the native groundwater (about 0.5). The effluent passage through the soil-aquifer system caused, during a certain period of time, a considerable reduction of the sodium concentration and, in parallel, an increase in hardness due to cation exchange between Ca and Mg on the one hand and Na on the other hand. The deviations of the concentrations of these three cations, measured in a certain well from those expected according to the effluent percentage in the well, confirmed that the responsible mechanism is indeed cation exchange [1].

The SAR of the water in Group 1 wells (e.g., wells 61 and 63) was much lower than that of the recharge effluent during a certain period of time, because of the lag in the rising of sodium to the concentrations found in the effluent an the corresponding increase in Ca and Mg concentrations (Figure 15.10). After a certain period of time the sodium concentrations

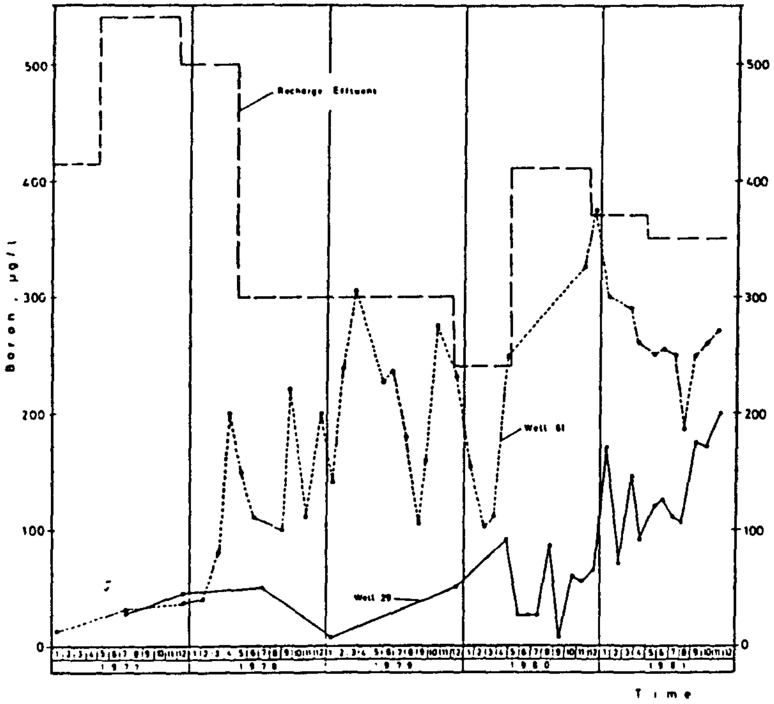

Figure 15.9 Boron before and after groundwater recharge.

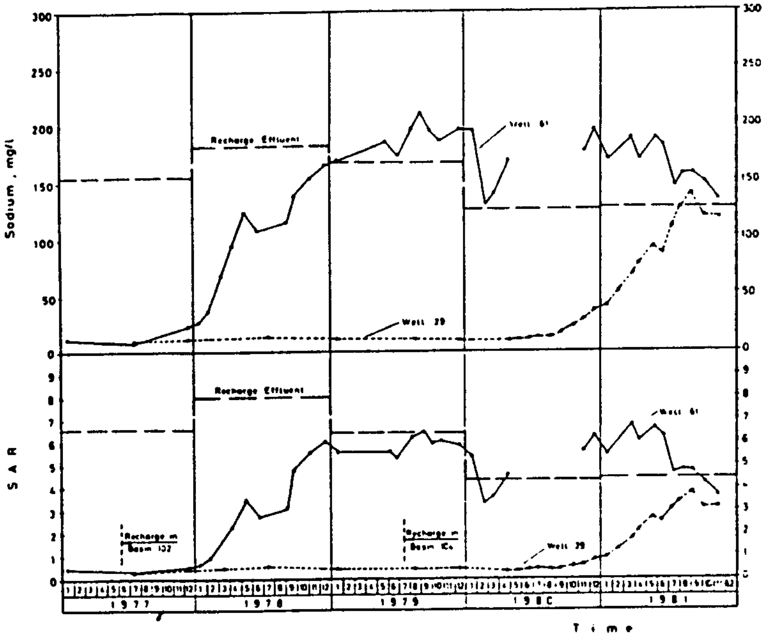

Figure 15.10 Sodium and SAR before and after groundwater recharge.

and hardness (and consequently also the SAR) in the groundwater reached values similar to those in the recharged effluent, thus indicating completion of the cation exchange process in this section of the aquifer. In 1981 the SAR increased in some of the Group 2 wells, as a result of the reduced efficiency of the cation exchange process (Figure 15.10). This situation will continue until the exhaustion of the cation exchange process in the respective section of the aquifer.

Although the SAR reduction is temporary, it is of great importance, considering that the reclaimed water is to be used mainly for irrigation of agricultural crops. The effect of soil-aquifer treatment on the water quality characteristics discussed above is presented in Table 15.5, by means of concentration ranges in the recharge effluent and well water.

### Trace Elements and Toxic Substances

The concentrations of 16 trace elements and toxic substances have been determined twice a year in four representative observation wells: 29, 54, 61, and 63. The following conclusions about the behavior of these trace elements and toxic substances in the soil-aquifer system can be drawn from the data collected during 5 years of recharge:

**Table 15.5** Purification Effect of Soil-Aquifer Treatment

| *Parameter* | *Unit* | *Recharge Effluent* | *Reclaimed (Well) Water* |
|---|---|---|---|
| COD | mg/L | 60–90 | 5–15 |
| $KMnO_4$ consumption as $O_2$ | mg/L | 10–15 | 1–2 |
| $UV_{254}$ absorbance | $m^{-1}$ | 20–25 | 2–6 |
| Detergents | mg/L | 0.5–1.5 | 0.1–0.3 |
| Kjeldahl N | mg/L | 7–14 | 0.2–1.5 |
| Ammonia as N | mg/L | 1–8 | <0.02 |
| Nitrate as N | mg/L | <0.01–0.5 | 2–10 |
| Total N | mg/L | 7–15 | 2–12 |
| Phosphorus | mg/L | 1–3 | 0.02–0.04 |
| Boron | mg/L | 0.3–0.5 | 0.1–0.3 |
| Sodium | mg/L | 120–160 | 100–150 |
| SAR | — | 4–7 | 3–4 |

> The concentrations of Cd, Cr, Se, Cu, Mn, K, and phenolic compounds are higher in the recharge water than in the native groundwater. Passage through the soil-aquifer system reduces the concentrations of most of these elements virtually to the levels in the native groundwater (Table 15.6).
>
> The concentrations of As, Pb, Hg, Ba, Ag, F, and cyanide are about the same in the recharge water and in the native groundwater.
>
> The well casing and sampling methods interfere with the analysis of Fe and Zn.

**Table 15.6** Removal of Trace Elements and Toxic Substances from Recharged Water During Passage through the Soil-Aquifer System

| *Parameter* | *Units* | *Recharge Water*[a] | *Natural Groundwater*[b] | *Observation Wells 29, 54, 61, 63*[c] |
|---|---|---|---|---|
| Cadmium | μg/L | <4 | <2 | <2.3 |
| Chromium | μg/L | 15 | 7 | <4.5 |
| Selenium | μg/L | 9 | 1 | <1.5 |
| Copper | μg/L | 18 | 7 | <6.4 |
| Manganese | μg/L | 27 | 12 | 19 |
| Potassium | mg/L | 25 | 0.6 | 3.2 |
| Phenol | μg/L | 4 | <1 | <1.5 |

[a] Five-year average.
[b] Average of seven analyses in observation well 29.
[c] Average of 16 analyses of 100 percent recharged water.

## FUNCTIONS OF GROUNDWATER RECHARGE

In addition to its major purification effect, groundwater recharge, as practiced in the Dan Region Project, fulfills a series of additional functions: It provides seasonal and multiannual storage, it is a safety barrier against any unpredictable quality deterioration, it increases the system's reliability and has an important psychological effect, since the consumers will be supplied from groundwater wells and not from the treatment plant outlet.

## ACKNOWLEDGMENTS

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# REUSO DE AGUAS SERVIDAS EN ISRAEL

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## 1. INTRODUCCION

En áreas donde los recursos hídricos naturales son escasos, las aguas servidas representan un recurso de agua no convencional, que puede ser utilizado eficazmente para agricultura, industria, recreación y recarga de acuíferos. Además, al reutilizar las aguas servidas en lugar de descargarlas a cuerpos de agua receptores, se disminuye considerablemente el peligro de la contaminación de los recursos hídricos naturales.

El reúso potable de las aguas servidas después de un tratamiento muy avanzado, es también posible y se practica en ciertas situaciones. Sin embargo, este tipo de reúso se encuentra a veces con la oposición de entidades a cargo de la salud pública, debido a las incertidumbres relacionadas con posibles efectos perjudiciales a largo plazo, y con la resistencia psicológica de la opinión pública.

La agricultura de Israel -un país semiárido en el norte y árido en el sur- depende casi totalmente del riego. La demanda de agua para riego representa un 65% de la demanda nacional (Cuadro 1). Ante la escasez de recursos hídricos naturales, las aguas servidas urbanas son consideradas como una importante fuente de agua, especialmente para fines de riego agrícola. Desde 1970 se han puesto en servicio numerosos sistemas de reúso de aguas servidas urbanas y otros se están diseñando y construyendo. La importancia a nivel nacional que se le asigna al reúso de aguas servidas ha estimulado el desarrollo de sistemas innovadores de tratamiento para reúso y ha generado un nuevo concepto de "tratamiento para fines de riego", que tiende a aprovechar tanto el agua como los fertilizantes (materia orgánica y nutrientes) contenidos en las aguas servidas.

En la actualidad ya se está reusando en Israel más de 40% del volumen total de aguas servidas de las redes de alcantarillados urbanos. Esta cifra crecerá hasta casi el 65% en 1986-87 año en que se prevé el comienzo de la operación de la segunda etapa del proyecto de la Región de Dan (Cuadro 2).

El reúso de aguas servidas a escala nacional a un costo relativamente bajo, posibilitó posponer considerablemente las grandes inversiones requeridas para la desalinización del agua de mar, el último recurso de agua disponible de Israel (Cuadro 1).

## 2. POSIBILIDADES DE REUSO

Existen numerosas posibilidades de reuso de las aguas servidas urbanas en función del nivel de tratamiento lo cual determina la calidad del efluente obtenido. En el Cuadro 3 se presenta una clasificación general de estas posibilidades según tres calidades generales de efluentes (baja, alta y muy alta).

Los sistemas de reúso para riego se pueden clasificar en tres categorías (A, B y C) según el tipo de cultivo, las restricciones impuestas al sistema de riego y el riesgo para la salud pública, dependientes de la calidad del efluente (1). Las categorías A y B incluyen "riego restringido" mientras que la categoría C corresponde a riego no restringido para todo tipo de cultivos incluso hortalizas y frutas para consumo humano crudo (Cuadro 4).

Debido a sus limitaciones, el riego de categorías A y B es generalmente aplicable a zonas limitadas en las cuales los planes de cultivos y las áreas a ser irrigadas con efluentes se hallan bien definidos. En cambio, el riego de categoría C se adecúa a proyectos de gran envergadura, cuyos planes agrícolas incluyen o podrían incluir cultivos que demandan agua de alta calidad y/o cultivos de exportación.

En Israel, la mayor parte de las aguas servidas se destinan actualmente al riego del algodón. En el proyecto de la Región de Dan (el área metropolitana del Gran Tel Aviv) las aguas servidas se recargan al acuífero para reúso futuro especialmente en riego no restringido.

A continuación se describen varios sistemas innovadores de tratamiento para reúso desarrollados y aplicados en Israel y se presenta el proyecto más importante de ese país -el proyecto de reúso de la Región de Dan- cuya primera etapa se halla en operación desde hace varios años.

## 3. METODOS DE TRATAMIENTO PARA REUSO

### 3.1 Lagunas facultativas con recirculación (LFR)

El proceso de tratamiento de aguas servidas en lagunas de oxidación es un proceso sencillo y muy común en Israel. De un total de 350 plantas de tratamiento de diversas magnitudes que se encuentran en operación en Israel, unas 250 incluyen lagunas de oxidación que procesan aproximadamente el 50% del caudal total tratado (2). No obstante, el proceso de tratamiento en lagunas de oxidación posee dos limitaciones:

i) Dado que el oxígeno necesario para proveer condiciones aeróbias es generado por fotosíntesis, las instalaciones requieren extensas superficies para exponer las algas a la irradiación solar.

ii) El funcionamiento correcto de las lagunas de modo que no causen molestias ambientales depende en gran medida de condiciones fuera del control del operador (temperatura, radiación solar, tasa de desarrollo de las algas, comportamiento de los sedimentos en el fondo) lo que puede resultar en la generación de olores, especialmente en condiciones de alta carga orgánica.

Uno de los métodos desarrollados para evitar el desprendimiento de olores de las lagunas de oxidación es la recirculación del efluente de la última laguna a la zona de entrada de las aguas servidas crudas.

El complejo mecanismo de recirculación que mejora considerablemente el funcionamiento de las lagunas incluye: la mezcla inmediata del líquido que contiene algas y bacterias con aguas servidas crudas, agregado de líquido saturado de oxígeno a las zonas con demanda máxima de oxígeno, uniformización de la distribución de la carga orgánica sobre la superficie de las lagunas y dilución inmediata de las aguas servidas crudas.

Este mecanismo permite aumentar la carga orgánica por unidad de superficie de las lagunas, resultando ello en una menor superficie de las lagunas con recirculación.

Las lagunas con recirculación permiten también ejercer cierto control sobre el proceso por medio de la variación de la tasa de recirculación.

Las lagunas facultativas con recirculación consisten en tres o más unidades en serie, con una profundidad de aproximadamente 1.5 m (Cuadro 5). La carga orgánica en la primera laguna denominada primaria, es alta, mientras que en las siguientes lagunas denominadas secundaria y terciaria la carga es mucho menor (3).

En el fondo de estas lagunas, se acumula cierta cantidad de lodo, pero no es necesaria su remoción sino sólo una vez cada cinco a diez años.

En este tipo de lagunas la remoción total de materia orgánica soluble es similar a la obtenida en procesos más avanzados de tratamiento secundario (80-90%), pero el efluente contiene concentraciones elevadas de sólidos suspendidos, de los cuales una gran parte son algas.

En el Cuadro 6 se presentan los criterios principales de diseño de lagunas facultativas con recirculación. Las lagunas con recirculación de Tel Aviv fueron construidas y operan con base en dichos criterios. La experiencia de doce años de operación de esta planta demuestra que en verano es posible incrementar la carga orgánica en un 50% sin causar molestias.

Criterios similares han sido utilizados por TAHAL también para el diseño de una planta de lagunas facultativas con recirculación en la Ciudad de México, actualmente en construcción (4).

La operación de las lagunas puede verificarse por la determinación del potencial de producción de oxígeno por el método de "botellas blancas y negras", desarrollado en el curso de la operación de las lagunas de Tel Aviv. En este método se toman muestras del efluente de las lagunas primarias (capa superficial, al medio día) en botellas transparentes (blancas) y en botellas opacas (negras).

Las botellas transparentes permiten la fotosíntesis lo que facilita el desarrollo de algas, mientras que las opacas la inhiben. En consecuencia, en las botellas blancas hay tanto demanda como producción de oxígeno, mientras que en las negras hay sólo demanda. Se mide la concentración de oxígeno disuelto en cada botella a intervalos de hasta dos horas. La diferencia entre la concentración de oxígeno disuelto en las botellas blancas y las negras refleja el potencial neto de producción de oxígeno en las lagunas, que debe ser positivo.

El método de las botellas blancas y negras permite no sólo prever la ocurrencia de un posible estado de emergencia en las lagunas sino también aumentar la carga orgánica de las lagunas (si el potencial neto alcanza valores positivos elevados). Por otra parte, si el potencial neto presenta valores bajos, se debe disminuir el flujo a las lagunas para reducir la carga orgánica aumentando, automáticamente, la tasa de recirculación.

## 3.2 Tratamiento en reservorios profundos (TRP)

Uno de los componentes importantes de un proyecto de reuso de aguas servidas para riego es el almacenamiento estacional que, debido a las fluctuaciones de la demanda agrícola de agua por un lado y el suministro casi constante de efluente durante el año por el otro, resulta generalmente imprescindible.

El tratamiento en reservorios profundos combina los procesos de tratamiento en lagunas facultativas con el almacenamiento estacional de efluentes. Un reservorio profundo en el marco de un sistema de tratamiento de aguas servidas constituye en realidad una laguna facultativa de gran profundidad (6-10 m) en la cual el nivel de agua varía durante el año (Cuadro 7).

Los objetivos de un eficiente reservorio profundo para tratamiento de aguas servidas son:

- Proveer el tiempo de detención necesario para el almacenamiento estacional.
- Lograr el máximo nivel de tratamiento.
- Evitar la creación de molestias ambientales, tales como malos olores.

La condición principal para evitar la generación de molestias ambientales es la presencia de oxígeno disuelto en el estrato superior del reservorio, en una concentración suficiente en relación con la carga orgánica (concentración de DBO) de las aguas servidas que llegan al reservorio.

La estimación de la carga máxima de DBO se basa en principios similares a los aplicados en el caso de lagunas facultativas de oxidación; sin embargo, el caso de los reservorios profundos es más complejo por el hecho de que la superficie del agua varía significativamente con las estaciones del año. Esto último resulta de la pendiente pronunciada de los taludes que determina una notable diferencia entre las áreas del fondo y de la superficie. Como consecuencia de la variación de la superficie se produce también una variación del valor permisible de la carga de DBO, por lo que el diseño debe tomar en cuenta las condiciones críticas existentes en el momento de llenado y vaciado del reservorio, considerando además las variaciones de la temperatura.

Diversas fórmulas desarrolladas con base en extensos estudios permiten diseñar un reservorio profundo de tratamiento o determinar la capacidad y modo de operación de reservorios existentes. Es preciso destacar que el diseño de un reservorio de este tipo depende mucho de las condiciones específicas del lugar, tales como la topografía, suelos, condiciones climáticas, distancia hasta la zona urbana más cercana, etc.

El tratamiento en reservorios profundos está incluido en numerosos esquemas pequeños de reúso, así como en los proyectos de reúso de las ciudades de Haifa y Jerusalén.

## 3.3 Tratamiento cal-magnesio con reúso de los coagulantes químicos

Un sistema de tratamiento muy eficiente utilizado tanto para la purificación de aguas servidas como para el ablande de agua potable es el tratamiento con cal a altos valores de pH.

Uno de los compuestos químicos importantes que se forma durante tal tratamiento es el hidróxido de magnesio $Mg(OH)_2$ que tiene capacidad de remoción de algas y de elementos tales como el boro.

La presencia de una concentración suficiente de iones de magnesio resulta imprescindible para lograr la clarificación eficiente de efluentes de lagunas de oxidación cargadas con algas (5). Cuando la concentración de magnesio en las aguas servidas no es suficiente, es necesario agregar al proceso sales de magnesio tales como $MgCl_2$, lo que aumenta el costo del proceso y complica su operación.

La recirculación del lodo de alto pH y rico en $Mg(OH)_2$ y su mezcla con el efluente permite resolubilizar el magnesio y así se puede evitar la adición de sales de magnesio (Cuadro 8). Asimismo, se disminuye la demanda de cal gracias a la utilización del exceso de cal en el lodo y al elevado pH del efluente obtenido en el tanque de mezcla (6).

Otra ventaja significativa de este proceso consiste en la conversión del lodo de alto pH rico en $Mg(OH)_2$, que es gelatinoso y de difícil desecación, en un lodo de bajo pH y sin $Mg(OH)_2$, que resulta más fácil de desecar.

Esta configuración nueva del proceso cal-magnesio, que ha sido estudiada durante varios años en laboratorio y planta piloto, está en operación a gran escala en el Proyecto de la Región de Dan (6).

### 3.4 Tratamiento suelo-acuífero (TSA)

El sistema TSA, tal como su nombre lo indica, consiste en la purificación del efluente en el curso de su percolación a través del suelo y su flujo en el acuífero (7).

En el TSA, las aguas servidas reciben cierto tratamiento (a nivel primario o secundario) antes de su recarga al acuífero; el sistema suelo-acuífero provee un tratamiento adicional a nivel muy elevado por medio de una combinación de procesos físicos, químicos y biológicos.

El sistema consiste en campos de recarga operados intermitentemente en los cuales el efluente se infiltra al acuífero; allí el efluente avanza hacia los pozos de bombeo rodeando la zona de recarga que lo extraen para reuso (Cuadro 9).

El sistema TSA comprende tres instalaciones principales:

(i) Campos de recarga

(ii) Pozos de observación

(iii) Pozos de recuperación.

Campos de recarga. Los campos de recarga deben ubicarse en suelos de alta capacidad de infiltración (arena).

La operación de los campos de recarga se efectúa mediante inundación y secado intermitentes a efectos de permitir el mantenimiento de tasas elevadas de infiltración en los estratos superiores del suelo y también la penetración del oxígeno en el suelo que favorece el proceso de purificación biológica-aeróbia.

El ciclo de inundación y secado puede variar de acuerdo a los objetivos del sistema. Por ejemplo, períodos cortos de inundación aseguran condiciones aeróbias más favorables para la descomposición de materia orgánica y la nitrificación, mientras que períodos largos de inundación favorecen la desnitrificación.

La relación entre el tiempo de secado y el de inundación depende de las condiciones climáticas de la zona (precipitación, evaporación, humedad relativa), pero oscila generalmente entre 1 a 1 y 3 a 1.

El ciclo operativo adoptado en el Proyecto Dan es de 24 horas de inundación y 48 horas de secado. Es preciso aclarar que la variación de este patrón no afecta las dimensiones de los campos de recarga y de las tuberías de distribución.

Luego de cierto período de operación de los campos de recarga, la tasa de infiltración tiende a disminuir. A fin de rehabilitar la tasa de infiltración se requieren operaciones de mantenimiento con maquinaria adecuada para limpiar el estrato superficial del suelo.

El parámetro de diseño más importante para los campos de recarga es la carga hidráulica, que difiere de la tasa de infiltración instantánea en que toma en cuenta los períodos de secado, períodos de mantenimiento, etc. La carga hidráulica es en realidad la tasa anual promedio de infiltración.

La tasa de recarga de efluentes en campos de recarga está generalmente relacionada con la tasa de infiltración de agua clara y depende principalmente de la calidad del efluente recargado (especialmente su turbidez) y del ciclo de operación de los campos de recarga.

El desarrollo de algas en los campos de recarga que puede ocurrir en ciertas condiciones puede limitar la capacidad de infiltración.

Pozos de observación

Para controlar el proceso TSA se requiere una serie de pozos de observación de diámetro pequeño ubicados entre los campos de recarga y los pozos de recuperación. La finalidad de estos pozos es el muestreo del efluente que ya ha sido sometido al proceso TSA antes de su llegada a los pozos de recuperación. La profundidad de los pozos de observación así como las técnicas de muestreo empleadas son factores importantes que afectan la confiabilidad de los resultados correspondientes a las muestras de agua extraídas de estos pozos.

Si hay una gran diferencia entre la salinidad del efluente recargado y la del acuífero natural, resulta relativamente fácil determinar el porcentaje de efluente recargado contenido en cada muestra de agua subterránea.

Pozos de recuperación

Estos pozos tienen la doble función de bombear el agua recargada después de que ha sido sometida al proceso TSA y crear una barrera hidráulica que confina la zona del acuífero asignada al TSA separándola del resto del acuífero.

Debido al hecho de que el efluente recargado desplaza el agua subterránea natural y la "empuja" hacia los pozos de recuperación, éstos bombean agua subterránea natural al principio de la operación; más adelante el agua bombeada contiene un cierto porcentaje de efluente que se va incrementando hasta que en la etapa final los pozos de recuperación bombean agua que en su mayor parte consiste en efluente recargado que ha sido sometido al TSA.

## Pre y Post-tratamiento

La incorporación del proceso TSA en esquemas de reuso debería consistir de los siguientes elementos adicionales: pre-tratamiento antes del TSA por medio de procesos que no sólo no reiteran a los que ocurren en el suelo sino que los mejoran, y post-tratamiento luego del TSA a través de procesos que pueden mejorar aún más la calidad del agua bombeada del acuífero. El agua obtenida del pretratamiento y TSA es de alta calidad y puede ser suministrada para usos no potables, mientras que el agua luego de post-tratamiento puede ser suministrada aún para usos potables, incluyendo recarga de acuíferos potables (Cuadro 10).

Algunos de los procesos que parecen apropiados para servir como pretratamiento son: sedimentación primaria, precipitación química primaria (sedimentación primaria con ayuda de compuestos químicos) y tratamiento biológico de bajo costo tal como lagunas de oxidación o lagunas aireadas. La selección del método de pre-tratamiento más apropiado depende de tales factores como: disponibilidad de terreno, costo de energía, origen y concentración de las aguas servidas. En caso de que la disponibilidad de terreno sea un problema, las lagunas de oxidación deben ser complementadas con algún proceso de separación de algas, a fin de reducir el riesgo de colmataje de los campos de recarga.

En caso de que el área de recarga se halle situada en la vecindad de zonas urbanas, el pre-tratamiento podría ser dictado por la necesidad de evitar molestias ambientales.

Allí donde ya existe una planta de lodo activado, la incorporación del TSA puede eliminar la costosa etapa de aeración o alternativamente, permitir el incremento de la carga orgánica de la planta. Ello permitiría aumentar el volumen de aguas servidas tratadas sin necesidad de ampliar la planta. Reduciendo el nivel de pre-tratamiento y añadiendo algún tipo de post-tratamiento, el sistema puede ser optimizado en su conjunto con referencia a costos y calidad del efluente.

Los procesos que parecen apropiados como post-tratamiento son: adsorción con carbón activado granular para eliminación de compuestos orgánicos de biodegradación difícil; intercambio de iones y osmósis inversa para la eliminación de iones específicos y salinidad total; ablande y desinfección.

## 4. EL PROYECTO DE LA REGION DE DAN (ZONA METROPOLITANA DEL GRAN TEL AVIV)

El Proyecto de Reuso de las Aguas Servidas de la Región de Dan consiste en el tratamiento de las aguas servidas urbanas de la zona metropolitana del Gran Tel Aviv, la recarga del efluente tratado al acuífero utilizando el sistema TSA, y el reuso de este efluente altamente depurado, mezclado con agua subterránea natural y con agua de la red nacional de abastecimiento. Esta agua se utilizará últimamente para fines agrícolas e industriales y para usos municipales excepto el consumo humano; en la actualidad parte del agua recuperada en este proyecto se utiliza para fines potables (8).

El proyecto persigue dos finalidades principales: producir volúmenes importantes de agua aprovechable, ayudando así a compensar la diferencia existente entre la creciente demanda de agua del país y la limitada disponibilidad de sus recursos hídricos naturales; y detener la contaminación de las playas de Tel Aviv y del Mediterráneo, causada por la descarga de aguas servidas.

En su última etapa, a fines de este siglo, el proyecto servirá a una población total de 1.5 millones de habitantes y contará con una capacidad de más de 100 millones de $m^3$ por año. Este proyecto es el mayor y más avanzado de los sistemas de reuso de aguas servidas de origen urbano planeados para los años venideros en el marco del plan nacional de desarrollo de recursos hídricos no convencionales de Israel.

La primera etapa del Proyecto, que cubre las zonas sur de la región metropolitana de Tel-Aviv, opera en su totalidad, desde 1977. La segunda etapa del Proyecto, que cubre las zonas norte y este de la región, está ya casi completamente construida y su funcionamiento se iniciará en 1986-87.

En la primera etapa del Proyecto, más de 25 millones de $m^3$ son bombeados al sitio de la planta de tratamiento ubicado a unos 20 km al sur del centro de Tel Aviv (Cuadro 11).

Las aguas servidas son sometidas a tratamiento biológico en lagunas facultativas con recirculación, y a tratamiento químico por el proceso cal-magnesio a altos valores pH seguido por la detención del efluente tratado con cal en lagunas de pulido, cuyas funciones principales son: reducción de la concentración del amoníaco y reducción de la pH sin aireación mecánica y sin adición de productos químicos (Cuadro 12).

El alto grado de purificación logrado por el sistema TSA permitió la reducción del nivel de pre-tratamiento sin afectar la calidad final del agua recuperada. Desde 1985, parte del efluente (aproximadamente un cuarto del volumen total) pasa de las lagunas de oxidación directamente (sin tratamiento con cal) a la última laguna de pulido, de donde el agua se bombea a los campos de recarga.

El efluente así tratado es recargado al acuífero por medio del proceso TSA arriba descrito (9).

En la etapa final del proyecto, cuando los pozos de recuperación bombearan casi exclusivamente agua recargada, ésta se distribuirá sólo para usos no potables, mediante una red independiente que actualmente se encuentra en construcción.

Hasta llegar a dicha situación, la mayor parte del agua recargada se almacena en el acuífero, en el cual se halla desplazando el agua subterránea natural hacia los pozos de recuperación. En consecuencia, la mayor parte de estos pozos aún bombean agua subterránea natural. Cuatro pozos ubicados relativamente cerca de los campos de recarga bombean una mezcla de efluente recargado y agua natural. Sin embargo, el porcentaje de agua que origina del efluente recargado en la red de distribución es inferior al 5% debido a la dilución con agua procedente de otros pozos y con agua del Acueducto Nacional.

Los resultados de los análisis de calidad de agua en las diversas etapas de tratamiento (promedios obtenidos en 1984) se presentan en el Cuadro 13 para orgánicos y nutrientes y en el Cuadro 14 para metales pesados y elementos tóxicos (10).

La combinación de procesos biológicos y físico-químicos y la incorporación del proceso TSA aseguran una alta calidad de agua, que es adecuada para todos los usos no potables. La adición del proceso de tratamiento con carbón activado granular al agua bombeada por los pozos de recuperación, que funciona en la actualidad como proyecto piloto en uno de los pozos del proyecto, permitirá también el reuso potable futuro de este agua, si y cuando fuese necesario. El objetivo principal de este proceso es de reducir la concentración de las sustancias orgánicas disueltas (11).

## 5. CONCLUSIONES

1) Es factible del punto de vista tecnico-económico reutilizar las aguas servidas para agricultura, industria, recreación y recarga de acuíferos.

2) En ciudades ubicadas cerca del mar, u otros cuerpos receptores de agua, el reuso de las aguas servidas persigue dos finalidades: producir agua aprovechable para diversos usos y detener la contaminación de las playas y del mar o de los recursos hídricos.

3) Algunos de los procesos de tratamiento adecuados para reuso son: lagunas facultativas con recirculación (LFR), tratamiento en reservorios profundos (TRP), tratamiento cal-magnesio con reuso de los coagulantes químicos y tratamiento suelo-acuífero (TSA).

4) El tratamiento suelo-acuífero (TSA) puede ser utilizado, cuando las condiciones hidrogeológicas lo permiten, como el núcleo del sistema de reuso. Este sistema de bajo costo y sencilla operación es muy confiable de punto de vista de la calidad del efluente final y además provee también el almacenamiento estacional y multi-anual necesario especialmente para el reuso agrícola.

5) Los procesos de pre-tratamiento y post-tratamiento a ser utilizados conjuntamente con el proceso TSA deben ser determinados cuidadosamente, tomando en cuenta la capacidad de tratamiento del sistema suelo-acuífero.

6) La combinación pre-tratamiento y TSA puede asegurar el suministro de agua de muy alta calidad para usos no-potables. El post-tratamiento (por ejemplo por medio del proceso de absorción con carbón activado) puede aún asegurar el suministro de agua potable, si fuese necesario.

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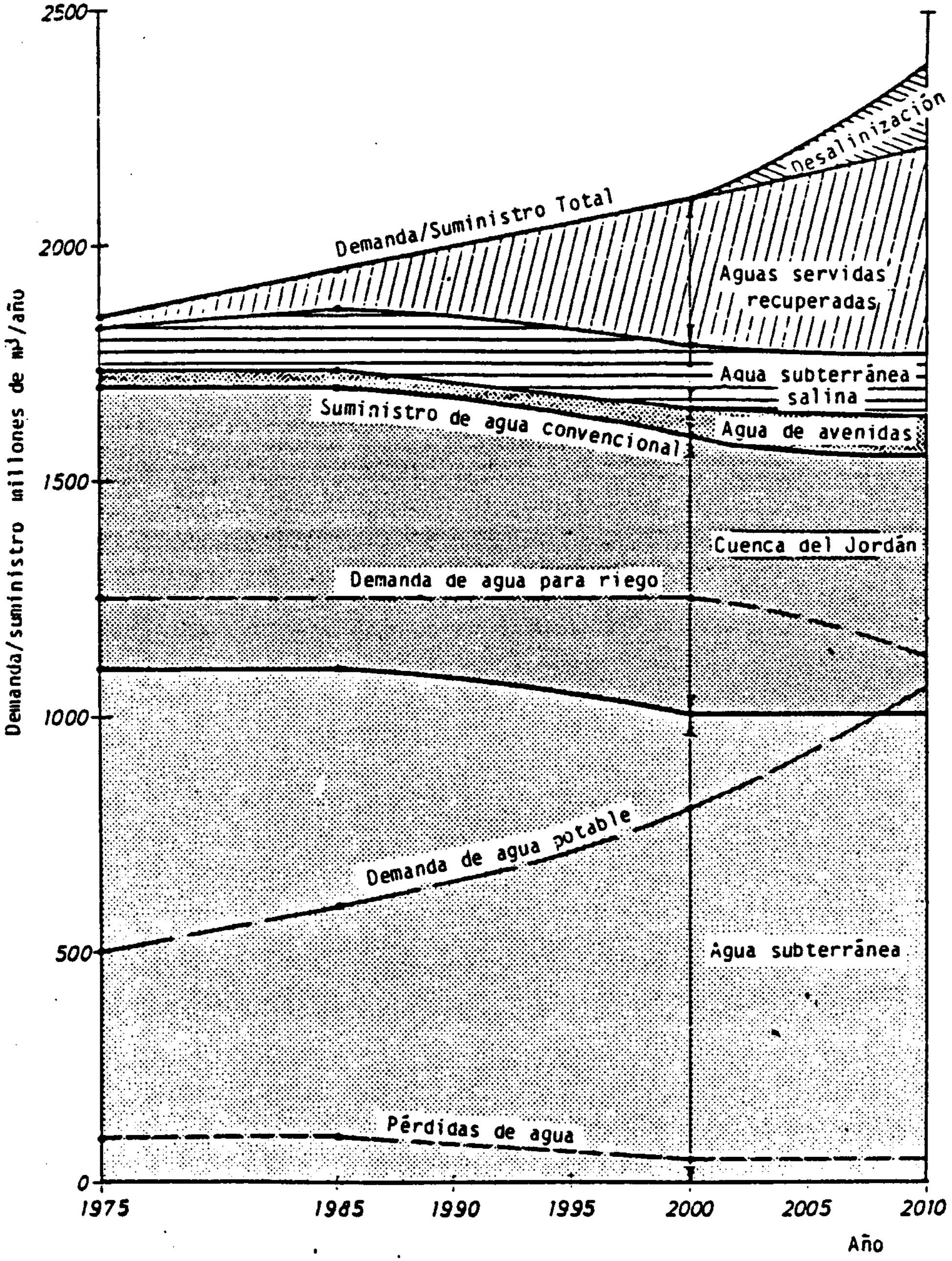


CUADRO 1: DEMANDA DE AGUA Y FUENTES DE SUMINISTRO

## Cuadro 2: Reuso de Aguas Servidas en Israel

| | millones de $m^3$/año |
|---|---|
| Potencial de aguas servidas urbanas | 250 |
| Aguas servidas con alcantarillado urbano | 220 |
| Aguas servidas con tratamiento | 140 (190) |
| Descarga al mar | 70 (20 |
| Descarga a corrientes de agua | 60 |
| Reuso | |
| - Riego | 70 |
| - Proyecto Región Dan | 20 (70) |
| Reuso total | 90 (140) |
| Porcentaje del total con alcantarillado | 41% (64%) |

Las cifras en paréntesis incluyen la segunda etapa del proyecto de la Región de Dan, que será puesto en operación en 1986-87.

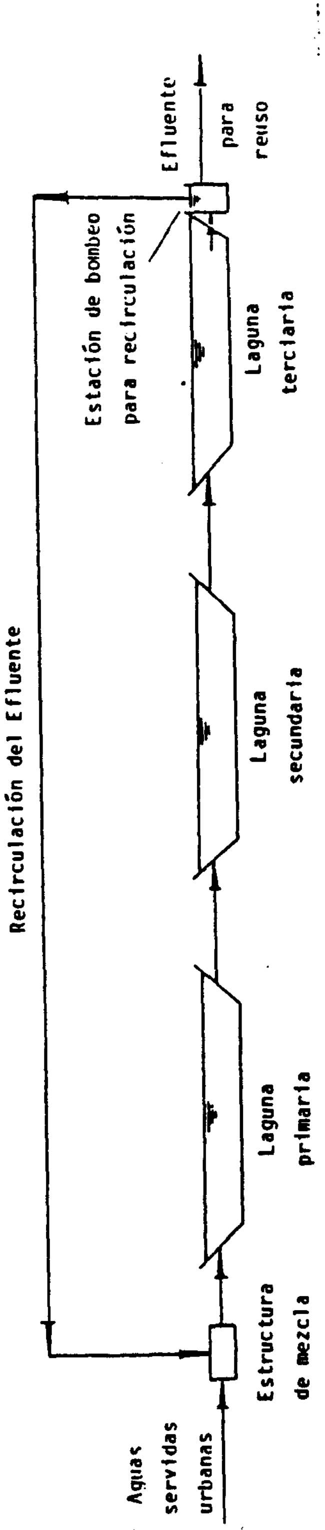
Cuadro 3: Posibilidades de Reúso de Aguas Servidas

| Categoría de reuso | Tipo de reúso | Calidad General Requerida |
|---|---|---|
| Reuso Agrícola | Riego restringido | Baja |
| | Riego no restringido y abrevamiento del ganado | Alta |
| | Uso general en la finca | Muy alta |
| Reuso Industrial | Usos secundarios | Baja |
| | Agua de proceso | Alta |
| | Agua de proceso para la industria alimenticia | Muy alta |
| Reuso Recreacional | Especializado o de contacto parcial con el cuerpo | Baja |
| | General o de contacto completo con el cuerpo | Alta |
| Reuso Municipal | No doméstico (riego de parques) | Baja |
| | Doméstico no potable | Baja |
| | Potable, indirecto (recarga del acuífero) | Alta |
| | Potable, directo (cañería a cañería) | Muy alta |

Cuadro 4: Categorías de Cultivos Regados con Efluentes Tratados

| ATEGORIA DE CULTIVO | CATEGORIA "A" | CATEGORIA "B" | CATEGORIA "C" |
|---|---|---|---|
| DEFINICIONES DE CULTIVOS Y RESTRICCIONES | 1. Cultivos no destinados al consumo humano (por ejemplo, algodón).<br>2. Cultivos que habitualmente son procesados por calor o desecación antes del consumo humano (granos, oleaginosas, remolacha azucarera).<br>3. Hortalizas y frutas producidas exclusivamente para envasado u otro tipo de procesamiento que destruye efectivamente los patógenos.<br>4. Cultivos forrajeros u otros cultivos para la alimentación animal que son desecados al sol antes del consumo.<br>5. Riego de parques en áreas cercadas inaccesibles para el público (viveros, bosques, cinturones verdes). | 1. Pasturas, cultivos forrajeros frescos para consumo.<br>2. Cultivos para consumo humano que no tienen contacto directo con aguas servidas, siempre que no se comercialicen las cosechas caídas (frutales, cultivos en estacas, tomate, pepino, etc. y vid regada por inundación o por goteo).<br>3. Cultivos para consumo humano que habitualmente se sirven cocidos (papa, berenjena, remolacha).<br>4. Cultivos para consumo humano que se sirven crudos pero cuya cáscara no es consumida (melón, cítricos, banana, nueces, maní, etc.) | 1. Todo cultivo para consumo humano que habitualmente se sirve crudo y que se cultiva en contacto directo con efluentes (hortalizas, tales como: lechuga, tomate, etc. y frutas).<br>2. Riego de parques y prados con libre acceso al público inmediatamente tras el riego, siempre que durante el riego por aspersión estas áreas se encuentren cercadas y no haya acceso libre a por lo menos 15 m de la zona regada. |
| IDAD REQUERIDA DE EFLUENTE | BAJA | INTERMEDIA | ALTA |

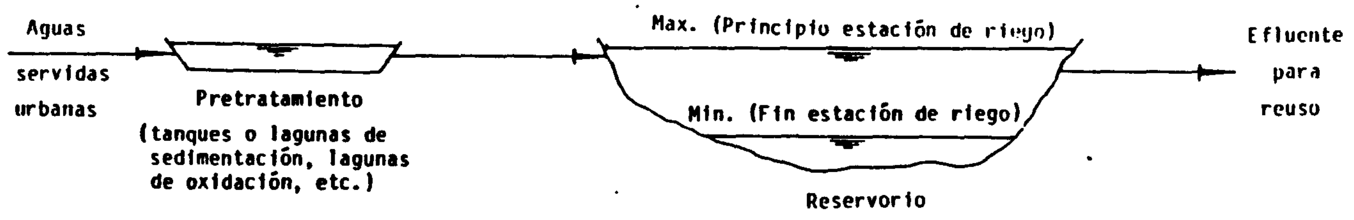
Fuente: TAHAL - Reúso de las aguas servidas del sur de Lima, Perú (1).



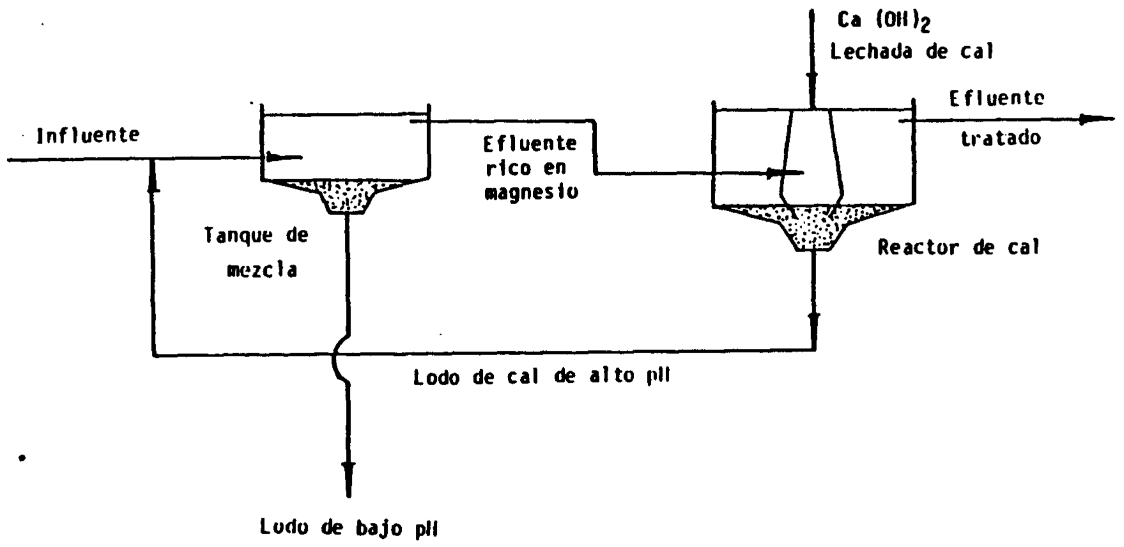
CUADRO 5: ESQUEMA DE TRATAMIENTO EN LAGUNAS FACULTATIVAS CON RECIRCULACION (LFR)

Cuadro 6: Criterios de Diseño para Lagunas Facultativas con Recirculación

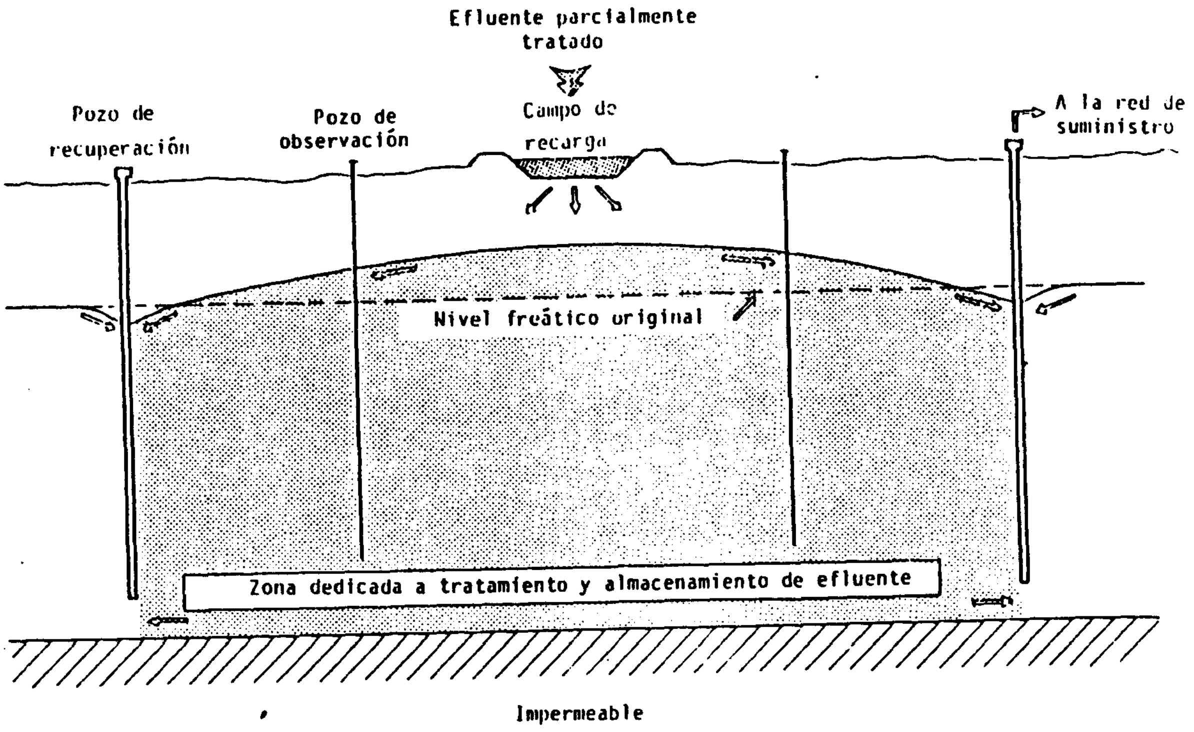
| PARAMETRO | UNIDAD | VALORES RECOMENDADOS |
|---|---|---|
| Profundidad | m | 1.2 - 1.8 |
| Carga orgánica en la laguna primaria | Kg DBO/d/ha | 400 - 600 |
| Carga orgánica promedio en todas las lagunas | Kg DBO/d/ha | 170 - 200 |
| Tasa de recirculación | - | 1.25 - 2 |



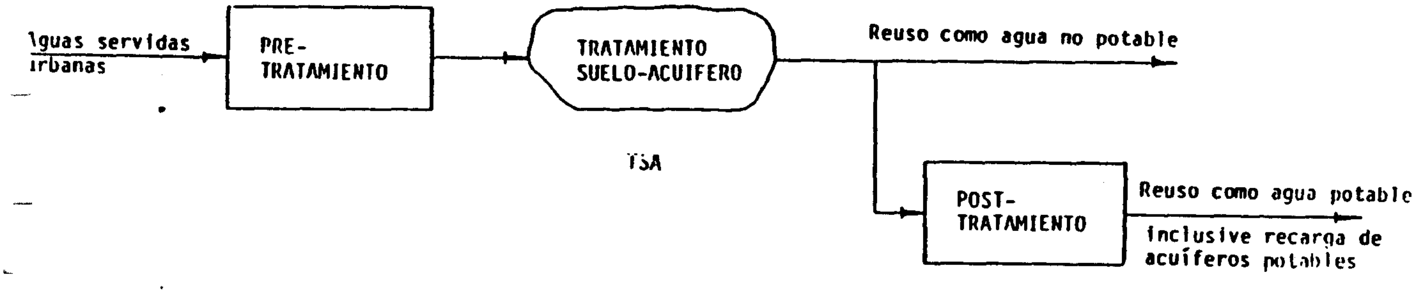
CUADRO 7: ESQUEMA DE TRATAMIENTO EN RESERVORIOS PROFUNDOS (TRP)



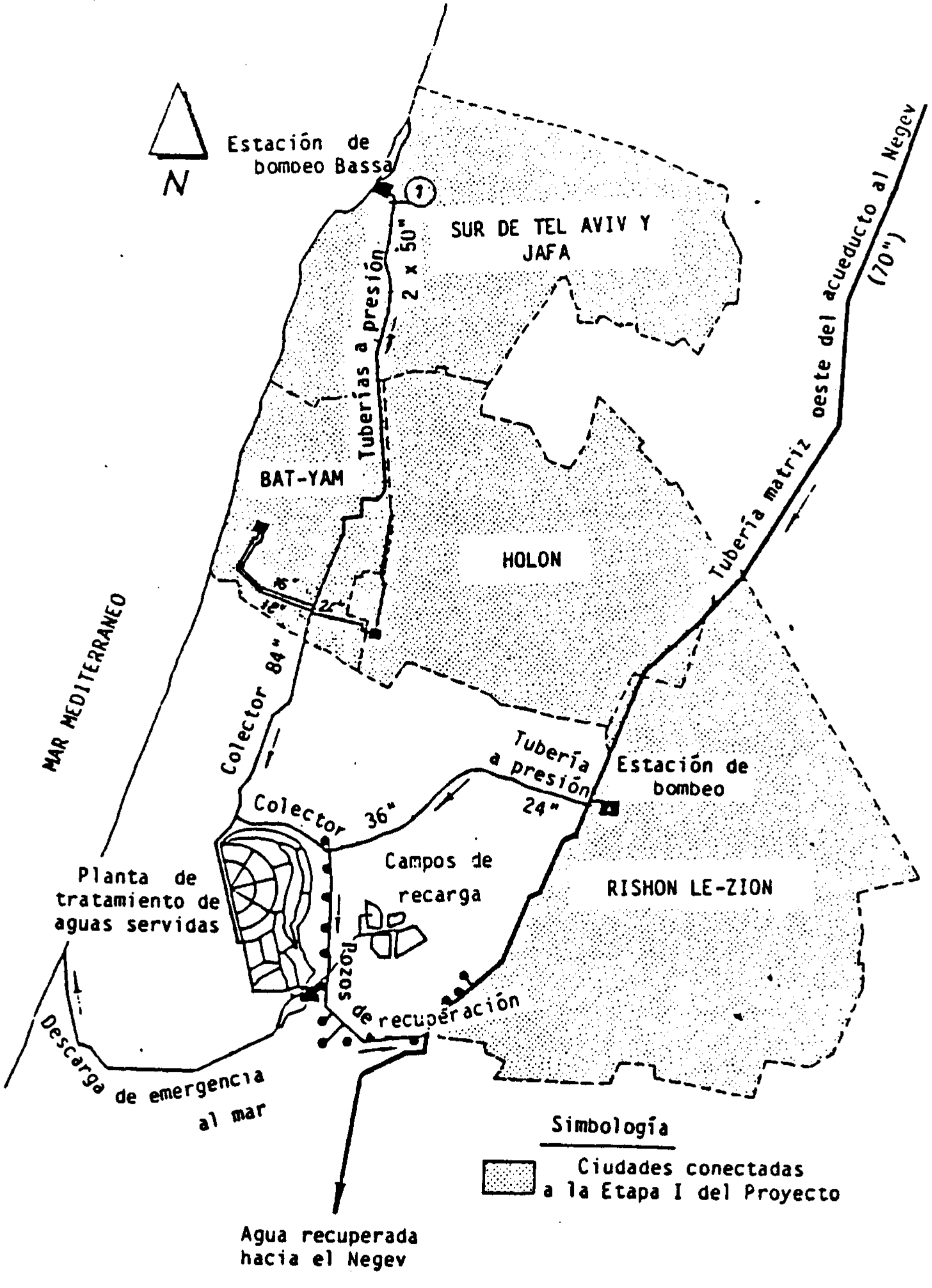
CUADRO 8: ESQUEMA DE TRATAMIENTO CAL-MAGNESIO CON REUSO DE LOS COAGULANTES QUIMICOS



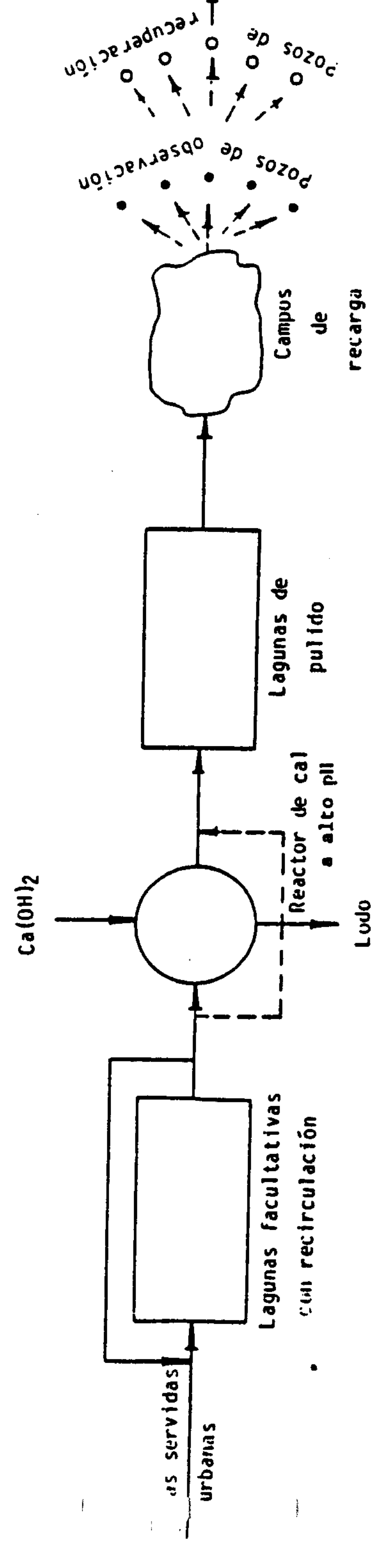
CUADRO 9: ESQUEMA DE TRATAMIENTO SUELO-ACUIFERO (TSA)



CUADRO 10: ESQUEMA DE TRATAMIENTO BASADO EN TSA PARA REUSO POTABLE Y NO POTABLE



CUADRO 11: PROYECTO DAN - ETAPA I; PLAN GENERAL



CUADRO 12: PROYECTO REGION DAN - ESQUEMA DE TRATAMIENTO

## Cuadro 13: Proyecto de la Región de Dan - Calidad de Agua en Varias Etapas de Tratamiento
## Compuestos Orgánicos y Nutrientes

| Parámetro | Concentración, mg/l | | | | |
|---|---|---|---|---|---|
| | Aguas servidas crudas | Efluente lagunas oxidación | Efluente tratamiento con cal | Efluente lagunas pulido (agua de recarga) | Aguas recuperadas de pozos de observación o de bombeo |
| Sólidos en suspensión | 240 | 305 | 140 | 20 | 0 |
| $DBO_5$ | 285 | 135 | 40 | 14 | 0.5 |
| $DBO_5$ filtrado | 115 | 15 | 15 | 4 | 0.5 |
| DQO | 590 | 435 | 150 | 65 | 12 |
| DQO filtrado | 225 | 90 | 55 | 40 | 12 |
| TOC | 175 | 125 | 55 | 20 | 3.5 |
| Detergentes | 12 | 4 | 3 | 1.1 | 0.2 |
| Amoníaco como N | 35 | 25 | 25 | 5 | 0.02 |
| Nitrógeno Total | 60 | 60 | 38 | 11 | 6 |
| Fósforo | 12 | 11 | 4 | 1.5 | 0.03 |
| pH (unidades) | 7.6 | 7.8 | 11.5 | 9.2 | 7.8 |

Lagunas Facultativas con recirculación LFR

Tratamiento Cal-Magnesio

Lagunas de Pulido

Tratamiento Suelo Acuífero TSA

## Cuadro 14: Proyecto de la Región de Dan - Calidad de Agua en Varias Etapas de Tratamiento
## Metales Pesados y Elementos Tóxicos

| Parámetro | | Concentración, mg/l | | | | |
|---|---|---|---|---|---|---|
| | | Aguas servidas crudas | Efluente lagunas oxidación | Efluente tratamiento con cal | Efluente lagunas pulido (agua de recarga) | Aguas recuperadas de pozos de observación o de bombeo |
| Boro | B | 500 | 500 | 350 | 250 | 250 |
| Cadmio | Cd | 25 | 3.7 | 0.9 | 0.4 | 0.3 |
| Cromo | Cr | 180 | 32 | 10 | 4 | 3 |
| Cobre | Cu | 150 | 36 | 12 | 6 | 6 |
| Fenol | Ph | 77 | 11 | 8 | 8 | 3 |
| Manganeso | Mn | 59 | 40 | 17 | 15 | 15 |
| Mercurio | Hg | 0.9 | 0.9 | 0.9 | 0.7 | 0.2 |
| Niquel | Ni | 125 | 55 | 25 | 19 | 8 |
| Plomo | Pb | 39 | 13 | 10 | 7 | 6 |
| Selenio | Se | 6 | 6 | 5 | 4 | 2 |
| Zinc | Zn | 715 | 267 | 167 | 118 | 108 |

Lagunas Facultativas con recirculación LFR

Tratamiento Cal-Magnesio

Lagunas de Pulido

Tratamiento Suelo Acuífero TSA

# Land Application of Waste — An Accident Waiting to Happen[a]

by Charles C. Johnson, Jr.[b]

## ABSTRACT

Half the population depends on ground water for domestic uses. Use is increasing 25 percent per decade. Ground water is generally used with little or no treatment.

Some persons would transfer the discharge of our waste products from contaminated surface streams to the land and thus relatively clean ground waters.

No standards exist that protect ground-water quality. Research necessary to give assurance that natural interaction of waste water and soils will remove, to acceptable levels, potentially harmful contaminants, organic and inorganic, that permeate today's waste streams and today's health concerns, has not been done.

Success reports on land treatment of waste water have not evaluated deterioration of ground water from organic contamination. Most waste waters contain synthetic organics in varying concentrations. EPA recommends their reduction in drinking water to the lowest possible level.

Most instances of ground-water contamination have been discovered after drinking water is contaminated. Unless the public is willing to treat ground water as it does water from surface streams, greater control of land disposal practices must be exercised. Current practice does not indicate the necessary controls are contemplated or recognized. It follows that the widespread use of the land treatment alternative is, in reality, an accident waiting to happen.

The title assigned for this session does not limit one to a discussion of waste-water treatment practices and their residuals, although I suspect this was the original intention. Inasmuch as ground-water protection is the underlying theme of this National Symposium, let's take a brief look at the tremendous waste produced in this country that helps to place our nation's vast ground-water resources in jeopardy. Each year major U.S. industries treat about 5,000 billion gallons of waste water before discharging it to the environment (U.S. EPA, 1977a). Of this volume, about 1,700 billion gallons are pumped to oxidation ponds or lagoons for treatment or as a step in the treatment process. Another 5,000 billion gallons per year of municipal waste water is discharged to the environment, with an estimated 730 billion gallons of this amount discharged to the land. In the United States, municipal sludge production amounts to about 5 million dry tons per year. Industrial sludge production is believed to be many times this amount. EPA (1977b) estimated annual solid waste production at 136.1 million tons in 1975. In addition we must deal with the millions of tons of gaseous wastes that are produced annually; the untold hundreds of million tons of mine tailings disposed of each year; and the tremendous volumes (more than 24 million barrels per day) of oil field brines produced each day. The discharge to the environment of such large volumes of waste with varying concentrations of toxic and hazardous substances must have a detrimental effect on the quality of our nation's water resources, both surface and ground-water supplies.

Because of the proclivity for waste production in this country, and as a result of the rather indiscriminant waste disposal practices that prevail, some basic concepts regarding water-supply protection have been destroyed, and some new ones that affect established policies in the water-supply field have surfaced. The paragraphs that follow will discuss some of these as they relate to land application of waste water and protection of ground-water quality.

The Public Health Service Drinking Water

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[a] Presented at The Fourth National Ground Water Quality Symposium, Minneapolis, Minnesota, September 20-22, 1978.

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Discussion open until July 1, 1979.

Standards (USPHS, 1962) required that water supplies be obtained from a "protected source." In the case of ground water, the standard defined "protected source" as water resulting from the natural purification by infiltration through soil and percolation through underlying material and storage below the ground-water table. If the source was not adequately protected by natural means, the supply is to be adequately protected by treatment. Ground water for human consumption is seldom treated except for disinfection. It is generally agreed that these standards were designed to protect against bacteriological contamination and that this definition of protected source provides insufficient protection against the massive volumes of toxic and hazardous wastes that are discharged to our environment today.

It must be recognized that the number of protected underground sources are diminishing. The remaining acceptable sources are further threatened by policies being initiated without supportive studies in terms of today's health concerns, and promoted under the desire for "zero discharge" of waste water to surface waters.

Well meaning, but poorly advised persons would have us transfer the discharge of our waste products from our surface streams, which are already contaminated to the land and thus, subsequently to the ground water, which in both instances are relatively clean. This would be an excellent idea if complete reliability could be placed upon the natural action of most soils to remove the potentially harmful contaminants, organic and inorganic, that seems to permeate today's waste stream and today's health concerns. The fact is that we can not. A search of the literature has failed to reveal one instance where any qualified expert has agreed that the natural filtration and percolation of today's waste-water stream through the soil produces a water that without question is safe to drink. That is not unusual. Neither does anyone recommend that waste water receiving tertiary treatment followed by dilution with water of drinking quality is fit to drink. On the other hand, the literature is replete with examples of ground-water contamination that results from current practices—even those projects considered by some to be successful land treatment operations.

While some will dismiss these studies as being representative of archaic practices, others will recognize that they provide information that we need to know. For instance, a preliminary report discussing the subsurface migration of hazardous chemicals (Geraghty and Miller, 1977) showed substantive movement of heavy metals, cyanide, arsenic, selenium and organics through the soil and into the ground water.

A study of the long-term effects of land application of domestic waste water at Hollister, California (EPA, 1978a) revealed substantially higher levels in the total coliform count per 100 ml over control wells; also it indicated increased cadmium levels, and lead levels that sometimes exceeded drinking water standards. Further, the report stated that "the greatest void of information remaining with respect to land treatment systems is that of persistent or refractory organic compounds. Uncertainties regarding health effects from transport of these materials through the soil from land-applied waste water must be answered before essential design criteria can be established. Quantification of basic scientific data on organic substances of known or suspected toxicity and determination of safe underground travel distances are major areas where research is needed."

A report just recently released by EPA (1978b) discusses environmental changes from long-term land application of municipal effluents. In my opinion, what the report says about the impact of the practice in two communities—Bakersfield, California, and Lubbock, Texas—on ground water is most supportive of my concern for the paucity of data related to this practice and its impact on ground-water quality. The report says that "very little" is known with regard to ground-water changes underlying the sewage-effluent-irrigated farm at Bakersfield. What is known suggests that high levels of nitrate are in the ground water under the farm. By comparison it notes that ground water under farm land several miles to the east are also high in nitrates. The report states that only slightly more information is available on ground-water quality at the Lubbock irrigation farm. Total dissolved solids of 1692 ppm were one-third higher than comparison wells, and nitrates of 50 ppm were 6 times higher. From the few data available, it appears likely that the long-term effluent irrigation operation (3 farms operated for 38, 19 and 6 years) has caused increased dissolved solids and nitrate concentrations in the ground water underlying the farms.

It can be pointed out that even the quality of the ground water under the country's most heralded land treatment system—the waste-water management system serving Muskegon County, Michigan—must be viewed with a note of caution. Following a preliminary survey of toxic llutants at the site, the report (EPA, 1977c) stated the presence of low levels of organics in the final effluent indicates the need for definitive information concerning the movement and fate of organic pollutants in the subsurface. I believe we all know that the State of Michigan does not allow the ground water at this site to be used for drinking water.

Other States also have exhibited concern for the potential health effects of ingesting over long periods of time, ground water contaminated with the waste water following its application to land. California convened a panel of experts to advise it on this question (State of California, 1976). Their report concluded that "areas of uncertainties regarding health effects can not be resolved because basic scientific knowledge is lacking." It noted that "monitoring and surveillance have not been directed at health aspects."

The California State Health Department has completed a draft version of proposed ground-water recharge regulations for use when reclaimed waste waters are used to augment underground potable water supplies through spreading (AWWA Research Foundation, 1977). Among other things these draft regulations require that the waste water receive carbon absorption treatment (30 minutes detention time), and percolate through an unsaturated aerobic zone of undisturbed soil for at least 10 feet vertically.

Almost daily we are reminded of the perils to our ground water that can result from land application of wastes. Just recently the Communicable Disease Center of the Public Health Service (1978) reported at least 759 cases of gastroenteritis associated with leakage from a municipal sewage lagoon in southern Missouri that affected the aquifer in Arkansas as well. The lagoon was over a porous limestone formation which permitted rapid movement of ground water. Hindsight tells us a lagoon system probably should not have been used at this site under these conditions, *but it was.* It is a very poignant reminder that most instances of ground-water contamination have been discovered only after drinking water has been contaminated.

The EPA recent report to Congress on waste disposal practices effects on ground water (EPA, 1977a) offers some interesting insight into this subject. Among other things it notes that:

- Ground water has been contaminated on a local basis in all parts of the nation and on a regional basis in some heavily industrialized areas, precluding the development of water wells.
- Degree of contamination ranges from a slight degradation of natural quality to the presence of toxic concentrations of such substances as heavy metals, organic compounds, and radioactive materials.
- Removing the source of contamination does not clean up the aquifer once contaminated. The contamination of an aquifer can rule out its usefulness as a drinking water source for decades and possibly centuries.
- Existing technology alone cannot guarantee that soil attenuation alone will be sufficient to prevent ground-water contamination from a waste disposal source.
- Land disposal of waste is not environmentally feasible in many areas.
- Existing Federal and State programs address many of the sources of potential contamination, but they do not provide comprehensive protection of ground water.

A review of EPA's *Land Treatment Manual* by C. Winklehaus was carried in the *Journal of the Water Pollution Control Federation* (1978). The commentary in the review in large measure summarizes some of my concerns on this treatment alternative. Mr. Winkelhaus notes that there is a tendency to overlook the fact that land treatment systems, unlike "conventional systems," have relatively open boundaries through which water and solids can pass quite freely—once applied, the waste water and pollutants are largely beyond control. Can anyone assure us that the EPA policy on land treatment as contained in the Administrator's memorandum of October 3, 1977 (Costle, 1977), and further amplified by the draft Program Requirements Memorandum (EPA, 1978c) by Mr. Cahill dated May 10, 1978, provides the safeguards required to protect our ground-water resources? In my opinion, the answer is no.

The Assistant Administrator for Water and Hazardous Materials for EPA seems to have verified these concerns in a statement (Jorling, 1978) before the Committee on Environment and Public Works of the U.S. Senate. In that statement he said "concern for ground water has emerged relatively recently as a major environmental issue. There is a great deal yet to be learned about the fate and transport of contaminants below the surface; the practices that represent the greatest threat to this national resource; and the economics of alternative ways of disposing of wastes in a manner more protective of the environment. Another reason for proceeding carefully is the sheer number of facilities that seem to have the potential for an adverse impact on the quality of ground water. Literally hundreds of thousands of wells, surface impoundments, ditches and landfills used by industry, municipalities, farmers and other private individuals are involved. Prudence dictates careful preparation in designing programs to bring these practices under control and in incurring the probable social and economic costs involved."

It is accepted knowledge that contaminants of known and unknown character and concentrations reside in all sewage effluents. The examples that have been cited in this paper certainly show that sewage effluents do percolate through the ground to the ground-water table. Contaminants in sewage without a doubt have public health significance. Little is known about the health significance of trace levels of synthetic organic contaminants that have been identified in the ground water under some of the land treatment systems. The Administrator of EPA has publicly stated that the presence of trace levels of synthetic organic chemicals in drinking water may be hazardous to the health of persons. Further, the EPA has proposed a treatment standard for public water supplies so that these contaminants can be reduced to the lowest practicable level. If EPA's statements and actions are to be taken seriously, why should anyone promote a waste-water treatment practice that adds even small increments of these contaminants to an otherwise safe water supply?

There is no doubt in my mind that from strictly a technological viewpoint we can design land disposal systems of many types that will dispose of waste water, allow the production of agricultural crops, extend the development and use of pasture land, provide for certain recreational pursuits and establish bird sanctuarys, all at a profit when only measured by the flow of dollars into the cash register. The overriding question is, where should this be done; what quality of waste water should be applied to the land; and what degree of degradation should be permitted in the quality of our ground water? In the absence of more definitive answers to these questions than those implied by current EPA policy, I would prefer the policy I understand is used by the British, i.e., only water of drinking water quality should be returned to ground-water aquifers from which drinking water will be extracted. Otherwise we must admit that we proceed in ignorance and we cannot be surprised at some future time when reality tells us the accident has happened.

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# Audience Response to Session III — Land Application of Waste

**Stephen A. Smith, 1423 S. College Ave., Tempe, AZ 85281:** The advantages of using the soil for the application and treatment of wastes are well-known; however, although we probably have the technology to design a fail-safe system, we may not be able to afford it. Accidents can happen due to improper design, inadequate maintenance, or a lack of a complete understanding of the soil-water system.

Ground-water zoning, and the delineation of areas of possible, potential, or actual ground-water degradation, is a possible solution to this dilemma. We can, in effect, have our cake (land treatment) and be able to eat most of it (or drink the water). By siting land treatment and disposal facilities in areas designated as zones of potential degradation, we serve notice that a contamination potential exists, and special measures are necessary to ensure the safety of water supplies. For example, development in the area would have to be accompanied by public or community water systems with an outside source, or any wells drilled in the zone would have to meet rigid requirements, with sufficient casing and depth to seal off any contaminated ground water due to the waste disposal facility (zones could be three-dimensional).

To a limited extent, the zoning concept is already being applied. In Wisconsin, bacteriological contamination of ground water is a fact in a few, scattered areas where creviced dolomite is overlain by thin soils. In these areas, which are delineated on maps or described in terms of section, township, and range, drillers have been notified that they are required to emplace as much as 200 feet of casing in order to seal off the upper, contaminated portion of the aquifer. In other words, zones of actual ground-water degradation have been defined, and measures have been taken to ensure bacteriologically safe drinking water. However, these are not areas where additional land disposal facilities are encouraged; they would aggravate what is already a bad situation.

For planning future land treatment and/or disposal systems and zones of potential degradation, we obviously want to keep the zones small. Case histories of contamination incidents in the literature document the difference in the extent of contamination between incidents which have taken place in recharge areas and those which have occurred near discharge areas. I suggest that we site our land treatment and disposal facilities as near as possible to ground-water discharge areas. This will accomplish some important objectives. If an accident does happen (and Murphy's Law says that it will), the actual zone of degradation is small, and we won't waste valuable hydrogeologists' time tracking a plume across a county (let the engineers track it down a stream). In addition, if the facility is subsequently modified to correct the situation, the zone of contaminated ground water will be renovated in a relatively short time.

Mr. Sheaffer has suggested that we use floodplains for the operation of land treatment systems. Provided that a floodplain site can meet the criteria for soils and ground water set forth in most State regulations, it would be an excellent choice if we accept the discharge area concept of degradation zoning. Obviously, floodplains would be a poor choice for land disposal facilities such as solid waste sites, for which adequate protection from periodic flooding would be difficult and expensive to provide.

**David E. Lindorff, Assistant Geologist, Illinois State Geological Survey, 425A Natural Resources Bldg., Urbana, IL 61801:** It is perhaps obvious that an important part of any land application program is proper operation and maintenance. No matter how well designed a land disposal project may be, serious problems can still develop from lack of or improper maintenance and operation. I have visited a number of industrial spray irrigation facilities and found that, frequently, they are a low priority in terms of maintenance. We, therefore, need to be aware of this problem, not only for land disposal of waste, but for other types of waste disposal as well.

**Elmer E. Jones, Jr., Agricultural Engineer, USDA, Beltsville Agricultural Research Center, Beltsville, MD 20705:** Kenneth Wright implied that on-site subsurface disposal was not a satisfactory land application technique. In the last 10 years, tremendous advances have been made in on-site disposal technology. Johannes Bouma and others with Small Scale Waste Management Project at the University of Wisconsin have made four major contributions to the science of subsurface disposal, (1) Application of soil moisture tensiometry to subsurface disposal system evaluation; (2) Quantification of interface barrier resistance; (3) *Development of effluent purification criteria,* and (4) Development of innovative loading procedures to improve purification, reduce interface barrier resistance, and maintain or improve soil porosity. While the Wisconsin research has demonstrated that it is possible to design systems for higher loading rates than recommended in the *Manual of Septic Tank Practice,* I would recommend using lower rates, in effect making the subsurface disposal system an irrigation system to maximize utilization of water and nutrients by vegetation. On-site disposal can be the most economical and desirable land application technique.

Dr. Sheaffer placed considerable emphasis on the use of underdrain systems. There are sites where underdrainage is essential to satisfactory performance; however, there are other situations where increased ground-water recharge should be a major factor in encouraging greater use of land application. Some major rivers serving large metropolitan areas are deficient in surface storage for low flow augmentation. Every home or building built will increase runoff and reduce ground-water recharge. As development proceeds the rivers tend to become flashy, higher peak discharges and lower base flows. Land application of waste water to increase ground-water storage can have two very beneficial effects, reduction of waste-water discharge to the river during critical low flow periods, and increased base flow from ground water.

**Robert D. Sinclair, Western Ground-Water Consultants, St. Norbert, Manitoba, Canada:** In looking at this problem as a hydrogeologist and as a sanitary engineer, I see it from both sides. We are bound to have some sort of pollution.

If we are looking at any kind of waste, we have the problem of the sugar in the tea. You put the sugar in, and we have a problem trying to get it out again. I think this is the big problem with a lot of the sewage we're trying to put on the land. For example, heavy metals, if we look at the plating industry, it would be much easier and simpler in a lot of cases to tackle the problem at the source instead of trying to handle it after it is in the sewage and diluted down a couple hundred thousand times. It makes it impossible to get it out. I think we would be better spending our money in terms of looking at this type of alternative versus trying to figure out how much sludge or sewage we can put on the land and how much heavy metals we can accept in a crop. It may be easier to just get the problem out right at the start and avoid this whole mess we've got into.

In terms of adding stuff on the land we have to look at what the land can do. What sort of crops are we taking off? We're looking at crops that pass through the food cycle, we're looking at crops that can be biodegraded, so what kinds of things should we be looking at putting back on the ground as these alternatives. We're looking at food processing waste for one. There are some problems maybe with caustics in some of them but in general, if we look at waste like pulp and paper and food processing or even human waste, we are looking at some materials that can be recycled back on the land that don't have the impact of such things as heavy metals and plastics that we have in industries. They can't be handled by the soil structure. It may take them out but if they are not biodegrading they are just accumulating, and this problem with time is going to find itself 20 years down the road where we won't be able to grow the crops. They look okay now but it may be a problem in a few years.

If we look at sewage treatment, basically it is one of removing carbon. When we throw sewage in the river, we are killing the fish and causing zones of pollution. So we've gone to removing carbon which eliminates this problem. What's happening now is we don't remove too much of the nutrients in terms of phosphorus and nitrogen, and in some cases we find that algae will just take that nitrogen and phosphorus and put it right back in the stream. So in terms of sewage treatment, we haven't done anything except transfer it down the stream a little ways.

I'll give one example of what happened in Winnipeg. There was a town that was on ground water for quite a long time and could have continued it but the engineers decided to go to river water. You can see it, and it's a lot easier to use so they went to it. Now during low flows, the water going into that plant was about 50-50 sewage/river water which a lot of the people in town didn't think was too good. Now they are finding during some tests there may be problems with viruses.

If we are looking at adding things into the environment which we have to, we can only add it into the water, the air or the land and if we look at most of our waste, they are either on the land or water; we have to be able to integrate the two and have a managed system that will allow acceptable use of the environment.

**Virginia Jamison, Suntech Group, Marcus Hook, PA:** Sometimes I feel as if I should have a very big inferiority complex because I work for a big, bad villainous oil company. We have seven refineries in the U.S. and Canada, and we produce an awful lot of waste water and an awful lot of sludge, and we have to get rid of that. We can't dump it in the oceans, or in the streams, or in the rivers. The government laws say we can't. We have to pay and it is an expensive proposition to have it hauled away and then incinerated. Then we've got an air pollution problem.

Dick Raymond and I have spent the past ten years trying to convince our company that there is a safe way to get rid of this waste and it is land application. I think we've convinced our company of this and they are doing something about it. We are what we call "biofarming." We are applying our waste on the land and getting rid of it. I think we meet all of Dr. Sheaffer's criteria.

In the first place we don't have a storage problem because our crop doesn't cease in the winter. Our crop is bacteria and they biodegrade in the winter as well as in the summer. I don't think that, as Dr. Johnson says, "we're an accident about to happen," because we have very strict, stringent regulations and we monitor these land applications. If we monitor them correctly and put on the proper amounts of material, there is no reason why we will contaminate ground water. We have been doing this for five years now and we are not the only company. Most of the oil companies are using land farming. We feel as if it is an economical process; therefore, our company is happy. We feel that it's a natural process; therefore, we meet government regulations and it is a very safe process.

I hope that our government doesn't outlaw land farming. If it does, the oil companies have spent an awful lot of time and money in perfecting this process for nothing.

# Wastewaters in the Vadose Zone of Arid Regions: Hydrologic Interactions[a]

by John F. Mann, Jr.[b]

## ABSTRACT

In truly arid regions there is essentially no direct penetration of rainfall. Recharge to ground water is not only infrequent, but extremely localized, occurring only where surface runoff has been channelized or ponded. Over those vast desert areas covered by sparse xerophytic vegetation, the scant rainfall has little or no chance of becoming ground-water recharge. Such water is quickly dissipated by capillarity-assisted evaporation, or through rapid evapotranspiration by short-lived annuals. Where perennial xerophytes cover the ground surface, the extensive shallow root systems quickly utilize all of the rainfall stored in the soil. Beneath the infrequently moistened soil zone is the lower part of the vadose zone, extending to water tables which are usually at depths of tens to hundreds of feet. Almost always these vadose zones have moisture contents well below field capacity.

Regardless of the cause, these dry vadose zones are capable of holding additional water, at least up to field capacity. And no water-carried pollutants can reach the water table from the ground surface until a pre-wetted path has been formed for the entire vertical distance. A practical use of this water-holding capacity can be made in the design of wastewater tailings ponds, with predictable safety and with great economic benefit. However, only in predictable geologic conditions, and in limited amounts can the use of this water-holding capacity be recommended. On the other hand, to make no use whatsoever of these great natural dry sponges would be an economic waste.

## INTRODUCTION

The term "vadose water" was originally used by Posepny (1894) to include all of the water in the zone of aeration. Later, Meinzer (1939) divided the zone of aeration into three parts and proposed a series of terms of Greek origin to apply to the contained waters. For all of the water in the zone of aeration he proposed the term "kremastic water." He proposed "rhizic water" for the water in the soil zone, "anastatic water" for the water in the capillary fringe, and "argic water" for the water in the intermediate zone. Unlike "phreatophyte" which has become a commonly used term among hydrologists, Meinzer's adaptations from the Greek for waters in the zone of aeration have never caught on, and few ground-water geologists normally use them. As regards research activities, the intermediate portion of the zone of aeration must be considered a true orphan (Bean *et al.*, 1967). The surficial materials to the depth of root penetration (soil, hydrologically speaking) have been the subject of intensive research by soil scientists. Some agriculture-oriented hydrologists such as Blaney (1933) have systematically calculated deep penetration of rainfall primarily from a soil-moisture approach. The water which is not disposed of by evapotranspiration or stored in the soil zone is *presumed* to reach the water table. The soils engineer, to whom soil may be anything which is not hard rock, is concerned mainly with strength characteristics, but does collect samples of the intermediate portion of the vadose zone and makes moisture determinations. Actually, a great deal of soil moisture information has been collected by soils engineers in arid regions, but remains in their files because the information was not relevant to anticipated problems. Thus we find the vadose zone of arid regions an inadvertent victim of non-emphasis, with considerable data existing, but with few organized attempts to study and understand its hydrologic aspects. From laboratory studies, Palmquist and Johnson (1962) have demonstrated the mechanism of movement of water in vadose zones, and suggested the possible use of arid

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vadose zones for nuclear waste disposal. Winograd (1974) raised the possibility of storing high-level radioactive wastes within containers in the thick unsaturated zones of arid regions.

## THE VADOSE ZONE IN ARID REGIONS

It is generally recognized that there is no direct recharge by rainfall through the soil zones of arid regions. Xerophytic vegetation, by definition, grows in an environment of sparse soil moisture. Following a rainstorm, such vegetation quickly removes the water stored in the soil until the plant's capacity to remove water is equalled by the soil's tenacity for holding water. In this environment of great competition for soil moisture, the perennial xerophytes are typically widely spaced and shallow rooted. In addition, short-lived annuals quickly remove moisture from the shallow soils in places where they can compete successfully against capillarity-nourished evaporation and the far-flung root systems of the perennial xerophytes. Many xerophytes are capable of utilizing more water than reaches them as direct rainfall. Examples can be seen in the Mojave Desert of California where the most abundant xerophyte is creosote bush (*Larrea divaricata*). Individual plants growing close to paved roads are commonly two to three times as large as those growing on the normal alluvial surfaces. The paved roads typically have shallow drainage ditches on both sides into which is directed the rain which falls on the impervious pavement. Those creosote bushes whose roots are able to reach the augmented rainfall in the soil below the ditches grow with unusual vigor. As yet, no research has been done on the question of whether there is penetration of rainfall beyond the rooting depth in these ditches. Quantitatively, such recharge to ground water, if any, would be small.

Studies of recharge in arid regions, particularly by the United States Geological Survey in Arizona and Nevada show that the main recharge to the alluvial basins occurs from the beds of flowing streams within a short distance after those streams have left the mountains. Winograd (1974) has provided us with the following summary:

> "Hydrologic, geomorphic, and pedologic evidence suggest that little or no recharge (namely, infiltration of precipitation to the water table) occurs beneath interfluves (inter-stream areas) in the arid and semiarid portions of the Southwest under present climatic conditions."

In truly arid regions the above generalization seems to hold, but as pointed out by the writer previously (Mann, 1957) there is considerable penetration of rainfall (especially in very wet years) in semiarid regions. As average annual rainfall increases, a point should be reached where direct penetration of rainfall may be expected to start. Many other variables such as the permeability of the soil, angle of slope, and nature of vegetation would be involved. In a study of the Mojave River basins in California, Chun and Weber (1967) used an average annual rainfall of 8 inches as the upper limit of the zone of no direct rainfall penetration. The same figure of 8 inches was used by Malmberg (1967) in studies in Pahrump Valley.

In those areas where there has been no downward passage of infiltrating rainfall for unknown thousands of years—perhaps since the ending of the last Pluvial period—moisture levels have dropped far below field capacity. This condition has been confirmed in numerous borings supervised by the writer in the Mojave Desert area. Where air rotary rigs are used, these dry vadose zones are characterized by lost circulation problems and by dust blowing out of the hole. The mechanism by which the vadose zone drops from the field capacity condition it is assumed to have had at the time of the last infiltrating water to its present low-moisture condition is poorly understood. Evaporation by air movement accompanying changes of barometric pressure is probably a factor. The production of hydrated weathering products is another. Minor, but perhaps reversible, changes may be due to vapor transfer.

The depth of the dry vadose zone is highly variable. Winograd (1974) speaks of depths of 100 to 300 meters in local areas. Regardless of the depth, there are in places thick zones with moisture levels far below field capacity. And in considering the applications of liquids to interfluve soils we may liken these vadose zones to huge dry sponges. Before liquids can reach the water table they must travel a path which has been pre-wetted to field capacity. The volumetric pellicular film capacity which can be suspended above the water table could be viewed as a permanent storage reservoir for certain liquid wastes.

## MOVEMENT OF WATER IN DRY VADOSE ZONES

Some interesting field tests bearing on the mechanism of movement of water in dry vadose zones were conducted during 1970 at the site of a coal-fired generating plant in southern Nevada. One of the wastes to be disposed of was coal ash. Prior to the start of operation of the plant, a question was raised as to whether rain, falling upon ash

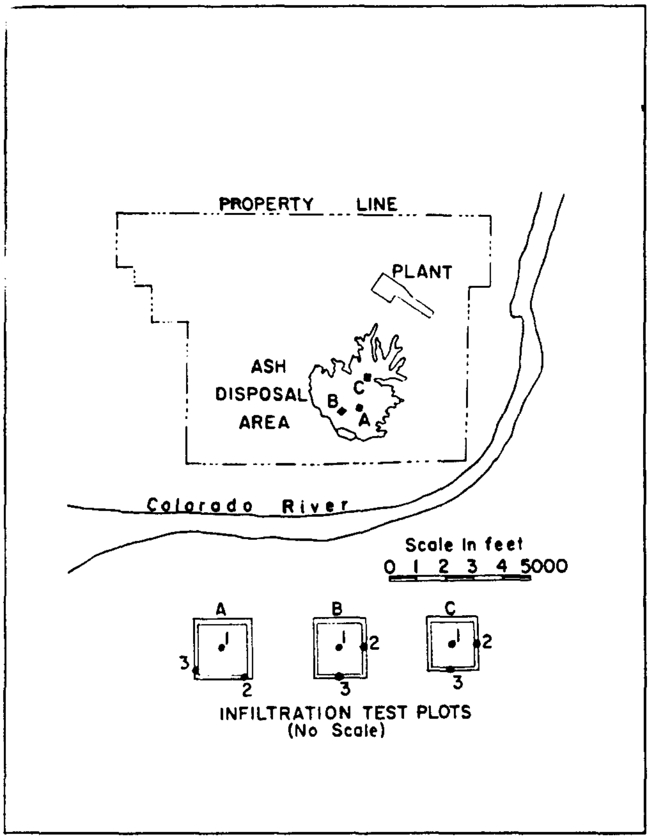

**Fig. 1. Locations of infiltration test plots near coal-fired generating plant (after Glenn A. Brown).**

which had been spread on the ground surface, would cause pollution of the underlying ground waters, which in turn were thought to be tributary to the Colorado River. It was known from laboratory tests that ash leachate had a total dissolved solids content in the range of 3000 to more than 5000 parts per million. Prior to disposal of the ash, three infiltration test plots were constructed (Figure 1). Each plot was about 10 feet by 10 feet (3 m X 3 m) and about 18 inches (0.5 m) high. At each plot three holes were drilled and equipped with 1 ⅝-inch (4.13 cm) shelby tubes. The central holes were 40 feet deep, and plywood sheets were placed around the holes to prevent disturbance to the natural soil by the drilling rig. At each plot 2 holes were drilled in the berms to depths of 25 feet and also equipped with 1 ⅝-inch (4.13 cm) shelby tubes.

The ash disposal area is in a dissected alluvial surface extending easterly from the Newberry Mountains. Several generations of terrace deposits overlie an extensive sequence of silty and clayey lake beds.

Prior to the flooding of the test plots, the natural soil moisture was measured in each hole by means of a Nuclear-Chicago Model P19 moisture probe. Such neutron-scattering moisture meters have been used by soil scientists for many years (Nixon and Lawless, 1960) and have been confirmed as acceptable substitutes for the older, more laborious methods of determining soil moisture (de Boodt, Moerman and de Boever, 1969). Both the neutron probe measurements and standard laboratory tests on core samples indicated that the moisture conditions in the vadose zone, above the water table at a depth of about 85 feet (26 m), were considerably below field capacity.

More specific results are shown for Hole A-1 (Figure 2). The materials encountered in the entire 40 feet of hole consisted of fine to coarse sand with 10 to 20 percent gravel. A 6-inch (15 cm) plug of concrete was poured around the tube to prevent direct entry of water down the outside of the tube. Water was introduced to the plot once, to a depth of 5.25 inches (13.3 cm). Its downward progress was then monitored by means of the neutron probe. On the first day, the top 4 feet (1.2 m) were nearly saturated. By the second day, some of the water from the top 4 feet (1.2 m) had moved down to 10 feet (3 m). There appears to be a translocation of gravity water from the shallow zones and a downward progression of a pellicular front in the deeper zones. The rate of reduction of

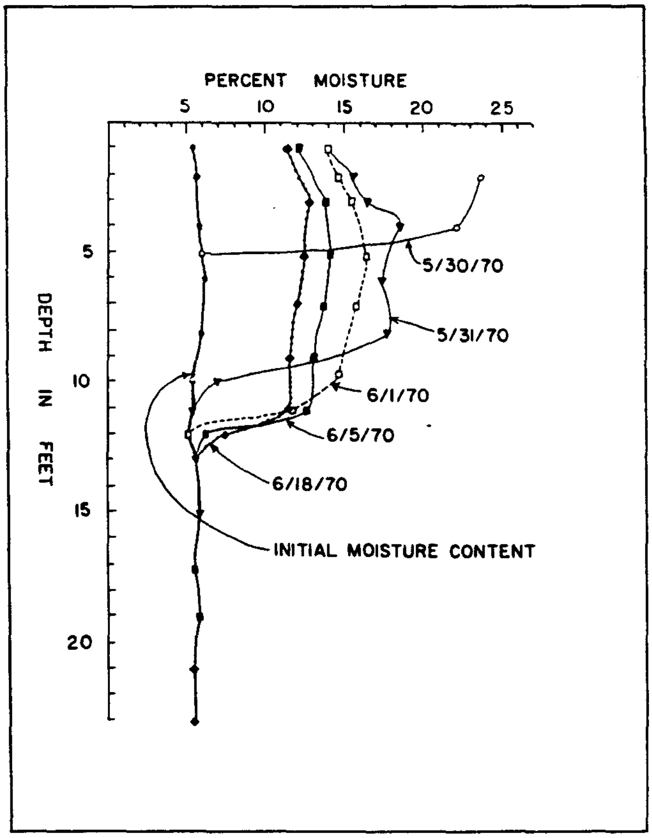

**Fig. 2. Progress of wetting front in Hole A-1 (after Buena Engineering).**

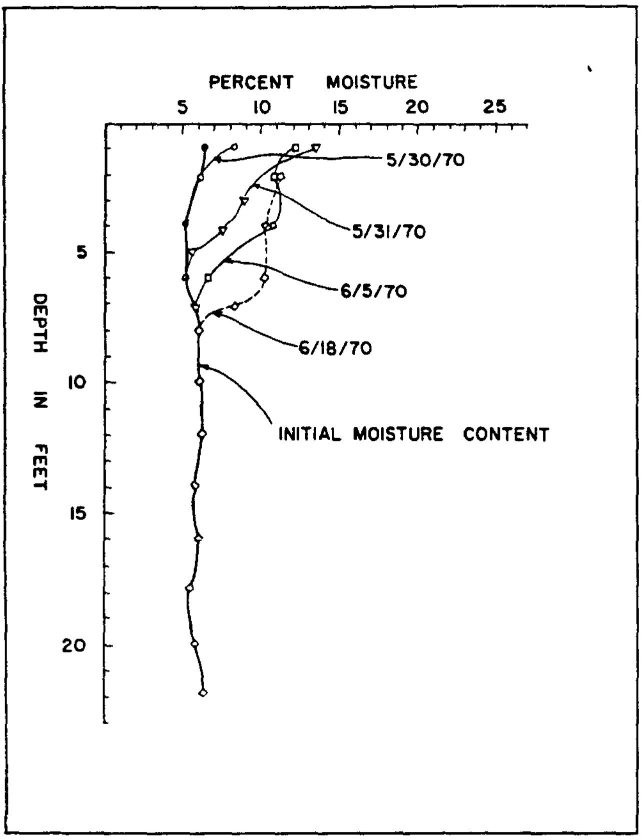

**Fig. 3. Changes of moisture content in Hole A-2 (after Buena Engineering).**

moisture content in the shallow zones slows with time as there is downward drainage. Almost none of the applied water reached to depths greater than 12 feet (3.7 m), below which the natural moisture conditions prevailed. From this diagram, in a relatively homogeneous geologic situation, one could suggest that the field capacity was 10 or 11 percent, but that factors operating in the arid vadose zone had reduced the moisture content to about 5 percent.

The curves for Hole A-2 (Figure 3) show a vastly different picture. This shows some lateral movement beneath the berm, but only to a depth of 7 feet (2.1 m). The low percentages of moisture suggest that capillarity, rather than gravity saturation, is the dominant wetting mechanism.

A comparison of Figures 2 and 3 indicate a downward vertical movement of the water, with only minor lateral movement. In this relatively homogeneous geologic material, such a movement would be expected from the laboratory experiments of Palmquist and Johnson (1962). Departing from this homogeneous and permeable situation, it is possible to predict the effects of certain types of geologic heterogeneities. If there are nearly horizontal low-permeability layers, perching is to be expected, along with greater lateral spread of the saturated zones. Such low-permeability layers will not only slow the downward rate of movement but there is a strong possibility that a greater percentage of water will be required to bring the lower-permeability materials up to field capacity.

## DISPOSAL OF WASTES OF DRY VADOSE ZONES

The capacity of dry vadose zones to tie up liquid wastes as pellicular films and wedges can be utilized to great economic advantage and with predictable safety. Figure 4 illustrates the simplified concept of controlled leakage. At the prospective site of a tailings pond for liquid wastes which are not chemically acceptable for direct recharge to the water table, a drilling and coring program would have to be undertaken to determine natural geologic and moisture conditions. From this information the amount of leakage permitted could be calculated, of course with an adequate safety factor.

Industrial tailings ponds are often of huge dimensions—hundreds of acres in extent. The costs of low permeability liners are enormous. Within this economic framework, there is justification for

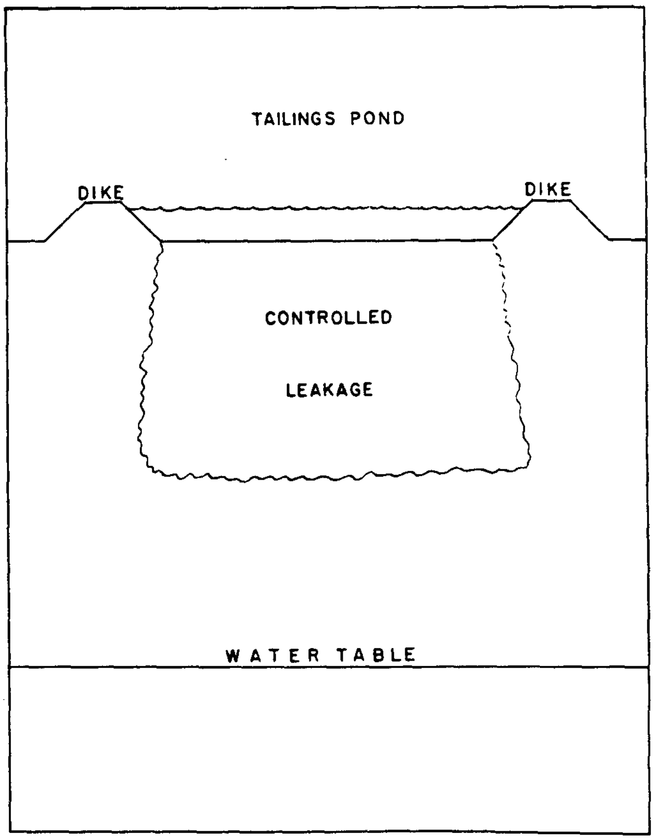

**Fig. 4. Diagrammatic distribution of moisture beneath a tailings pond under a controlled leakage approach.**

considerable testing and experimentation based on sound geologic and hydrologic principles.

In the Mojave Desert of California, studies were made for the installation of a 129-acre tailings pond for the evaporation of highly saline wastes from a large chemical plant. Local clays were available to construct a compacted low permeability liner. Laboratory tests indicated that a permeability of 0.002 feet (0.06 cm) per year could be achieved. The clay layer (compacted in two lifts and 0.83 feet [25 cm] thick) would be covered by a permeable protective blanket to avoid desiccation of the clay layer if the ponds were emptied. Pre-flooding with fresh water was planned to provide a reservoir of fresh water within the protective blanket which could saturate the clay and negate capillary forces within the clay. The saline tailings water would then follow the fresh water through the clay. Under a "zero leakage" concept there is no provision for pellicular storage in the underlying dry vadose zone; on the contrary, the useful life of the pond is considered to end when the tailings water reaches the bottom of the base (clay) layer. Pond life as a function of total pond head, using a design permeability of 0.002 feet (0.06 cm) per year and an effective porosity of 0.33 is shown in Figure 5 for different thicknesses of the impervious clay layer. A pond built with a clay layer 0.83 feet (25 cm) thick and a head of 10 feet (3 m) is expected to have a useful life of about 12 years, at which time a new tailings pond would have to be completed and ready to receive tailings water. However, by adopting a controlled leakage approach, the useful life of the pond could be increased. As this pond receives solid wastes (slurry) as well as liquids, it will progressively lose its capacity for seasonal storage of liquids. Its useful life could be extended further if the dikes could be raised to permit a higher theoretical head. Actually, during the deposition of the water-carried solids, there are laid down nearly horizontal clay layers, so that the effective head operating on the base layer may be less than would be indicated by Figure 5.

Under the design criteria used, Figure 6 shows the relationships among total pond head and base (clay) thickness as it relates to the rate at which water will move through the base (clay) layer. Using such data, along with information as to moisture content of the underlying vadose zone, it is possible to predict the downward movement of the wetting front and to stop it at a safe distance above the water table.

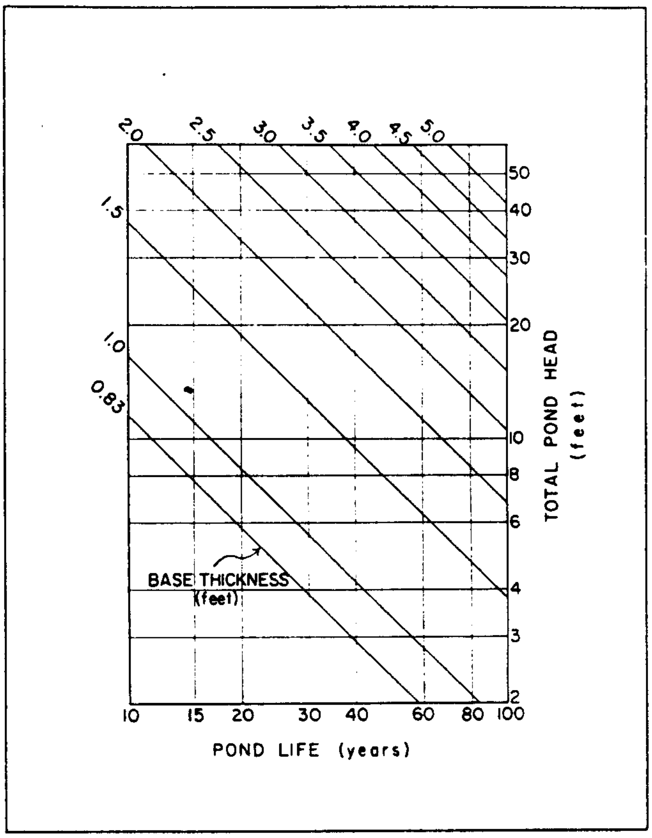

Fig. 5. Relationships among pond liner thickness, total pond head, and pond life under a zero leakage concept (after Moore and Taber).

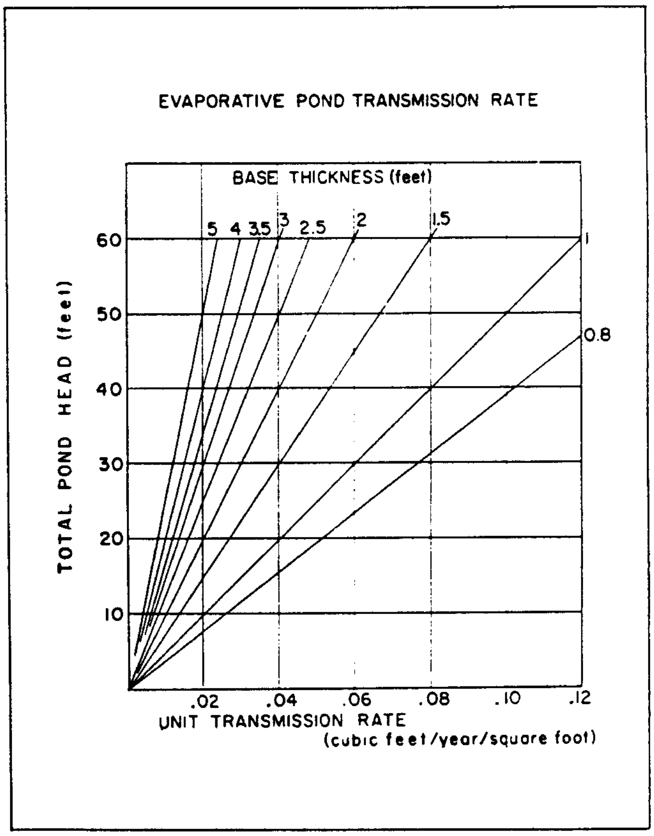

Fig. 6. Rate of leakage through clay layers of various thicknesses at various total pond heads (after Moore and Taber).

## MONITORING

The key to acceptance of a controlled leakage approach is monitoring. Classical monitoring of the water table downgradient from the tailings pond may be unacceptable if such monitoring results in the detection of a pollution problem only after it has reached unmanageable proportions. The facts demonstrating direct downward vertical movement suggest monitoring wells within the pond itself, or such monitoring of perched lateral flow as would be indicated by the geologic conditions. With such wells and a neutron probe, the downward movement of the wetting front could be followed, and discharge of the effluent could be stopped where desired.

## ACKNOWLEDGEMENTS

For making available basic data used in this paper, and for many helpful suggestions, the writer would like to thank Mr. Glenn A. Brown, Consulting Geologist, Los Angeles; Mr. R. F. Moore, of Moore and Taber, Consulting Engineers and Geologists, Anaheim, California; and Buena Engineering, Ventura, California. However, neither their cooperation nor helpfulness should be construed as agreement with all of the writer's conclusions, or any of them.

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## DISCUSSION

**The following questions were answered by John F. Mann, Jr., after delivering his talk entitled "Wastewaters in the Vadose Zone of Arid Regions: Hydrologic Interactions."**

**Q. by G. F. Hendricks.** *Please redefine the term "field capacity." How is it determined?*
**A.** Field capacity describes the condition of a soil after gravity drainage. It is a term most often used in agricultural circles, and similar to specific retention.

**Q. by Ron Barto.** *Do you think that the low field capacity is due to evaporation?*
**A.** Evaporation is a possibility. On a longer-term basis, hydration as feldspars are weathered to clays, is another.

**Q.** *In the case of the Nevada Ash Pit, at about what level of natural rainfall would the vadose zone reach field capacity?*
**A.** There are some suggestions that average annual rainfall as little as 8 inches per year would result in some deep penetration of rainfall on coarse permeable soils. Tighter soils and hot season rainfall would probably place this value higher.

**Q. by Jack E. Sceva.** *How do you evaluate the potential for the development of perched ground-water zones and the lateral movement of leakage away from the site?*
**A.** By thorough coring of the site and determination of permeabilities. Low permeability layers may be expected to cause perching and lateral spread.

**Q. by K. Childs.** *Shouldn't the integrity of the suggested leaking pond be verified by a ground water and soil zone monitor network, or don't you feel it is necessary?*
**A.** I would suggest both neutron probe monitoring within the pond and water-table monitoring downgradient.

**Q. by Dan Kimball.** *Are there any ground-water observation wells located around the ash pond at the Mojave plant and, so, have they shown any changes in ground-water quality?*
**A.** No changes in ground-water quality in the monitor wells have been related to the disposal of ash.

**Q. by Bill Bellis.** *With all the core samples – what is the mineralogy?*
**A.** There has been visual logging of the cores and cuttings but no detailed mineralogical study.

**Q. by Ed Meiser.** *What is the danger or likelihood of ocalized, pre-wetted paths developing as routes of direct communication between the source of wastewater application and the water table?*
**A.** In usual alluvial environments, this should be no problem. Concern should be expressed in areas of cracking due to compaction or tectonics. A thorough subsurface investigation must be made.

**Q. by Jan Turk.** *In your "no leakage" example, fresh water was used to saturate the clay lining before the highly saline waters were allowed to pass through the clay. Have you noticed any shrinkage of the clay when the brine arrives (i.e. osmotic desiccation)?*
**A.** The brine is discharged as a slurry, which causes the bottom to be covered by deposited clays. Furthermore, the clay is covered by a permeable layer, pre-wetted with fresh water. Under these conditions, desiccation is unlikely.

**Q. by Jack Robertson.** *Does the clay lining of ponds produce any ion filtration effects?*
**A.** No liquid has shown in the underdrains, so it hasn't been possible to check this.

**Q. by D. G. Boyer.** *What is your type of clay? Bentonite or non-expanding clay? What is the concentration and make-up of your saline fluid?*
**A.** The clay is not highly expansive. The saline fluid is a high sodium brine with 50,000 to 100,000 parts per million total dissolved solids.

**Q. by C. W. Fetter.** *Homo sapiens have existed, so far, through a number of climatic changes. During the late Pleistocene, the "arid" southwest had heavy precipitation. Does waste disposal in the arid zone present potential problems to future generations if the climate shifts to a humid one? Long-forgotten contaminants might become mobile.*
**A.** The possibilities of a climatic change to wetter (or drier) over a span of thousands (rather than hundreds) of years are a consideration. Covering with a low permeability blanket has been suggested as a means of preventing future leaching by infiltrating rainfall or flood flows.

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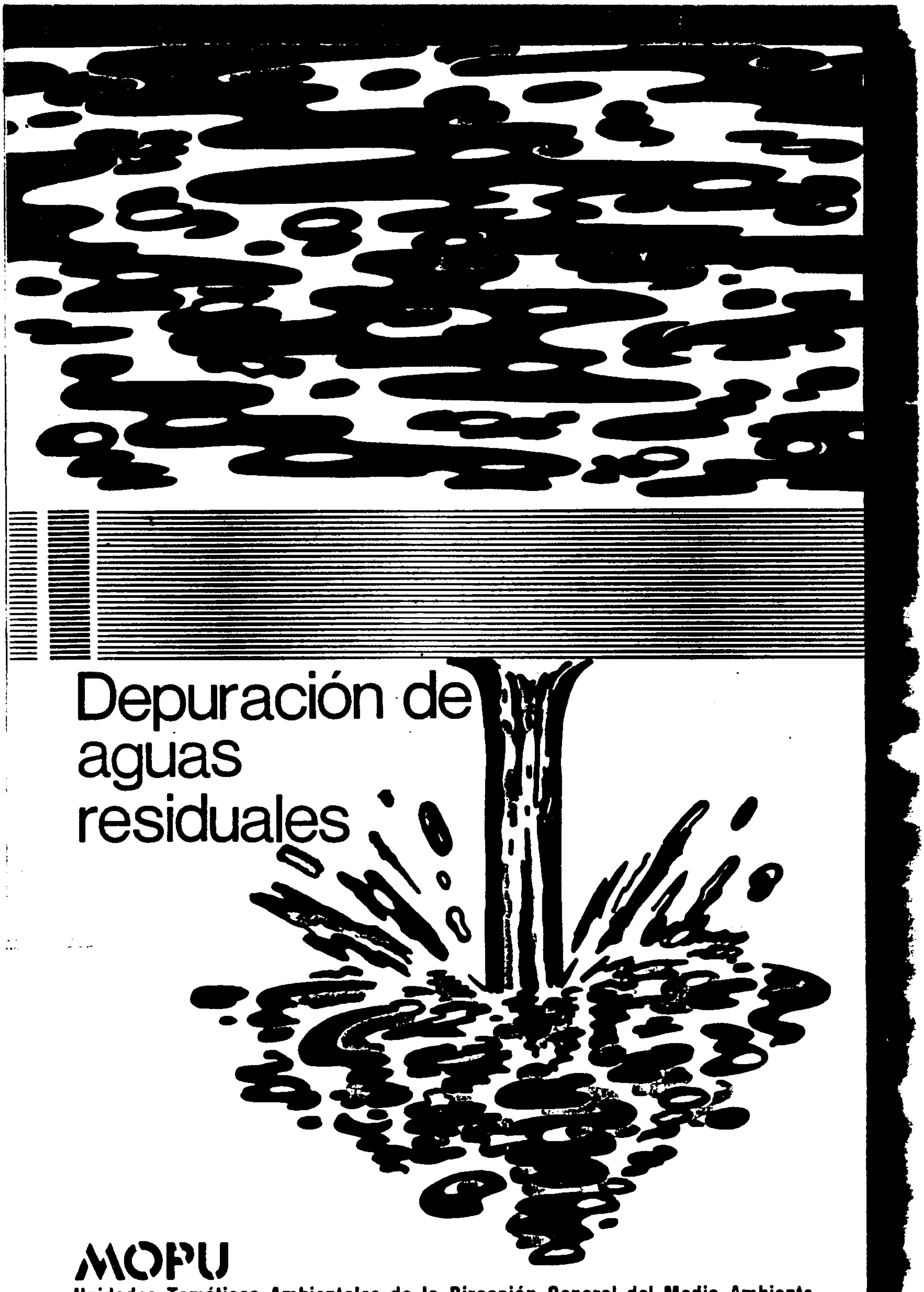
## Objectives of NWWA

The objectives of this association shall be: to assist, promote, encourage, and support the interests and welfare of the water well industry in all of its phases; to foster, aid and promote scientific education, standards, research, and techniques in order to improve methods of well construction and development, and to advance the science of ground-water hydrology; to promote harmony and cooperation between well contractors and scientific agencies relative to the proper development and protection of underground - water supplies; to encourage cooperation of all interested groups relative to the improvement of drilling and pumping equipment; to encourage, serve, assist and promote closer cooperation among the existing State water well contractors' associations and to foster the development of such associations in States where they do not exist; to collect, analyze, and disseminate to the public facts about the role of the water well industry in the economy of the nation; and to advance generally the mutual interests of all those engaged in the water well industry, in their own and the public welfare.

NWWA Constitution



# Depuración de aguas residuales

MOPU
**Unidades Temáticas Ambientales de la Dirección General del Medio Ambiente**

y Urbanismo.

3o

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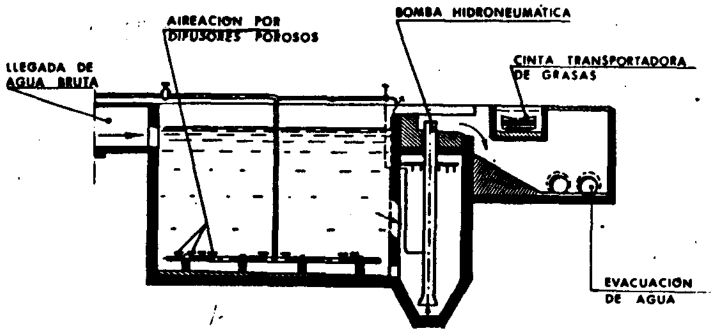
# INDICE

## 6.4. Desengrasadores (separación de grasas y aceites)

El proceso de separación de grasas y aceites, tal y como se entiende en un pretratamiento, consiste en la eliminación de este tipo de sustancias siempre que se encuentren libres en el agua residual, no pudiendo utilizarse cuando dichas sustancias se encuentren disueltas o en forma coloidal.

En aguas urbanas el desengrase es conveniente (indispensable si no hay decantación primaria) y se efectúa satisfactoriamente en combinación con el desarenado (tranquilización de una fracción de la superficie líquida en un desarenador aireado y recogida de las grasas y espumas por el vertido o rascado).

El equipo lleva una zona de aireación en la que se inyecta aire por su parte inferior y una zona tranquila donde se acumulan las grasas en la superficie. El tiempo de permanencia del agua es de 10 a 15 minutos a caudal medio. En estas condiciones, puede conseguirse una eliminación del 80% de las materias grasas. Puede ser necesario el empleo de un desengrasador separado cuando la calidad del agua bruta exige la eliminación de gran cantidad de grasa. En este caso se calcula la zona tranquila para una velocidad ascensional de 15 a 20 m/h.



DESENGRASADOR- DESACEITADOR

# 7. Tecnologías de bajo coste

La aplicación de tecnologías denominadas blandas al tratamiento de las aguas residuales urbanas, vienen desarrollándose en todo el mundo desde hace décadas. Encuentran un campo de aplicación especialmente adecuado en las pequeñas y medianas colectividades debido fundamentalmente a:

- Su buen rendimiento en eliminación de patógenos.
- Su facilidad de operación y mantenimiento.
- Buena integración en el medio rural.

En España se ha demostrado que los sistemas de depuración convencionales aplicados a municipios pequeños y medianos, han tenido muchos problemas de funcionamiento estando en la actualidad parados o abandonados en su gran mayoría, esto es debido, no a que los procesos no sean los adecuados, ya que se ha demostrado que son perfectamente válidos, sino a lo elevado de sus costes de explotación.

Como alternativa a los sistemas de depuración convencionales se están implantando en España, los sistemas de depuración de bajo costo de explotación en pequeños y medianos municipios.

Estas tecnologías que se desarrollarán a continuación se concretan en: lagunaje, filtros verdes, lechos de turba y contractores biológicos rotativos.

## 7.1 Lagunaje

Las lagunas de estabilización constituyen la solución idónea para aquellos núcleos de población donde: el coste del suelo no es muy elevado, las cargas orgánicas pueden fluctuar, hay escasez de personal cualificado y por lo tano la inversión en costes de explotación del sistema, debe ser baja.

El coste de la eliminación de $DBO_5$ en procesos de lagunaje, es normalmente menos de la mitad que en los sistemas de tratamiento convencional (lodos activados).

El concepto de lagunaje o lagunas de estabilización, se aplica a cualquier estanque o grupo de estanques diseñado para llevar a cabo un tratamiento biológico. La materia orgánica que llevan las aguas residuales, se estabiliza (disminuye la $DBO_5$) mediante procesos anaerobios o aerobios según sea la laguna.

### 7.1.1. *Lagunas anaerobias*

El proceso en este tipo de lagunas tiene lugar mediante una fermentación anaerobia. Este proceso se descompone en dos grandes etapas. En la primera, un grupo especial de bacterias formadoras de ácido facultativas, descomponen las cadenas complejas de la materia orgánica en ácidos grasos, aldehidos y alcoholes. En la segunda etapa, otro grupo de bacterias estrictamente anaerobias, formadoras de metano, transforman los productos intermedios en gas metano, amoniaco y anhídrido carbónico e hidrógeno. El proceso anaerobio, lo mismo que el aerobio transforma carbono, nitrógeno y fósforo en protoplasma celular.

Estas lagunas, están formadas por estanques con más de 2 metros de profundidad, en los que se produce una estabilización parcial de la materia orgánica, pero que dependiendo del medio receptor, puede ser suficiente para el vertido de las aguas tratadas.

El principal inconveniente del tratamiento anaerobio, es la posibilidad de que produzcan malos olores, si el diseño no es correcto.

El diseño de lagunas anaerobias debe fundamentarse en cargas volyméticas entre 0,1 y 0,4 Kg. $DBO_5/m^3/día$.

Los valores alrededor de 0,1, deben ser utilizados en zonas donde la estación fría es prolongada y alrededor de 0,4 donde hay unos valores anules de temperaturas templados (27-30 °C).

El volumen de una laguna se puede calcular mediante la siguiente fórmula:

$$V = \frac{Li \cdot Q}{\lambda v}$$

donde:

$V$ = Volumen de la laguna en $m^3$
$\lambda v$ = carga volumétrica $grDBO_5/m^3/d$ (100-400)
$Li$ = concentración del influente $DBO_5$ mg/l.
$Q$ = caudal de entrada $m^3/d$.

Teóricamente no hay límite para la profundidad de la laguna anaerobia, pero una profundidad de 4 m., puede considerarse como óptima desde el punto de vista del tratamiento.

Las lagunas anaerobias deben ser construidas, en general, en paralelo, si bien puede ser necesario el construir una tercera unidad por las condiciones de carga, por lo que podría ser necesario que dos de las lagunas funcionen en serie. Los problemas de olor pueden surgir si el contenido en azufre (formación de sulfídrico) del influente es superior a 100 mg/l.

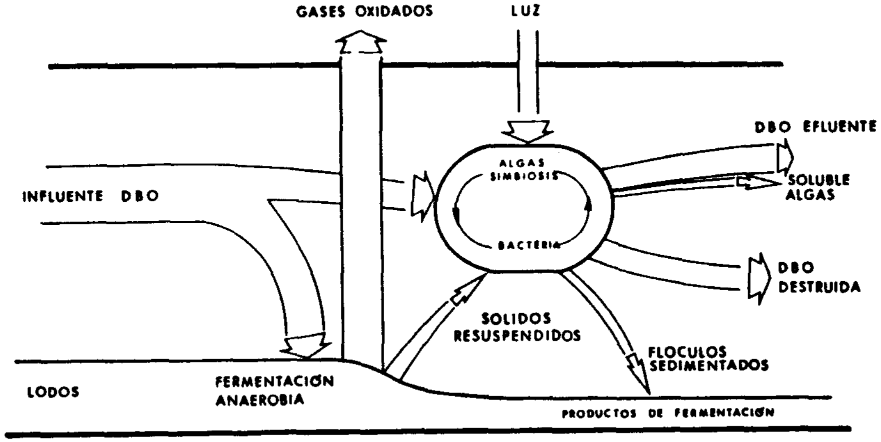
En el peor de los casos el rendimiento de una laguna anaerobia es el de un decantador primario (30% $DBO_5$; 60% S.S.), pero es normal alcanzar rendimientos superiores al 50% en eliminación de $DBO_5$.

### 7.1.2. *Lagunas facultativas*

En las lagunas facultativas hay una capa superior aerobia, una zona central que contiene bacterias facultativas y una zona de fondo de fangos que es anaerobia. La mayor parte de las lagunas de estabilización contienen lagunas facultativas.

El proceso aerobio de la materia orgánica hace que el carbono se utilice como fuente de energía para los microorganismos que al ser respirado produce anhídrido carbónico. Los microorganismos que participan son bacterias fundamentalmente, pero también hay hongos y protozoos. Estos microorganismos utilizan el carbono restante, así como el fósforo y el nitrógeno para formar nuevas células.

El oxígeno que utilizan las bacterias es producido mediante la actividad fotosintética de las algas durante el día, y la aireación atmosférica durante la noche.



ELIMINACIÓN DE DBO EN UNA LAGUNA

Las lagunas facultativas pueden ser las primeras de una serie o seguir a las lagunas anaerobias.

La profundidad de las lagunas facultativas puede variar de 1 a 2 metros, variando la relación entre longitud y anchura entre

1:1 y 2:1. La variación de la temperatura en las diferentes estaciones del año, puede obligar a hacer más profundos los estanques.

En cuanto al diseño, se han desarrollado una serie de fórmulas que pueden ser utilizadas para el cálculo de lagunas. A continuación vamos a desarrollar alguna de ellas. Estas las aplicaremos en el ejemplo de La Solana, al final del capítulo.

*Método del Banco Mundial:*

$\lambda = 20T - 60$

$\lambda$ = ratio de carga Kg $DBO_5$/Ha/día

T = temperatura mensual media mínima

$$A = \frac{10 \times Li \times Q}{\lambda}$$

A = Area de la laguna facultativa en $m^2$

Li = Concentración del influente en mg/l de $DBO_5$

Q = Caudal de entrada $m^3$/día

— Método basado en la carga por unidad de superficie. Los datos experimentales indican que pueden generalizarse determinados valores de carga orgánica en función de la superficie y del tiempo atmosférico.

| **Carga de superficie Kg $DBO_5$/Ha/d** | **Habitantes por Ha** | **Tiempo de retención (días)** | **Condiciones ambientales** |
|---|---|---|---|
| Menos de 10 | Menos de 200 | Más de 200 | Zonas muy frías |
| 10 - 50 | 200 - 1000 | 200 - 100 | Clima estacional frío, hielo en invierno, templado verano |
| 50 - 150 | 1000 - 3000 | 100 - 33 | Clima templado y semi tropical |
| 150 - 300 | 3000 - 7000 | 33 - 17 | Clima tropical |

Conviene ser muy cuidadoso con estos valores a la hora de diseñar el proyecto.

*Método de Ringuelet:*

$$Tr = \frac{1}{K} \frac{Ce}{Cs} - 1$$

Ce = $DBO_5$ de entrada mg/l

Cs = $DBO_5$ de salida mg/l

K = Constantte

Tr = Tiempo de retención

*Método de Hermann y Gloyna:*

$$L_p = \frac{L_o}{K_T R_T + 1}$$

$L_o$ = $DBO_5$ de entrada mg/l

$L_p$ = $DBO_5$ de salida mg/l

$K_T$ = Velocidad de descomposición a temperatura T

$R_T$ = Tiempo de retención a temperatura T

$$\frac{K_{35}}{K_T} = \theta^{(35-T)}$$

T = Temperatura de funcionamiento de La Laguna.

$\theta$ = Coeficiente de reacción de 1,085

$K_{35}$ = Velocidad de descomposición a 35ºC es de 1,2

*Método sudafricano:*

$$L_p = \frac{600}{(0,18d + 8)}$$

$L_p$ = DBO de salida mg/l

d = profundidad (m)

### 7.1.3. *Lagunas de maduración*

En estas lagunas se supone que toda la masa de agua está en condiciones aerobias. Son estanques de poca profundidad (0,3 - 1 m) con una producción máxima de algas. Se diseñan fundamentalmente para eliminar patógenos y reducir la $DBO_5$ a los niveles mínimos, ya que el grueso de la materia orgánica ha debido de ser estabilizada en las lagunas anaerobias y facultativas previas.

El objetivo principal de las lagunas de maduración es producir un efluente de alta calidad. Con los estanques de maduración no se persigue aliviar el trabajo de los estanques facultativos o de plantas biológicas convencionales que se hayan quedado escasos de tamaño.

El tiempo de retención de una laguna de maduración así como el número de lagunas, está condicionado por el grado de depuración bacteriana que se quiere. La laguna de maduración debe proporcionar un período de retención de siete a diez días con una profundidad de un metro.

Para el diseño de una laguna anaerobia se utiliza el concepto de considerar la eliminación coliformes fecales como una ecuación de primer orden dado por la fórmula:

$$B_e = \frac{B_i}{1 + K_{B(T)} \cdot t^+}$$

$B_e$ = n.º de coli. fecales /100 ml de efluente
$B_i$ = n.º de coli. fecales/100 ml en influente

$K_{B(T)} = 2{,}6 \cdot (1{,}19)^{(T-20)}$

$K_{B(T)}$ = Constante en T.ºC/día
$t^+$ = Tiempo de retención

Para una serie de lagunas anaerobia + facultativa + maduración, la eliminación total de coliformes fecales sería:

$$B_e = \frac{B_i}{(1 + K_{B(T)} \cdot t^+_{an})\ (1 + K_{B(T)} \cdot t^+_{fac})\ (1 + K_{B(T)} \cdot t^+_{mad})}\ .$$

$t^+_{an}$ ; $t^+_{fac}$ ; $t^+_{mad}$ = Tiempo de retención de las lagunas anaerobia, facultativa y de maduración

## 7.1.4. *Recomendaciones generales de proyecto*

En este apartado se hacen una serie de consideraciones al proyecto de lagunaje que no pretenden ser exhaustivas.

La ubicación de una instalación de tratamiento por lagunaje puede estar predeterminada por alguna decisión anterior como puede ser el emplazamiento de un colector de desagüe. La nautraleza de la zona influye mucho en la economía global del proyecto de construcción y de la explotación de la planta.

Las características del terreno deben ser tales que la laguna o lagunas previstas satisfagan los requisitos de forma y tamaño fijados. Debe tenerse especial cuidado con las zonas residenciales próximas. Las normas relativas a la distancia de las viviendas dependen de las normas locales, del clima y del tipo de tratamiento previsto, así en Israel se ha previsto una distancia mínima de 1-2 km y en Estados Unidos de 0,5-1 km entre las

lagunas y las zonas residenciales.

Debe preverse una carretera o camino de acceso a la planta de tratamiento.

El sistema de estanques debe estar protegido contra las inundaciones.

El terreno debe ser suficientemente extenso para proteger las ampliaciones previstas.

En general es necesario instalar un pretratamiento con rejas de gruesos, finos y desarenador. El desarenador es imprescindible cuando la red de alcantarillado recoge a la vez aguas residuales y pluviales.

Siempre que sea posible conviene instalar un dispositivo de medida de caudal. Si no se mide el caudal de entrada, es imposible determinar la carga de un sistema de lagunaje, y si no se mide el caudal de salida no se pueden evaluar las pérdidas por filtración y evaporación.

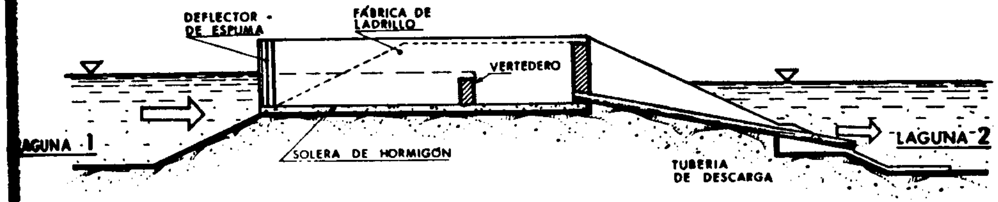
Las entradas de agua a las lagunas deben estar sumergidas, y salvo en instalaciones pequeñas deben ser múltiples. La tubería de entrada puede estar enterrada o montada sobre soportes asentados en las proximidades del punto de descarga. En lagunas anaerobias la tubería de entrada se debe instalar en el centro del estanque, debiendo prolongarse el punto de descarga un metro o dos hacia arriba para reducir las posibilidades de obstrucción por el fango. Una descarga superficial cerca de un dique contribuye a la aparición de malos olores.

La erosión del agua puede evitarse si la tubería de conexión entre estanques descarga horizontalmente cerca del fondo.

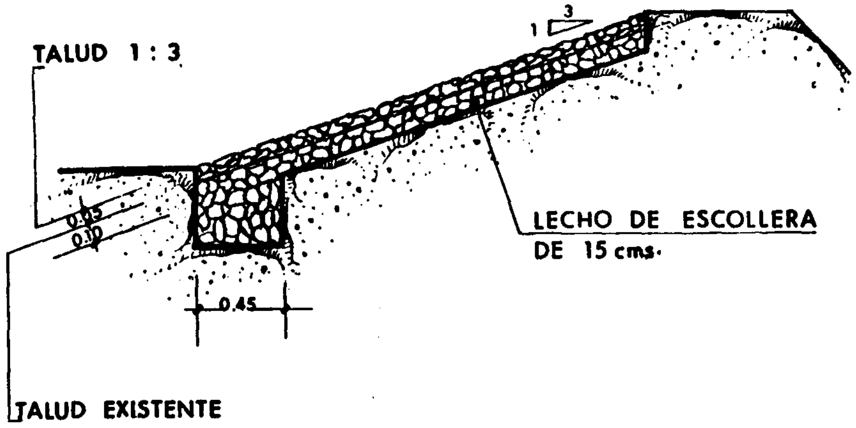
En el proyecto debe tenerse en cuenta la dirección del viento dominante ya que éste puede dar lugar a la formación de olas y, por tanto, movilización y mezcla de agua que puede llegar a las capas de fondo.

El fondo de la laguna debe estar impermeabilizado, poniendo especial atención en los puntos donde había árboles.

La parte interior de los diques debe estar protegida contra la erosión causada por la lluvia, el viento y el oleaje dentro de la laguna. Las lagunas deben estar dispuestas a sotovento del núcleo y con el eje mayor perpendicular a los vientos dominantes.



DETALLE DE UNA CONEXION ENTRE LAGUNAS



SECCIÓN DE DIQUE CON PROTECCIÓN DE ESCOLLERA

Siempre que sea posible, se deben disponer interconexiones entre las lagunas de forma que puedan trabajar independientemente instalando un by-pass.

*Ventajas e inconvenientes del sistema de lagunaje*

Las ventajas se pueden concretar en:

- Bajo coste de construcción y explotación.
- Las necesidades de aporte energético artificial son mínimas.
- No requiere personal cualificado para el mantenimiento y la operación.
- Altos rendimientos de eliminación de contaminantes orgánicos y sólidos en suspensión.
- Los lodos estabilizados hay que evacuarlos cada 5-10 años.
- Absorbe fácilmente las variaciones de carga debido al efecto regulador de las lagunas.
- El efecto regulador permite almacenar agua que por sus características sanitarias (laguna de maduración), es apta para el riego.
- Es un proceso natural que se integra fácilmente en el medio rural.

Los inconvenientes fundamentales son:

- Se necesita una gran cantidad de terrenos.
- Si el vertido del agua tratada es a un embalse, puede crear problemas de eutrofización por la gran cantidad de algas que lleva en suspensión.
- Puede haber pérdida de agua por evaporización.

### 7.1.5. El sistema de lagunaje de La Solana.

La Solana es una población de la provincia de Ciudad Real, situada a 14 Km. de Manzanares, en la Carretera Nacional número 430, en dirección a Albacete, y que cuenta en la actualidad con una población de 14.000 habitantes.

La actividad principal de La Solana es la agricultura, existiendo industrias de transformación de productos agrícolas (bodegas, almazaras). El resto de las industrias (confección, fabricación de aperos agrícolas, etc...) tienen poca demanda de agua y sus vertidos no presentan contaminantes especiales.

La Solana cuenta con una red de saneamiento unitaria que, por medio de dos colectores principales de 1,20 × 0,60 vierte las aguas a 700 m. del casco en un paraje conocido con el nombre de La Serna, de donde se evacuan por medio de un canalillo en tierra que llega hasta el rio Azuer.

Siguiendo distintos métodos y teniendo en cuenta factores externos, se estudió la posible evolución de la población sacando como conclusión que está muy estabilizada y que en el año 2000 sería un máximo de 14.500 habitantes.

Las estadísticas confeccionadas por el Ayuntamiento señalan, como es normal en este tipo de poblaciones, una gran diferencia de consumo de agua entre verano e invierno y un aumento, pequeño, pero constante en la dotación. Los datos actuales son: consumo de invierno 76 l./hab./día, y 125 l./hab./día en verano. Dado el tipo de población y las dificultades de abastecimiento, se ha previsto para el año 2000 un consumo, constante en toda época de 250 l./hab./día.

Para el buen funcionamiento de la planta era necesario evitar el vertido de los alpechines a la red general ya que su alta $DBO_5$ y DQO (100.000 mg./l.) y la formación de una película superficial de grasas impediría la correcta oxigenación del agua. Debido a esto se proyectaron redes de saneamiento independientes para las almazaras que vierten los alpechines a dos balsas de acumulación y evaporización.

Se realizaron estudios de suelos y geotécnicos que marcaron limitaciones en el uso de tierras, según procedieran de distintos horizontes, y en la profundidad máxima a excavar.

Los datos de diseño de las lagunas se concretan en la siguiente tabla:

| | **BALSAS ANAEROBIAS** | **BALSAS FACULTATIVAS** | **BALSAS AEROBIAS MADURACION** |
|---|---|---|---|
| FUNCION | REDUCCION ANAEROBIA DE M.O. | REDUCCION FACULTATIVA DE M.O | ELIMINACION GERMENES PATOLOGICOS Y REDUCCION AEROBIA DE M.O. |
| CARGAS DE DISEÑO (DBO) | 100-140 grs./$m^3$/día | 10 grs. $m^2$/dia | |
| DEPURACION PREVISTA (DBO) | 50% (MEDIA) 75% verano<br>40% invierno | HASTA 25-50 mg /l | HASTA 15 25 mg./l |
| DEPURACION REAL (Oct. Nov.) | 78% | HASTA 25-50 mg /l | HASTA 15 30 mg./l |
| COLOR | GRIS OSCURO | VERDE GUISANTE | VERDE CLARO |
| NECESIDAD DE LIMPIEZA | CADA 2-3 Años | Cada 10-15 Años | |
| ELIMINACION GERMENES PATOGENOS | | | > 99'99 % |

M.O. = Materia orgánica

Los procesos de depuración considerados fueron tres:

a) proceso meramente facultativo.

b) proceso anaerobio + facultativo; en una primera fase se le añade una depuración más por medio de balsas de maduración.

c) proceso de recirculación.

Se evaluaron para su cálculo cinco métodos diferentes: Sudafricano, Macgarry y Pescod; empírico de la India; empírico Israelí y el Oswald y Gotaas. De todos ellos los que mayores superficies exigían eran el sudafricano y el israelí, por lo que, para calcular del lado de la seguridad, fueron los métodos de cálculo elegidos.

Los cálculos se realizaron para una primera fase (final de 1984) y para la fase final (año 2000) y con una limitación de que las aguas depuradas en las peores condiciones de carga orgánica y climatológicas, tuvieran una $DBO_5$ menor que 40 mgr./l.

De los cálculos realizados bajo las condiciones anteriores y por los métodos citados se dedujo que el proceso merámente facultativo (a/) exigía, en la fase final, mayor superficie de la disponible, por lo que fue rechazado.

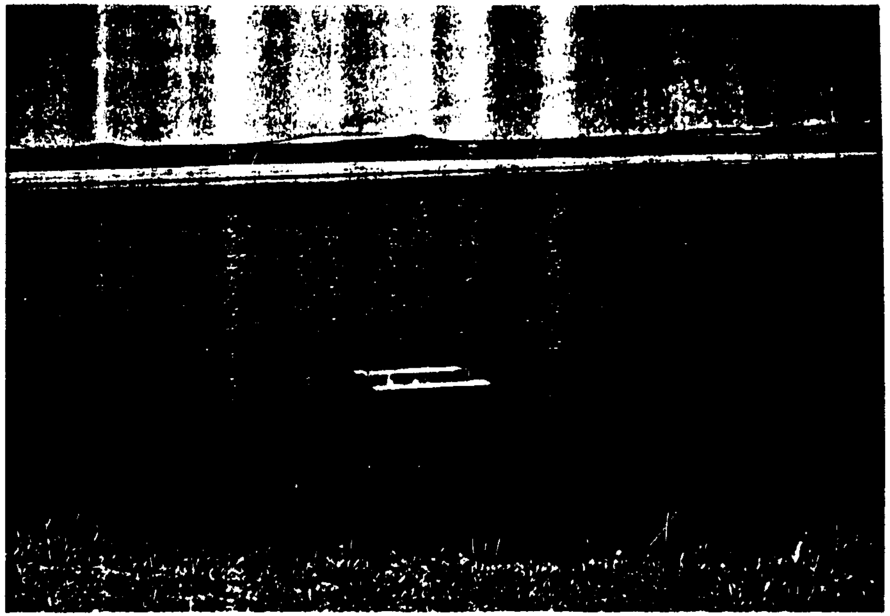
Dado que las superficies exigidas por los dos procesos restantes (b/ y c/) eran muy similares (algo mayor los del proceso de recirculación), se decidió dar una mayor flexibilidad a la planta y, sobredimensionando las lagunas facultativas del proceso b/, permitir en un futuro el cambio de proceso eliminando las lagunas anaerobias y recirculando las aguas desde la salida de las lagunas de maduración secundarias a la entrada de la laguna facultativa primaria.

Las lagunas quedaron dimensionadas de la siguiente forma:

4 lagunas anaeróbicas de 0,105 Has./Ud.

2 lagunas facultativas de 1,19 Has./Ud.

2 lagunas de maduración (*) de 1,33 Has./Ud.



Laguna de maduración. (Foto: Autor)

En la primera fase, funcionan dos lagunas anaerobias, dos facultativas y dos de maduración. Progresivamente y de acuerdo con el aumento de aportación de carga orgánica y de volúmen de vertidos, entrarán en funcionamiento la tercera y cuarta lagunas anaerobias. De igual forma, se transformará la primera de las lagunas de maduración en facultativa y, más adelante, la segunda. Es decir el dimensionamiento de las lagunas es tal, que permite el crecimiento de la planta depuradora en forma modular.

Si se llegase en determinado momento a una sobrecarga de la planta y debido a ello se produjese un problema de olores, podría cambiarse el proceso al de recirculación. Este proceso asegura la eliminación de olores y da una calidad menor de las aguas residuales (40 mgr./litro $DBO_5$ en verano y 60 mgr./litro $DBO_5$ en invierno). El cambio de un sistema a otro se realiza en poco tiempo, con sólo las siguientes obras:

— Instalación de una bomba con capacidad para 1,5 Q (caudal medio).

— Instalación de tubería de presión desde la bomba a la entrada de las lagunas primarias.

— Instalación de tubería que conecte la salida de las lagunas primarias a las secundarias.

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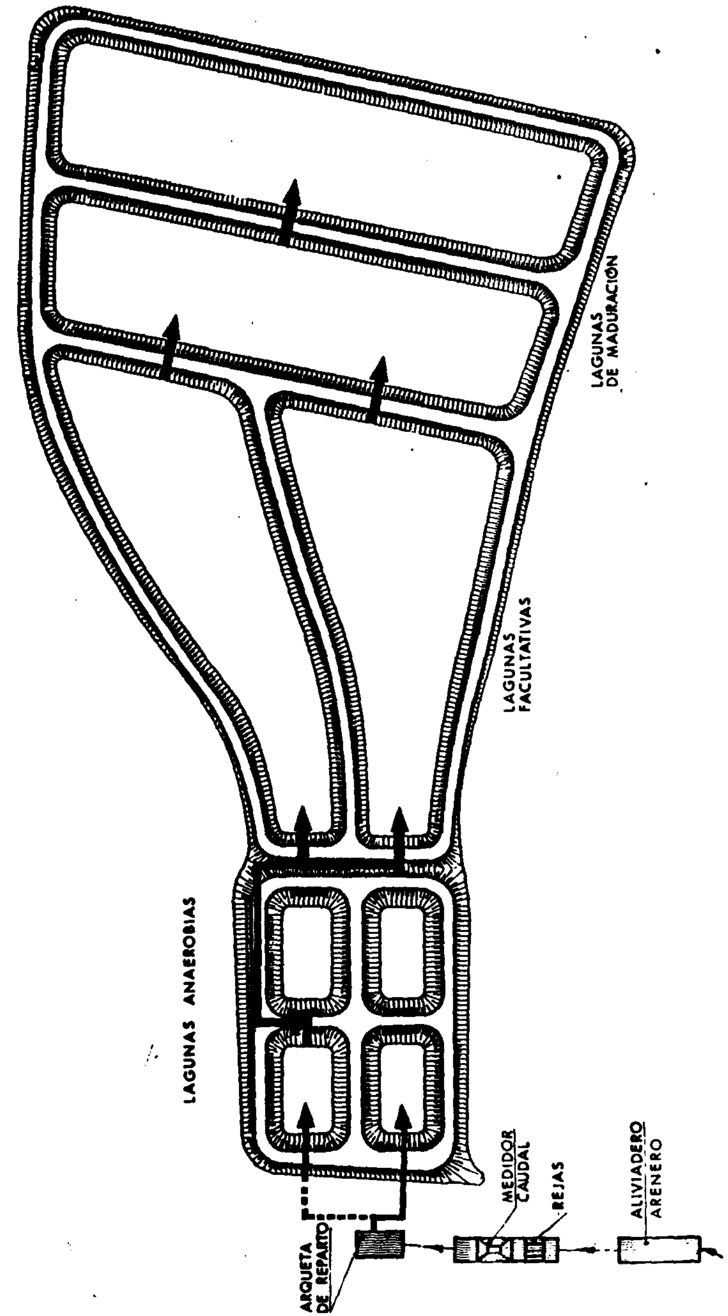
(*) Estas lagunas pasarán a facultativas cuando el incremento de vertidos lo exija.



LAGUNAJE EN LA SOLANA (CIUDAD REAL)

— Conexión directa desde el canal Parshall a las lagunas primarias.

Los gastos de inversión en la planta depuradora han sido de 20.000.000 de ptas., lo que supone 1.000 ptas./hab. equivalente. Esta cifra tan baja ha sido debida fundamentalmente a las buenas condiciones del terreno, lo que ha permitido el que no hubiera que impermeabilizar.

Posteriormente y debido a problemas de erosión en los taludes, ha sido necesario realizar una obra complementaria de estabilización de los taludes con escollera que supone un incremento de inversión de alrededor de 100 ptas./hab. equivalente.

Pero, aún siendo el sistema ventajoso por su bajo coste de instalación, lo es aún, mucho más, por sus costes de conservación, mantenimiento y operación. Este tipo de planta no necesita personal cualificado para su explotación, no precisa aporte de energía artificial y, al no existir, elementos mecánicos no exige mantenimiento ni reparaciones costosas.

El presupuesto anual requerido, confeccionado con datos contrastados con informes y documentación del Banco Mundial, es:

| | | |
|---|---|---|
| 1 operario-vigilante (media jornada) | 500.000 | pts. |
| 1 operario-vigilante media jornada (80d/año) | 200.000 | " |
| Herbicidas, larvicidas | 150.000 | " |
| Maquinaria en repaso muros y taludes | 200.000 | " |
| Varios | 100.000 | " |
| TOTAL | 1.150.000 | " |

Siendo el volumen tratado a finales de 1984 se cifra en 285.600 m³, el coste imputable por operación, conservación y mantenimiento fue de 4,20 ptas./m³ en 1985. Siguiendo el ritmo de enganche actual, 357.000 m³, lo que da un coste de 3,70 ptas./m³ en 1986. Dado que con este coste se logran similares eficiencias a las de las plantas de tipo convenional, la ventaja del sistema es patente.

## 7.2. El filtro verde

Un filtro verde es un terreno cubierto de cultivos agrícolas o plantaciones forestales (generalmente esta última), sobre el que se sitúan periódicamente aguas residuales procedentes de núcleos urbanos, con el fin de conseguir su depuración mediante la acción conjunta del suelo, microorganimos y plantas, mediante una triple acción física, química y biológica.

La instalación de filtros verdes, tiene por objeto el aprovechar las aguas residuales de poblaciones que podrían encontrar dificultades en la instalación y, sobre todo, en el mantenimiento de una depuradora de tipo convencional o de otros sistemas depuradores como el del lagunaje con extracción de lodos.

Se pretende que el suelo no sólo actúe como un filtro físico, sino que también trabaje biológicamente produciendo una oxidación bioquímica sobre buena parte de las sustancias contenidas en suspensión de las aguas residuales, de modo que sean asimilables por la vegetación y las que no lo sean pasen al acuífero en forma y proporción que no supongan peligro de deterioro para el mismo.

La depuración de las aguas residuales aplicadas al suelo tiene lugar en los horizones superiores de éste, en una capa biológicamente activa cuya profundidad no puede rebasar 1,20 metros. En este suelo vivo las aguas que lo atraviesan lentamente al percolar, sufren dos procesos simultáneos: un filtrado mecánico que retiene los sólidos en suspensión no degradables, aún los más finos como colorantes de tintorería o detergentes, cuyas espumas desaparecen y una oxidación bioquímica bacteriana que mineraliza la materia orgánica.

El oxígeno necesario para esta mineralización de la materia orgánica proviene del aire contenido en esta capa activa del suelo en la que ocupa del orden del 50% de su volumen lo que equivale a un contenido de unos 600 Kg. de oxígeno por Ha., que se renueva naturalmente por difusión cada 75 minutos.

La oxidación de la materia orgánica es producida por bacterias aerobias que se hallan en el suelo formando una ténue película que envuelve las partículas de éste, que son capaces de entrar en acción a partir de 0° C de temperatura en presencia de agua y de materia orgánica adherida a la película que forman. En suelos de textura no gruesa la superficie de esta película está evaluada en cifras del orden de 5.000 m² por metro cuadrado de superficie de terreno.

La descomposición bacteriana de la materia orgánica da lugar a que el nitrógeno pase de formas amoniacales a nitritos y nitratos. Este efecto se hace patente en la vegetación espontánea de los campos de aplicación que se compone casi exclusivamente de plantas nitrófilas y en la ausencia de las leguminosas capaces de fijar el nitrógeno atmosférico en los módulos de sus raíces.

El fósforo se oxida y da con las bases fosfatos que parte se inmobilizan y parte son retenidos por la vegetación y parte percolan; en presencia de cal queda inmovilizado, y es preciso abonar los cultivos con fósforo a partir de la aportación del contenido en las aguas residuales.

La flora intestinal que transportan las aguas residuales domésticas encuentra en el suelo condiciones hostiles de variaciones de temperatura y abundancia de oxígeno que le son nocivas; sus relaciones con la población bacteriana del suelo no son bien conocidas, pero los análisis de las aguas de percolación por debajo de suelo activo muestran que a efectos sanitarios la depuración natural producida por éste podrían ser similares a los logrados por un tratamiento secundario convencional.

Con las altas temperaturas se intensifica la actividad vegetativa y bacteriana. La temperatura de las aguas residuales, más elevada que la del terreno en invierno, debe producir un alargamiento de la actividad vegetativa. Todo ello contribuye a incrementar la extracción de nutrientes aportados por las aguas residuales.

Los estudios ya realizados sobre la composición de las aguas residuales urbanas dan como resultado su posible utilización para el riego de choperas, una vez eliminados los sólidos más gruesos y algunas sustancias fácilmente separables.

Las aguas que lleguen a las choperas, deberán estar desprovistas de los elementos sólidos que queden retenidos por las rejillas, que se colocarán con esta finalidad y en la medida de lo posible desarenadas y desengrasadas.

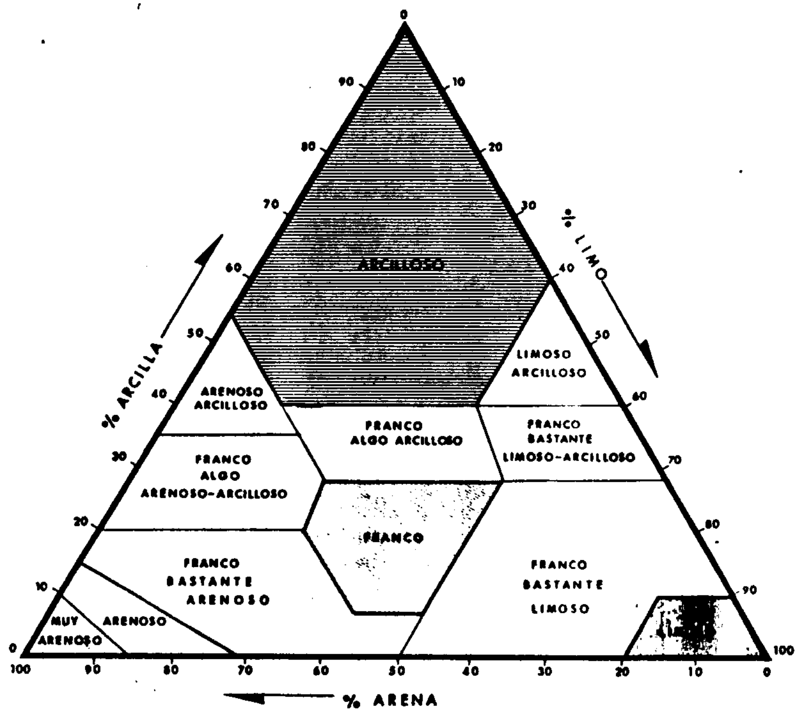
Las aguas residuales urbanas pretratadas deben ser distribuidas en riego por gravedad a lo largo del año sobre un terreno nivelado y compartimentado en la forma denominada en agricultura riego a manta por pie. Las cargas de riego anuales pueden variar si la vegetación a establecer es arbórea para producción de madera, de 500 a 2.800 mm. de agua residual, lo que supone riegos semanales de 10 a 50 mm. La práctica de este riego no difiere de las que se practica en cultivos agrícolas de regadíos más que los riegos se mantienen durante todo el año. No implica consumo de energía si no se precisa bombeo desde el colector al campo de aplicación.

El ámbito de utilización de la depuración natural establecido por la Dirección General de Obras Hidráulicas en el Informe Técnico Previo para Depuración de Aguas Residuales por Aplicación al Terreno (Filtro Verde) es para:

- Poblaciones con un límite superior de 25.000 habitantes.
- De efluentes que no contengan sustancias nocivas para los cultivos.
- En Municipios en los que existen superficies de terreno del orden de 1 Ha. para cada 200 hab. que

pueden ser susceptibles económicamente de transformación en regadío.

En cuanto a características granulométricas de los terrenos, los suelos arcillosos, arenosos o muy arenosos no son recomendables.



Un tratamiento selvícola esencial lo constituye el laboreo del suelo para romper la costra de sólidos contenidos en las aguas residuales que se acumulan en la superficie del terreno, facilitando una mejor aireación del suelo y aumentando la capacidad de infiltración de las aguas. Así se consigue un suelo mucho más activo biológicamente, ya que se favorece la penetración y circulación del oxígeno y el incremento de su microflora y microfauna.

Experiencias realizadas por el Insituto Nacional de Investigaciones Agrarias demuestran que efectuando laboreos se llega a triplicar la producción de madera en los primeros 12 años. Aunque en una chopera regada con aguas limpias sería suficiente la realización de un laboreo al año, en el caso de regarse con aguas residuales resulta aconsejable ejecutar dos o tres laboreos anualmente.

La labor debe hacerse en un sólo sentido, prescindiendo de

la cruzada que se aconseja para otras choperas, ya que así se facilita que las filas de chopos paralelas a la labor queden en unas fajas de tierra no movida por la labor, que servirán como caballones de separación de las parcelas de riego. Estos caballones irán teniendo más entidad a medida que se realicen las distintas labores.

Otra labor selvícola indispensable para conseguir maderas de buena calidad y aptas para el desarrollo, que son las que alcanzan mayores precios en el mercado, está constituida por las podas. Esta operación deberá realizarse empleando las técnicas de poda sobradamente conocidas, para no producir daños al árbol, periódicamente durante los primeros años, hasta dejar un fuste limpio de 8 m. de altura.

En cuanto a plagas y enfermedades cabe decir que debe extremarse en el futuro la vigilancia para atajar desde el primer momento posibles ataques de plagas o enfermedades.

Tendrá mucha importancia, para nuevas repoblaciones, elegir los clones resistentes a las plagas actuales.

Dentro de este punto hay que considerar los siguientes:

- Adaptación al medio. No todos los clones resultan adecuados. Pero si hay repoblaciones anteriores nos pueden indicar los clones que se han desarrollado satisfactoriamente en la zona.
- Adaptación al sistema de filtro verde. En principio no existe suficiente experiencia para asegurar que clones son más adecuados para actuar en una parcela dedicada al filtro.
- Posibilidades de desarrollo. A mayor desarrollo obtendremos más alto rendimiento maderable y por tanto económico, y a la vez mayor capacidad de evapotranspiración.
- Elección del clon para que la chopera sea lo más resistente posible a plagas y enfermedades.

En España existen ejemplos de múltiples ataques de plagas y enfermedades en estos casos. Sin embargo muchas veces han sido debidos a defectos de instalación de los filtros en suelos no adecuados para chopera, a retrasos en la llegada del efluente y falta de agua de riego como alternativa, a que en los efluentes verían industrias productoras de vinazas y otros defectos de instalación o mantenimiento.

En cuanto a la plantación de chopos hay dos tipos de marco el de 5 X 5 m. y el de 6 X 5 m.

Estudios recientes permiten estimar que con marco de plantación de 6 X 5 m. en lugar del habitual 5 X 5 m. se pueden obtener rendimientos económicos al menos de un 10% superiores y una actividad del filtro verde un 2% más alta. Es decir unos 300 árboles por Ha.

Mayores espaciamientos podrían dar lugar a deformaciones de la sección, haciéndola más elíptica y, por tanto, disminuyendo el rendimiento en la transformación industrial de los rollizos, hecho estudiado por el INIA por J. V. Orensanz y otros.

. La chopera, con los riegos que se van a dar, rápidamente se empradizará. El aprovechamiento de los pastos no sólo podría suponer un beneficio económico suplementario (en el filtro verde de 50 Ha. de Villarrubia de los Ojos, Ciudad Real, se ha asignado los pastos anuales en 500.000 ptas., es decir, a razón de 10.000 ptas./Ha.) sino también ayuda a extraer las sales que la vegetación herbácea ha obtenido del suelo.

No podrá tenerse el ganado lanar en la parcela hasta el extremo de que agote los pastos, pues entonces mordisquearía la corteza de la base de los chopos. No es aconsejable la introducción de ganado vacuno por la tendencia que tiene a rascarse con los troncos, lo que también podría ocasionar daños.

El laboreo del suelo previsto afectará a dos tercios de la superficie (ya que se deja una faja de 1 m. a cada lado de la hilera de chopos). Del terreno labrado no se obtendrá más que la mitad de hierba de la que se produce en la zona sin labrar en una chopera regada con aguas limpias y abonadas. Eso supondrá una producción menor en una tercera parte de la que se obtendría por Ha. de una chopera no labrada.

Se estima que la cárga por Ha. que puede soportar una chopera es de 2 a 3 cabezas de ganado lanar por Ha. durante todo el año con una renta de 7.500 a 11.000 ptas. por Ha. y año

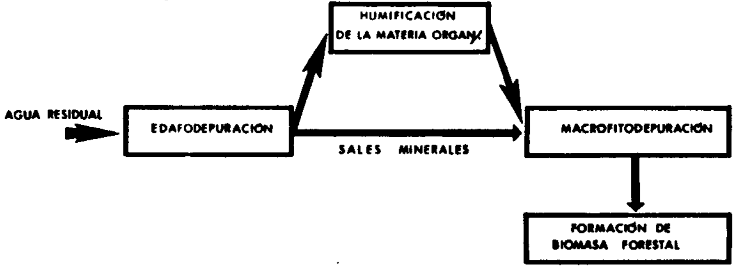
### 7.2.1. *Comportamiento del filtro verde*

En el período invernal los árboles tienen paralizadas sus savias y funciones vegetativas y no toman agua y nutrientes del suelo. La situación en que se encuentra un filtro durante esos meses es la de letargo desde un punto de vista vegetal, pero en un filtro verde concurren simultáneamente dos procesos de depuración natural, que se superponen a lo largo el año. Dichos procesos son: la macrofitodepuración y la edafodepuración.

La macrofitodepuración, es el proceso de depuración natural realizado por la macrofitas, al tomar la materia orgánica, los nutrientes y las sales minerales del agua para su propio metabolismo. Macrofitodepurácion será así la realizada por el chopo, el jacinto de agua, las espadañas y otras macrofitas. Microfitodepuración es, en cambio, la realizada, por ejemplo, por las microalgas en los lagunajes.

La edafodepuración, es el proceso de depuración natural realizado por el suelo, al constituirse en soporte inerte de microflora y fauna bacteriana que permite reproducir a gran escala y en función de su permeabilidad, los mecanismos de depuración que se desencadenan en un lecho bacteriano convencional.

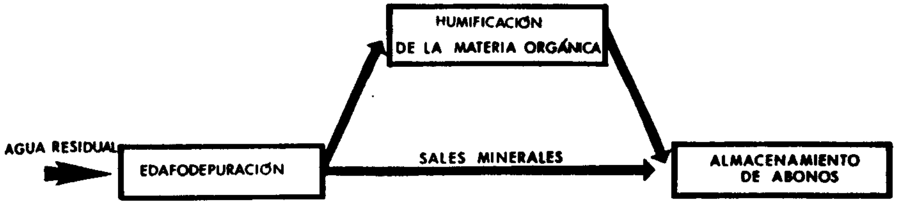
En el funcionamiento del filtro erde durante el período Primavera-Otoño, macrofitodepuración y edafodepuración, funcionan en régimen interdependiente, reteniendo el terreno materia orgánica de las aguas residuales, cuya degradación posterior por vía biológica origina ácidos húmicos y fúlvicos, que son asimilados por las raíces convirtiendo así contaminación en biomasa forestal. El terreno retiene materias minerales, igualmente asimiladas por las raíces.



FILTRO VERDE PRIMAVERA-OTOÑO

En el período invernal, la macrofitodepuración se interrumpe, pero no así las etapas anteriores del proceso, que prosiguen con normalidad su actividad, almacenando reservas de materia orgánica y de sales minerales en el terreno, que se movilizarán por el arbolado al comenzar éste su período vegetativo.

Este caso se puede asimilar a la práctica agrícola de realizar abonados de fondo en invierno, ya que la primavera traerá demandas importantes de los mismos ante las exigencias de punta, que significan las primeras etapas de crecimiento de los cultivos.



FILTRO VERDE OTOÑO-INVIERNO

## 7.3. Lechos de turba

El elemento esencial de una instalación de depuración de este tipo, es un lecho de turba a través del cual circula agua residual. Dicho lecho descansa sobre una delgada capa de arena, soportada a su vez por una capa de grava; un dispositivo de drenaje recoge el efluente en la base del sistema que puede realizarse simplemente, practicando una excavación que en caso necesario se aisla del terreno circundante mediante una lámina impermeable o cualquier otro método adecuado.

La turba es un material que es el resultado de la acumulación de materia vegetal y de su mayor o menor degradación biológica en condiciones de exceso y de falta de oxígeno.

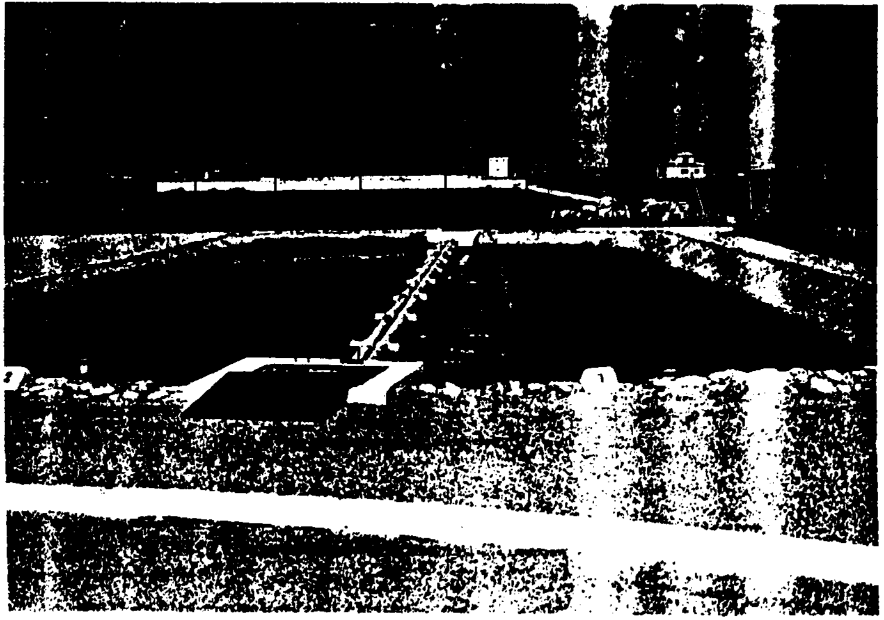
Este material, abundante en Europa y en América del Norte, se utiliza corrientemente en jardinería y para la enmienda de suelos.

El fenómeno básico que rige la depuración de un agua residual en un lecho de turba obedece a que estos materiales poseen cualidades absorbentes y de formación de complejos con respecto a las sustancias disueltas y las coloidales. Este fenómeno se completa con una retención mecánica de las materias en suspensión y con una depuración biológica.

El proceso de tratamiento consta de un pretratamiento y/o tratamiento primario que elimina los sólidos gruesos y una buena parte de los sólidos en suspensión y un tratamiento secundario que es el propio lecho de turba.

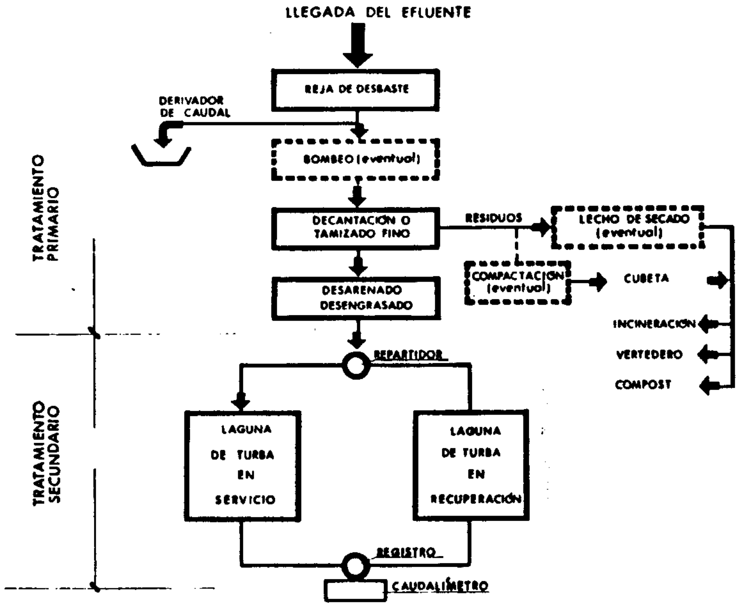
Los resultados de la depuración obtenida son, en general muy satisfactorios siendo los rendimientos medios del siguiente orden:

$DBO_5$ : 85-90%
DQO : 80-95%
S.S. : 90%



Lechos de turba. (Foto: Autor)

**FILTRO DE TURBA**
**ESQUEMA DEL PROCESO**



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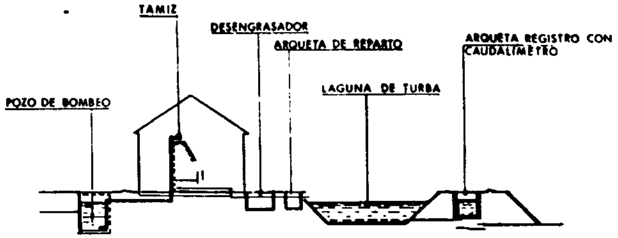

# Ground-Water Chemical Quality Management by Artificial Recharge[a]

by **Harry I. Nightingale and William C. Bianchi**[b]

## ABSTRACT

The effectiveness of basin ground-water recharge at the Leaky Acres Recharge Facility in Fresno, California for improving the regional ground-water quality was studied as 65,815,000 m$^3$ of high-quality surface water was recharged from 1971 through 1975. Observation wells at the facility showed some variability in chemical parameters associated with each recharge period. The long-term decreases in salinity could be described by a power law decay curve fitted by regression analysis.

Without a special network of observation wells outside the facility, scientific evaluation of the enclave of recharged water is not possible. A practical evaluation of water-quality changes is possible from producing water wells around the facility. However, the pumping well discharge-time variations, well depth, aquifer sequence, and prior use of surrounding land must be considered, since all of these factors affect the pumped-water quality and its seasonal variability. Recharge at Leaky Acres had noticeably decreased the ground-water salinity for a distance of up to 1.6 km in the direction of the regional ground-water movement.

## INTRODUCTION

Ground-water chemical quality has frequently been degraded as a result of man's activities and use of this resource (Kaufman, 1974). The causes of degradation and alternative projects to prevent it or perhaps improve the chemical quality are unique for each ground-water basin (Cofer and Owen, 1975; Schmidt, 1975a).

Artificial ground-water recharge is an alternative procedure for consideration in ground-water quality management, especially when low-salt surface water is economically available and the geology is favorable. Even wastewater is currently being considered for recharge, but more research is needed before its utilization for recharge to aquifers for unlimited use becomes a reality (Baffa, 1975; Bouwer, 1974; Cofer and Owen, 1975).

In 1970, the City of Fresno, California, which is completely dependent on ground water for its domestic and industrial supply, initiated a basin-type artificial recharge project. Surface water from the Kings River is delivered at a cost of $8.107 per 1000 m$^3$ ($10 per ac-ft). This water is available because urbanization is replacing irrigated agriculture. The recharge facility, Leaky Acres, is located about 7.2 km upgradient from the main pumping depression which is apparent in the regional ground-water gradient due to the urban well field. During the first 5 years (1971 through 1975) 65,815,000 m$^3$ (17.388 billion gal) of low-salt water has been recharged at Leaky Acres. Various research investigations associated with the large basin-type recharge facility were conducted during this time (Nightingale and Bianchi, 1973; Bianchi and Lang, 1974; McCormick, 1975; Nightingale, 1975; Bianchi and Nightingale, 1975).

Evaluating the effectiveness of a basin recharge facility based on the regional ground-water quality is analogous in many respects to evaluating

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[a] Water Management Research, Western Region, Agricultural Research Service, U.S. Department of Agriculture, Fresno, California 93726. In cooperation with the City of Fresno Department of Public Works—Water Division. Presented at The Third National Ground Water Quality Symposium, Las Vegas, Nevada, September 15-17, 1976.

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Discussion open until June 1, 1977.

contamination enclaves from surface waste disposal sites (LeGrand, 1965; Kimmel and Braids, 1974; Palmquist and Sendlein, 1975).

We could not economically justify determining the detailed geological and ground-water hydrological factors needed to develop a theoretical shape of an enclave of high-quality recharged water for this specific site. We expected the ground-water chemical quality to improve without bacteriological changes in the ground water. Therefore, like at Leaky Acres, evaluating most ground-water quality changes, associated with large recharge facilities, will depend upon sampling the existing wells and the variations accompanying: multiaquifer extraction, well depths, different usage, and non-ideal areal distribution. This report considers five years of ground-water quality data associated with recharge at Leaky Acres as determined under the above conditions.

## STUDY AREA AND METHODS

Geologically, the Leaky Acres Recharge Facility is located on a compound alluvial fan (Page and LeBlanc, 1969) composed of vertical and lateral heterogeneous layers of clay, silt, and sand. Water movement in the fan sediments is vertically impeded by the less permeable clay and silt horizons, as indicated by perched water, while lateral transmission is semiconfined specifically to the sandy horizons. The geology and stratigraphy in the upper 30 m at Leaky Acres has been well defined (Nightingale and Bianchi, 1973; Bianchi and Nightingale, 1975). Generally, there are three low permeability layers at 4, 11, and 20 m depths; each is about 1 to 3 m thick. In 1975, these layers limited the facility's average infiltration rate to 14.2 cm/day. The ground-water mound dissipates rapidly after each recharge period and is indicative of productive aquifers, as indicated by Schmidt (1975b). Well yields usually exceeded 126 l/sec (2,000 gpm); specific capacities averaged about 25 l/sec/m (120 gpm/ft) of drawdown and transmissivities averaged more than 2,480 $m^2$/day (200,000 gpd/ft).

Figure 1 presents the location of Leaky Acres and the location and use classification of peripheral wells used to evaluate the influence of recharge on the regional ground-water quality. Behnke and Haskell (1968) conducted a ground-water nitrate study in this area. The urban development south of E. Dakota Avenue (Figure 1; except for Fresno Air Terminal) consists of lot sizes of about 557 $m^2$ (6,000 sq ft), each with an individual septic tank disposal system so the nitrate contamination in the area was about 25 mg/l. Since north of E. Dakota Avenue is mostly abandoned farm land with few home septic tanks because of the clear-zone for the airport, nitrate levels were usually less than 10 mg/l.

The initial areal distribution of ground-water specific electrical conductivity (SEC) in the study area (Figure 2) was established in 1971 from data collected in the summer (May through August) before recharge. The rural area north of E. Dakota Avenue usually had SEC < 200 µmhos/cm.

The surface water being recharged at Leaky Acres is a low salinity, Ca-Mg$HCO_3$ type water, delivered from the Kings River (29 km east of Fresno) to the north side of Leaky Acres by the Fresno Irrigation District's Gould Canal. Samples for analyses were collected at inlets to the uppermost basins on the same schedule as the ground-water sampling at Leaky Acres.

Changes in ground-water quality beneath Leaky Acres were observed by sampling ten 20.32 cm I.D. PVC plastic-cased water-quality wells midway between top and bottom of slotted section below the third slowly permeable layer. The frequency of sampling of the wells at Leaky Acres had decreased from twice a week in 1971, to once a week in 1972, and semimonthly in 1973 through 1975. Ground water from 25 to 30 producing peripheral wells of various use-classification was usually sampled semimonthly, unless they were temporarily out of service, (i.e. irrigation well during the winter), or monthly if they were

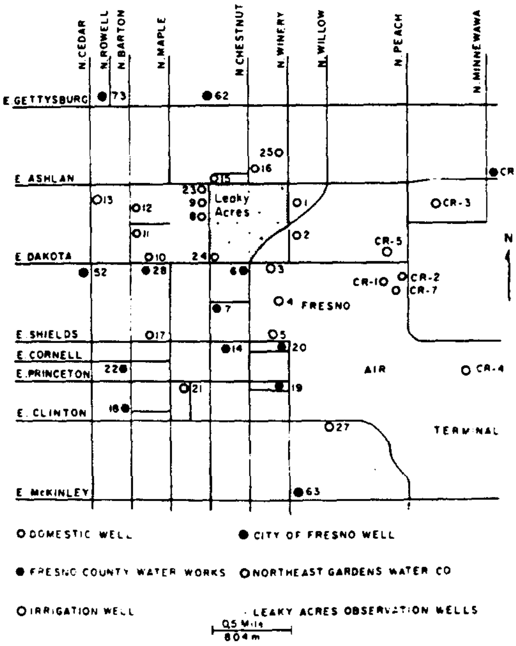

Fig. 1. Location of Leaky Acres Recharge Facility and peripheral ground-water quality monitoring water wells.

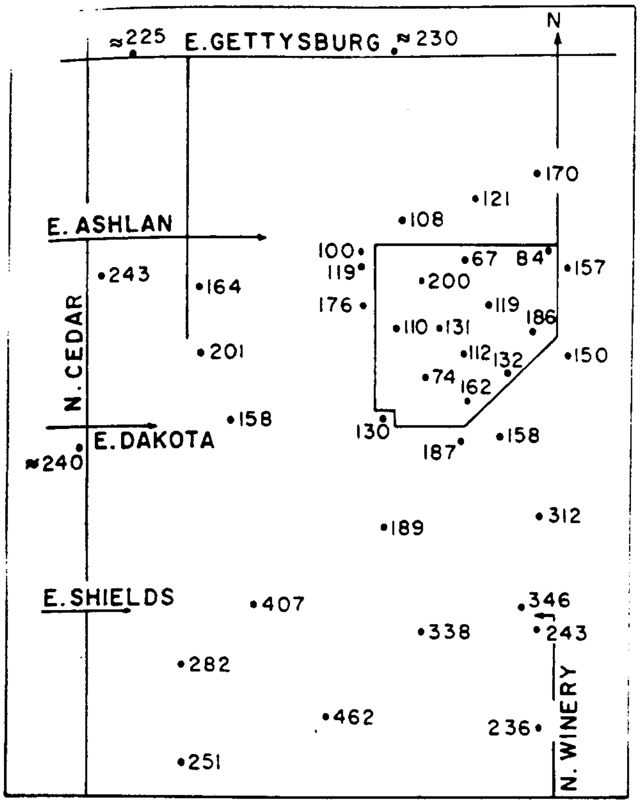

Fig. 2. Initial (May through August 1971) mean ground-water specific electrical conductivity, µmhos/cm, for study area.

fairly constant in quality. Water samples were immediately analyzed for their nitrate, chloride, and specific electrical conductivity (U.S. EPA, 1974). Ground-water turbidity and temperature changes were also measured. Major cations and anions were analyzed once yearly. For this study area, the total dissolved solids (TDS, mg/l) can be estimated accurately from the specific electrical conductivity (SEC, µmhos/cm) when in the 20 to 650 µmhos/cm range, by the regression equation: TDS = (SEC + 22.79)/1.62.

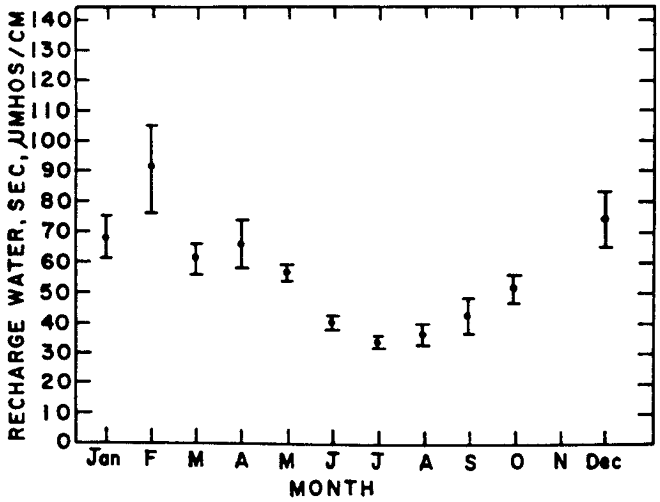

Fig. 3. Monthly mean specific electrical conductivity of the recharge water for 1971 through 1975 recharge periods. No recharge in November.

Table 1. Selected Canal Water Quality Parameters. Monthly Mean for 1971 Through 1975

| *Month* | $Cl^-$ *mg/l* | $NO_3^-$ *mg/l* | *FTU*[*] | *Temp. C* | *D.O.*[+] *mg/l* |
|---|---|---|---|---|---|
| Jan. | 2.51 | 2.77 | 13.0 | 8.8 | 8.00 |
| Feb. | 3.34 | 6.00 | 13.7 | 11.0 | 7.88 |
| Mar. | 1.58 | 1.21 | 8.7 | 13.0 | 9.80 |
| Apr. | 1.94 | 1.50 | 4.8 | 14.9 | 9.60 |
| May | 1.08 | 1.98 | 4.2 | 16.9 | 9.77 |
| June | 0.87 | 1.45 | 4.1 | 20.1 | 8.70 |
| July | 0.79 | 1.25 | 4.6 | 20.3 | 9.38 |
| Aug. | 0.72 | 0.95 | 2.8 | 19.8 | 9.00 |
| Sep. | 1.25 | 1.81 | 3.0 | 18.8 | 9.70 |
| Oct. | 0.78 | 1.28 | 3.6 | 16.6 | 9.34 |
| Nov. | – | – | -No Recharge- | – | – |
| Dec. | 2.65 | 1.78 | 4.5 | 10.0 | 8.00 |

[*] Turbidity in Formazin Turbidity Units.
[+] Dissolved oxygen.

## RESULTS AND DISCUSSION

### Canal Water Quality

The salinity of the canal water as evaluated by the SEC was usually between 30 and 70 µmhos/cm (Figure 3) during the recharge periods. The higher values resulted from runoff of some winter and spring storms. Table 1 presents the monthly mean chloride, nitrate, turbidity, temperature, and dissolved oxygen for 1971 through 1975. These data establish the excellent quality of the water before recharge.

### Ground-Water Quality Trends – Beneath Leaky Acres

Figure 4 summarizes the ground-water SEC trends during five years of recharging 65,815,000 $m^3$ of high-quality water. Shortly after the beginning of each annual recharge period, there was some increase in ground-water SEC, but a new, low, SEC was attained by the end of the recharge period. The fluctuations in the Spring of 1973, shows the impact of turning the water on and off because of turbid canal water or shut-down periods in the western basins because of insect problems. The jump in SEC during August and September 1974, is in response to a gypsum treatment over tile lines used for water collection for a recharge well study.

Because of the variability (Figure 4) caused by irregular recharge periods, a power law decay curve was fitted to the monthly mean SEC data by regression analysis. The fit is satisfactory with the

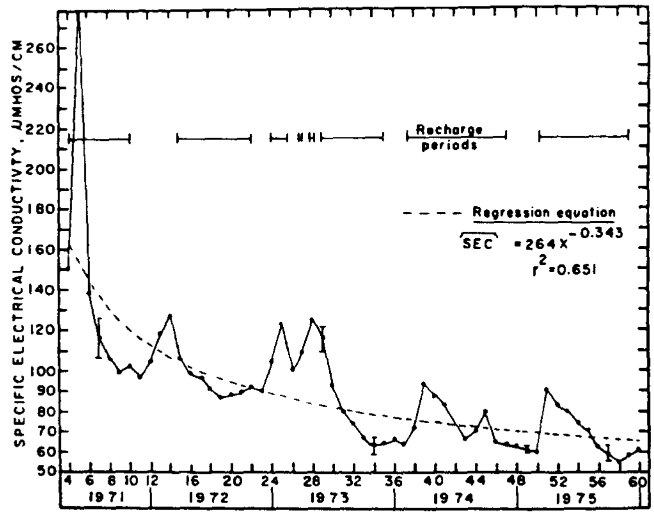

**Fig. 4. Response in ground-water SEC beneath Leaky Acres as related to annual recharge periods. Each data point is the mean of all observation wells and sampling times for a month. In the regression decay curve equation, X is the number beginning with X = 1, for January 1971.**

coefficient of determination, $r^2$ equal to 0.651. The regression equation would predict a ground-water SEC of 44 µmhos/cm by the December of 1980, 120 months after recharge started. This is reasonable if there is no recharge during December through February, when only water that is slightly higher in salinity is available. Essentially, the quality of the ground water beneath Leaky Acres is nearly in a steady-state equilibrium with the recharge-water quality.

Monthly mean ground-water nitrate levels beneath Leaky Acres are presented in Table 2. The nitrates in the soil profile before recharge were essentially leached out during the first recharge period. The slightly higher nitrate levels in the spring resulted partly from mineralization of organic matter during the winter dry period and partly from higher nitrate levels (5-15 mg $NO_3^-$/l) in the canal water when it contains surface runoff water from local storms. Fluctuations in the ground-water chloride levels were similar to the nitrate fluctuations. Before recharge the chloride level averaged 2.8 ± 0.3 mg/l for all wells at Leaky Acres, by 1975 the high spring level was 1.7 ± 0.1 mg/l and season's low was 0.7 ± 0.1 mg/l.

Periodically, the concentration of the major cations and anions in the ground water were determined. Table 3 presents the results only for the June 1971 and October 1975 samplings. Between these times the concentrations are between those shown. Fluctuations in these chemical parameters within a given recharge period are similar to the fluctuations in SEC shown in Figure 4. The milliequivalents per liter ratio of $Ca^{2+}$ to $Mg^{2+}$ has increased from 0.61 to 1.16 during the five-year period. The $Ca^{2+}$ to $Mg^{2+}$ ratio for the canal water for the 1975 data was 1.13, about the same as that for the ground water. The soluble silica concentration was within the 30 to 50 mg/l which is typical of the regional ground water. The pH has remained between 6.7 and 7.0. The dissolved oxygen content of the ground water averaged 8 to 9 mg/l at the start of a recharge period and decreased to 5 to 7 mg/l by the end of a recharge period.

## Ground-Water Quality Trends — Beyond Leaky Acres

In evaluating the chemical data to determine the significance of recharge at Leaky Acres on the surrounding regional ground-water chemical quality, we must remember several factors: (a) The domestic wells are shallow, usually 30 to 40 m deep, open-bottom, and the volume of pumped water for house and yard use is small (as compared with public supply and irrigation wells), and the

**Table 2. Ground-Water Nitrate, mg/l, Trends Beneath Leaky Acres. Monthly Mean, and for Some Months the Standard Deviation of Mean**

| *Month* | *1971* | *1972* | *1973* | *1974* | *1975* |
|---|---|---|---|---|---|
| Jan. | — | 3.5 | 6.2 ± 0.9 | 4.9 | — |
| Feb. | 29.2 ± 8.3* | 2.4 | 4.5 | 5.0 | — |
| Mar. | — | 6.6 | 3.2 | 5.1 | 4.0 ± 0.5 |
| Apr. | — | 3.2 | 3.2 | 4.2 | — |
| May | — | 3.5 | 2.8 | 2.5 | — |
| June | 15.3 ± 4.9 | 2.4 ± 0.5 | 2.3 ± 0.4 | 1.6 ± 0.1 | 1.7 ± 0.1 |
| July | 12.6 | 1.8 | 3.2 | — | — |
| Aug. | 5.3 | 3.9 | 2.5 | — | — |
| Sept. | 3.6 | 2.7 | 2.0 | 1.0 | — |
| Oct. | 3.2 ± 1.0 | 2.4 ± 0.8 | 1.7 ± 0.2 | — | 0.7 ± 0.1 |
| Nov. | 2.3 | 2.7 | 1.1 | — | — |
| Dec. | 3.5 | 8.1 | 1.2 | — | 1.2 |

* Before recharge.

3. Change in the Mean and Standard Deviation of the Mean of Selected Water-Quality Parameters for All Wells at Leaky Acres Between 1971 and 1975

| *Quality Parameter mg/l* | *June 2, 1971* | *Oct. 28, 1975* | *Percent Decrease in Mean* |
|---|---|---|---|
| $Na^+$ | 6.95 ± 1.32 | 2.77 ± 0.14 | 60.1 |
| $K^+$ | 1.98 ± 0.20 | 0.82 ± 0.02 | 58.6 |
| $Ca^{2+}$ | 5.97 ± 1.04 | 4.89 ± 0.26 | 18.1 |
| $Mg^{2+}$ | 5.90 ± 0.60 | 2.55 ± 0.09 | 56.8 |
| $HCO_3^-$ | 69.67 ± 5.23 | 36.12 ± 1.68 | 48.2 |
| $Cl^-$ | 1.85 ± 0.21 | 0.70 ± 0.04 | 62.2 |
| $NO_3^-$ | 12.6 ± 4.52 | 0.73 ± 0.09 | 94.2 |
| $SO_4^{2-}$ | 38.6 ± 2.4 | 7.6 ± 1.1 | 80.3 |
| $SiO_2$ | 48.0 ± 3.9 | 37.7 ± 3.4 | 21.4 |

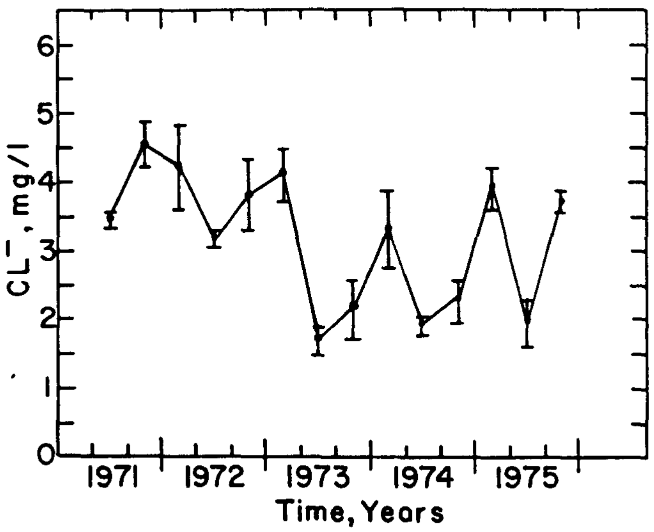

Fig. 5. Ground-water mean chloride concentration for each third of a year for peripheral well (PW) No. 7.

observed annual variability in water quality is low. For example, in 1974 periphery well PW-10 had an SEC coefficient of variability (Cv) of 2.9%. The domestic wells penetrate only the upper aquifers and can be polluted if they are in dense, septic tank areas, like PW-17, with a mean $NO_3^-$ level of 45.3 mg/l with Cv of 7.5% in 1974, as compared with domestic well PW-11 (outside the dense septic tank area), with a mean 7.6 mg $NO_3^-/l$ in 1974. (b) The urban area, which had only septic tanks, was converting to the Fresno Sewer System, and most connections were completed by the end of 1974. Thus, this pollution source will eventually be eliminated. (c) The public-supply wells are much deeper (75 to 100 m) and usually not perforated above 36 to 45 m. Fluctuations in pumped-water quality were observed in the public-supply wells in this area in response to seasonal use demand and drawdown. For example, PW-21 had a 1974 mean $NO_3^-$ level of 32.3 mg/l with Cv of 27.1% and PW-7 had Cv of 80.3% variability.

The mean ground-water chloride concentration for the PW-7 (Figure 5), shows that during the summer months of maximum pumping, lower chloride levels (or SEC) were observed. This variability, which was not the same for all public-water wells complicates the long-term evaluation of effect of Leaky Acres' recharge. Regression analysis is probably the best practical way to make long-term evaluations. The seasonal cyclic variability is not great and for long-term effects could be disregarded.

Table 4 presents the results of power curve regression analysis for ground-water SEC for wells at various distances from Leaky Acres, as a function of time. The coefficient of determination for PW-6, 7, 14, and 21, in a south to southwest direction decreased with distance from Leaky Acres. This statistical method of describing trends in ground-water quality associated with basin-type recharge seems acceptable, considering expected life of the recharge facility, which is probably less than 30 years, and the unjustified costs of a scientific ground-water hydrological and quality study.

In the past ground-water nitrate levels in the public-supply wells of this area of high septic tank

Table 4. Ground-Water SEC Time-Trends Predictions for Peripheral Wells at Various Distances and Directions from Leaky Acres Based on Power Law Curve Regression Analysis of 1971 Through 1975 Data

| | | | | | | *Estimated SEC, $\hat{y}$, µmhos/cm for Dec.* | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| *PW NO.* | *Approx. Distance m* | *Approx. Direction* | *No. of Obs.* | *Coef. of Deter., $r^2$* | *Regression Equation* $y = ax^b$* | *1971 x=12* | *1972 x=24* | *1973 x=36* | *1974 x=48* | *1975 x=60* | *1980 x=120* |
| 2 | 200 | SE | 93 | 0.835 | $\hat{y} = 356\,x^{-0.36}$ | 146 | 113 | 98 | 88 | 82 | 64 |
| 6 | 200 | S | 68 | 0.693 | $\hat{y} = 389\,x^{-0.29}$ | 189 | 155 | 138 | 126 | 118 | 97 |
| 7 | 400 | SSW | 75 | 0.592 | $\hat{y} = 346\,x^{-0.24}$ | 190 | 161 | 146 | 137 | 130 | 110 |
| 14 | 800 | SSW | 85 | 0.217 | $\hat{y} = 387\,x^{-0.064}$ | 330 | 316 | 308 | 302 | 298 | 285 |
| 21 | 1200 | SSW | 104 | 0.106 | $\hat{y} = 619\,x^{-0.13}$ | 448 | 409 | 388 | 374 | 363 | 332 |
| 11 | 800 | W | 83 | 0.404 | $\hat{y} = 246\,x^{-0.12}$ | 183 | 168 | 160 | 154 | 150 | 138 |

* $\hat{y}$ = estimated specific electrical conductivity, µmhos/cm, after x-months from January 1971 when x = 1.

Table 5. Mean Ground-Water Nitrates, mg/l, for May through August, During Maximum Pumping

| *Peripheral Well No.* | *Use* | *Approximate Distance from Basins, m* | *Nitrates, mg/l* | | | | |
|---|---|---|---|---|---|---|---|
| | | | *1971* | *1972* | *1973* | *1974* | *1975* |
| 2 | Domestic | 200 | 7.4 | 8.2 | 3.4 | 3.0 | 2.0 |
| 16 | Domestic | 200 | 4.3 | 2.3 | 2.5 | 4.1 | 2.6 |
| 8 | Domestic | 200 | 8.2 | 12.2 | 5.9 | 2.8 | 2.1 |
| 6 | Public | 200 | 14.2 | 15.1 | 9.1 | 7.4 | 6.3 |
| 4 | Irrigation | 400 | 34.1 | OFF | 30.9 | OFF | 32.8 |
| 7 | Public | 400 | 14.4 | 14.7 | 8.9 | 6.1 | 4.9 |
| 10 | Domestic | 600 | 9.9 | 12.1 | 11.0 | 10.9 | 7.5 |
| 20 | Public | 800 | 18.4 | 21.4 | 16.7 | 22.1 | 26.2 |
| 14 | Public | 800 | 30.0 | 30.1 | 31.5 | 22.8 | 35.8 |
| 17 | Domestic | 900 | 43.8 | 44.9 | 39.6 | 44.7 | 44.5 |
| 11 | Domestic | 800 | 6.0 | 7.1 | 5.0 | 7.4 | 6.6 |
| 19 | Public | 1200 | 18.7 | 23.5 | 19.9 | 28.8 | 28.4 |
| 21 | Public | 1200 | 35.1 | 36.9 | 31.2 | 33.7 | 31.9 |
| 18 | Public | 1600 | 23.4 | 25.1 | 23.3 | 27.6 | 24.5 |

density have received considerable attention. Table 5 presents summer (May through August) trends in mean ground-water nitrate levels, ranked according to approximate distance from the recharge basins. In general, only wells less than about 600 m have shown a nitrate decrease, and initially these were low in nitrates, because they were upgradient from the dense septic tank area. The nitrate levels of pumped water in the septic tank area have not greatly changed during this observation period. Unsaturated flow of poor quality water from the soil around the abandoned septic tanks probably will continue for several years because of necessary watering of lawns and gardens.

Figure 6 shows the areal distribution of the 1971-1975 percent decrease in ground-water salinity as evaluated by SEC from data for May through August (maximum pumping period). The decrease in SEC south of E. Shields Avenue is evident, but as shown in Table 5 the nitrate level apparently has not yet been noticeably reduced by recharge, probably because of the high septic tank density.

The Leaky Acres Ground-Water Recharge Facility demonstrates that ground-water quality can be improved or managed by basin-type recharge using high-quality water. Scientific evaluation of the enclave and movement of the recharged water is difficult without a specially built network of expensive observation wells and monitoring equipment. All recharge facilities should be monitored, which is possible if there are producing water wells around the facility. However, consideration must be given to their pumping capacity, delivery time schedule, depth, and zones of extraction. The historic and present land use around the recharge facility, and the stratification will affect the chemical quality of the pumped water and its seasonal variability.

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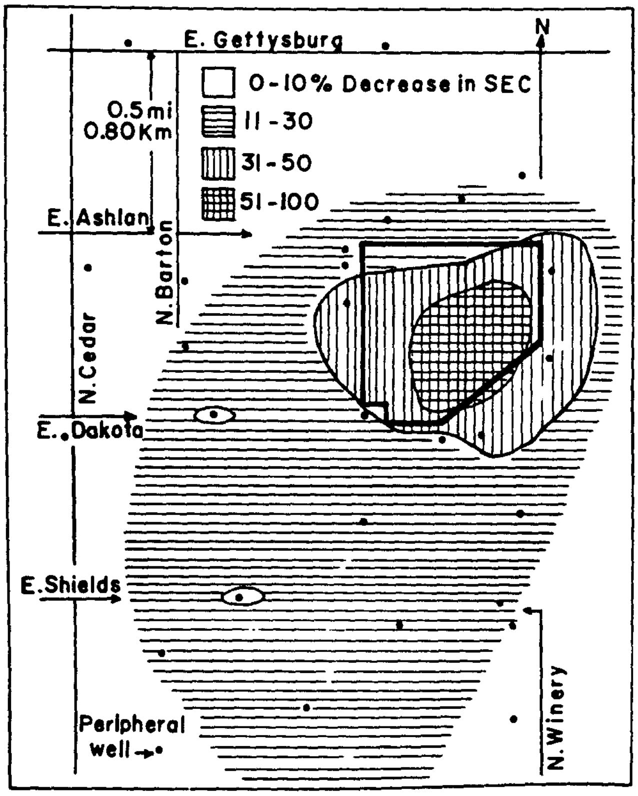

Fig. 6. Percent decrease in ground-water specific electrical conductivity (SEC) between 1971 and 1975, based on May through August data for these years.

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## DISCUSSION

**The following questions were answered by Harry I. Nightingale after delivering his talk entitled "Ground-Water Chemical Quality Management by Artificial Recharge."**

**Q. by Abe Kreitman.** *Has the recharge capacity of the basins changed with time? If so, what is the nature of the change and what is the nature of the rehabilitative procedures employed?*
**A.** During the first five recharge periods the measured average infiltration rate for all basins has not statistically changed. The basin soils have been disked once during the winter drying period to mix the accumulation of organic material into the soil. We feel that disking should not be done unless necessary to prevent surface sealing due to accumulation of organic or inorganic sediments.

**Q. by J. Brown.** *Will the amount of recharge water available for the Leaky Acres project be affected by continued dry conditions in California or are quality criteria the only reasons for discontinuing recharge?*
**A.** During the dry year of 1976, water was made available for recharge through October even after the Fresno Irrigation District had to shut off surface-water deliveries to farmers on July 15. Perhaps this would indicate a higher-use priority, especially when the farmers also have ground water available for irrigation in water-short years. For the second part of the question, the historical chemical quality of the surface water available to Fresno for recharge would indicate that its quality would probably never be a criteria for discontinuing recharge.

**Q. by Leonard Konikow.** *Would you recommend the use of a deterministic model for studying artificial recharge problems?*
**A.** Yes, but only from the point of view that you have a real need and can justify the cost. There is no question that each recharge site is unique, and quantity and quality modeling is beneficial to our understanding recharge problems, but the problem of cost is difficult to justify.

**Q. by K. E. Childs.** *(a) Specifically what was the source of the recharge water? (b) Were the actual zones receiving recharge water monitored or were the monitoring wells in a different horizon?*
**A.** (a) The source of recharge water is the Kings River about 18 miles (29 km) to the east of Leaky Acres and is delivered by the Fresno Irrigation District's Gould Canal. (b) The zones receiving recharge water were actually monitored at Leaky Acres by observation wells perforated in the lower soil zones beneath the basins. Outside the bounds of Leaky Acres, domestic, public supply, and irrigation wells were monitored and they extracted water from many

different sandy zones, which for wells further away from Leaky Acres, probably are not continuous with the zones monitored beneath the basins.

**Q. by L. A. Swain.** *Do you have any evidence that shows whether the better quality water recharged is vertically mixed throughout aquifer depth or floats on top of older ground water?*
**A.** The construction of the water wells monitored around Leaky Acres precludes the determination of vertical mixing based on chemical data of pumped water. The density difference between the native ground water and the recharged water is small.

**Q. by Mike Kaczmarek.** *In areas of short water supply where ground water is a major source of drinking water and municipal supply, where will you obtain recharge water?*
**A.** The management of the ground-water quality and quantity in a given area does demand the input of water of equal or better quality to maintain the quantity of water in storage, as well as an output of excess salt, be it to the ocean or a salt lake. The need for input by recharge should be first minimized by area-wide water conservation, especially keeping precipitation within the area, and maximum re-use of waste waters, then high quality surface water should be imported. If additional water is not economically available, then the people of the urban area should determine how they can continue to exist within their limited water supply and management capabilities.

**Q.** *What was the problem (need for recharge), and what does it cost?*
**A.** The problem in Fresno, California, is typical of urban areas in arid climates in that the water table has been falling as use has increased and septic tanks have resulted in a slow deterioration in ground-water quality in some areas. For the Leaky Acres Recharge Facility, and assuming 12,000 acre-feet per year of recharge, the total users costs are $16.98 per ac.-ft., which is composed of $4.56 per ac.-ft. for recharge operations, $10.00 per ac.-ft. for purchase of water (Bureau of Reclamation), and $2.42 per ac.-ft. for energy costs only to recover the water. For Leaky Acres there is a secondary benefit to the facility, namely benefits associated with Federal Airport clear zone land purchase contributions. If this benefit is considered, then the recharge costs are $3.53 instead of $4.56 per ac.-ft., which would reduce the total users' costs to $15.95 per ac.-ft.

# TECHNICAL DIVISION ANNOUNCEMENTS

## NWWA TECHNICAL DIVISION CALLS FOR PAPERS FOR 1977 CONVENTION PROGRAM

Technical Division Chairman Robert Minning, and Vice Chairman and Program Director Harry LeGrand, are planning the finest Technical Division Education Program NWWA has ever held. In an attempt to greatly increase Technical Division attendance at our NWWA Convention, the 1977 Program in Boston will be extended to two days, Wednesday, September 14, which is the last day of the Exposition, and Thursday, September 15.

The full line-up of Technical papers is now being sought. Subject emphasis will be placed on the following areas:

- Fractured-rock hydrology.
- Glacial and alluvial hydrology.
- Environmental, political, and legal constraints on ground-water development.
- Relation of land-use planning to ground-water hydrology.

Members wishing to deliver papers on one or more of these subjects should send a 300-500 word summary to Harry LeGrand, 331 Yadkin Drive, Raleigh, North Carolina 27609. The deadline for submission of paper summaries will be April 30. The final paper selection will be made prior to the end of May.

The Technical Division of the National Water Well Association is the professional society of ground-water hydrologists, geologists, and engineers. We hope our Technical Division members will all soon value our National Convention as their most important annual professional meeting. The opportunity to give and hear important scientific presentations on ground-water hydrology should become a common bond that will aid our efforts to enhance the development and protection of this great natural resource.

Boletín Geológico y Minero. T. LXXXVIII-III. Año 1977 (223-240)

# Utilización de las aguas residuales urbanas para riego y recarga artificial

Por J. PORRAS MARTIN (*)

**RESUMEN**

Se describen la composición y aplicaciones del agua residual urbana tratada para riego y recarga artificial de acuíferos; se tratan con especial énfasis los problemas de contaminación relacionados con dichos usos y el papel del suelo como filtro físico, químico y biológico.

**ABSTRACT**

Characteristics and composition of treated sewage and its use for irrigation and artificial recharge of aquifers are described with special emphasis on associated pollution problems; the role of soil as physical, chemical and biological filter is analyzed.

## I. INTRODUCCION.

De acuerdo con varios de los índices de aridez más conocidos, una buena parte del suelo español es considerado como zona árida o semiárida, como consecuencia de dos factores climáticos fundamentales, la precipitación anual y la temperatura. Los valores que presentan estos parámetros en la Península, especialmente en el Sur y en ciertas zonas de Levante, hacen que los recursos hídricos totales, superficiales y subterráneos en dichas zonas sean relativamente escasos, especialmente en lo que a las demandas para regadío se refiere; dado que las condiciones de suelo y temperatura en muchas de las áreas a las que nos referimos son favorables a un elevado rendimiento agrícola en régimen de regadío, los campesinos han buscado el agua afanosamente, y como consecuencia, y a falta de unos recursos superficiales suficientes, se está produciendo la sobreexplotación de las reservas en muchos de los mantos acuíferos subterráneos existentes, algunos de los cuales quedarán agotados en unos pocos años.

Sin embargo, existe una posibilidad de paliar estos problemas, aunque tal vez sólo en pequeña medida y después de minuciosos estudios. Se trata del reciclado o reutilización de las aguas residuales domésticas producidas en los núcleos urbanos, que, una vez sometidas a tratamiento conveniente, pueden utilizarse, como en seguida veremos, para regar e incluso para recargar los acuíferos subterráneos, disminuyendo así los peligros de sobreexplotación.

Del agua que el hombre consume para usos domésticos, urbanos, comerciales, etc., en una ciudad con sistema de alcantarillado extendido a toda la

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zona urbana, más de un 80 por 100 se recoge en los colectores y es normalmente considerada como un agua inutilizable, contaminada y de la que es necesario desembarazarse de la manera más expeditiva posible, con o sin un tratamiento de purificación que la haga menos perjudicial para el medio ambiente.

Sin embargo, estas aguas pueden ser, en ocasiones, superiores en calidad química a las aguas subterráneas de muchos acuíferos costeros, y en cuanto a su contenido en materia orgánica y microorganismos, el adecuado tratamiento, por una parte, y la propia acción del suelo y de los terrenos permeables, por otra, pueden conseguir una total purificación eliminando los problemas de contaminación. Por otro lado, las aguas residuales urbanas contienen una cierta cantidad de nutrientes para las plantas (compuestos de N y P) que las hacen, en este aspecto, muy adecuadas para regar. Según STEVENS (1972), ya en el año 1559 se utilizaban las aguas fecales para regar, en la Baja Silesia, y en la ciudad de Berlín se compraban, en 1869, grandes extensiones de terreno para regarlo con aguas fecales sin tratar, plan que continuaba en operación en 1935. Muchos otros ejemplos, históricos y actuales, demuestran que la utilización de las aguas residuales para regar ha sido considerada desde muy antiguo como un método de eliminación y tratamiento de las mismas que, a la vez, producía unos beneficios adicionales.

Sin embargo, el estudio científico y sistemático de las posibles aplicaciones útiles de estas aguas y el análisis de la capacidad del terreno y de las formaciones acuíferas, como elementos purificadores, es relativamente reciente. En líneas generales, el problema se suele enfocar hoy día tal vez más como eliminación de un residuo que como reciclado de un recurso hídrico que ha sufrido una degradación en su calidad; no obstante, en muchos casos, se consideran ambos aspectos al mismo tiempo, obteniéndose entonces las mayores ventajas en la operación.

En los siguientes apartados se van a estudiar brevemente los aspectos más importantes en relación con el tema, comenzando por los tratamientos a que un efluente urbano puede ser sometido, los métodos de eliminación y los problemas de contaminación que pueden plantear, para continuar analizando el papel del suelo y de los terrenos permeables como elementos purificadores, pasando después a estudiar específicamente las ventajas e inconvenientes del riego con aguas tratadas y de la recarga artificial de acuíferos. Se consideran, en cada caso, los elementos fundamentales para la adecuada gestión de ambas operaciones.

## II. CARACTERISTICAS DEL EFLUENTE URBANO. METODOS DE TRATAMIENTO.

Hemos llamado aguas residuales urbanas a las producidas en pueblos y ciudades por las operaciones domésticas (lavado, eliminación de excretas, etcétera), comerciales, de servicios (lavado de calles, escorrentía urbana, etc.) e industriales dentro de la ciudad. En ciertos casos, parte de estas aguas residuales, sobre todo domésticas, se recogen en instalaciones individuales o de reducida capacidad (pozos negros, fosas sépticas, etc.), y en otros existe un sistema de alcantarillado donde se reúnen y coleccionan todas estas aguas y se conducen hacia el punto o puntos finales de vertido, sometiéndose, en ocasiones, a diferentes grados de tratamiento, previos al vertido definitivo. Este vertido puede realizarse en: cursos de agua superficial, acequias de riego, acequias especiales de drenaje de aguas fecales (que, a su vez, vierten en alguno de los puntos mencionados), el mar mediante emisarios submarinos, estanques o lagunas de infiltración, terrenos de regadío, barrancos secos, etc.

Las aguas fecales recogidas en las alcantarillas presentan una composición muy variable de unos lugares a otros, y aun a lo largo del año e incluso del día. En general, se puede considerar que existen cuatro grupos de componentes, que son los que determinan sus peculiares características:

— Sales disueltas.
— Materia orgánica.
— Partículas y sólidos en suspensión.
— Microorganismos.

Las aguas fecales pueden someterse a diversos grados de tratamiento. El más simple de ellos, que denominaremos tratamiento primario, consiste en un desbaste de los sólidos más gruesos y una sedimentación primaria, gracias a los cuales se pueden eliminar alrededor de un 90 por 100 de los sólidos decantables y un 50 por 100 de los sólidos en suspensión. La cantidad de materia orgánica, expresada en la demanda bioquímica de oxígeno (DBO), puede reducirse en un 35 por 100, aproximadamente (STEVENS, 1972). Fósforo, nitrógeno y sales inorgánicas disueltas no sufren apenas modificaciones.

El tratamiento secundario, que siempre va precedido de un proceso primario, consiste fundamentalmente en una purificación biológica, en la que los agentes de tratamiento son microorganismos, que, en régimen aeróbico, consumen materia orgánica contenida en el agua residual. Se emplean diversos métodos para proporcionar el oxígeno necesario a esta actividad microbiana; entre los más empleados se encuentra el de *lodos activados*, en el que se insufla aire mediante difusores que crean un movimiento circulatorio en el líquido y facilitan el contacto, y el de *lechos filtrantes*, en el que se hace pasar el agua residual a través de un macizo de gravas, en el que se favorece la oxigenación y, por tanto, la descomposición aeróbica.

En condiciones óptimas de operación se puede eliminar hasta un 90 por 100 de la DBO y reducir en un 85 por 100 la cantidad de sólidos en suspensión mediante los tratamientos secundarios. Se elimina también parte del nitrógeno, pero muy poco de P y prácticamente nada de las demás sales solubles inorgánicas.

El líquido que sale de las plantas de tratamiento recibe el nombre de efluente; estudiaremos a continuación los grupos principales de sustancias que componen dichos efluentes.

*La cantidad de sales disueltas* depende de la previamente existente en las aguas de abastecimiento de la ciudad. Si en la zona que se considera no existe una gran actividad industrial, o bien si las industrias no vierten sus líquidos residuales en la red de alcantarillado, las aguas fecales pueden contener, en líneas muy generales, una cantidad total de sales disueltas que sobrepasa en 200-400 mg/l a la que existía en las aguas de abastecimiento. Dicho incremento puede ser mucho mayor (más de 1.000 mg/l) si se vierten residuos industriales; en este caso, los iones metálicos pesados son un problema potencial de contaminación a añadir a los normalmente existentes.

La American Chemical Society (1969) presenta un cuadro de las sales añadidas al agua en el uso doméstico (Tabla I).

TABLA I

*Constituyentes inorgánicos añadidos a través del empleo doméstico del agua*

| Constituyentes | Incremento medio de la concentración, mg/l |
|---|---|
| Grupo 1: | |
| Sodio | 70 |
| Potasio | 10 |
| Calcio | [illegible] |
| Magnesio | 7 |
| Cloruros | 75 |
| Bicarbonatos | 100 |
| Sulfatos | 30 |
| Sílice | 15 |
| Dureza (como $CO_3Ca$) | 70 |
| Alcalinidad (como $CO_3Ca$) | 85 |
| Grupo 2: | |
| Fosfatos | 25 |
| Amonio ($NH_4^+$) | 20 |
| Nitratos | 10 |
| Nitritos | 1 |

Es interesante indicar, de acuerdo con HUNTER y KOTALIK (1973), que prácticamente ninguno de los compuestos del Grupo 1 se elimina durante los procesos normales de tratamiento a los que se suelen someter las aguas fecales, mientras que los del Grupo 2 sufren diversas transformaciones durante el tratamiento.

En general, los fosfatos pasan a ortofosfatos y también son eliminados en parte durante el tratamiento secundario. Nitratos y nitritos se producen por la oxidación microbiana del amoníaco, y éste es debido a la hidrólisis de la urea y a la descomposición biológica de los compuestos orgánicos nitrogenados. En ocasiones, los efluentes no contienen cantidades apreciables de nitratos o nitritos, estando el N en forma orgánica o amoniacal. Parte del nitrógeno puede ser eliminado en forma de $N_2$ gas por oxidación del amonio ($NH_4^+$).

Como más tarde veremos, el N es un elemento importante a considerar, beneficioso para el uso agrícola del efluente, pero perjudicial para las aguas subterráneas cuando la concentración en $NO_3^-$ es elevada y aquéllas se usan para abastecimiento público. Se puede admitir que un efluente urbano contiene 10-35 mg/l de N, parte en forma orgánica, parte en forma amoniacal y parte en forma oxidada, como nitratos. En lo que respecta al fósforo, se encuentra normalmente en cantidades de

5-10 mg/l de P en los efluentes urbanos. Desde otro punto de vista, puede considerarse que la aportación humana al efluente se cifra en unos 8 a 10 gramos/habit./día de N, 1 gr. de P y 6 gr. de K.

*La materia orgánica* contenida en las aguas fecales es causante de olores y color en las mismas, aparte de incluir compuestos tóxicos y ser medio de vida de microorganismos, que, al consumir el oxígeno disuelto en el agua, hacen que la vida en la misma sea imposible para peces y otras especies acuáticas dependientes del $O_2$ disuelto.

La materia orgánica se presenta en forma de compuestos solubles y de partículas sólidas, tanto en el agua fecal como en el efluente. **Bunch** y colaboradores (1961) presentan el siguiente cuadro para la materia orgánica de un efluente secundario (lodos activados) en los Estados Unidos (Tabla 2).

TABLA 2

*Composición de un efluente en una planta de lodos activados* (**Bunch**, 1961)

| Constituyentes | Porcentaje del total de la DQO |
|---|---|
| Extraíbles en éter | 10 |
| Proteínas | 10 |
| Carbohidratos y polisacáridos | 5 |
| Taninos y ligninas | 5 |
| Detergentes aniónicos (MBAS) | 10 |
| Sin identificar | 65 |

Otros autores citados por **Hunter** y **Kotalik** (1973) presentan diferentes cuadros con las composiciones de diversos efluentes, donde, en general, se demuestra que alrededor del 50-70 por 100 de la materia orgánica contenida en el líquido que sale de una planta de tratamiento se encuentra en forma disuelta, el 6-9 por 100 en forma de sólidos en suspensión finos y el 25-39 por 100 en forma de sólidos gruesos en suspensión.

En cualquier caso, y desde el punto de vista de los problemas de contaminación que se pueden producir por la utilización posterior del efluente sobre el terreno, los compuestos orgánicos más peligrosos son los estables; es decir, aquellos que son resistentes a la degradación biológica. El efecto a largo plazo de estos compuestos sobre la salud humana no es conocido; entre ellos se cuentan los sulfonatos de alquil-benceno, algunos insecticidas, etcétera.

En líneas generales, se admite que la reducción de la materia orgánica, expresada en DBO, que tiene lugar en el tratamiento secundario, es de un 90 por 100; un efluente bien tratado puede presentar una DBO de 25 mg/l, con una demanda química de oxígeno (DQO) de unos 70 mg/l.

*Los microorganismos* presentes en las aguas fecales son de muy diversos tipos y proceden generalmente de la orina y heces de hombres y animales. Un cierto número de ellos son patógenos y pueden atravesar las plantas de tratamiento sin alteración, excepto cuando se procede a una desinfección final del efluente, como más adelante veremos.

Según **Foster** y **Engelbrecht** (1973), los principales organismos patógenos presentes en las aguas residuales pueden dividirse en cuatro grupos (Tabla 3).

TABLA 3

*Grupos más importantes de organismos patógenos en las aguas residuales*

1. *Bacterias*
    - Salmonella
    - Shigella
    - Mycobacterium
2. *Protozoos*
    - Entamoeba histolytica
    - Naegleria
3. *Parásitos helmintoides*
    - Ascaris
    - Ancylostoma
    - Necator
    - Taenia
    - Trichuris
4. *Virus*

De entre todos estos grupos, son las bacterias las mejor conocidas y estudiadas, y los virus, los que mayor incertidumbre plantean en cuanto a su número y, sobre todo, su eliminación mediante desinfección y retención por el terreno.

A estos organismos patógenos suele acompañar una flora mucho más abundante, algunas de cuyas especies son exclusivamente de las aguas fecales y, gracias a esta circunstancia, suelen utilizarse como indicadores de contaminación por dichas aguas.

Entre los diversos microorganismos de este tipo, los que se usan con más frecuencia son los que se incluyen en la denominación general de "Coliforme fecal", y entre ellos, los del grupo Escherichia Coli, cuyo origen es exclusivamente fecal. La aparición de una sola E. coli en 100 ml. de agua es evidencia de contaminación.

A su paso por las distintas fases del tratamiento, primario o secundario, de las aguas residuales urbanas, los microorganismos, tanto los patógenos como los inocuos, pueden sufrir una serie de cambios, aunque, en general, se acepta el hecho de que los procesos de tratamiento hasta ahora mencionados no son eficaces en la remoción de la fauna microbiana patógena. De hecho, los tratamientos secundarios fomentan el desarrollo de microorganismos aeróbicos que descompongan la materia orgánica en componentes minerales sencillos. En cualquier caso se puede concluir que algunos organismos perjudiciales sufren reducciones en su número, sobre todo en el tratamiento secundario, pero el efluente nunca está libre de los mismos.

Por estas razones, los procesos de tratamiento se suelen completar con una operación de desinfección, consistente en la mayor parte de los casos de una adición al efluente de cloro en forma de gas $Cl_2$ disuelto en agua, cloraminas, ácido hipocloroso, hipoclorito sódico o peróxido de cloro. El efecto germicida es producido por el gas cloro libre y su eficacia es mundialmente conocida. Aunque no podemos entrar en un excesivo detalle, conviene indicar que, dependiendo de una serie de circunstancias (tiempo de contacto, temperatura, pH, naturaleza de las materias orgánicas), la acción del cloro puede no eliminar el 100 por 100 de los organismos patógenos. Por otra parte, el cloro añadido al efluente puede tener efectos diversos sobre la composición química del mismo (aumento de los cloruros. formación de cloraminas y cloro-benceno, etc.).

Se admite, en general, que la inactivación de los virus es uno de los objetivos más difíciles de conseguir en la cloración, aunque una prolongación del tiempo de contacto y un incremento en la concentración pueden conseguir el efecto apetecido. En cualquier caso, y a pesar de la cloración, cabe esperar que gran parte de las plantas de tratamiento produzcan un efluente con cierta cantidad de microorganismos, algunos de ellos patógenos, cuya evolución es necesario seguir en los usos posteriores de dicho efluente, para evitar problemas de contaminación.

## III. PAPEL DEL SUELO Y DE LAS FORMACIONES ACUIFERAS.

Como indicamos anteriormente, el efluente de las plantas de tratamiento puede eliminarse de muy diversas maneras. Una de las más frecuentes es el vertido del mismo en un curso de agua superficial, donde, si tienen lugar una serie de condiciones favorables (agua previamente no contaminada, buena aireación, etc.), la materia orgánica presente en el efluente se degrada aeróbicamente, el agua residual se mezcla con agua natural, y al cabo de un cierto recorrido, el agua que circula por el río puede ser perfectamente adecuada para gran número de aplicaciones útiles. Ha tenido lugar un proceso de autodepuración. Sin embargo, cuando gran cantidad de aguas residuales urbanas e industriales, tratadas o sin tratar, se vierten en un curso de agua, este pierde su poder autodepurador, se consume el oxígeno disuelto por las bacterias aeróbicas, y el río se convierte en una alcantarilla, sin oxígeno para mantener la vida acuática.

Una alternativa a este método es la eliminación del efluente por vertido sobre el terreno, aprovechando la capacidad depuradora del mismo, haciendo que parte del agua se infiltre y llegue más o menos depurada, hasta el manto acuífero subterráneo. En esta alternativa existen tres aspectos íntimamente ligados: el efecto depurador del suelo sobre los componentes perjudiciales del efluente, la contaminación de las aguas subterráneas, y los beneficios producidos por la aplicación del efluente, bien para regar o bien para recargar deliberadamente, el manto acuífero y poder utilizar después el agua recargada.

Estudiaremos a continuación desde un punto de vista general, el efecto autodepurador del suelo y formaciones permeables para más adelante analizar los aspectos específicos del riego y la recarga artificial y las ventajas e inconvenientes (contaminación, etc.) en relación con ellos.

### III.1. EL SUELO COMO FILTRO FÍSICO, QUÍMICO Y BIOLÓGICO.

El suelo se puede definir como la parte superior de la capa de materiales no consolidados que cubre la superficie terrestre. Es una mezcla de partículas minerales, materia orgánica, aire y agua, en proporciones muy diversas. Cuando sobre el suelo se aplican efluentes urbanos tiene lugar una serie de fenómenos físicos, químicos y biológicos durante la circulación vertical del efluente a través de los poros de aquél; estudiaremos brevemente dichos fenómenos:

Entre los *fenómenos físicos* el más importante es sin duda el efecto filtrante del suelo, que se manifiesta en la retención de los sólidos en suspensión que aún quedan en el agua residual urbana tratada. Esta acción filtrante puede verse afectada negativamente cuando los poros del suelo se taponan, impidiendo la circulación del líquido. En general, esto ocurre cuando la superficie del suelo se mantiene inundada con el efluente durante bastante tiempo. En este caso se forma sobre aquélla, o como máximo, en los primeros 3 cm., una especie de capa de naturaleza a la vez física y biológica (Mc. GAUHEY y KRONE, 1967), formada a favor de las condiciones anaeróbicas que se dan después de largos períodos de inundación. La porosidad, sin embargo, puede reconstruirse en ocaciones por el establecimiento de condiciones aeróbicas, es decir, dejando secar al aire la superficie taponada (THOMAS, 1973).

En el caso de que se utilice el efluente para regar, los problemas de taponamiento parecen no haberse manifestado; es en la recarga artificial mediante estanques de infiltración donde este problema puede presentarse. En la inyección de aguas residuales urbanas en sondeos, tema al que más adelante prestaremos especial atención, este problema de colmatación o taponamiento es uno de los más difíciles de resolver, dada la imposibilidad de proporcionar la aireacción suficiente.

Entre los *efectos químicos* que tienen lugar en el suelo cuando se aplica al mismo un efluente urbano, sea por inundación o por irrigación, los más importantes pueden resumirse en tres grupos: a) Cambio iónico, b) Adsorción y precipitación, c) Alteración química.

Se define cambio iónico como el proceso reversible por el cual se produce un intercambio de aniones y cationes entre la fase sólida y la fase líquida, o bien entre dos fases sólidas en íntimo contacto. En el suelo y en los terrenos permeables, el intercambio de cationes es el fenómeno predominante; de hecho, en agricultura cobra una especial importancia la sustitución del $Ca^{++}$ del suelo por $Na^+$ contenido en las aguas de irrigación. El índice conocido por SAR (Sodium Adsorption Ratio), que se obtiene (mequ./litro).

$$SAR = \frac{Na^+}{\sqrt{\dfrac{Ca^{++} + Mg^{++}}{2}}}$$

es una medida del peligro de salinización del suelo: una excesiva cantidad de $Na^+$ produce un efecto de dispersión e impermeabilización del suelo muy perjudicial para las plantas. En general, los efluentes normales no transportan elevadas cantidades de Na como para producir graves problemas de este tipo al aplicarse para irrigación; sin embargo, todo depende de la composición inicial de las aguas de abastecimiento de la zona urbana de la que se recoge el efluente y de la cantidad de sales añadidas durante el uso del agua.

La adsorción y la precipitación son fenómenos distintos, pero con la suficiente similitud como para poder ser considerados al mismo tiempo. Adsorción es la retención, en una fina capa, de moléculas de gas, sustancias disueltas o líquidos, por la superficie de cuerpos sólidos en contacto con ellos. Precipitación es la producción, por reacción química, de un compuesto poco soluble que cristaliza. El efecto final de ambos procesos es la eliminación de ciertos compuestos del líquido que circula a través del suelo. Especial interés en el caso de los efluentes urbanos aplicados al terreno presentan los fenómenos de adsorción de gases, fosfatos, sulfatos y cationes metálicos pesados. Según BOWER (1974), el suelo posee una tremenda capacidad para fijar y almacenar fosfatos, que, generalmente no llegan hasta el manto acuífero; aún en gravas y arenas gruesas, y si el efluente es de naturaleza alcalina, gran parte del *P* se elimina del mismo, sobre todo si hay suficiente Ca como para formar precipitados insolubles de fosfato cálcico. En lo que respecta a los metales pesados (Zu, Cu, Fe, Mn, etc.) que pueden encontrarse en algunos efluentes procedentes de zonas industriales, la información sobre la posible adsorción de los mismos por el terreno no es conclusiva, aunque varios autores describen casos de adsorción parcial de algunos de ellos, citados en ELLIS (1973).

Por lo que respecta a las alteraciones químicas que sufren los efluentes en el suelo, conviene no olvidar que muchas de ellas son consecuencia de una actividad biológica. Especialmente los compuestos orgánicos son descompuestos, bien en condiciones aeróbicas, con aporte de oxígeno, o bien anaeróbicas. Las reacciones aeróbicas pueden sintetizarse: (California D. W. R., 1969).

Compuestos orgánicos (C, H, O, N, S)$+O_2\rightarrow$

$\rightarrow CO_2 + H_2O + N_2$ +nitratos+sulfatos.

En realidad, el nitrógeno orgánico se transforma primero en $NH_3$ o en $NH_4^+$, que rápidamente se oxida bien a $N_2$ o a nitratos, que al ser solubles pasan al líquido percolante y pueden representar un problema para la calidad de las aguas subterráneas.

La descomposición anaeróbica tiene lugar cuando no existe oxígeno suficiente y entran en juego las bacterias anaeróbicas. Las reacciones se pueden resumir:

Compuestos orgánicos:

$$(C.\,H.\,O.\,N.\,S) - O_2 \longrightarrow CO_2\uparrow + CH_4\uparrow + H_2\uparrow + NH_4^+ + SH_2\uparrow$$

Estas reacciones son productoras de los gases malolientes, normalmente asociados a los efluentes urbanos.

Entre otros efectos bioquímicos importantes desde el punto de vista de la contaminación de los mantos acuíferos se encuentra el de la desnitrificación, por el cual ciertas bacterias usan el oxígeno de los nitratos para descomponer la materia orgánica, formándose gas $N_2$ u óxido nitroso, que se pierden en la atmósfera. Este proceso requiere la presencia de nitratos y de carbono orgánico, en condiciones anaeróbicas.

Veamos ahora el efecto que el suelo ejerce sobre los *microorganismos* (algunos de ellos patógenos) que los efluentes urbanos arrastran cuando la desinfección final no ha sido eficaz o no se ha llevado a cabo en absoluto. Las reacciones iniciales del suelo frente a los organismos presentes en el efluente son la retención de los de mayor tamaño en forma semejante a un filtro, y la adsorción por las partículas del suelo, que tiene lugar fundamentalmente en la superficie del mismo. Después, y aunque los procesos no son bien conocidos dada la imposibilidad de observación y medidas directas, existen una serie de factores que eliminan, al cabo de recorridos más o menos largos en la zona no saturada o ya en el manto acuífero, a los microorganismos en plazos más o menos largos de tiempo. Entre estos factores se cuenta la competencia por los elementos nutrientes entre los microorganismos que habitan normalmente en el suelo, la producción de toxinas microbicidas, la lisis enzimática, la existencia de bacterias parásitas o de protozoos predadores, etc.

En general, las bacterias y los protozoos son los organismos más fácilmente retenidos por el suelo y las formaciones permeables, mientras que los virus parece ser que pueden recorrer distancias más largas, aunque a la larga también quedan detenidos. Estas premisas son aplicables a suelos y a formaciones permeables no consolidadas (arenas, gravas, limos, etc.), pero no se cumplen en formaciones con fisuras o en zonas karstificadas (calizas, dolomías, etc.), donde los microorganismos pueden recorrer grandes distancias rápidamente sin sufrir alteración.

Romero (1970) sintetiza, de acuerdo con diversos estudios del movimiento de los microorganismos fecales en el suelo y subsuelo, las más importantes conclusiones relativas al tema; entre ellas:

— Para cualquier grado de poder depurador del suelo o de la formaciones acuíferas, la eliminación de microorganismos depende sólo de la distancia, y no del número de ellos existente en el efluente que se infiltra o del volumen de éste.

Los materiales permeables más adecuados para esta eliminación son los de grano fino (arenas, limos) con apreciables contenidos en arcillas.

— En un sistema ideal de este tipo, la máxima distancia que recorren los microorganismos antes de morir o ser absorbidos por el terreno oscila entre 20 y 30 m.

— En la zona no saturada, los microorganismos quedan retenidos con mayor facilidad que cuando se incorporan al manto acuífero.

— La naturaleza del suelo sobre el que se aplica el efluente juega un papel primordial en el subsiguiente movimiento de las bacterias.

— Las distancias recorridas y la pervivencia de bacterias y virus en las aguas subterráneas puede ser mayor si éstas van cargadas de elementos nutrientes.

En base a estos estudios, el mismo autor presenta diversos cuadros con las distancias de seguridad recomendables a mantener entre los sondeos de abastecimiento y las diversas fuentes de contaminación biológica. Este aspecto, así como el de la contaminación de los vegetales que se consumen crudos y son regados con efluentes urbanos, será discutido más adelante.

## IV. EL RIEGO CON EFLUENTES URBANOS.

Desde el punto de vista del agricultor, el agua que sale de una planta de tratamiento de aguas residuales urbanas contiene, en condiciones normales, unos 200-400 mg/l más de sales disueltas que el agua de abastecimiento de la zona urbana. Como es de escerar que el TSD (total de sales disueltas) de ésta sea relativamente bajo (el Código Alimentario considera 750 mg/l como límite conveniente y 1.500 mg/l como límite tolerable), se puede decir que, salvo en casos especiales, la cantidad total de sales que lleva el efluente no va a presentar graves problemas de tolerancia por parte de las plantas. Las cantidades de nitrógeno presentes en el efluente oscilan entre 10 y 35 mg/l, presentándose éste en forma orgánica, amoniacal y como nitratos. Las cantidades de fósforo, generalmente como fosfatos u ortofosfatos, oscilan entre 5 y 10 mg/l de P. La presencia de estos elementos en el efluente es una ventaja, desde el punto de vista del aporte de fertilizantes a las plantas. Si se ha procedido a una desinfección para eliminar gérmenes patógenos, los niveles de $Cl_2$ residual en el efluente pueden oscilar entre 0 y 2-3 mg/l, lo que no suele presentar problemas para el regadío. En definitiva, la calidad del efluente suele ser adecuada para regar y, desde este punto de vista, no se plantean problemas, excepto cuando al sistema de alcantarillado se vierten aguas residuales industriales con elevadas cargas de sales o elementos tóxicos para las plantas. Si ocurre esto, cada problema es un caso particular, que debería ser estudiado por separado. De todas formas, debería prohibirse a ciertas industrias (y de hecho, se hace) verter directamente sus residuos en la red de alcantarillado general.

Considerada la calidad del efluente, vamos a contemplar ahora los aspectos que influyen en su utilización para regar. Fundamentalmente, el problema consiste en regar con la mayor cantidad posible de efluente, de forma que éste no cause problemas en superficie (encharcamiento, etc.) ni en profundidad (contaminación), obteniéndose el mayor beneficio agrícola (incremento en la producción) posible. Estos aspectos están en relación muy esrecha, pero vamos a tratar de desarrollarlos desde dos puntos de vista: el del beneficio agrícola y el de la contaminación de las aguas subterráneas. Será inevitable la repetición de algunos conceptos, pero el contraste entre ambos modos de ver el problema es altamente formativo.

En primer lugar, hay que considerar que el éxito y eficacia de cualquier operación de riego con efluentes urbanos se basa en la correcta planificación de la misma; si lo que se pretende es eliminar la mayor cantidad posible de efluente, es necesario emplear las dotaciones más elevadas que no produzcan problemas de contaminación. Si lo que se busca es obtener el máximo beneficio de los nutrientes que existen en el efluente, la aplicación del mismo al terreno debe satisfacer tanto el consumo neto por evapotraspiración de las plantas como las necesidades de estas en nutrientes (N y P).

En el diseño de una operación de esta clase se pueden destacar, entre otros, como factores fundamentales:

a) El volumen y cadencia de producción del efluente.

b) La existencia a distancias razonables de zonas regables.

c) El tipo de suelo y las condiciones topográficas e hidrogeológicas.

d) Los factores climáticos, precipitación, temperatura, etc.

e) El tipo de cosechas.

Para fijar ideas, puede considerarse que en una zona urbana bien abastecida, el efluente producido por unas 100 personas sería suficiente para regar 1 Ha de terreno, si aquél se produjese al ritmo requerido por las plantas. Sin embargo, siempre existen discrepancias entre el ritmo de producción de efluente en la planta de tratamiento y las necesidades de agua y nutrientes por parte de las cosechas que se riegan con los mismos. Esto puede traducirse en un exceso de efluente en invierno o un déficit en verano, o ambos a la vez, en cuyo caso hay que buscar recursos de agua suplementarios en verano, y en invierno tener previsto un método alternativo de eliminación (al río, al mar), o bien aplicar al terreno cantidades muy superiores a las normalmente necesarias, dando lugar a elevados índices de percolación hacia el manto acuífero (si es que existe) o bien al encharcamiento del terreno con los correspondientes problemas de drenaje. Si gran parte del efluente percola y si la superficie piezométrica del manto acuífero libre se encuentra a poca profundidad, el efecto purificador del suelo y de la zona no saturada puede quedar muy reducido y las aguas subterráneas pueden recibir cantidades apreciables de contaminantes, tales como materia orgánica y microorganismos.

Un caso favorable puede darse en una zona semiárida de atractivo turístico, donde las mayores producciones de efluente se suelen dar en los períodos secos, que es cuando las plantas necesitan mayor aporte de humedad.

Por otra parte, si el efluente se aplica de forma que la evapotranspiración de las plantas y la retención en el suelo lo consuman casi completamente, se presentan los clásicos problemas agrícolas de regadío con alta eficiencia, es decir, concentración excesiva de sales en el suelo y necesidad de lixiviarlas para regenerar las propiedades de aquél.

Por lo tanto, en estas operaciones es necesario mantener un equilibrio entre el volumen aplicado, la cantidad de N y P que las plantas toman del efluente, el flujo de retorno y la capacidad del terreno para purificar y retener los elementos nocivos para la calidad de las aguas subterráneas. Trataremos algunos de estos aspectos más adelante, al hablar de la contaminación.

En lo que respecta al tipo de terrenos más adecuado, **Parizek** (1973) afirma que el suelo natural ya existente, o bien materiales no consolidados de grano fino (arenas o limos) son los más adecuados para ser regados con àguas residuales tratadas, mientras que los materiales bien drenados, como por ejemplo los suelos producidos por alteración mecánica de rocas fisuradas, son menos adecuados, dadas las elevadas capacidades de infiltración, que no favorecen la retención y purificación del efluente. En suelos relativamene poco permeables, pueden darse casos de encharcamiento: sin embargo, es en ellos donde se produce una mayor eliminación del N, que, como más adelante veremos, es uno de los elementos que pueden contaminar las aguas subterráneas.

Dado que la mayor parte de los procesos de modifación del efluente, de los que hablamos en el apartado 3, tienen lugar en el primer metro de espesor del suelo o poco más, sería muy conveniente que en las zonas regadas, aquél tuviese dicho espesor como mínimo, para así poder asegurar un mejor tratamiento natural del efluente que percola.

En lo concerniente a las características hidrogeológicas, las más importantes son el tipo de formación permeable y la profundidad de la superficie piezométrica. Respecto al primer punto cabe repetir lo expuesto anteriormente; si la percolación tiene lugar lentamente (terrenos de permeabilidad media o baja) la acción biológica y de retención de gérmenes patógenos, así como la capacidad de las plantas para tomar N y P se ven muy favorecidas, proporcionando un mejor tratamiento al efluente què percola. Si la permeabilidad es elevada (rocas fisuradas o karstificadas) sólo cabe contar con la dilución y la dispersión para aminorar la contaminación que va a sufrir el acuífero. La profundidad a que se encuentra la superficie libre del manto es importante en relación con la capacidad purificadora del terreno. Conviene tener presente, además, que dicha superficie fluctúa a lo largo del tiempo en una misma zona, y que su profundidad puede modificarse mediante el bombeo en pozos o sondeos. En general, cuanto más profunda se encuentre, menores serán los problemas de contaminación y de drenaje, aunque, en ocasiones, se pueden admitir niveles próximos a la superficie (2 ó 3 metros) si el suelo y las cantidades aplicadas de efluentes son adecuados.

Si se considera el riego con efluentes urbanos desde el punto de vista de la posible contaminación, hay que considerar dos aspectos distintos: en primer lugar, existe la posibilidad de que algunos microorganismos queden retenidos en la superficie de ciertos vegetales, que luego se van a consumir crudos (lechugas, tomates, etc.), pudiendo producir enfermedades en el hombre o en los animales. En segundo lugar, los efluentes, al percolar hacia el subsuelo, pueden alcanzar el manto acuífero, incorporándose al mismo y contaminando el agua subterránea que se bombea de dicho manto.

En el primer caso, varios autores citados por **Foster** y **Engelbrecht** (1973) ofrecen los resultados de diversos experimentos: **Rudolfs** (1951) observó que la supervivencia de Salmonella y Shigella sobre tomates regados con efluentes que contenían dichos organismos no era superior a los siete días; sin embargo, los bacilos M. Tuberculosum sobrevivieron durante tres meses en una cosecha de rábanos (**Musehold**, 1900). En general, se admite que pueden existir problemas de contaminación por gérmenes patógenos de los vegetales que se consumen crudos, cuando éstos se han regado con efluentes dentro de los dos o tres meses anteriores a la recolección, por ello se recomienda dar los últimos riegos con aguas normales.

La contaminación de las aguas subterráneas debida al riego con efluentes urbanos puede producirse por la incorporación al flujo subterráneo de diversos agentes, tales como el total de sales disueltas, el N (generalmente, en forma de nitratos), los microorganismos, metales pesados y compuestos orgánicos no degradables. Estudiaremos cada uno de ellos:

*Total de sales disueltas.*

En el riego con cualquier tipo de agua (no sólo aguas residuales), la fracción de ésta que no es consumida por las plantas ni retenida en el suelo, percola hacia el manto acuífero, y en ella van concentradas prácticamente todas las sales que inicialmente contenía. Esto tiene como consecuencia la llegada al acuífero de un agua (retorno en el regadío) muy concentrada en sales, y, en consecuencia, la calidad general del agua subterránea puede irse degradando lentamente. Este problema es especialmente importante en zonas áridas, donde, además, el agua subterránea vuelve a bombearse, tanto para abastecer poblaciones como para regar, reciclándose y cargándose paulatinamente de sales.

Con todo, este problema no es exclusivo del riego con efluentes, y nos atrevemos a decir que es el menos importante de los que potencialmente pueden darse por esta causa.

*Nitrógeno y fósforo.*

Como ya hemos indicado en anteriores apartados, el N se encuentra en el efluente en forma orgánica, amoniacal y oxidada (nitratos). En el suelo, y debido a los procesos bioquímicos ya mencionados, una fracción apreciable se transforma en nitratos. Por una parte, las plantas absorben el N en esta forma, y por otra, los iones $NO_3$ son muy móviles, y el agua que percola los arrastra fácilmente, en disolución, hacia el manto acuífero. Prácticamente, todo el N que, a la larga, llega al manto acuífero, lo hace en forma de nitratos, ya que los compuestos orgánicos y amoniacales, bien quedan retenidos (adsorción o intercambio iónico) o bien se oxidan a nitritos, primero, y a nitratos, después. Los nitritos son relativamente inestables y pasan rápidamente a nitratos. Las aguas subterráneas pueden contener, en condiciones naturales, hasta 10 mg/l de $NO_3^-$, y cualquier cantidad mayor indica una contaminación, bien por la eliminación de aguas o productos residuales, o bien por el empleo de fertilizantes nitrogenados en las zonas donde el acuífero se recarga. El Código Alimentario Español considera como límite para aguas potables un máximo de 30 mg/l en $NO_3^-$, viéndose este límite excedido en muchas zonas agrícolas españolas. Un exceso de $NO_3^-$ en el agua de bebida puede producir en los recién nacidos una metahemoglobinemia; la hemoglobina de la sangre se transforma en metahemoglobina, que no es eficaz en el transporte de oxígeno, y la piel de los niños adquiere un tono azulado. El límite clínico se encuentra alrededor de los 90 mg/l de ion $NO_3^-$; sin embargo, la resistencia individual parece ser muy variable.

En el regadío con efluentes urbanos, el nitrógeno y sus compuestos han de vigilarse atentamente, ya que mediante una adecuada planificación es posible lograr que las plantas eliminen gran parte del mismo, consiguiéndose, al mismo tiempo, una mejor fertilización y la eliminación de un problema de contaminación. De hecho, una de las grandes ventajas del riego estriba precisamente en que las plantas, retirando N y P, purifican eficaz y beneficiosamente el efluente. Según Bower (1971), la cantidad de N aplicada al regar con un efluente urbano al ritmo de una pulgada por semana durante seis meses (equivalente a unos 6.250 $m^3$/Ha/año), puede ser de unas 100 a 200 libras por acre (112 a 234 Kg/Ha/año) y teniendo en cuenta que las cosechas pueden eliminar desde 60 Kg/Ha (trigo) hasta 600 Kg/Ha (césped, frecuentemente cortado), puede comprobarse que la capacidad de purificación es muy elevada.

En diversos estudios prácticos y experimentales de riego con efluentes tratados, se han observado índices de remoción del N por las plantas de hasta un 60 por 100 respecto al N presente en el efluente (Parizek y Myers, 1968).

Por otra parte, cuando se aplican cantidades grandes de efluente sobre el terreno, tales que las plantas sólo absorben pequeñas proporciones de N, sería interesante poder favorecer la desnitrificación, que, como vimos en el apartado 3, requiere condiciones anaeróbicas y la presencia de carbono orgánico como alimento de las bacterias desnitrificantes. Este fenómeno es importante, como veremos más adelante, en los casos de recarga artificial por estanques; en el regadío normal, las condiciones el suelo y zona no saturada suelen ser aeróbicas, y apenas tiene lugar la desnitrificación.

Otro aspecto interesante en relación con el nitrógeno es que cantidades excesivas del mismo aplicadas a ciertas cosechas, puede tener efectos adversos sobre las mismas; por ejemplo, en ciertas frutas se reduce el tamaño y la producción; en las patatas puede disminuir el contenido en almidón, etcétera. Esto debe tenerse en cuenta al aplicar fertilizantes nitrogenados al terreno. El riego con efluentes urbanos tratados, hace innecesario, en gran parte de los casos, la adición de fertilizantes de este tipo.

En lo que al fósforo se refiere, los problemas de contaminación son mucho menos importantes, debido a que los fosfatos son absorbidos, fijados en el suelo con gran facilidad, o bien precipitan en medio alcalino y en presencia de Ca; los casos de presencia de fósforo procedente de aguas residuales o de fertilizantes, en las aguas subterráneas son muy raros. Por eso, el P no se suele considerar como un problema grave en lo que a contaminación de aguas subterráneas se refiere.

*Microorganismos.*

Tal y como se expuso en el apartado 3, el movimiento de los microorganismos en el suelo y subsuelo se ve afectado por una serie de fenómenos de retención, adsorción, competencia, etc., que hacen que el camino que pueden recorrer aquéllos en las aguas subterráneas, sea relativamente reducido; por ello es de especial importancia tener bien controlada la situación, distancia de los sondeos de abastecimiento urbano, respecto a la zona de regadío, y la dirección del flujo subterráneo, ya que bacterias y virus nunca se mueven en dirección contraria a éste. En general, se admite que las capas superiores del suelo vegetal son las más eficaces en la retención de los organismos patógenos que puedan llevar los efluentes urbanos, y que únicamente en el caso de regar suelos de muy poco espesor, situados inmediatamente sobre rocas fisuradas, pueden los microorganismos recorrer distancias superiores. En general, para suelos y formaciones de grano fino puede considerarse que la distancia de seguridad es de unos 100 metros, mientras que no existen estudios sistemáticos de las distancias recorridas en formaciones fisuradas, por lo cual es necesario extremar, en este caso, las precauciones. La mayor seguridad se obtiene, en cualquier caso, mediante una adecuada y bien vigilada operación de cloración (desinfección) del efluente antes de ser usado para regar. ROMERO (1970) sintetiza en dos cuadros (figs. 1 y 2) las distancias recorridas por micoorganismos en casos reales y los límites de seguridad.

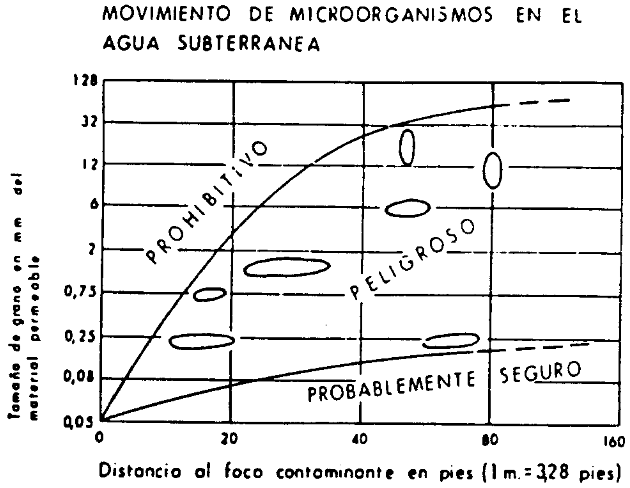
MOVIMIENTO DE MICROORGANISMOS EN EL AGUA SUBTERRANEA

Figura 1

Movimiento de la contaminación biológica en las aguas subterráneas. Distancias máximas descritas en casos reales (Romero, 1970)

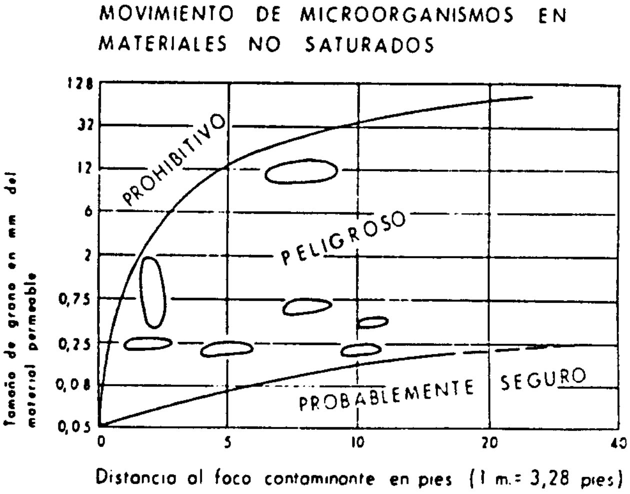
MOVIMIENTO DE MICROORGANISMOS EN MATERIALES NO SATURADOS

Figura 2

Movimiento de la contaminación biológica en materiales no saturados (Romero (1970)

*Metales pesados.*

En ocasiones, los efluentes contienen ciertas cantidades de iones de metales pesados, que pueden representar un peligro para la calidad de las aguas subterráneas. En general, la información sobre los cambios que sufren estos elementos, tales como Zn, Cu, Re, Mn, Ni, Cr, Pb, etc., es muy escasa. Parte de los iones parece ser que son absorbidos por el suelo y otros quedan atrapados gracias a un intercambio iónico, el Zn, entre ellos.

*Materia orgánica.*

En el suelo y zona no saturada, la materia orgánica presente en el efluente con el cual se riega, sufre una serie de transformaciones químicas promovidas por las bacterias aeróbicas del terreno.

El resultado, como se explicó en el apartado 3, es la transformación de dichas sustancias en sales minerales (sulfatos, nitratos, etc.), gases ($CO_2$, $N_2$, etc.) y agua. Si el efluente alcanza el manto acuífero transportando aún materia orgánica sin degradar, tienen lugar después de cierto tiempo, otra serie de reacciones producidas por bacterias anaeróbicas, que también transforman la materia orgánica en sales minerales y gases ($CH_4$, $SH_2$, etc.). Estos procesos son más lentos que los anteriores, y por ello, si hay pozos de abastecimiento cerca de la zona de regadío, puede que en alguno de ellos el agua presente color y olores desagradables. Sin embargo, el problema de la materia orgánica biodegradable es menos importante a largo plazo que el de aquellas sustancias resistentes, que no se descomponen bajo la acción microbiana y cuyos efectos sobre la salud de hombres y animales a largo plazo, no son conocidos. Un ejemplo típico es el de los antiguos detergentes no biodegradables, entre cuyos componentes se encontraban los sulfonatos de alquil-benceno. Hoy día, las disposiciones legales, en España concretamente, exigen el uso de detergentes biodegradables. También ciertos insecticidas se pueden incluir en el grupo de compuestos orgánicos estables potencialmente contaminantes.

El estudio de estos compuestos es especialmente importante cuando las aguas residuales se emplean para recargar deliberadamente un acuífero cuyas aguas se vayan a usar para el consumo humano, como veremos en el apartado siguiente.

## V. RECARGA ARTIFICIAL.

Cuando de un manto acuífero subterráneo se bombea agua a un ritmo superior al de la recarga natural del mismo, se produce el consiguiente desequilibrio, y los niveles del agua en los sondeos y pozos descienden sin recuperarse año tras año en magnitudes que dependen del grado de desequilibrio (diferencia entre los volúmenes bombeados y la recarga natural) y de las características de almacenamiento de la roca permeable. En algunas zonas del Sureste español, este ritmo de descenso en los niveles llega a alcanzar en ocasiones hasta los 10 metros por año.

Una forma de paliar este problema y aprovechar al máximo la capacidad de almacenamiento de las estructuras subterráneas permeables, es la recarga artificial de las mismas con aguas de procedencia exterior al sistema, o con aguas que saldrían del mismo sin tener un aprovechamiento adecuado.

En ocasiones, y en mantos acuíferos no sobreexplotados, se dispone en invierno de ciertos excedentes de aguas superficiales, que normalmente se perderían en el mar sin utilización, y que, recargándolas en la formación permeable, podrían permitir un aumento de las disponibilidades globales de agua, aprovechando la capacidad de almacenamiento del volumen no saturado de los poros o fisuras de la formación acuífera.

También se puede utilizar la recarga artificial para proporcionar a aguas de superficie o a aguas residuales tratadas, un tratamiento natural adicional que elimine sólidos en suspensión y microorganismos, obteniendo al mismo tiempo las ventajas de un mejor aprovechamiento de los recursos.

La recarga artificial, por fin, puede utilizarse como método de lucha contra la contaminación por intrusión de aguas saladas en los acuíferos costeros.

Los métodos para recargar agua en una formación permeable son de dos tipos, generalmente: balsas o estanques de infiltración y pozos o sondeos; en los primeros el agua a recargar llena los estanques y se infiltra a través del fondo y de las paredes de aquellos percolando a través de la zona no saturada hasta alcanzar la superficie libre del manto acuífero. En los segundos, el agua de recarga se hace llegar directamente al manto acuífero a través de la perforación que lo atraviesa. Como se comprende, el primer método sólo es posible en acuíferos libres en los que la formación permeable tenga continuidad hasta la superficie del terreno y la permeabilidad vertical sea aceptable. Los sondeos se emplean en acuíferos cautivos o cuando la permeabilidad vertical se ve interrumpida por razones geológicas, y también cuando la disponibilidad del terreno donde implantar las balsas de infiltración es muy reducida.

Las aguas de recarga pueden ser de muy diversas procedencias: aguas superficiales en régimen normal, aguas torrenciales, aguas residuales procedentes de plantas de tratamiento, etc.

En el presente estudio vamos a concretar la atención en el caso específico de las aguas procedentes de las plantas de tratamiento secundario de los residuos líquidos de tipo urbano y doméstico; no insistiremos en la composición de dichas aguas puesto que se ha analizado en apartados precedentes.

## V.1. RECARGA MEDIANTE ESTANQUES DE INFILTRACIÓN.

Se trata del método más sencillo para recargar acuíferos libres y el método más adecuado para proporcionar un tratamiento terciario al efluente urbano cuando la formación es de tipo granular. Se considera que la permeabilidad vertical del material debe ser superior a unos $10^{-4}$ m/s para que la operación tenga éxito. En los estanques de infiltración, cuya superficie puede ser desde unos cientos de metros cuadrados hasta varias hectáreas, se pueden conseguir volúmenes de infiltración de hasta unos 5 $m^3$/día por $m^2$ de superficie, aunque los ritmos más corrientes se encuentran por debajo del metro cúbico por día y metro cuadrado de superficie. La infiltración tiene lugar no sólo por el fondo, sino también por los lados del estanque (fig. 3). Si la superficie libre del manto se encuentra a cierta profundidad, el agua infiltrada sufre en la zona no saturada la serie de transformaciones físico-químicas y biológicas de las que ya hemos tratado en el apartado 3. La superficie libre del acuífero se eleva en la zona bajo el estanque al recibir el agua infiltrada, y en ocasiones llega a alcanzar el fondo del estanque, en cuyo caso la capacidad de infiltración se reduce considerablemente, y las ventajas proporcionadas por la circulación en la zona no saturada se pierden. En el caso de recarga con agua residual urbana tratada, debería tratar de mantenerse el nivel piezométrico del agua subterránea por lo menos dos o tres metros por debajo del fondo del estanque de infiltración.

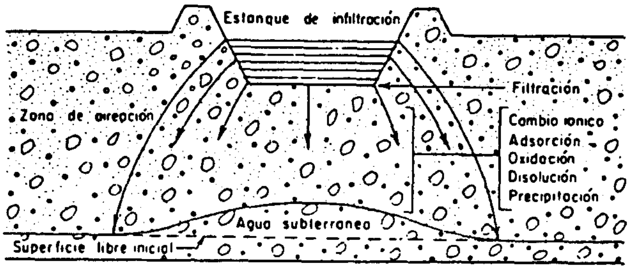

Figura 3

Recarga por estanque

Con el tiempo se va formando en el fondo del estanque una capa colmatante de naturaleza físico-biológica, que va reduciendo el caudal infiltrado hasta casi anularlo. Esta capa se puede eliminar mediante su exposición al aire, es decir, vaciando el estanque y no utilizándolo durante varias semanas; la capa se oxida y el fondo vuelve a recuperar su permeabilidad. Al cabo de cierto tiempo, es necesario remover el fondo, ya que la permeabilidad no se recupera al 100 por 100 con la exposición al aire. En ocasiones, el fondo se cubre de un filtro artificial de grava que favorece la acción biológica y química y que retiene materias en suspensión. Dicho filtro ha de ser también regenerado con cierta frecuencia.

Cuando se recarga un efluente urbano por este procedimiento, prácticamente todo el N presente en aquel puede pasar al acuífero, ya que las plantas no lo eliminan. Para reducir al mínimo este aporte es necesario estimular el fenómeno de la desnitrificación. Ciertas bacterias, en presencia de carbono orgánico y condiciones anaeróbicas, reducen el nitrato a gas $N_2$ y a óxido nitroso, que eventualmente se pierden hacia la atmósfera. Por lo menos 0,7 mg. de carbono orgánico se requieren para reducir 1 mg. de N en forma de nitrato (JOSEPH y LIDDELL, 1976).

Por lo tanto, para favorecer esta descomposición es necesario reunir carbono orgánico y nitratos en condiciones anaeróbicas. Si en un estanque se aplica efluente en pequeñas cantidades y con frecuencia, el suelo (zona no saturada) mantendrá unas condiciones principalmente aeróbicas, y prácticamente todo el N amoniacal presente en el efluente pasará a $NO_3^-$. Sin embargo, cuando el estanque se mantiene lleno durante períodos relativamente largos, el oxígeno se gasta y el $NH_4^+$ ya no se oxida a nitrato, siendo absorbido por las arcillas y materia orgánica del suelo. Los nitratos sufren una desnitrificación, y como consecuencia de ambos procesos, el agua que llega al manto acuífero lleva menos cantidad de N. Sin embargo, las arcillas pueden verse saturadas de $NH_4^+$, si el estanque se mantiene lleno durante mucho tiempo, y por ello conviene dejarlo secar antes de que esto ocurra. Con el oxígeno aportado por la aireación el $NH_4^+$ adsorbido se oxida a $NO_3^-$, y en la siguiente inundación del estanque es arrastrado junto con el $NO_3^-$ no desnitrificado. Esto produce una "punta" temporal en la concentración en $NO_3^-$ del agua que se infiltra. En cualquier caso, estas reacciones son muy dependientes del tipo de condiciones hidrogeológicas del terreno y de la composición del efluente, por lo que la descripción precedente ha de considerarse más como una posibilidad que como una regla general.

## V.2. RECARGA MEDIANTE SONDEOS.

Cuando la permeabilidad vertical de los materiales no permite una recarga mediante estanques, se puede recurrir a la recarga por sondeos, donde se aprovecha la permeabilidad horizontal del material acuífero (fig. 4). El agua de recarga puede inyectarse a presión o bien utilizando la altura proporcionada por la superficie del terreno sobre la superficie piezométrica inicial del manto acuífero. El sondeo de recarga tiene la ventaja de que puede ser utilizado al mismo tiempo para recargar en invierno y para bombear el agua recargada en verano. Sin embargo, cuando se trata de inyectar efluentes urbanos tratados, los problemas en la inyección pueden ser insalvables. Las cantidades apreciables de sólidos en suspensión que arrastran los efluentes de las plantas de tratamiento secundario, la materia orgánica y los microorganismos, son agentes que suelen producir la colmatación de las rejillas del sondeo, las cuales no pueden ser aireadas fácilmente para limpiar como en el caso de los estanques. Incluso en la recarga de aguas de buena calidad se produce la colmatación de las rejillas (REBHUN y SCHWARZ, 1968). Sin embargo, si la formación permeable es fisurada, estos problemas pueden verse reducidos, al menos durante cierto tiempo. La recarga directa de un efluente en la zona saturada del acuífero. no permite varios de los procesos purificadores mencionados en el apartado 3, y por ello es menos recomendable, como método de eliminación, y de cara a la posible contaminación bacteriológica, que los estanques de infiltración o el riego; no obstante, en el seno del acuífero, y después de un cierto recorrido, se produce también una cierta purificación.

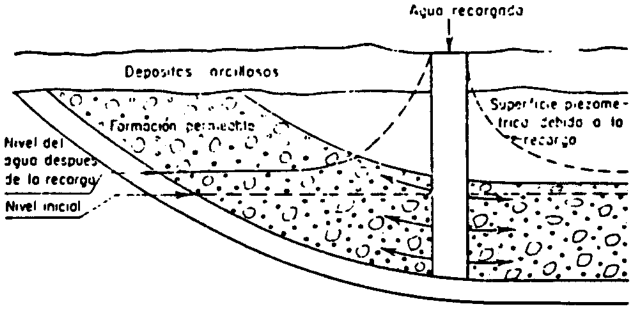

Figura 4

Sondeo de inyección

En diversas ocasiones, se ha procedido a inyectar aguas residuales urbanas después de un tratamiento terciario con objeto de detener la instrusión del agua marina en acuíferos costeros. En California (MITCHELL y SAMPLES, 1967) se han realizado experimentos de inyección con un efluente terciario, es decir, sometido a una filtración y cloración, en un acuífero arenoso cautivo. Estas pruebas de inyección han demostrado que es posible inyectar el efluente mencionado, y que al cabo de un recorrido de unos 100 metros en el acuífero, se producirían reducciones apreciables en el contenido en materia orgánica. Las bacterias y virus quedaban retenidos bien en la filtración previa o bien en el acuífero. De todas formas, el coste del tratamiento terciario y los costes de operación y mantenimiento de los sondeos, así como la legislación posterior desaconsejaron el montaje de la operación en régimen permanente. Conviene tener en cuenta que el agua inyectada en este caso, cumplía prácticamente todos los requisitos para aguas de bebida salvo en lo concerniente a total de sólidos disueltos, extracto en cloroformo, color y sulfonatos de alkil-benceno.

SCHICHT (1971) menciona el hecho de que el taponamiento de los poros en la formación permeable (en su caso unas areniscas consolidadas) es debido a la precipitación de materiales subsecuente a la cloración del efluente. Este precipitado consistía en fosfato cálcico y silicato cálcico. La adición de hipoclorito sódico como desinfectante hacía aumentar el pH, disminuyendo la solubilidad, por lo que fue necesario añadir ClH después de la cloración.

Todos los datos expuestos conducen a una serie de conclusiones generalmente no favorables a la inyección de efluentes mediante sondeos. Por no comentar más que aquellas que se refieren a la capacidad física de las formaciones para absorber el efluente, diremos que en prácticamente todos los casos mencionados en la literatura las inyecciones se efectuaban con efluentes a los que se había aplicado un tratamiento terciario, generalmente filtración por diversos métodos: microfiltros, tierra de diatomeas o filtros rápidos de arena, seguida de una retención en depósito y una cloración. Unicamente conocemos un caso, en la isla de Mallorca, en el que un efluente secundario, sin tratamiento posterior, es absorbido en una batería de sondeos, en un acuífero calizo fisurado de alta permeabilidad sin producir más que ligeros problemas de taponamiento que se resuelven bombeando periódicamente los sondeos.

## V.3. LA RECARGA ARTIFICIAL DE EFLUENTES COMO ELEMENTO DE GESTIÓN.

En los epígrafes 5.1 y 5.2, se han comentado algunos de los aspectos físicos de la recarga artificial de efluentes, y conviene ahora analizar el papel que este procedimiento puede jugar en la gestión integrada de los recursos y las posibles implicaciones en cuanto a contaminación.

La recarga artificial de aguas residuales tratadas es, fundamentalmente, un método de reciclado de un recurso hídrico que ha sufrido un deterioro en su calidad debido a una primera utilización. Parte de la degradación que ha sufrido el agua se elimina durante el tratamiento, y parte durante su introducción y movimiento en el acuífero. Sin embargo, la restitución de la calidad original es prácticamente imposible, sobre todo en términos económicos. Por esta razón, en los países en los que la recarga artificial de efluentes se practica no como un simple método de eliminación, sino para optimizar la gestión de los recursos, no se contempla la reutilización en un 100 por 100 del agua recargada para abastecimiento humano.

Una solución muy adecuada y que se practica en diversas partes del mundo, es la recarga para crear barreras de agua dulce que detengan la intrusión del agua en acuíferos costeros y poder así aprovechar éstos mucho mejor.

Otra práctica frecuente es la mezcla de efluentes con aguas superficiales naturales para obtener un agua de recarga en la que las concentraciones de ciertos elementos contaminantes (nitratos, compuestos orgánicos estables, etc.) queden muy reducidas; por supuesto, que la mezcla de los efluentes inyectados con el agua subterránea existente en el manto acuífero produce también esta dilución, aparte de los efectos de purificación ya estudiados.

Otra alternativa es la recarga de acuíferos o zonas de los mismos cuyas aguas se bombean fundamentalmente para riego u otros usos distintos del abastecimiento humano. Esta manera de emplear el efluente puede compaginarse con el uso directo del mismo para regar, recargándolo en épocas en que no sea necesario el riego del terreno.

Una situación teóricamente ideal para el reciclado de efluentes urbanos mediante recarga, sería el caso de un manto libre costero con buena permeabilidad y problemas de intrusión salina producidos por el bombeo de aguas utilizadas para regar. El uso alternativo de un efluente procedente de un tratamiento secundario para regar y para recargar tendría por resultado la recuperación de la superficie piezométrica, la mejora en la salinidad del agua subterránea y la posibilidad de un aprovechamiento mejor de las aguas subterráneas dulces del acuífero tierra adentro.

Las limitaciones que se imponen actualmente a la recarga de efluentes, dejando aparte la mayor o menor dificultad física de hacer llegar el efluente hasta el manto acuífero, son debidas a los posibles problemas de contaminación. Suponiendo que los nitratos y el total de sales disueltas sufran una dilución al mezclarse con las aguas subterráneas, quedan aún los microorganismos patógenos resistentes, los metales pesados y los compuestos orgánicos estables, como ya se comentó en el apartado 4 al hablar de la contaminación por el riego con efluentes. Muchos efluentes urbanos reciben aguas residuales industriales, y ciertos elementos y compuestos de las mismas atraviesan las plantas de tratamiento de aguas fecales sin alteración, por lo tanto, la evaluación de los riesgos producidos por estos elementos es indispensable en cualquier operación de recarga artificial con efluentes tratados.

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# Wastewaters in the Vadose Zone of Arid Regions: Geochemical Interactions[a]

by **Donald D. Runnells**[b]

## ABSTRACT

Because of increasingly stringent laws governing discharge of fluid wastes to surface waters, the alternative of discharge to the subsurface has become attractive. The physical-chemical processes that prevail in the subsurface are not well understood, but they are clearly not identical to processes of purification in surface waters. For example, in the subsurface the process of oxidation may be of little value in significantly reducing the concentration of discharged contaminants; in contrast, oxidation plays an important role in purifying surface waters. Eleven physical-chemical processes can be identified as having potential value for purifying wastes discharged to the subsurface, as follow: dilution, buffering of pH, precipitation by reaction, hydrolysis, oxidation or reduction, filtration, volatilization, biological assimilation, radioactive decay, membrane filtration, and sorption.

Discharge to the vadose zone may be a safe means of disposal of wastes in arid regions. But it is necessary to carefully test the suitability of a particular site for a particular waste. Processes of purification in the vadose zone can be incorporated into a workable plan of discharge if adequate studies and safeguards are employed. Regulations governing subsurface discharge should take into account the physical-chemical processes that may act to purify the waste fluids. In one set of experiments, a soil from Sulfur Springs, New Mexico was capable of removing large quantities of dissolved molybdenum and copper from a synthetic mill water, and the soil was able to quantitatively retain the copper during subsequent leaching by fresh and metal-free mill waters. Such studies permit rational plans of discharge to be developed.

## INTRODUCTION

Until fairly recent times we have generally assumed that ground water was well protected from contamination. And in many areas this seemed to be a fairly realistic view. Now, however, we know that the apparent purity of ground water was the result of our failure to *look* for pollution. With increasing pressures of population, and with incessant prodding from new regulations, water scientists have begun to fully recognize the widespread occurrences of contamination of ground water. Table 1 shows chemical components that have been demonstrated as being present in documented cases of contamination of ground water throughout the world (summarized from several sources, including Furiman and Barton, 1971; Scalf and others, 1973; Miller and others, 1974; Cole, 1974; van der Leeden and others, 1975). Each of these cases demonstrates that the protective capacity of subsurface materials is finite, and that this protective barrier can be overwhelmed by excessive discharge of virtually any contaminant.

The means by which contaminants may enter ground water are many and varied. Figures 1 and 2 illustrate some of the more obvious sources and pathways of contamination of ground water. The details of the hydrology of movement of

**Table 1. Chemical Components Present in Documented Cases of Contamination of Ground Water Throughout the World**

- chromium
- cadmium
- zinc
- lead
- fluoride
- iron
- barium
- manganese
- nickel
- silver
- molybdenum
- boron
- uranium
- mercury
- aluminum
- lithium
- cyanide
- copper
- selenium
- chloride (and hydrochloric acid)
- sulfate (and sulfuric acid)
- nitrate
- detergents
- radium and other radioactive wastes
- phenols
- alcohol
- gasoline
- leachate from landfills
- pesticides
- herbicides
- solvents

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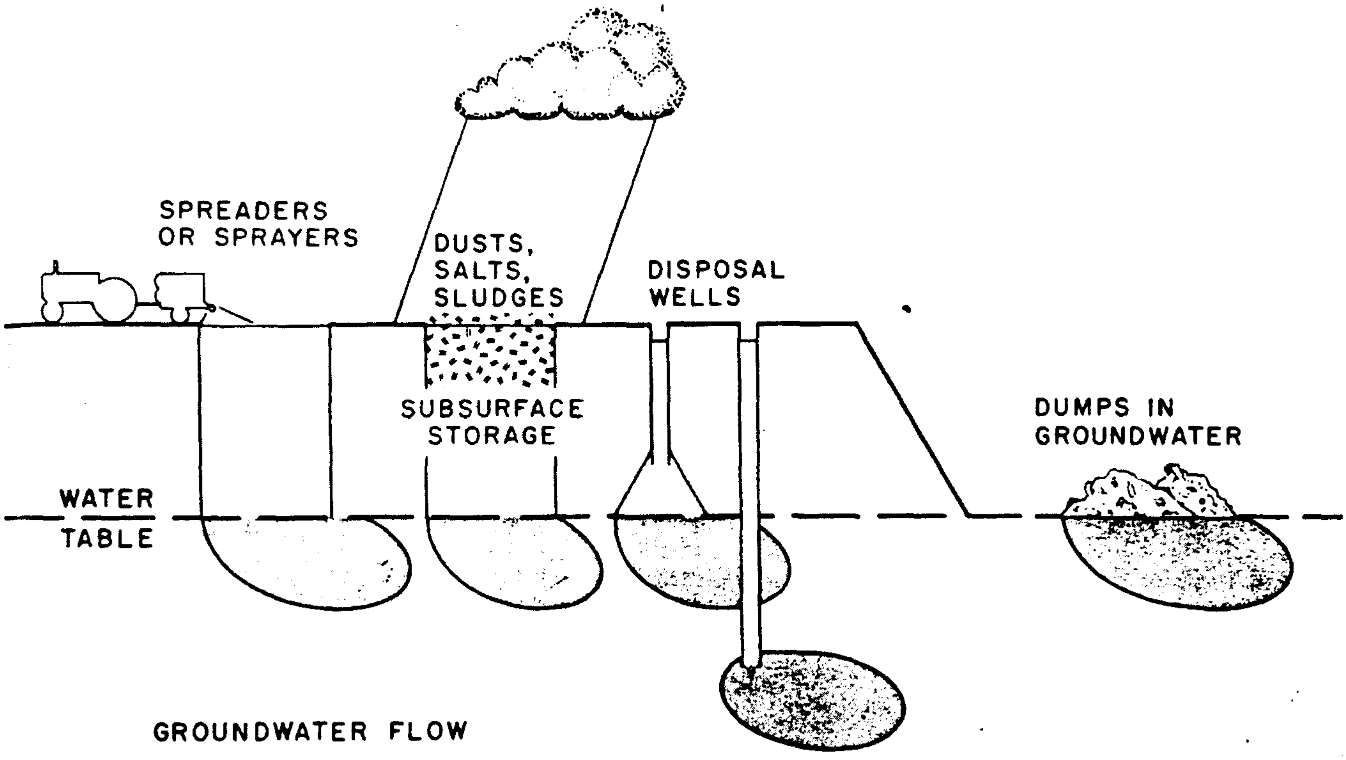
POSSIBLE SOURCES OF GROUNDWATER POLLUTION

Fig. 1. Summary of some obvious pathways by which contaminants may enter ground water.

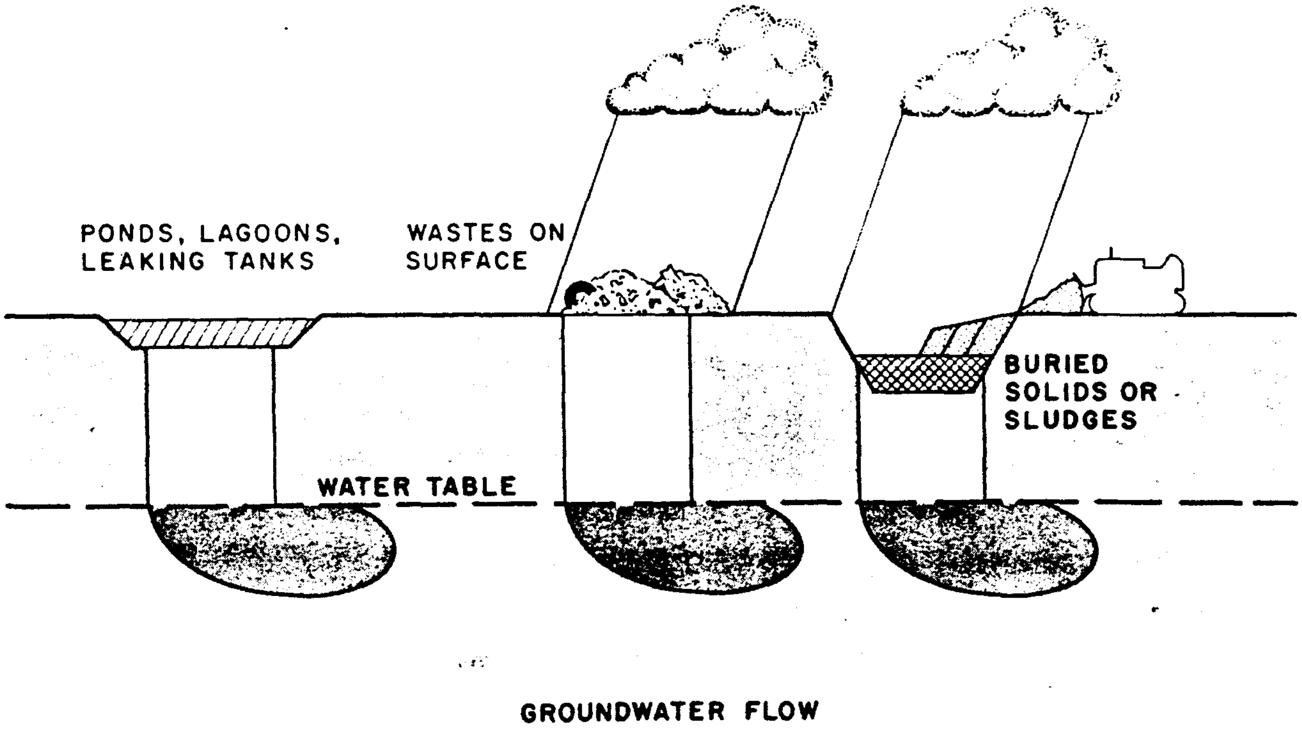
POSSIBLE SOURCES OF GROUNDWATER POLLUTION

Fig. 2. Additional illustrations of possible means by which ground water can be contaminated.

contaminants in the subsurface is treated in other papers in this Symposium and will not be discussed here.

Despite the fact that we are aware of hundreds of documented cases of ground-water contamination, the sobering fact is that there certainly exist thousands of similar cases of which we remain blissfully unaware. Miller and others (1974) made the important observation that most instances of ground-water contamination become known because of obvious leaks or spills at the surface, or because of complaints from subsequent users. Contamination of ground water is generally *not* discovered because of monitoring or routine chemical analysis. Many States in the past have not required analyses of water supplies for many toxic substances, such as barium, selenium, and silver, despite the listing of such substances in the 1962 Drinking Water Standards of the U.S. Public Health Service.

## POSSIBLE PROCESSES OF PURIFICATION

### Introduction

Having now emphasized the fact that ground water can surely become contaminated, let us look at some of the processes which offer protection to ground water, even in the face of significant discharges of potential contaminants to the subsurface. In Table 2 are listed eleven physical-chemical processes that may, under favorable circumstances, afford significant protection to the chemical purity of ground water. These eleven processes are listed together for the sake of completeness, but all are surely not equally effective.

The possible role and importance of each of these processes is discussed briefly in the following sections. Special emphasis is placed on their role in purifying wastes that might be discharged to the vadose zone in arid regions.

**Table 2. Eleven Physical-Chemical Processes That May Operate in the Subsurface to Purify Fluid Wastes**

1. Dilution.
2. Buffering of pH.
3. Precipitation by reaction of wastes with indigenous waters or solids.
4. Precipitation due to hydrolysis.
5. Removal due to oxidation or reduction.
6. Mechanical filtration.
7. Volatilization and loss as a gas.
8. Biological assimilation or degradation.
9. Radioactive decay.
10. Membrane filtration.
11. Sorption.

### Dilution

The first process listed in Table 2, dilution, is well understood as it occurs in surface waters. However, we cannot rely on dilution to be effective over the short term in the vadose zone, especially in an arid region. And in any location, we must expect dilution of wastes *below* the water table to take place far more slowly than in surface streams. Indeed, the one outstanding characteristic of contamination of ground water is the slow rate at which it is diluted and dispersed by the natural flow system. Fryberger (1972) has presented convincing data and arguments to show how expensive and difficult it may be to reclaim a contaminated aquifer, even considering natural attenuation due to flow and dilution.

### Buffering of pH

Natural ground waters in arid regions generally exhibit pH values between about 6 and 9. The upper limit is established in nature by the reaction between carbon dioxide gas and either limestone or caliche in the soil. The lower limit is an educated guess at the pH that might be found in shallow ground water from non-reactive rocks, such as quartz sandstone, or in ground water issuing from a mountain meadow. Trost (1974) found a range in pH from 6.0 to 9.0 in 1392 samples of ground water from southern Arizona.

The pH is a critical factor in many reactions involving contaminants, including processes that affect the stability of solid minerals and precipitates in the subsurface. If a discharged fluid has a pH outside of the range of 6 to 9, it is possible that drastic chemical reactions may occur, including the dissolution of such natural solids as calcium carbonate, iron oxyhydroxide, and manganese oxyhydroxide. Such reactions can exacerbate problems of contamination of ground water.

### Precipitation by Reaction

This process could be quite important in purification of discharged fluids. A perusal of any handbook of chemistry will show that *in theory* it is possible to precipitate virtually any dissolved contaminant if the appropriate precipitating agent is present. In nature, however, the necessary precipitants do not commonly exist in sufficient quantities or favorable locations to cause removal of toxic substances to safe levels. For example, it is theoretically possible to reduce the concentration of dissolved sulfate in a discharge by mixing it with a natural barium-rich ground water. However, barium-rich ground waters are not common, so that particular theoretical control will generally not operate in nature. In natural ground waters and in the enclosing solids, only certain species are normally present and available for reaction in meaningful concentrations. These species include dissolved calcium, magnesium, sodium, potassium, bicarbonate, sulfate, chloride, and silica. In the surrounding porous solids we find the additional abundant components of iron, aluminum, and possibly carbonate and phosphate. Dangerous or undesirable contaminants that might be reduced to safe concentrations by reaction with these species include the following: arsenic (precipitation by iron, aluminum, and calcium), barium (by sulfate and carbonate), cadmium (by sulfate and carbonate), copper (by carbonate), fluoride (by calcium and phosphate), cyanide (by many metals, including iron), iron (by silica), lead (by carbonate, sulfate, and phosphate), mercury (reduced form precipitated by carbonate and chloride), molybdenum (by iron and aluminum at proper pH), silver (by chloride), zinc (by carbonate and silica), and radium (by sulfate and carbonate). Evidence that such precipitation reactions do occur and may control concentrations in ground water has been given by many researchers. For example, Dutt and others (1968) found that the concentrations of iron and zinc in the ground waters of the Tucson Basin are apparently controlled by dissolved silica and the pH. However, despite the numerous possibilities for purification offered by precipitation reactions, a great many examples of contamination of ground water exist involving species that might be expected to precipitate; such examples demonstrate the failure of precipitation to protect ground water in many cases.

### Hydrolysis and Precipitation

This type of reaction occurs when a dissolved contaminant reacts with water, with the release of either a hydrogen ion or a hydroxyl ion. The best known example involves the hydrolysis of dissolved ferric ion with precipitation of ferric hydroxide. On the acid side, molybdenum is thought to hydrolyze to bimolybdate ion and precipitate with compounds of iron and aluminum under acid conditions (LeGendre and Runnells, 1975). In an aerated environment, such as the vadose zone in an arid region, the concentration of dissolved iron can be held to very low values by hydrolysis and precipitation of ferric oxyhydroxide (Hem, 1970). However, in oxygen-deficient zones, such as commonly exist in the phreatic zone, high concentrations of iron can occur in the ferrous ($Fe^{2+}$) form. High rates of infiltration or flooding by waters into the vadose zone, such as might occur beneath a disposal lagoon, may produce anaerobic conditions and lead to solubilization of some metals.

In addition to ferric ($Fe^{3+}$) iron, several other toxic metals form highly insoluble hydroxides in the range of pH of 6 to 9. These include copper (above a pH of about 6.5), chromium (the +3 form only, above a pH of about 6), nickel (above a pH of 9), and zinc (above pH 7.5). For example, Dutt and others (1968) found that the concentration of copper in ground water of the Tucson Basin seems to be controlled by either cuprous ($Cu^{+}$) or cupric ($Cu^{2+}$) hydroxide.

The apparent importance of hydrolysis in both the vadose and phreatic zones is the result of the simple requirement that water be present, in contrast to other precipitation reactions that demand the presence of specific precipitating agents.

### Precipitation Due to Oxidation or Reduction

Relatively insoluble oxides of certain contaminants are known for copper, iron, manganese, mercury, and nickel. Aerobic conditions favorable for the development of these oxides will normally be found in the vadose zone and possibly in the upper parts of the phreatic zone. Deep in the phreatic zone or in swampy or flooded vadose environments, anaerobic conditions may prevail and lead to the mobilization of some species. Dutt and others (1968) in their work in the Tucson Basin found that rapid infiltration of domestic sewage caused a loss of oxidizing conditions in the vadose zone and led to increased movement of copper, zinc, manganese, nickel, and lead through the soil.

In some instances reducing conditions are most favorable for removing possible contaminants from water. One example is that of chromium, which is highly soluble and mobile in the oxidized state (+6, chromate or dichromate ion), but quite insoluble as the solid oxide or hydroxide of the reduced form (+3). Natural reducing conditions can also theoretically cause the formation of such native elements as arsenic, copper, mercury, selenium, silver, and lead, each of which is quite insoluble. We do find significant quantities of native copper, mercury, and silver in mineral deposits, and native selenium appears to be a stable phase in some soils (Goering and others, 1968). In reducing environments bacteria can convert dissolved sulfate to sulfide and dissolved nitrate to ammonia or nitrogen gas. Reaction of the sulfide with certain metals will produce highly insoluble precipitates, such as the sulfides of arsenic, cadmium, copper, iron, lead, mercury, molybdenum, nickel, silver, and zinc.

In terms of usefulness and reliability for purification, the processes of oxidation and reduction can be of significant value if the necessary environmental conditions are maintained and managed. Considerable reliance can probably be placed on the process of oxidation for purification of wastes in the vadose zone. Processes of reduction may be more risky because some undesirable species can be mobilized, such as iron (Apgar and Langmuir, 1971) and manganese. One well-known benefit of the process of reduction is the denitrification of dissolved nitrate from sewage effluent, with significant removal of nitrogen possible under a carefully programmed mode of disposal (Bouwer, 1974).

### Mechanical Filtration

The sixth process listed in Table 2, mechanical filtration, will generally be of little help in removing dissolved contaminants. Exceptions to this might occur following the formation of precipitates by means of one of the processes mentioned earlier. Such precipitates would be filtered out mechanically as the water moves through the porous medium. Iron is the species most likely to exhibit this behavior; it is well established that much of the total iron in aerated waters is present as particulate ferric hydroxide (Hem, 1970). Manganese probably behaves in a similar fashion. If these possible particulates sorb or include other dissolved species as they form, the filtration would remove these other species as well. Of course, most particulates originally present in the discharge would be filtered out quickly during downward movement through the vadose zone.

### Volatilization and Loss as a Gas

Some inorganic species can be effectively removed from discharged waters by volatilization. Perhaps the best known example is that of the bacterial reduction of dissolved sulfate to hydrogen sulfide gas, with loss of the hydrogen sulfide to the atmosphere (Kellogg and others, 1972). Mercury in solution can also be volatilized in anaerobic environments (Lagerwerff, 1972) or by reaction with dissolved humic acids (Alberts and others, 1974). Several organic compounds of arsenic are volatile, and escape of arsenic as a gas has been demonstrated for both aerobic and anaerobic soils (Woolson and others, 1971). Based on its similarity to sulfur, we might also expect selenium to be subject to volatilization (Lakin, 1973). And of course, the microbial reduction of nitrate to ammonia and nitrogen gas is well documented (Bouwer, 1974), although the failure of this mechanism to protect ground water is demonstrated by numerous examples of pollution of ground water by nitrate (Minear and Patterson, 1973).

Volatilization as a means of purification is poorly understood, and we probably run considerable risk in relying upon it to any great extent. The exceptions to this would be the conversion of dissolved sulfate and nitrate to gases; these processes are known to be effective if managed properly.

### Biological Degradation

Biological degradation or assimilation, the eighth process, is very important in removal of organic and biologic contaminants. Many organic substances would be removed or attenuated by biologic activity in the subsurface, especially in the oxidizing environment of the normal vadose zone. The biologic involvement of sulfate and nitrate have already been mentioned. In addition, arsenic, cyanide, mercury, and selenium are likely candidates for biologic fixation or volatilization. Molybdenum is strongly assimilated and concentrated by plants that are nitrogen-fixers (Johnson, 1966). And in their study in the Tucson area, Dutt and others (1968) found that grass assimilated and removed significant quantities of metals from infiltrating sewage effluent.

Biological processes could be of great value in managing discharges to the subsurface, but at present we know so little of the principles involved that each case must be studied and evaluated on its own.

### Radioactive Decay

This mechanism is of value in the management of radioactive wastes by means of storage in the subsurface. Winograd (1974) has recently discussed the attractiveness of storage of high-level radioactive wastes in the vadose zone of arid environments. It seems clear that storage in this environment is possible with a high degree of safety for periods of time from thousands to hundreds of thousands of years, during which time the wastes would lose much of their activity through processes of decay. There is a substantial history of disposal of radioactive wastes to the subsurface near nuclear facilities in the United States, and in humid regions serious problems of contamination of ground er can be expected. It is the lack of recharge to move contaminants to the ground water that makes disposal to the vadose zone in arid regions so attractive.

## Membrane Filtration

The tenth possible process of purification, embrane filtration, is a topic that is greatly in vogue at the present time in the literature of the geochemistry of subsurface waters. A summary of the phenomenon, given by Hanshaw (1972), points out that some observed data on the pressure and salinity of formational waters cannot be explained by simple gravitational flow of water or by dissolution of minerals. In such cases, the anomalous data may be related to osmotic pressures and filtration of salts across beds of shale. Pressure differentials of several hundred atmospheres can theoretically be generated across a membrane if fresh water exists on one side and a saturated brine on the other (Hanshaw, 1972).

In terms of purification of wastes in the subsurface, it is theoretically possible that a discharged wastewater could be diluted by osmotic transfer of water across a clay-rich aquitard. Conversely, one can speculate on the possibility of forcing waste fluids through a clay liner on the bottom of a disposal pond, anticipating some removal of dissolved salts and passage of a relatively more pure fluid. However, at this stage in development of the technology of disposal, careful pilot studies would be required to test the efficacy of such a scheme.

## Sorption

Finally, and importantly, ground water may be protected by various processes of sorption. Clays, metallic oxides and hydroxides, and organic matter can all be good substrates for sorption of various dissolved species. With the exception of fractured shale or siltstone, consolidated bedrock will generally not be very effective as a sorbent. Virtually every ionic species and many non-electrolytes will be sorbed and removed to some extent as ground water moves through an aquifer; of the ions, only chloride, and to a lesser extent sulfate and nitrate, seem to pass through soils and alluvium without significant sorption.

The troubling aspect of sorption as a means of purifying waste waters in the vadose zone is that the process can be highly specific in its action, both as to the dissolved substance and the solid substrate. Molybdenum offers an interesting example. Dissolved molybdenum is sorbed strongly by most soils that exhibit an acidic reaction with water, but molybdenum will move freely through alkaline soils (Katz and Runnells, 1974). Similarly, fluoride is sorbed much more extensively by acidic soils than by alkaline soils (Bower and Thatcher, 1967). Perhaps the most revealing study of the effect of pH on sorption of ions is that by Griffin and others (1976), in which they determined the extent of removal by clay minerals of a suite of heavy metals from synthetic solutions and natural leachates from sanitary landfills. They found a very great variation in the extent of sorption, depending on the metal involved and the pH of the solution.

It is clear that the processes of sorption depend on the type of contaminant and on the physical and chemical properties of both solution and porous medium. Under favorable circumstances sorption can be wonderfully effective in purifying waters, as evidenced by the extensive use of ion-exchange media in water treatment. One should keep in mind, however, that when a contaminant ion is sorbed, some other change must also take place to compensate for loss of the charged species from solution. In ion-exchange reactions, a different ion is released by the solid to the solution. The ions released to water from a previously uncontaminated soil or alluvium will almost certainly be less harmful than most of the contaminants of concern here.

We can place considerable reliance on the processes of sorption to help us protect ground water, but each case will be different and tests must be run to assure an adequate degree of sorption and retention.

## Summary

Eleven processes have been discussed, each of which is capable, under favorable circumstances, of contributing to the purification of liquid wastes discharged to the subsurface. However, hundreds of documented examples of ground-water contamination show that in specific instances the defensive mechanisms have been overwhelmed by discharged contaminants. It is reasonable to make an attempt to use the purifying properties of the subsurface as part of a plan of discharge, especially in the vadose zone in arid regions. However, such a plan of discharge must incorporate careful studies of the capacity and efficiency of the vadose zone for removing contaminants, and of possible later remobilization of the toxic substances.

## USING NATURAL PROCESSES OF PURIFICATION IN A PLAN OF DISCHARGE

### Introduction

Because of the potential value for purification and long-term storage of wastes in the vadose zone of arid regions, it seems reasonable to include the use of this zone in plans of discharge. In fact, Winograd (1974) presents a detailed analysis of such a plan for storage of radioactive wastes. As water scientists, we can envision similar useful applications in the discharge of other wastes. Bouwer (1974) has given an example of proper management of disposal of sewage effluent to the vadose zone in Arizona in order to maximize removal of dissolved nitrate. At the time of this writing, the New Mexico Water Quality Control Commission is considering, for possible adoption, new regulations for protection of ground water. These proposed regulations, developed chiefly by the New Mexico Environmental Improvement Agency, would permit a discharger to make use of a fraction of the purifying capacity of the vadose zone as part of his plan of discharge to the subsurface. The concept of using the vadose zone in this manner was suggested by the present writer, as a consultant to the New Mexico Environmental Improvement Agency. It is gratifying that most parties concerned, from industry on the one hand to environmental groups on the other, seem to agree that the vadose zone in arid regions can offer a safe repository for contaminants, provided that very careful study and monitoring are incorporated into the plan of discharge.

### Potential Capacity of the Vadose Zone for Purification

The capacity of the vadose zone can be large for removal and storage of contaminants. As an example of this large theoretical capacity, let us consider a hypothetical discharge of dissolved cadmium from a metal-plating plant. The assumptions involved in this example are summarized in Table 3. The concentration of 3.7 mg/l dissolved cadmium is that actually reported by Lieber and others (1964) for the well-known example of ground-water contamination on Long Island. It would be dangerous to allow a discharger to use the total capacity of the vadose zone for sorption of contaminants from his wastewater, so for purposes of this example we will assume that State regulations allow him to use 25 percent of the estimated total sorptive capacity of the unconsolidated mantle beneath his discharge pond. Based on the data listed in Table 3, the discharger could store approximately 380,000 pounds (172,400 kg) of cadmium in the vadose zone. This is a *lot* of cadmium. At a concentration of 3.7 mg/l, this amount corresponds to about 38,000 acre-feet (4.7 × $10^{10}$ liters) of wastewater that could be discharged from the one-acre pond.

**Table 3. Assumptions Used in Hypothetical Example of the Capacity for Sorptive Removal of Dissolved Cadmium from Discharge to the Vadose Zone in an Arid Region**

*Assumptions:*

1. Unconsolidated mantle with 10 weight percent clay.
2. Cation exchange capacity of 50 milliequivalents per 100 grams (0.22 pounds) of clay.
3. Density of 2 grams/cc (124.9 lbs/cubic foot).
4. Thickness of mantle = 100 feet (30.5 meters).
5. Area of disposal = 1 acre (0.405 hectare).
6. Efficiency of sorption is 100 percent.
7. Only 25 percent of the total sorptive capacity is utilized.

Of course, the preceding simplistic example ignores competition or enhancement of sorption by other ions, other chemical reactions, possible channelized flow of the wastewater through the vadose zone, and a great many other complicating factors. The example does point out, however, that a significant degree of purification of wastewater and storage of dissolved contaminants is possible under the conditions that exist in an arid region.

### Dangers and Necessity for Study

Having emphasized the great potential value of the vadose zone for discharge, we must temper our enthusiasm by recognizing that the vadose zone has a finite capacity for purification, and that there is always a danger of leakage and contamination of ground water. Referring back to Table 1, we can see that it is possible to overwhelm the protective mechanisms in the subsurface with any dissolved contaminant. This knowledge warns us of the necessity for thorough and competent studies of the hydrologic and chemical characteristics of a proposed site *before* discharge begins. A proposed plan of discharge must also allow for a wide margin of error in the measurements required, including such parameters as heterogeneity in the hydraulic properties of the earth materials at the site. Finally, an adequate scheme for monitoring must be included, together with contingency plans in the event that the discharged contaminants do not behave as expected.

### Necessary Measurements and Testing

Our knowledge of the specific physical-chemical processes that obtain in the subsurface nitive. We cannot predict from theory what will happen when a particular contaminant in a particular fluid matrix is discharged into a specific type of soil. We do have some general knowledge of the principles involved, as discussed in an earlier section of this paper, but we remain ignorant of many of the specifics. Now that the Drinking Water Act is a reality, we can expect to see a rapid increase in research and knowledge concerning the movement of contaminants through earth materials. In fact, it would be possible to list several new publications that deal specifically with this subject. By far the most complete and satisfying such report known to this writer is that by Weir and others (1975), prepared for the Electric Power Research Institute. The purpose of the work reported by these workers was to determine if toxic ions would be leached from ash and sludge discharged into ponds by electric utilities, and to determine if such ions would move through underlying soils into ground water. Another excellent study, with greater emphasis on the hydrologic factors, is that by Papadopulos and Winograd (1974).

In order to have confidence in the probable efficacy and safety of a plan of discharge to the vadose zone, a great many factors must be considered. For example, one must have a fairly complete knowledge of the hydrogeologic conditions and homogeneity of the earth materials at the proposed site of disposal. In order to obtain such information it will certainly be necessary to conduct fairly extensive field studies, possibly including drilling, to determine the rate and paths of movement of the fluid discharge. An example of one aspect of the work that may be necessary can be found in the study of seepage of effluent from septic tanks, published by Bouma and others (1972). Hajek (1969) has given a good summary of some of the technical aspects of the tests and calculations that should be done to understand and predict the chemical interactions of wastewaters with soils. And finally, a potential discharger must look to the future, realizing that "purification" of wastewaters by the vadose zone really represents *storage*, either long-term or short-term. For some chemical species, such as phosphate, zinc, and copper, the storage may be permanent because these ions can become fixed in the structure of minerals and not be available for later release by percolating solutions (Ellis and Knezek, 1972). In other cases, contaminants that have been discharged and stored in the vadose zone may be readily available for leaching and remobilization by the next soaking rainfall, or by a subsequent change in the chemical composition of the discharged fluid. Tests must therefore be run, not only to determine the extent of removal of contaminants from the discharge, but also to determine the possibility of later remobilization and flushing to ground water. With regard to storage, Winograd (1974) has discussed the selection of a site for disposal to best avoid exhumation by the normal processes of erosion or remobilization by climatic change over periods of hundreds of thousands of years.

Discharge of any kind may pose a long-term threat to the environment, and at present, our knowledge of interactions in the vadose zone does not permit us to make unequivocal predictions from theoretical principles. Much work must be empirical, and teams of scientists and engineers must be involved in devising the plan of discharge. Perhaps most importantly, each type of discharge and potential site must be treated individually, on a case-by-case basis. No one can claim that the job of insuring non-destructive disposal of wastes to the vadose zone will be easy, but neither can they claim that the job is impossible.

### An Example of Geochemical Testing

As part of the task of developing possible regulations to govern discharges to the subsurface in New Mexico, this writer was asked to present an example of how the geochemical portion of the testing might be conducted. The work was not intended to be complete, but was designed instead to illustrate the minimum information that would be appropriate. A very brief summary is presented here.

The soil chosen was from Sulfur Springs, New Mexico. It was dark brown in color, contained rootlets, and represented a composite of the upper six inches of the profile. A 1:2 soil:water slurry yielded a potentiometric pH of 6.4. The soil was air-dried and sieved to (−) 2 mm to remove the coarse, non-reactive fragments. A small column of soil [approximately 23.2 cc (1.42 cubic inches)] was prepared in a chromatography tube. The test solution was gravity-fed into the column at various rates, from about 0.5 to 1.6 ml/min (corresponding to a velocity of about 4 to 12 feet/day). Saturated conditions were maintained, together with a relatively high rate of flow, because of limitations of time. An actual case would require slower rates of flow and an unsaturated column. Aliquots of the effluent were collected in volumes of about 9.5 ml (0.0025 gallons) by means of an automatic fraction collector. The porosity of the soil column was estimated in two ways: (1) addition of water to saturation and reweighing in the column and (2) measurement of volume of water withdrawn from a graduated cylinder by suction into the soil column. Density was determined by weighing and measuring the volume of soil column. The porosity was 48 ± 4 percent, and the density was 0.88 ± 0.03 gm/cc (55 ± 2 lbs/cubic ft). The pore volume of the column was 11 ± 1 cubic centimeter (0.00039 cubic feet). The errors involved in this crude study are obvious, such as the disturbance and sieving of the soil, and the rough estimate of porosity and density. For an actual plan of discharge it would probably be necessary to obtain cores of the soil; ASTM methods would be used to measure the porosity and density of the core material.

The test solution was made up to simulate the aqueous discharge from a metallurgical flotation mill. The aqueous matrix was a saturated solution of gypsum, initially spiked with 1.0 ppm (mg/l) dissolved copper and 2.1 ppm (mg/l) dissolved molybdenum. The initial concentration of copper was made considerably higher than might be expected in an actual mill effluent; this was done because initial tests indicated that the soil had a large capacity for removal of copper from the test solution. The initial solution and the aliquots of effluent were analyzed for copper and molybdenum by atomic absorption and a thiocyanate colorimetric procedure, respectively. Standard additions were employed to test for sensitivity and interference in the copper analysis.

The results for molybdenum are shown in Figure 3. The results for copper are not shown because copper was generally below the limits of detection in the effluent during most phases of the study (the detection limit for dissolved copper was 0.01 parts per million).

As shown in Figure 3, an initial period of leaching by distilled water (simulated rain water) failed to release detectable amounts of molybdenum (less than 15 parts per billion dissolved). The concentration of dissolved copper was approximately 0.05 ppm in the effluent during this initial leaching, which continued for 378 ml (34 pore volumes). This initial period of leaching shows that neither the concentration of copper nor molybdenum released by heavy applications of fresh water to Sulfur Springs soil poses a threat to ground water. Next, in Figure 3, the column was leached with approximately 26 pore volumes (288 ml) of metal-free synthetic mill water. In this case, dissolved copper in the effluent averaged less than 0.01 ppm and molybdenum less than 15 ppb. Again, these results show that extensive leaching with metal-free mill water fails to leach significant quantities of either metal from the soil.

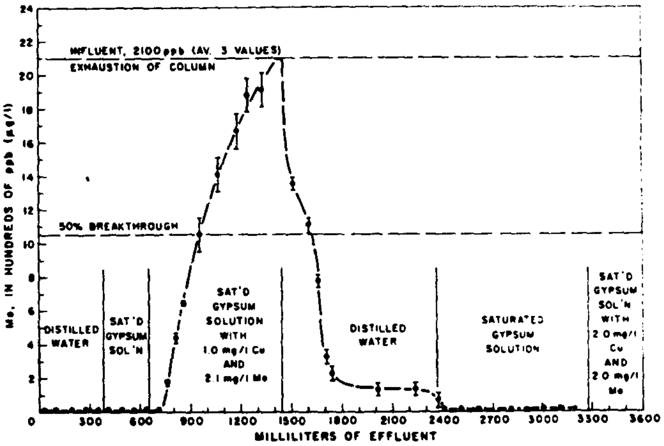

**Fig. 3. Results of experimental study of leaching, removal, and remobilization of molybdenum using synthetic mill water and a New Mexico soil. Analytical error shown by vertical bars; error is approximately the diameter of the circles along the abscissa.**

At a volume of 661 ml in Figure 3 the metal-spiked water was introduced into the column, and 72 pore volumes were passed through. No detectable copper (less than 0.01 ppm) ever appeared in the effluent. Molybdenum was first detected after 66 ml (6 pore volumes) of spiked water had flowed through the soil. Fifty percent breakthrough of molybdenum occurred after passage of 32 pore volumes. The capacity of the soil to remove molybdenum was completely exhausted after passage of 72 pore volumes of mill water (792 ml). The difference in behavior of these two metals is caused by their ionic form in solution; molybdenum occurs as an anion ($MoO_4^{--}$), whereas copper is present as a cation ($Cu^{++}$) under these conditions. This soil had a very large capacity for removal of copper.

In Figure 3, after exhaustion of the column for molybdenum, the soil was again leached with simulated rainfall and metal-free mill water to test for desorption and remobilization. Of the 792 micrograms of copper introduced, leaching by 165 pore volumes of distilled water and synthetic mill solution released only about 19 micrograms; this means that the removal of copper by this soil approaches being irreversible. The initial leaching of the soil, prior to the introduction of the metal-rich mill water, released as much copper. In contrast, leaching of the loaded column with 83 pore volumes of distilled water remobilized nearly 60 percent of the approximately 700 micrograms of molybdenum previously added. The rate of release of molybdenum was decreasing exponentially with continued leaching the distilled water, as shown in Figure 3,• indicating that a very large volume of water would be required to remove all of the added molybdenum. Interestingly, upon initiation of leaching of the loaded soil with metal-free mill water, the concentration of molybdenum in the effluent fell to about 10 parts per billion, continuing to the end of this portion of the experiment with an additional 82 pore volumes (about 3280 ml on Figure 3). Calculations suggest that this decrease in desorbed molybdenum was probably due to the precipitation of solid $CaMoO_4$ in the soil due to the high concentration of dissolved calcium in the water.

Finally, in Figure 3, a new spiked mill water, containing 2 ppm copper was passed through the column in an attempt to cause breakthrough of the dissolved copper; however, breakthrough did not occur, even after a calculated addition of 256 ppm copper to the solid phases in the soil column.

The results of this study show that the soil from Sulfur Springs, New Mexico, has an enormously high capacity to remove copper, and that the removal is essentially irreversible. A relatively high capacity for removal of molybdenum is also exhibited, with 50 percent breakthrough at 32 pore volumes of spiked mill water. This type of information would comprise an important portion of a rational plan of discharge.

Space does not permit development here of expressions to relate the data of Figure 3 to a real situation. However, it can be shown from considerations of the distribution of the ions between the liquid and solid phases (Hajek, 1969; Tamura, 1972; Wierenga and others, 1975) that thousands of years may be required for the molybdenum in this study to reach an assumed water table at a depth of 30 meters (98 feet) in the Sulfur Springs soil, depending on the rate of discharge and infiltration. Copper would require much longer to reach the ground water.

## SUMMARY

In summary, I have tried to demonstrate that there are valid chemical and physical reasons for looking toward the subsurface, especially the vadose zone in arid regions, for disposal of some types of wastes. Processes that operate in this environment can, under favorable circumstances, greatly attenuate and detoxify some wastes. On the other hand, we know of too many examples of contamination of ground water to believe that the overlying mantle of soil and sediment affords absolute protection. With these facts in mind, we must try to balance the need for protection of ground water against the economic realities of the use of water by industry, municipalities, and the general public.

## ACKNOWLEDGMENTS

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## DISCUSSION

**The following questions were answered by Donald D. Runnells after delivering his talk entitled "Wastewaters in the Vadose Zone of Arid Regions: Geochemical Interactions."**

**Q. by Leonard Wood.** *Rain water is not distilled water. Have you used rain water in your column experiments?*
A. No, I have not tried to exactly duplicate the chemistry of rain water in the column studies. However, I do not believe that the observed variations in the chemistry of rain water would make a great deal of difference under most circumstances. The exception might be in regions of serious air pollution in which sulfuric acid becomes the principal control on the pH of rain water.

**Q. by Mike Kaczmarek.** *Would substantial increases in the Mo loading in the soil column result in comparable increases of Mo in the leachate resulting from "rain water" (distilled water)?*
A. Yes, the greater the initial loading, the greater would be the slug of Mo released from a soil when remobilization due to leaching does take place.

**Q. by E. E. Jones, Dale Ralston and Harry Nightingale.** *What percent of pore space was saturated in your experiments?*
A. This particular column was filled from the bottom to drive out air, then maintained in a fully saturated condition, so the pores were nearly 100 percent filled with water.

was done because a limited amount of time was available and I had to maintain high rates of flow. A different experiment could be conducted under non-saturated conditions, in which the rate of flow would be substantially slower.

**Q. by Harold Meiser.** *What experiments are you planning for the future?*
**A.** We now have underway a program to test for leaching trace elements from spent oil shale, and the reactivity of these elements with soils of the oil shale region in Colorado.

**Q. by Neil Jaquet.** *How do the volumes of distilled water used in your column studies compare to rates of precipitation in the area of concern?*
**A.** Experiments such as I described are designed to yield basic information, such as the distribution coefficient, $K_d$. Such information can then be used in equations involving the specific rates of precipitation and infiltration for an area. Thus, it is not necessary to model the *amount* of rainfall and infiltration, just the chemistry and rates of flow.

**Q. by Jon O. Nowlin.** *Assuming contaminant "removal" by cropping, how is the crop to be disposed of?*
**A.** The three advantages in cropping would be: (1) to convert the dissolved contaminants into solid form, or (2) to consume and transform such contaminants as nitrate, BOD, etc., or (3) to concentrate the contaminants from a large amount of fluid into a smaller volume of crop. However, in some cases, cropping would still offer no advantages because you still could not dispose of it.

**Q. by K. E. Childs.** *Can these tests be used to evaluate other areas, soils, and chemicals, or are the results too qualitative for this?*
**A.** The results are quantitatively useful for the particular parameters chosen, but new tests would have to be run for other discharges, soils, etc.

**Q. by P. K. Saint.** *Can one quantify the sorption capacity of the soil and predict the time of passage of an ion through the vadose zone? How about desorption later on?*
**A.** Yes, the results can be used for predicting migration, and references are given in the paper to illustrate this. Figure 3 in the paper shows the results of desorption studies.

**Q. by Logan Kuiper.** *What harmful effects would result if the contaminants reached ground water in the case considered?*
**A.** Molybdenum at this concentration (2.1 mg/l) could be toxic to ruminant animals, and the TDS (saturated gypsum solution) would degrade the quality of the ground water. Finally, the 1.0 mg/l copper would probably not be harmful to health, but would be undesirable because of taste.

**Q.** *How do different types of soils affect the leaching process?*
**A.** The processes of removal and leaching are strongly dependent on the type of soil.

**Q. by G. F. Hendricks.** *Any study of pick-up of copper by plants?*
**A.** There have been several such studies by others in the past, but this was not part of our program.

**Q. by Don Lundy.** *What mechanism retained the copper?*
**A.** I really did not get into the study of specific mechanisms. The experiments were designed only to give a general indication of capacity for removal and retention.

**Q. by Buck Steingraber.** *What happened to the sulfate in your percolating water—did it come through in the original concentration?*
**A.** The specific conductance that was measured in the effluent suggested that the sulfate passed through the soil with little or no removal.

**Q.** *What time equivalency in years of precipitation would be required to equal the amount of solution put through the soil column?*
**A.** For a semi-arid region, with perhaps 2 centimeters of recharge per year, the total volume of solution percolating through each cubic centimeter of soil would correspond to about 10 saturated pore volumes (assuming 20% porosity) per year. In this study I passed about 300 pore volumes through the saturated column, corresponding to 30 years of recharge. However, the ions in the experiment were retarded by the soil and would move much more slowly than the water.

# Land Application of Waste — Important Alternative[a]

by John R. Sheaffer[b]

## ABSTRACT

Land treatment uses a combination of processes to manage and beneficially use waste water. It represents a revolution in sewage treatment because it (1) transforms sewage treatment from a single purpose activity into a multipurpose activity, (2) changes sewage treatment construction grants from subsidies into investments in the production of food and fiber, and (3) requires the participation of a variety of disciplines to implement successfully. Because it is revolutionary to the sewage treatment field, three situations have developed. First, it is displacing traditional technology at a record-breaking pace. Second, its logical appeal to thinking decision makers has created a situation in which the policy makers are ahead of many technicians. Third, it is attacked with a fervor heretofore unknown in the sewage treatment field.

Land treatment has logged an enviable track record in the United States. Existing systems have produced a high quality effluent at economically competitive prices. In addition, in terms of relative risk, the threat to environmental quality from a land treatment system compares favorably with advanced waste treatment systems.

## INTRODUCTION

Land treatment of waste water is a controversial issue. A part of the controversy stems from misunderstanding. As Mark Twain once said, if two minds disagree, it's for one of two reasons. Either they are using the same words to mean different things, or they are using different words to mean the same thing. This characterization describes many discussions and articles relating to land treatment. In general, discussants are perceiving different systems as they debate the performance level of land treatment.

To initiate a meaningful discussion of land treatment, it is necessary to identify first the elements of a complete system. This does not mean that a variety of abbreviated versions of land treatment system do not exist. Rather, it suggests that one should not expect an abbreviated version to function like a complete system.

There may be good reasons to abbreviate land treatment systems. These are policy decisions. However, the performance data from an abbreviated system should not be used to evaluate a complete system. When this is done it is an incorrect technological transfer. Such incorrect transfers have taken place and have influenced planning decisions.

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## COMPONENTS OF A LAND TREATMENT SYSTEM

A complete land treatment system consists of a number of processes which manage and use the waste water. There are six basic components. Each component is discussed briefly.

The first component is a network of pipes to collect the waste water. This collection and transport system conveys the waste water to a selected location. The location could be contiguous to the producing area 5 miles away, or conceivably 100 miles away.

The second component provides pretreatment. Thus, a land treatment system does not spread raw sewage on the land. Rather a proper level of mechanical and biological treatment is provided before application to the land. The pretreatment reduces the BOD level of the sewage to prevent the creation of nuisance conditions.

The third component provides storage. This component provides flexibility with respect to when waste water will be applied to the land. This eliminates conflicts with respect to planting, harvesting, and irrigation. Also, it avoids the necessity of irrigating during the nongrowing season. If the pretreated waste water is to be used primarily to irrigate and fertilize crops it must be stored during the nongrowing season. The storage facility must be designed to manage potential leakage. This can be done by constructing facilities to intercept the leakage and return it to the storage basin, controlling ground-water movements by wells and drainage ditches, or lining the storage basin with an impervious layer. Disinfection of the treated and stored waste water takes place as it leaves the storage basin.

The fourth component is an irrigation site on which crops can be grown. When selecting the site, the hydrogeology and the soil characteristics should be evaluated. There is a double interest in these evaluations. First, there are the general questions regarding the use of the site for irrigation, e.g., salinity, waterlogging, and the buildup of sodium in the soil. Second, there are the questions regarding the capability of the site to purify the stored and pretreated waste water.

Initially, a field description of the site must be developed. The soil must be evaluated with regard to texture and be separated into horizons with depth. This information on soil needs to be accompanied by observations regarding infiltration, permeability, depth to zone of saturation, and direction and rate of flow of ground water.

The following group of analyses will help to establish the potential for prolonged irrigation of the soil and to evaluate its potential to purify the waste water: (1) cation exchange capacity (c.e.c.), (2) pH, (3) calcium carbonate ($CaCO_3$) if exceeding 0.1 percent, (4) particle size distribution, (5) total organic carbon, (6) total organic nitrogen, (7) exchangeable Ca, Mg, K, and Na, (8) total soluble salts, (9) chlorides, and (10) bulk density. (These parameters are gleaned from the analyses of land treatment systems provided by G. W. Leeper, consultant in agricultural chemistry to Sheaffer & Roland, Inc.) The importance of these parameters is discussed briefly in the following passages:

*1. Cation exchange capacity.* This has a role in both spheres of irrigation and purification. In irrigation, it is helpful in estimating the impact of a known amount of sodium in the irrigating water. In purification, it is used in estimating the load of heavy metal which the soil may safely carry. A useful figure that is quoted here, and one with some experimental backing, is that if the pH is 6.5, then 10 percent of c.e.c. may be held by zinc and other heavy metals. (If c.e.c. is 20 m.e. per 100 g, this means 650 ppm of zinc.)

*2. pH.* There is also a dual role for pH. A pH exceeding 8 is taken into account in assessing the impact of sodium in water; while any information about pH is useful in describing a soil's chemistry—a low pH implies low reserves of nutrients, in particular. In renovation, one desirable (or essential) side of a moderately high pH is the increased ability to hold heavy metals (Leeper, 1978). Another concern is phosphate. It is held by calcium at pH above 6 and by iron and aluminum at pH below 6.

*3. Calcium carbonate.* Information on $CaCO_3$ is in any soil study. The qualification "above 0.1 percent" is merely to avoid analyzing for trivialities. Calcareous soils have their own peculiarities in availability of trace elements. In terms of renovation, $CaCO_3$ is a powerful buffer against both heavy metals and phosphate. Against zinc, 1 percent $CaCO_3$ would certainly hold 6500 ppm zinc. Against phosphate, 1 percent $CaCO_3$ would hold 1900 ppm phosphorus.

*4. Particle size distribution.* This is a numerical way of recording what the soil survey reports as sand, loam, clay loam, clay, etc. The percentage clay is the most important single value in this analysis. It can be linked to the conduction of water through the soil, and to the c.e.c. though different kinds of clay differ greatly in c.e.c.

5. *Total organic carbon.* This provides an estimate of total organic matter (given by multiplying by 1.72). While this figure is largely determined by climate it also depends on management and generally increases with fertility. Organic matter contributes to c.e.c. much more than does clay, and it holds heavy metals, atom for atom, more strongly than clay. This need not be evaluated below 2 feet.

6. *Total organic nitrogen.* This is easily determined and gives a little additional information to organic carbon, in terms of fertility. The carbon nitrogen ratio (C/N) is usually of the order 10 to 12 to 1. Any reduction in this ratio indicates a greater likelihood of liberating ammonia. This need not be evaluated below 2 feet.

7. *Exchangeable Ca, Mg, K, and Na.* These exchangeable metals are primarily of interest in coping with high proportions of sodium. They are of primary interest to the irrigator.

8. *Total soluble salts.* This is of interest to the irrigator.

9. *Chloride.* This is of interest to the irrigator.

10. *Bulk density.* This information is needed to make mutual transformations of parts per million and kilograms, per hectare or pounds per acre.

The fifth component is a growing crop. This provides a living filter with the potential to recycle nutrients. The crops selected must be compatible with the soil, climate, and the wastewater characteristics. A soil system with a growing crop can recycle nutrients and can extract some substances that should be confined and contained in the environment. Cadmium, an example, is an element in sewage which is efficiently extracted by crops. However, the crop keeps the cadmium out of its seed and deposits it in its stems and foliage.

The sixth component is an underdrainage system. This important component can be either natural (an area of ground-water discharge) or installed (drain tiles or wells) or a combination of the two. The purpose of the underdrainage system is to protect both the living filter and the aquifer. The living filter is protected from waterlogging and excessive salt buildup by the drainage network. The aquifer is protected because of the capability to collect and recycle any pollutants which may have broken through the living filter.

Many abbreviated land treatment systems do not have an underdrainage system. In essence, they have uncontrolled recharge. The Muskegon County waste-water management system can be used to illustrate how an underdrainage system will function. The United States Geological Survey in conjunction with the State of Michigan analyzed the underdrainage system at Muskegon County (U.S.G.S., 1978). A digital model analysis was undertaken. This analysis showed that if the effectiveness of the tile to collect drainage is reduced by 75 percent—a severe planning assumption—large areas within the land treatment site would become waterlogged. However, the effect outside the waste-water site would be negligible. With this type of information, conjecture concerning the potential effects a complete land treatment system would have on off-site wells is academic, when a properly designed underdrainage system is provided.

It is important to note that many systems referred to as land treatment are not designed properly and do not contain all of the components of a complete system. In some instances, the treatment plant does not work so the effluent is conveyed to a nearby field and discharged. This is not a land treatment system. Similarly an industry with a seepage bed does not have a land treatment system. The performance of such a system should not be used to evaluate the performance of a land treatment system.

## LAND TREATMENT IS SITE SPECIFIC

There is danger in repeating the design of a successful land treatment system at a new site. In this respect, land treatment differs significantly from the more conventional treatment plant technology. One should not repeat the Muskegon design throughout the country. A land treatment system must be designed to fit specific site characteristics. Even outspoken opponents of land treatment agree that it is possible to design a land treatment system that would not result in the pollution of underground-water resources.

To illustrate, one critic stated, "the impression should not be left that no waste materials can or should be placed on, in, or under the ground surface. Given the proper hydrogeological conditions and using appropriately designed facilities, there are situations when selected wastes can be disposed of into the ground without appreciably modifying the quality of the potable ground water." (Johnson, 1978).

## TECHNOLOGICAL CHANGES

There are a number of significant technological changes inherent in a land treatment system. Essentially land treatment transforms sewage treatment from a single purpose activity into a multipurpose activity, changes sewage treatment construction grants from subsidies into investments in the production of food and fiber, and requires the participation of a variety of disciplines to implement successfully.

A land treatment system constitutes a change from a single purpose to a multiple purpose program. A land treatment system provides agricultural open space near urban areas. It helps to preserve agriculture—a stated goal of many urban regions.

Land treatment can be integrated with floodplain management. The Department of the Army observed that the authorities in the Federal Water Pollution Control Act Amendments of 1972 regarding the acquisition of sites for land treatment of waste water when combined with the authorities of Section 73 of the Water Resources Development Act of 1974 offer an outstanding opportunity for multiple uses of floodplains while preserving green space and providing recreational opportunities. The Army spokesman inquired, "why not use our floodplains in urban areas for crop production, golf courses, forests, and other uses which can capitalize on the nutrients in our waste water and provide tertiary waste treatment at the same time?" (Ford, 1975).

Land treatment impacts on energy. The nitrogen in a year's flow of domestic waste water in the United States requires the equivalent of two and a quarter billion gallons of crude oil to replace as fertilizer.

The implementation of a land treatment system allows a community to transfer the cost of sewage treatment (a social inflationary cost) to the positive side of the ledger (an investment in the production of future food and fiber). This is the rationale behind the 10 percent bonus in Federal construction grants for land treatment systems. When the Federal government supports a land treatment system over a conventional treatment plant, the construction grant shifts from a Federal subsidy to an investment in the production of future food and fiber.

To design a land treatment system that is site specific will involve a number of disciplines. The design of such systems requires soil scientists, hydrogeologists, agriculturalists, chemists, biologists, and engineers. Physicists should play a role since land treatment is dealing with the basic laws of matter and thermodynamics. Land treatment seeks to use the forces in nature for the benefit of humanity. To do so successfully requires the involvement of several disciplines.

## EFFECTS OF LAND TREATMENT

Land treatment is replacing traditional technology at a rapid rate. A decade ago, land treatment was not considered seriously. Land treatment was viewed by some as a movement back to the Dark Ages. It was viewed as a return to honey buckets.

However, dramatic changes are now underway and land treatment is in the forefront. It is the encouraged alternative because it has been successful. In the light of its success, many policy makers have become the leaders or advocates of the approach.

Many persons in the engineering profession have chosen not to lead. This reluctance was observed by Eugene T. Jensen in 1971. Mr. Jensen told an American Society of Civil Engineers National Specialty Conference at Los Angeles: "I am ashamed to admit that . . . the old 'pros' in the field of water pollution control appear to be lagging. The people and Congress appear to have swept by us." The reaction to his challenge was somewhat predictable. Simply remove Mr. Jensen as Operations Chief of the Water Quality Control Office of EPA. The transfer took care of Mr. Jensen, but fortunately it did not take care of land treatment. Congress recognized the validity of his statement and has moved land treatment along legislatively as evident by the passage of the Clean Water Act of 1977 (P.L. 95-217). Having abdicated the leadership role, some in the engineering profession have chosen to attack land treatment. Articles have crept into the trade journals with titles or headings like "Land Disposal: A Giant Step Backward;" "Land Disposal: The Paper Tiger;" and "Land Disposal: The Environmental Blunder of the 20th Century." This terminology gives these authors away. They are so engrained with a disposal philosophy that land treatment is beyond their comprehension. The management and use of pollutants as resources out of place is not understood. Thus, these critics tend to gravitate to their familiar turf—disposal.

In the planning process, the attack on land treatment takes a different course of action. Here, technical distortions are introduced into the data to color the outcome.

A review of alternative treatment systems in

**Table 1. Key Chemical Raw Materials Advertised as Being Available from Michigan**

| *Organics* | *Inorganics* | *Minerals* |
|---|---|---|
| Benzene | Air gases | Cement |
| Butadiene | Bromine/derivatives | Clays |
| Chlorinated solvents | Chlorine | Gypsum |
| Ethylene | Hydrochloric acid | Lime |
| Phenol | Magnesium oxide | Natural gas |
| Propylene | Phosphoric acid | Petroleum |
| Styrene | Sodium carbonate | Salt |
| | Sodium hydroxide | |
| | Sulphuric acid | |

a New England State was undertaken. One system included a traditional treatment plant. The other system called for land treatment. Both of them were going to chlorinate their effluent. The study concluded that the two systems would cost essentially the same, and suggested that it would probably be easier to build a traditional treatment plant since there has been more experience in building such plants. When the cost breakdown of these systems is evaluated, a great disparity in estimating appears. The cost of the chlorination facility at the treatment plant was listed at $20,000. On the other hand, the cost of the chlorination facility at the land treatment system was listed at $360,000 (Town of Falmouth). This is simply one of many blatant examples of technical distortions which are being used in an effort to stymie the move toward land treatment.

## PERFORMANCE OF A LAND TREATMENT SYSTEM

Empirical information on the performance of a land treatment system can be gleaned from the monitoring of the Muskegon system. The excellent removal experience of traditional pollution parameters by the Muskegon County Wastewater Management System is well documented (U.S. EPA, 1977a). Therefore, emphasis here will be placed on performance with respect to organics.

The Muskegon system receives an unusually large assortment of organic compounds and this is not typical of a normal municipal system. Some of the exotic materials which enter the system is evidenced from the raw materials tabulated in Table 1. The industries which discharge their wastes into the system are tabulated in Table 2.

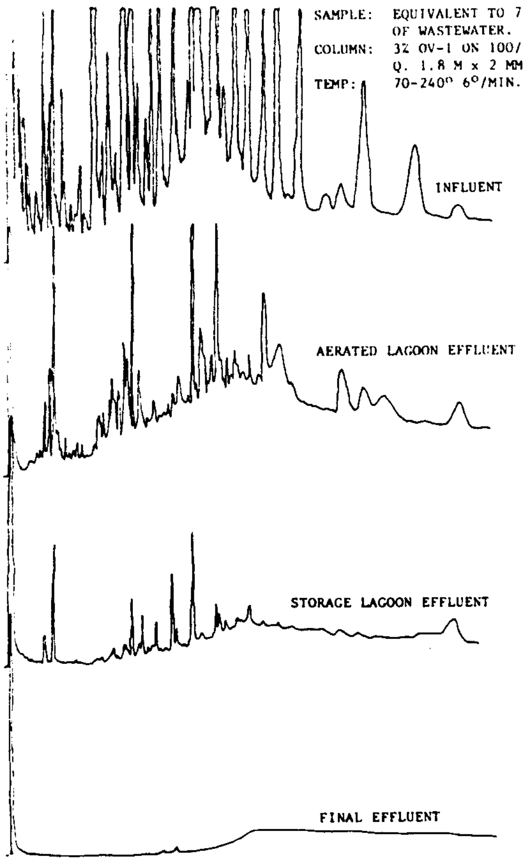
Another complicating factor is that the ground water in the nearby area has been contaminated by discharges from Lakeway Chemical Company's unsealed lagoons and Thermo-Chem Incorporated's seepage lagoons. In an effort to correct this ground-water pollution problem, two 8-inch purge wells are now in operation at the Lakeway Chemical site to pump the polluted groundwater to the Muskegon County Wastewater Management System for purification (communication with Andy Hogarth, Michigan Department of Natural Resources, August 10, 1977). The area north of the lagoons has shown steady improvement. However, the number of purge wells needed to correct the situation is likely to increase to six 8-inch wells before the ground-water pollution will be brought under control. There are two interesting observations that can be drawn from this experience. First, the Muskegon system is deluged with assorted organic discharges from industry each day (see Table 2). Second, it is serving as a pollution sink for a program that is seeking to clean up polluted ground water at a nearby industrial site. This ground-water cleanup program is enjoying a measure of success (W. Mich. Shoreline Regional Dev. Com., 1977).

The Robert S. Kerr Environmental Research Laboratory undertook a preliminary survey of toxic pollutants at the Muskegon system in August and September of 1976. The report emphasized that this was a preliminary survey conducted within a restricted time frame which considerably limited both sampling and analytical

**Table 2. Chemical Process Industries Discharging into the Muskegon Wastewater Management System**

| *Name* | *Employees* | *Products* |
|---|---|---|
| Burdick and Jackson Laboratories | 35 | Fine organic chemicals |
| East Shore Chemical Co., Inc. | 50 | Specialty chemicals |
| Lakeway Chemicals, Inc. | 140 | Dichlorobenzidine dihydrochloride, benzidine sulfate slurry |
| Story Chemical Corporation | 240 | Specialty fine chemicals |
| Webb Chemical Service Corporation | 25 | Industrial and laboratory chemicals |
| Thermo Chemical, Inc. | 25 | Disposal and reprocess chemicals |
| Thomas Solvent Company | 50 | Solvents |
| Fisons Limited[a] | – | Agrochemicals and pharmaceuticals |

[a] Not on line when survey was taken.



SOURCE: Robert S. Kerr; Environmental Research Laboratory, May 1977.

**Fig. 1. Comparison by gas chromatography of neutral extracts of waste waters from Muskegon system, August 10, 1976.**

efforts. Results from the analyses are presented in Figure 1. It concluded that:

> The Muskegon County Wastewater Treatment System was receiving for treatment waste waters consistently containing a great many organic pollutants of possible concern, including at least 11 compounds appearing on the EPA "List of Dangerous Pollutants." It is further apparent that, even though low levels of eight organic pollutants, including four toxic compounds, were indicated to survive the treatment sequence, the Muskegon System was relatively quite effective in removing organic pollutants from the waste water which it was treating. This is emphasized by the Figure which presents a comparison by gas chromatography of neutral extracts prepared from influent, aerated lagoon effluent, storage lagoon effluent, and final effluent samples. These chromatograms, which were obtained by chromatographing quantities of extract equivalent to 7.5 ml of each waste water, clearly show the very significant attenuation of organic pollutants across the system. It is very doubtful if any other types of treatment systems, with the possible exception of those utilizing heroic and very costly measures for polishing of final effluents would have been more effective than the Muskegon System in removing the organic pollutants occurring in the waste water being treated, especially since more than 60 percent of this waste water was comprised of industrial components. The presence in the final effluent of atrazine, trimethylisocyanurate, and those eight compounds which survived the entire treatment sequence is significant primarily because these substances necessarily traversed 5-12 ft (1.5-3.66 m) of sandy soil to reach the tile carrying the final effluent from the site. This comprises further evidence that organic pollutants, including chlorinated compounds of suspected toxicity, may survive and move significantly in the subsurface under proper conditions. Hence, the need is reiterated for developing definitive information concerning the movement and fate of organic pollutants in the subsurface environment in order that waste disposal methods which employ the subsurface as a pollutant receptor may be utilized to their full potential with minimum impact on ground water (U.S. EPA, 1977b).

This research did not show any evidence of ground-water pollution. Rather it simply showed that with heavy applications of waste water on a sandy soil, small quantities of organic compounds moved through 5-12 feet of sandy soil into the underdrainage system. It is necessary always to distinguish between ground water and a controlled underdrainage system when discussing a land treatment system.

The scientists called for more research. This is proper in light of the myriad of organic compounds which enter the environment.

The removal of chloroform by the Muskegon System is presented in Table 3. The influent averaged 870 parts per billion. A maximum contaminant level of 100 parts per billion for total trihalomethanes including chloroform is suggested for drinking water. The concentration which appears in the drainage tiles at Muskegon averages 6 parts per billion.

## CONCLUSION

Land treatment systems provide an opportunity to view sewage treatment as an investment in the production of food and fiber. They can be viewed as an investment in the future, rather than an increase in social costs. Land treatment provides

**Table 3. Removal of Chloroform by the Muskegon Land Treatment System (in parts per billion)**

| *Chloroform from sample date* | *Influent* | *Drainage water*[a] | *Percent removed* |
|---|---|---|---|
| 8-10-76 | 425 | 3 | 99.3 |
| 8-11-76 | 440 | 3 | 99.3 |
| 8-12-76 | 480 | 1 | 99.8 |
| 9-7-76 | 360 | 13 | 96.4 |
| 9-8-76 | 2645 | 10 | 99.6 |
| Average | 870 | 6 | 99.3 |

[a] Maximum contaminant level (MCL) 100 parts per billion for total trihalomethanes including chloroform.

our nation with a positive program to deal with a negatively perceived material, sewage. In addition, clean water is achieved. It is very doubtful if any other type of treatment system could be as effective as the Muskegon land treatment system in purifying water. Biases of the past pale when the merits of land treatment are evaluated objectively. We need to know more about it.

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# A Study of the Reuse of Reused Water

**by Edgar D. Smith[a], Robert M. Sweazy[b], George A. Whetstone[c] and R. H. Ramsey[d]**

## ABSTRACT

Lubbock is located in the center of the semiarid High Plains of Texas. Due to a shortage of adequate water-based recreational facilities in the region, the City Planning Department developed a scheme for converting an eight-mile intracity canyon into a linear park containing a series of small recreational lakes. Funds amounting to over 11 million dollars were committed to the project by a local bond issue and various governmental organizations.

The proposed lake makeup water is unique. Never before has a serious attempt been made to develop recreational lakes utilizing urban runoff and twice-used reclaimed municipal waste water as the sole source of supply. The majority of the makeup water will be obtained from wells on a farm which has received the major portion of Lubbock's secondary treated sewage since 1938 and used it for irrigation purposes.

In order to ascertain the suitability of utilizing this reclaimed sewage effluent for recreational purposes, the Texas Tech University Water Resources Center with the aid of an OWRT grant constructed a model system designed to simulate the lake project. Research performed to date includes algal growth potential studies, water quality analyses, bacteriological assays, viral assays, and fish life studies.

The practice of spreading sewage effluent on cropland or pastures has recently been accorded renewed attention. It is an effective means of withholding some of the plant nutrients which are not removed in secondary sewage treatment processes—and thus of aiding in the elimination of algae growths and other types of pollution in downstream waters. It should become increasingly popular as a serious consideration given to meeting the federally mandated 1985 deadline for zero discharge of pollutants from point sources.

The original motive for the employment of land-spreading of effluent was that of conservation of water and of the fertilizer values present. These concerns have increased in urgency over the years as populations have grown while the amount of water available has decreased due to legal and economic restrictions and to the exhaustion of fossil aquifers. In the late 1890s, Rafter (1897, 1899) studied the potential and the problems of sewage irrigation and devoted nearly 50 pages to a plant-by-plant description of sewage farms in the United States and Canada. He concluded that "the most efficient purification of sewage can be attained by its application to land. On properly managed sewage farms the utilization of sewage is not prejudicial to health." More recent studies have demonstrated that land application of sewage effluents is, indeed, a viable and accepted method of disposal albeit characterized by potential pollution problems to underground waters.

## EFFLUENT USE AT LUBBOCK, TEXAS

Lubbock, located on the High Plains of West Texas, has an economy based almost entirely on crop production. However, since the average annual precipitation is only approximately 0.46 meters (18 inches) while potential evapotranspiration exceeds 2 meters (79 inches), this agriculture is heavily dependent upon mining irrigation water from the underlying Ogallala formation, a fossil aquifer which underlies much of the High Plains area. Less than 5 percent of the 6 to 8 million acre-feet (740,000 to 990,000 hectare-meters) of water pumped from the aquifer each year is replenished via natural recharge. Consequently, during the last 30 years the water table in the Ogallala aquifer has been dropping at a rate of 1 to 3 feet (0.3 to 0.9 meters) per year (Hinesly, *et al.*, 1978). Due to this rapid decline in the aquifer levels and increased energy costs associated with pumping the water to the surface from increasingly greater depths, many farmers are now considering reverting to dry-land farming operations.

In 1938 the city of Lubbock contracted with a local farmer, Frank Gray, to take its entire production of secondary sewage treatment plant effluent for use in irrigation. In the 40 years that this operation has continued, the population has increased from 30,000 to 180,000. While some of the effluent is now diverted for use as power plant cooling water, and while a small portion of

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Discussion open until January 1, 1980.

he city's sewage is now treated at a second plant (with the effluent being sold to Texas Tech University for irrigation), Gray still receives up to 57,000 cu m per day (2 × $10^6$ $ft^3$/day) of effluents which he utilizes for the irrigation of about 1,200 hectares (3,000 acres) of land under his control and for supporting part of the water needs for about 800 hectares (1,977 acres) of his neighbor's lands (Wells, *et al.*, 1977). Gray grows alfalfa, grain sorghums, wheat, and lint cotton. His production costs for these crops are considerably less than those on other farms since he uses very little supplementary fertilizer.

## THE CANYON LAKES

Cognizant of the area's need for more water-based recreational facilities, there being virtually none within a 100-kilometer (62-mile) radius of Lubbock, the City Planning Department, in 1967, developed a scheme (The Canyon Lakes Project) for conversion of a 14-kilometer (8.7-mile) intracity arroyo (Yellowhouse Canyon) into a linear 555-hectare (1,371-acre) park including a series of small recreational lakes. The park facilities provide for picnicking, hiking, athletic events, and wildlife observation, as well as scenic driving and cycling. The lake system also offers canoeing, fishing, sailing, and other secondary contact recreational sports.

Since runoff from rainfall would be inadequate to maintain a suitable water level in the proposed system of lakes, Freese, Nichols and Endress, consulting engineers of Fort Worth (1971), investigated possible sources of water supply. They concluded that the best with respect to both quality and economy was that stored beneath Gray's farm as a result of his excessive irrigation over the past 40 years.

Thus, the recharged effluent recovered from Gray's farm plus incidental storm runoff from surrounding areas were chosen to be the primary sources of supply for filling and maintaining the lakes at levels suitable for maximizing recreational benefits. The project is apparently unique in that this seems to be the only serious attempt that has been made to develop recreational lakes utilizing urban runoff and twice-used (domestic and irrigation utilization) reclaimed municipal waste water as the sole sources of water supply. Figure 1 schematically depicts the water reuse strategy for the Canyon Lakes Project.

To secure title to this water supply, the city negotiated an additional contract with Gray. Under its terms, the city agreed to drill and equip 27 wells and provide the associated pipelines [14 miles (23 kilometers)] varying in diameter from 24 to 18 inches (61 to 46 cm) required to extract water from beneath Gray's farm. This water is to be used for irrigation of city cemeteries and parks as well as for maintaining the water level in the Canyon Lakes system. The wells have been gauged at flow rates which have ranged from 0.28 to 1.12 $m^3$/min (74 to 296 gal/min) with a cumulative daily production of 20,000 $m^3$/day (706,000 $ft^3$/day). The city pays Gray $2.64 per thousand cubic meters (264,000 gal) for all water extracted from beneath his farm.

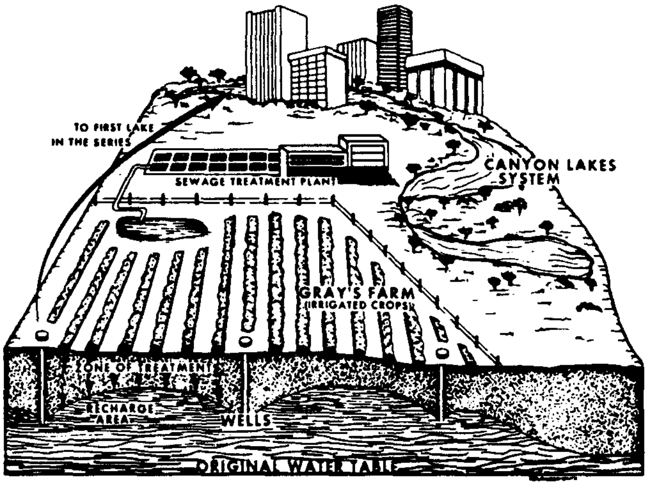

Fig. 1. Schematic of water reuse scheme.

## LAKE MAKEUP WATER

### Frank Gray's Farm

In grossly oversimplified terms, Gray's contract provides that the city will supply him with all the treated waste water from the Southeast Water Reclamation Plant on a daily basis, and that he will dispose of it in a manner that is acceptable to the local and State health authorities. For practical purposes, this means that the city pumps all its treated waste water to him, and he agrees to avoid letting any of it run off the surface of his land.

The contract has been highly beneficial to Gray in that he has had an abundant—indeed in most years, an overabundant—supply of "free" water that contains a good supply of plant nutrients to utilize in his crop production. Unfortunately for his sake, however, he has not generally been able to acquire additional land as rapidly as the city's production of treated waste water has increased. As a result, he has frequently been forced to utilize a portion of his land simply as a disposal area, and he has had to devote a significant fraction of his farm to the production of forage crops that are high-water users but that are not necessarily the

**Table 1. Characteristics of Effluent Applied on Land at Lubbock, Texas (Hinesly, *et al.*, 1978)**

| *Analysis in mg/l except as noted* | *Lubbock*[a] |
|---|---|
| Total hardness (as $CaCO_3$) | 290 |
| COD | 32-129 |
| BOD | 8-22 |
| Virus[+] | Probably present |
| Total organic carbon | 20 |
| Suspended solids | 8-19 |
| Total dissolved solids | 1194-1235 |
| Dissolved oxygen | 2-6 |
| Total N | 12-15 |
| $NH_4$-N | 2-4 |
| $NO_3$-N | 5-8 |
| Org.-N | 3-4 |
| Total P | 11-12 |
| Chloride | 318 |
| Calcium | 39* |
| Magnesium | 24* |
| Sodium | 344* |
| Potassium | 16* |
| Alkalinity | 236-262 |
| Sodium adsorption-ratio | 6 |
| pH, units | 7.4-7.5 |
| Fecal coliform, MPN per 100 ml | <200 |

[a] Freeze, Nichols and Endress Consulting Engineers Report, 1971. Sweazy and Whetstone, 1975. Wells, Sweazy, Gray, Jaynes, and Bennett, 1976 – Secondary effluent.

[*] Estimated from Northwest Lubbock treatment plant data reported by Sweazy and Whetstone, 1975.

[+] Bandy, *et al.*, 1977.

biggest money crops in the area (Wells, *et al.*, 1977). For many years prior to about 1970 he received an average of 1.5 $m^3$ (53 $ft^3$) of water per year for each square meter (10.8 $ft^2$) of land under his control while the normal water needs for cotton and grain sorghum, the principal crops grown in the area, exceed the annual precipitation by only 0.3 meters (12 in) to 0.5 meters (20 in). Since Gray is under contract to accept about three to five times as much effluent each year as his crops can utilize, he has developed a system of large holding ponds to help regulate the supply to coincide with irrigation demand.

Because of seepage from the holding ponds and infiltration from the applied waste water, which has historically greatly exceeded the crop requirements, the water table beneath his land has risen to within a few feet of the surface.

The quality of the waste water supplied to the Gray's farm is indicated in Table 1. It has been estimated that an average of 19,000 cubic meters per day (670,000 $ft^3$/day) of waste water percolates into the aquifer beneath the farm (Davenport, *et al.*, 1967). Seepage through the soil improves the quality of the water in many respects. The quality of the water underlying Gray's farm after percolating through the soil is indicated in Table 2. The water is clear, cool, colorless, odorless, and tasteless. It also meets secondary contact recreation requirements for coliform bacteria. The viral studies performed have failed to detect any virons in the water (Headstream, *et al.*, 1974).

The dramatic quality change associated with seepage through the soil (apparent from a comparison of Tables 1 and 2) occurs because the soil is a highly reactive system which has the ability to alter the composition of waste solutions added to it.

Properties inherent in the soil as a disposal medium include ion exchange capacity, filtration, buffering capacity, sorption, chemical fixation, microbial transformations, hydrodynamic and molecular diffusion, oxidation-reduction reactions, and precipitation.

In general it may be concluded that the combined action of the soil attenuation mechanisms mentioned above results in improved water quality in terms of bacterial and viral populations, BOD, ammonia, phosphorus, total organic carbon (TOC) and suspended solids. The powerful dissolution characteristics of water percolating through the soil, however, increase the total dissolved solids, hardness and $CO_2$ concentrations of the water. Figure 2 schematically depicts the fate of important water quality parameters as the water infiltrates into the underlying aquifer.

Lyon *et al.* (1952) defined soil ". . . as a natural body, synthesized in profile from a variable mixture of broken and weathered minerals and

**Table 2. Average Quality of Water from Well Field on Gray Farm* (Bandy, *et al.*, 1977)**

| | *Average* | *Maximum* | *Minimum* |
|---|---|---|---|
| pH | | 7.42 | 7.02 |
| Total hardness (as $CaCO_3$) | 693 | 996 | 464 |
| Total alkalinity (as $CaCO_3$) | 337 | 460 | 244 |
| Alkalinity (as $CaCO_3$) | 0 | 0 | 0 |
| Carbonates | 0 | 0 | 0 |
| Bicarbonates | 411 | 573 | 298 |
| Calcium | 127 | 195 | 82 |
| Magnesium | 92 | 124 | 47 |
| Sulfates | 297 | 468 | 196 |
| Chlorides | 406 | 570 | 304 |
| Fluorides | 3.8 | 5.8 | 2.2 |
| Potassium | 16 | 21 | 11 |
| Nitrate - N | 20 | 34 | 3.2 |
| Total phosphates | .12 | 1.56 | <.05 |
| Silica | 86 | 103 | 51 |
| Sodium | 269 | 361 | 137 |
| Carbon dioxide | 46 | 80 | 25 |
| Total dissolved solids | 1,798 | 2,155 | 1,487 |

[*] All values except pH are reported in mg/l.

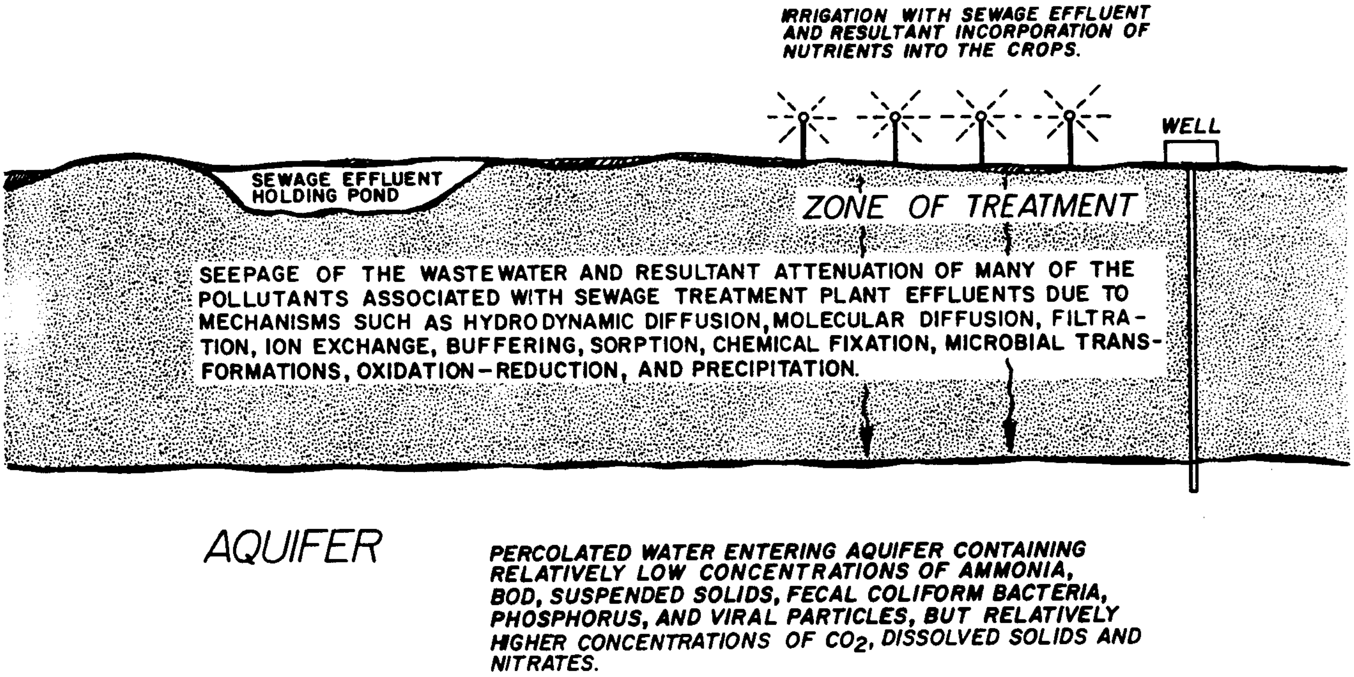

**Fig. 2. Fate of important water quality parameters associated with infiltration.**

decaying organic matter which cover the earth in a thin layer and which supplies, when containing the proper amounts of air and water, mechanical support and, in part, sustenance for plants." Although this definition infers an agricultural bias, it emphasizes the soil-plant nutrient relationship which underlies the reuse concepts of sewage effluent utilization for crop irrigation. The definition also emphasizes the essential components which form a soil system—minerals, organics, biological populations, solution, and air (Routson and Wildung, 1969).

It is not within the scope of this paper to review, in detail, the theory associated with these soil properties. The reader is referred to the EPA Technology Transfer publication (1976b) for an excellent discussion of soil pollutant attenuation mechanisms.

A few examples, however, should assist in the understanding of interactions with various substances often associated with secondary sewage treatment facility effluents. Solids contained in waste-water effluents will be filtered in the upper layer of soil. Bacterial removal from effluents passing through fine soils is quite complete. However, coarse, sandy, or gravelly strata; fractured rock; or limestone cavities may provide a passage for bacteria to travel several hundred feet from the point of application (EPA, 1976a).

Phosphorus exists in solution as an anion. However, in acid soil systems phosphorus reacts with aluminum and iron to precipitate out as insoluble compounds. In basic regimes phosphorus will react with calcium to precipitate as calcium phosphates (Routson, *et al.*, 1969). The removal of phosphorus is also a common result of percolation through clays or calcareous soils (Bandy, *et al.*, 1977). Soils in the Lubbock area are highly calcareous and have a practically inexhaustible capacity to react with and remove phosphates from water.

Nitrogen exists in the soil solution principally as ammonium ($NH_4^+$) and is relatively strongly sorbed by the soil solid phase due to cation exchange reactions common to many soil types. Nitrate either preexisting in the water or resulting from ammonium oxidation tends to remain dissolved and move with the soil solution. Thus, the addition of excessive concentrations of nitrate to the soil or nitrification occurring in the soil column may result in an increase in the concentration of nitrate ion in ground water. However, nitrate may also be converted to gaseous forms of nitrogen via microbial denitrification under anaerobic conditions and subsequently be lost to the atmosphere. Because nitrate has been demonstrated to be the causative agent of methemoglobinemia in children, its concentration in drinking water is limited in the EPA primary drinking water standards to 10 mg/l as nitrate nitrogen (EPA, 1976a).

One of the sites investigated in a study performed by Hinesly *et al.*, (1978) to obtain information about changes in chemical properties of soils subjected to long-term irrigation with sewage effluent, was the Gray farm. The researchers concluded, in part, that total concentrations of 15 elements and of organic matter indicate very few changes in the chemical status of soils associated with long-term sewage effluent applications at Lubbock. Other than increased concentrations of total and extractable P and total dissolved solids throughout the soil profile irrigated with sewage effluent for 38 years, no other differences in chemical composition of soils could be attributed to the use of waste water. The EPA-sponsored study states, however, that it appears that long-term disposal of sewage effluent on the farm may have increased dissolved solids and caused unacceptable concentrations of nitrates in ground water. This study concurs with a summary in an EPA publication (1976a) detailing design factors for land treatment of municipal waste-water effluents. The publication states, in part, that the most common public health concerns for land treatment systems are nitrogen and high total dissolved solids contamination of ground-water aquifers for water supply. A total dissolved solids limitation of 750 mg/ℓ is recommended for drinking water by Sorber (1973) since excessive concentrations of total dissolved solids can be harmful to people with cardiac or circulatory diseases.

The soils on the Gray farm at Lubbock, Texas, are classified as Acuff loam and Friona loam, with some Amarillo fine sandy loam along the north side of the farm. Most of the cultivated areas have slopes varying from practically level to about 3 percent, with runoff drainage to playas in the northern part, and to constructed ponds on the edge of Yellow House Canyon (mentioned earlier) in the southern part of the farm. All soil types have a horizon at 20 to 36 inches (51 to 91 cm) below the surface where $CaCO_3$ masses make up 30 to 40 percent of the soil volume. These soils are moderately permeable and have available water-holding capacities that range from 0.15 to 0.20 inches (0.38 to 0.51 cm) per inch (2.54 cm) depth. Evidently, no water impermeable layers exist between the soil surface and the Ogallala aquifer since surface applications of effluent have, over the years, resulted in higher water-table levels in the formation where it underlies the farm (Hinesly *et al.*, 1978). An annotated log of a typical test hole drilled on Gray's farm is presented in Table 3.

**Table 3. Geological Section in a Typical Test Hole on Gray's Farm (Engineering Department, City of Lubbock)**

| *Depth* | *Description* |
|---|---|
| 0- 10 | Buff sandy bentonitic clay |
| 20 | White sandy limestone |
| 30 | 20% white sandy limestone, 80% gray-buff fine calcareous sandstone |
| 40 | 10% light red fine-medium sand, 80% red & buff slightly-to-very argillaceous sandstone, 10% white sandy limestone |
| 50 | Red-buff argillaceous to calcareous fine sandstone |
| 60 | Same |
| 70 | Same |
| 80 | 20% white fine-medium sand, 80% buff very argillaceous sandstone |
| 90 | 70% white medium-coarse sand, 20% fine gravel, 10% buff argillaceous sandstone |
| 100 | White-yellow fine-medium-coarse sand |
| 110 | 40% same sand, trace fine gravel, 60% buff sandy clay |
| 115 | 20% same sand, trace fine gravel, 80% buff sandy clay, trace gray fine crystalline limestone |

Top Cretaceous: 115 ft (bottom of water-bearing formation)
Total Depth of Test Hole: 115 ft
Tubing: 113
Static Water Level: 35 ft

The aquifer under Gray's farm is a portion of the Ogallala Aquifer which serves as a source of municipal and agricultural water supply to portions of eight States. Though the aquifer is far from homogeneous, experience has indicated that the coefficient of storage and the specific yield may each be taken to be 0.15 in studies involving large tracts of land (Bell and Morrison, 1978).

### Model Studies Proposed

In order to evaluate the feasibility of utilizing the reclaimed waste water and runoff as makeup sources for the Canyon Lakes Project, the City of Lubbock contracted with the Texas Tech University Water Resource Center to serve in an advisory and research capacity during the planning, construction and operation of the project.

Texas Tech University is particularly well suited to explore the potential problems of using reclaimed treated waste water for the lakes since it has a similar store of percolated effluent beneath part of its experimental farm. The source of this effluent is the Northwest Water Reclamation Plant built by the city of Lubbock about one mile north of the Texas Tech University campus in 1962. This plant receives sewage from the campus and from portions of the city adjacent to the campus. At the time the plant was built, the city and Texas Tech negotiated a contract similar to the one the city already had with Gray. That is, Texas Tech agreed to accept all the effluent from the plant, and to utilize it for irrigation of agricultural lands on the campus.

The quantity of water produced by the Northwest plant has increased steadily from about 750 m$^3$ per day (26,000 ft$^3$/day) in 1962 to about 5,000 m$^3$ per day (177,000 ft$^3$/day) in 1975. It has remained roughly constant since that time. This water is stored in two unlined oxidation ponds with a total capacity of about three hectare-meters (24 acre-feet). On the Tech farm, as on Gray's farm, a large portion of the water from the ponds has infiltrated into the underlying aquifer and has raised the water table to within approximately 12 meters (40 ft) of the land surface. A well has been drilled near the oxidation ponds. The results of a typical analysis of the infiltrated effluent obtained from the underlying aquifer is shown in Table 4. The table also compares the quality of the well water on the Tech farm with that on Gray's farm. The similarity of the two waters prompted the Texas Tech Water Resources Center to develop a proposal for construction of a model on the University farm simulating the Canyon Lakes Project. Such a simulation could help anticipate and identify potential problems which might arise in the lakes due to the use of infiltrated effluent and surface runoff for lake makeup water. In particular, the high nitrate concentrations associated with the reclaimed well water provide nutrients for algae growth and are thus conducive to the manifestation of eutrophication problems in the lakes. Public health problems (viral and bacterial concentrations) are also associated with sewage effluents. Other factors considered to have an important bearing on the utilization of reclaimed sewage flows for lake makeup water include biochemical oxygen demand of dissolved and suspended organic compounds, suspended solid concentrations, heavy metals, toxics, dissolved oxygen, $CO_2$, and pH. These water quality parameters determine the safety and suitability of the water for recreational purposes and aquatic organism survival. Therefore, the specific objectives of the research associated with the reclaimed well water in the model lakes was to study algal growth potential, analyze water quality, make bacteriological assays associated with public health, and determine the suitability of such reclaimed waters for raising edible game and fish.

**Table 4. Average Quality of Water from Well Near Effluent Pond on Tech Campus and Gray Farm Well Water**

| *Parameter*+ | *Tech Farm Concentration* | *Gray Farm (Bandy et al., 1977)* |
|---|---|---|
| $NO_2$-N | 0 | – |
| $NH_3$-N | 0 | – |
| Organic-N | 0 | – |
| $NO_3$-N | 5.4 | 20 |
| Total dissolved solids | 2300 | 1798 |
| Hardness | 1350 | 693 |
| Ortho-phosphate | .04 | 0.12 |
| Dissolved oxygen | 2.9 | – |
| Alkalinity | 360 | 337 |
| $CO_2$ | 60 | 46 |
| Suspended solids | 9 | – |
| Total coliforms | < 10 | – |
| pH | 7.27 | 7.02-7.42 |
| COD | 91 | – |
| Iron | < .05 | – |
| Copper | < .05 | – |
| Calcium | 37 | 127 |
| Zinc | < 0.012* | – |
| Manganese | < 0.033* | – |
| Lead | < 0.26* | – |
| Nickel | < 0.073* | – |
| Cadmium | < 0.015* | – |
| Chromium | < 0.087* | – |
| Cobalt | < 0.063* | – |

* These values represent the sensitivity limits of the instrument below which concentrations of the elements may not be detected.

\+ All values except pH are reported in mg/l.

Such a model would provide the lead time necessary to allow effective remedies, if required, to be found and tested prior to actual filling of the recreational lakes. The proposal to construct and monitor such a simulation model was funded by OWRT of the U.S. Department of Interior. Water Resources Center researchers have had this work underway since September 1972.

## Simulation Model

Nine common-wall ponds, six feet deep (1.8 meters), each measuring 8 ft × 16 ft (2.4 × 4.9 meters) and having a volume of approximately 6,000 gallons (23,000 liters) were constructed near the lagoons on the Tech farm (Figure 3). The ponds were interconnected by valves in such a manner that they could be operated to simulate various numbers of lakes in series. Fifteen centimeters (6 in) of soil were placed on the bottom of each of the concrete tanks in order to better simulate the lake bottoms. Figure 4 is a photograph of the simulated lakes.

Makeup water for the concrete ponds was provided from the well located near the two unlined oxidation ponds. The result of a typical analysis of the Texas Tech farm infiltrated effluent obtained from the underlying aquifer is shown in Table 4.

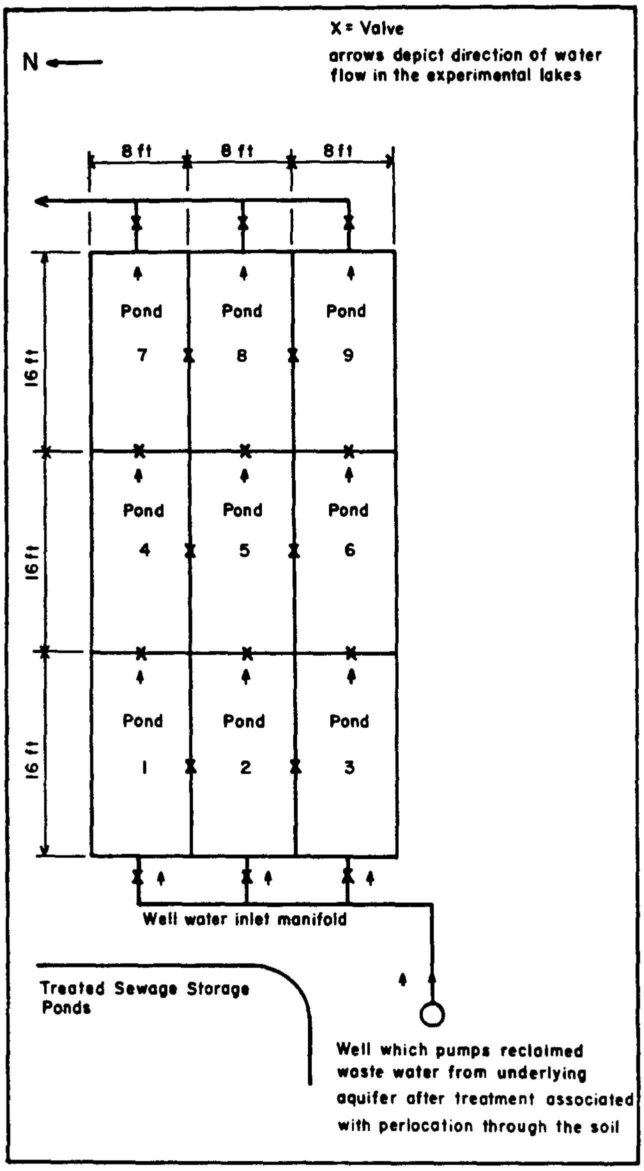

**Fig. 3. Model recreational lakes.**

### Algal Growth

On March 2, 1973, the ponds were first filled with well water. Three days later excessive growths of green filamentous algae appeared on the bottom of the first and second ponds in each of the three parallel series of three ponds each being tested. As growth continued, algal strands floated upward to form objectionable surface mats up to one-half inch (1.3 cm) thick. Growth was less in the second pond of each series. Algal growth in the third pond of each series was essentially absent.

By June 20, 1973, all the filamentous algae had disappeared. Subsequently blooms of diatoms occurred which manifested themselves either in the form of a fine brown scum or a floating brown gelatinous mat (Figure 5). These diatom blooms also greatly increased the turbidity of the water so that the bottom of the ponds became obscured. Diatom blooms recurred periodically, but were more prominent during warm weather. Blooms of a green unicellular algae were observed in some of the ponds in the early autumn.

### Algal Control

Several algicides were utilized to control algal growth in one series of ponds. Copper sulfate, while effective, was found to be uneconomical as a control strategy due to the high alkalinity of the water (360 ppm as $CaCO_3$) which led to precipitation of copper ions as $CuCO_3$. Research indicated that an improvement in economy can be obtained by dissolving copper sulfate in a solution of sodium citrate to form a copper citrate complex which remains soluble in waters with high alkalinity. Algal control by the sodium citrate method was achieved with 0.5 mg/l copper added every other day.

**Fig. 4. Photograph of model recreational lakes.**

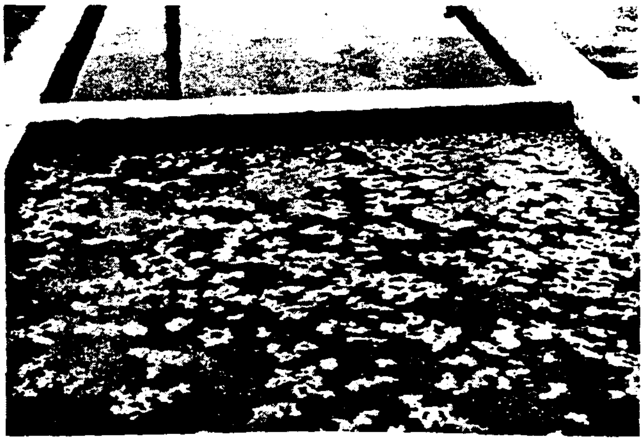

**Fig. 5. Algae bloom.**

The algicide Diuron, 3-(3,4-dichlorophenyl)-1, 1-dimethylurea, was found to be far more effective than copper sulfate. The addition of 0.2 mg/l every three days provided adequate control.

### Bacteriological and Viralogical Analysis

Coliform counts in the ponds remained below a MPN of 10/ml, thus meeting the secondary contact water recreational standards. Both fecal and total coliform counts in the well water remained less than 1/100 ml and viralogical tests were negative.

In an attempt to determine the effect of the reclaimed waste water on fish populations, several kinds (catfish, bass, *Tilapia*, minnows, sunfish hybid) were placed in the model lakes. Periodic seining of the ponds revealed that the catfish were doing well. The water regime also seemed to be conducive for catfish propagation as several generations of catfish were observed in the ponds. Surprisingly, the bass survived and grew somewhat, but no evidence of bass propagation was noted. Lack of suitable forage fish for the bass may have accounted for the lack of bass reproduction. The *Tilapia* managed to reproduce at least once. However, two major die-offs during the winter and spring of 1974-1975 indicate that the reported algae removing capabilities of the *Tilapia* may be curtailed in this geographic locality due to their apparent intolerance of the low temperatures. The hybrids become acclimated to their new environment but the minnows have apparently disappeared.

A major fish-kill followed the occurrence of an initial bloom of the crustacean *Diaphanosoma* and the pond snail *Lymanaea* in one of the ponds. Analysis of water quality data revealed no apparent reason for the preclusion of these two genera from the other eight ponds. Mid-morning analysis of the water in the affected pond indicated that it contained a dissolved oxygen concentration of 0.85 mg/l at the surface and 2.6 mg/l at the bottom. It is believed that the cumulative respiratory activities of the *Diaphanosoma, Lymanaea,* and algae, coupled with a relatively high-water temperature and lack of sufficient surface reaeration, resulted in a significant decrease of the dissolved oxygen concentration thus causing the fish-kill.

### Other Experimental Work

Throughout this research study, several chemical parameters indicative of water quality were monitored on a weekly or daily basis, in addition to the daily logging of physical observations, in an attempt to find a correlation between the appearance and disappearance of various algal species and the nutrient content of the pond water (Headstream, *et al.*, 1974). In particular, carbon dioxide, nitrate, phosphate, dissolved oxygen, chemical oxygen demand, dissolved solids and suspended solids, were determined routinely.

Additional experimental work in the model lakes has been related to algae growth potential, algae growth, and the effects of surface rain-water runoff containing nutrients associated with local farming activities.

## CONCLUSIONS

Continuing research performed with the simulation model at Texas Tech University indicated that the water recovered from Gray's farm will support several species of fish in the lakes, and will provide a much needed source of water-based recreation for the citizens of Lubbock. However, based upon incidences of excessive algae growth in the model ponds, Texas Tech University researchers predicted that the city will need to be vigilant and responsive in controlling excessive algal growth in the full scale lake system in order to maintain the lakes in an aesthetically desirable condition.

At present, several of the canyon lakes have been filled and stocked with fish and they are being utilized actively for recreation. As predicted the lakes have experienced algal blooms.

In February 1978, a new phase of Lubbock's waste-water reuse effort was initiated when Congress appropriated 5.6 million dollars for a pipeline to convey a portion of the waste water to farm land not previously irrigated with effluent, and 4.3 million dollars for a five-year EPA research and demonstration program to analyze the effects of land treatment of waste water on the new land as well as to continue and intensify studies on Gray's farm.

### Nonrecreational Benefits of the Project

The benefits, in addition to the provision of water-based recreation, which will result from this third cycle of use of a portion of the area's scarce water resources include the following:

1. the elimination of concentrations of unsightly and dangerous sources of pollution;

2. the conservation of a dwindling groundwater supply by use of reclaimed water for irrigation of parks and for industrial operations not requiring a potable-water supply;

3. the preservation of a significant amount of open space through the heart of an expanding metropolitan area;

4. the attainment of secondary economic gains due to factors such as increased income from tourism, higher real estate values and greater attractiveness of the city to new industry;

5. the creation of a positive upgrading influence upon adjacent, blighted areas through enhancement of their environment; and

6. the eventual strengthening of the Lubbock economic base.

In summary, domestic waste water produced by the city of Lubbock, Texas has been reused for more than 40 years. During this period of time an enormous quantity of twice-used water has infiltrated into the underlying aquifer. This water is now being recovered and put through a third cycle of reuse for recreational purposes. The project is an important step in demonstrating an economically feasible method of increasing water supplies in areas of shortage through water well technology and multiple reuse of existing water supplies.

## ACKNOWLEDGMENTS

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