Journal of Contaminant Hydrology, 8 (1991) 91–109
Elsevier Science Publishers B.V., Amsterdam

Application of the Monte Carlo simulation procedure to estimate water-supply well/septic tank-drainfield separation distances in the Central Wisconsin sand plain

Eric W. Harmsen[a], James C. Converse[b] and Mary P. Anderson[c]
[a]*Soil Science Department, North Carolina State University, Raleigh, NC 27695, USA,*
[b]*Agricultural Engineering Department, University of Wisconsin, Madison, WI 53706, USA,*
[c]*Department of Geology and Geophysics, University of Wisconsin, Madison, WI 53706, USA*

(Received February 19, 1990; revised and accepted December 17, 1990)

ABSTRACT

Harmsen, E.W., Converse, J.C. and Anderson, M.P., 1991. Application of the Monte-Carlo simulation procedure to estimate water-supply well/septic tank-drainfield separation distances in the Central Wisconsin sand plain. *J. Contam. Hydrol.*, 8: 91–109.

Using a three-dimensional groundwater-contaminant tracking model a number of Monte-Carlo simulations were performed to estimate the mean and standard deviations of the separation distances and minimum well depth required to avoid contamination of a pumping water-supply well by a nearby septic tank-drainfield. The range of conditions simulated represent those found in the Central Wisconsin sand plain. A sensitivity analysis revealed that the separation distances and minimum well depth are most sensitive to variations in the horizontal hydraulic conductivity, anisotropy ratio and horizontal regional hydraulic gradient. The mean and standard deviations of the theoretical safe separation distances and safe well depth are presented as contour diagrams that can be used for design purposes. An example is given for which the design separation distances and minimum well depth are determined for a hypothetical subdivision in the Central Wisconsin sand plain.

INTRODUCTION

Throughout the U.S., required water-supply well/septic tank-drainfield separation distances range from 15 to 85 m, but typically do not exceed 25 m (Yates and Yates, 1989). In the State of Wisconsin, a 15.24-m (50-ft) setback distance is required (Wisconsin Department of Labor and Human Relations, 1983). Although required separation distances may provide some protection, most codes (Wisconsin included) do not require that the water-supply well be located hydraulically up gradient from nearby drainfields. Consequently, in the residential subdivisions of the Central Wisconsin sand plain, water-supply wells located directly down gradient from drainfields are a common occurrence. Siting the well up gradient from the drainfield is an effective means of protecting the water supply from contamination. However, due to spatial constraints, placing wells up gradient from drainfields may not always be feasible.

In this paper, a rectangular well protection area (RWPA) is proposed. The RWPA is based on the combination of the long-term horizontal movement of contamination from a septic tank-drainfield and the maximum capture zone half-width for a well pumping for a specified period. This method provides detailed information on specific areas of a lot allowing one to judge where a contaminant source (e.g., the septic tank-drainfield) should not be placed. A minimum well depth based on the combination of the long-term vertical movement of contamination from a septic tank-drainfield and the upper half-dimension of a capture zone for a well pumping for a specified period is also considered.

METHODS

Monte Carlo analysis

A Monte Carlo analysis was performed using the three-dimensional contaminant tracking model described by Harmsen et al. (1991) to investigate the variability of the theoretical safe separation distances and well depth, for ranges of parameters typical of the Central Wisconsin sand plain. The approach used here is similar to that used by Carsel et al. (1988a) to define a safe distance down gradient of an agricultural field where pesticides entered the aquifer and travelled with the groundwater. The computer model used in this study, described by Harmsen et al. (1991), solves for the theoretical minimum horizontal and vertical distances between a rectangular contaminant source and a pumping well required to avoid well-water contamination. The aquifer is assumed to be unconfined, homogeneous, anisotropic and of infinite areal extent. The model considers advective contaminant transport only and ignores the effects of hydrodynamic dispersion.

The rectangle A-A′-B′-B, shown in Fig. 1, represents half of the proposed rectangular well protection area. Due to symmetry only the upper half of the actual rectangle is shown. The half-width dimension of the rectangle is defined as the safe Y separation distance (SYSD) and the distance from the well to the down-gradient edge of the rectangle (at A′-B′) is the safe X separation distance (SXSD). The distance from the well to the up-gradient end of the rectangle in theory should be infinite, but in practice corresponds to the up-gradient end of the residential lot or subdivision. The theoretical safe well depth (SWD) is defined in Fig. 2.

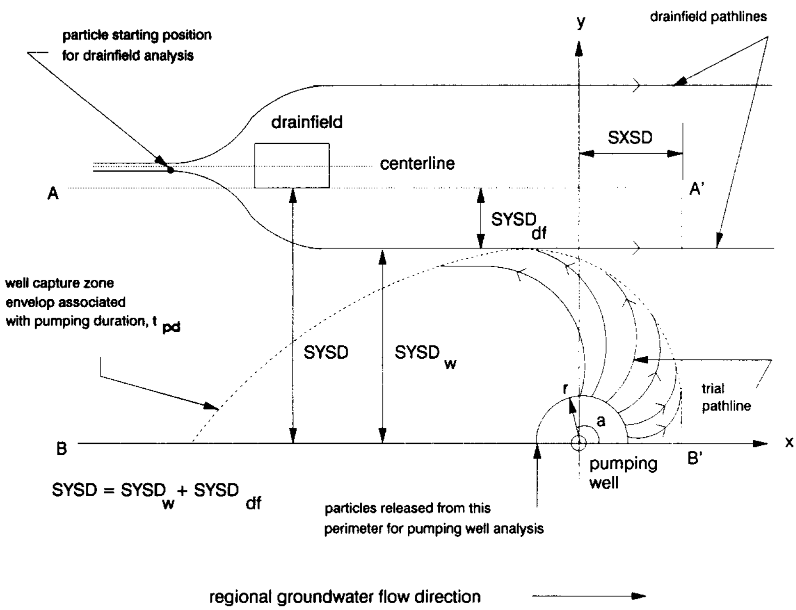

Fig. 1. Schematic diagram showing SYSD, SXSD, $\mathrm{SYSD_w}$ and $\mathrm{SYSD_{df}}$.

The purpose of using a Monte Carlo simulation was to approximate the probability distribution, mean and standard deviation for the model response variables: SYSD, SWD, $\mathrm{SYSD_{df}}$ and $\mathrm{SZSD_{df}}$, where the latter two variables are also defined in Figs. 1 and 2. In a Monte Carlo simulation, the generation of the response probability distributions requires running the model many times, each run referred to as a realization. Each realization depends on a different combination of input to the model and each combination of input is obtained by randomly selecting input values from their respective probability distribution functions.

The model input values were obtained by a constrained sampling scheme first developed by Mckay et al. (1979), called Latin hypercube sampling (LHS). LHS has been shown to yield more precise estimates of the response probability distribution than random sampling. The input values used were generated with a LHS computer program developed by Iman and Shortencarier (1984).

Table 1 gives the randomized input variables used in the analysis, their range, the type of probability distribution function used and the source of the information. The LHS computer program was used to generate combinations of the eight randomized variables, consisting of: horizontal hydraulic conductivity (K_h), anisotropy ratio (K'), effective porosity (n), regional hydraulic gradient ($i_{r,x}$), saturated aquifer thickness (D), drainfield recharge rate (R), drainfield width (W) and drainfield length (L).

TABLE 1

Range and characteristics of randomized input variables used in the Monte-Carlo simulations

Randomized variable[a]	Minimum	Maximum	Mean	Standard deviation	Distribution	Source
Horizontal hydraulic conductivity (K_h)	9.0	182.0	49.4	29.0	lognormal	Rothchild (1982)
Anisotropy ratio (K')	1.0	8.2	3.4	1.3	lognormal[b]	Rothchild et al. (1982); Weeks (1964); Dagan (1967)
Effective porosity ($1/n$)	2.6	6.4	4.1	0.7	lognromal[b]	Rothchild et al. (1982)
Regional hydraulic gradient ($i_{r,x}$)	-7.0	-5.1	-6.0	0.3	normal	Holt (1965)
Aquifer thickness (D)	7.5	60.0	34.0	8.7	normal	Rothchild (1982)
Drainfield recharge rate (R)	0.5	1.3	1.0	0.2	normal	Converse and Tyler (1987)
Drainfield width (W)	1.4	7.4	4.4	1.0	normal	Portage County (1988)
Drainfield length (L)	7.6	17.4	13.0	1.7	normal	Portage County (1988)

[a] K_h (m d^{-1}), D (m), R (m^3 d^{-1}), W (m), L (m).
[b] distribution assumed.

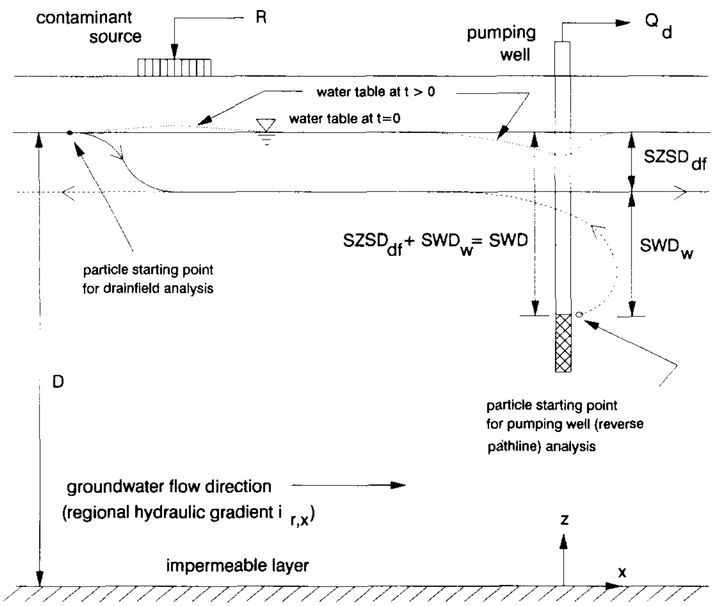

Fig. 2. Schematic diagram showing SWD and $\mathrm{SZSD_{df}}$.

During the sampling procedure, as a matter of convenience, n and $i_{r,x}$ were transformed to $1/n$ and $\ln(i_{r,x})$, respectively. For two of the variables, K' and $1/n$, lognormal probability distribution functions were assumed. The assumption of lognormality for these variables was based on limited data sets from the Central Wisconsin sand plain. The use of triangular distributions also could have been justified for these variables.

Safe separation distances and safe well depth

In this section, several simplifying assumptions which were used to reduce computer time and the scheme used to calculate the response variables SYSD, SXSD, SWD, $\mathrm{SYSD_{df}}$ and $\mathrm{SZSD_{df}}$, are discussed.

A homeowner survey, conducted at the Jordan Acres and Village Green subdivisions located in the Central Wisconsin sand plain near Stevens Point, Wisconsin (Fig. 3), was performed. Based on the responses of the 38 participants, the survey revealed that 80% of the homeowners watered their lawns. Of these, 40% had in-ground sprinkler systems of which, 12% used a single well for both domestic water use and lawn watering. Estimates of the size of this latter group, obtained from three local well installers, were 15%, 20% and 50%, respectively. Since this group of water users produce relatively large well capture zones (from running the sprinkler system) and consume water from these same wells, they can be considered the most vulnerable from a water

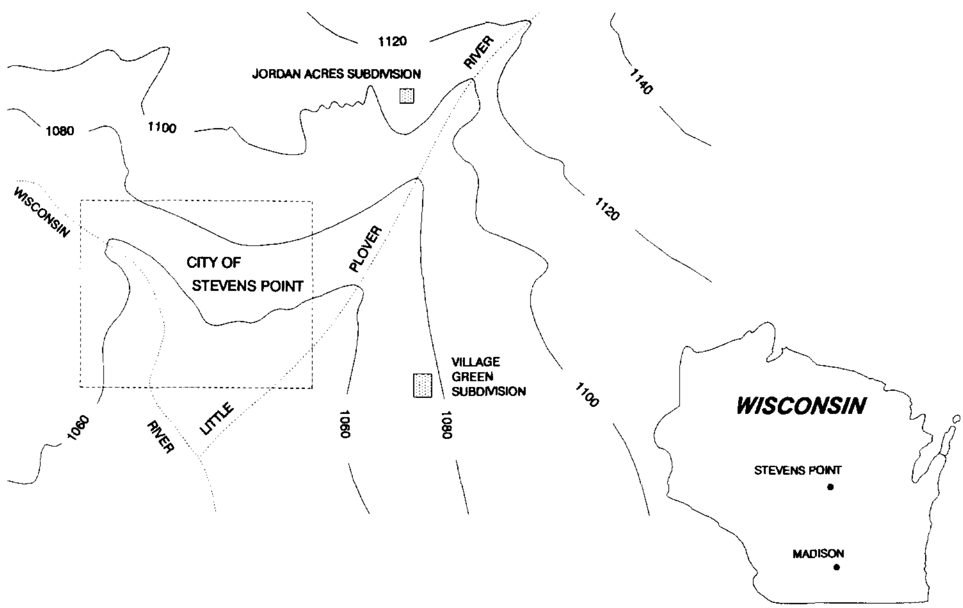

Fig. 3. Jordan Acres and Village Green subdivisions located near the City of Stevens Point, Portage County, Wisconsin. Water table elevation contours are shown in feet above mean sea level (after Holt, 1965).

quality standpoint. Two homeowners reported running their in-ground sprinkler systems 28 hours per week (4 hours per day, 7 days per week) and 30 hours per week (6 hours per day, 5 days per week), respectively.

From the local well installer survey it was learned that the typical discharge rate, Q, for the in-ground sprinkler systems, ranges from 6.8 to 11.4 $m^3 h^{-1}$ (30 to 50 $g\,min^{-1}$). The lower limit on Q would be for a homeowner who waters his lawn frequently with a hose connected to the domestic pump (e.g., $Q = 1.1\,m^3 h^{-1} = 5\,g\,min^{-1}$).

The high flow rate-high frequency pumping reported above is associated with the driest part of the year, ranging from one to three months during the summer when lawns located on soils with very low available water-holding capacity require frequent watering. Therefore, for Central Wisconsin sand plain conditions, the duration of pumping (t_{pd}) and consequently the period of time contaminants may be moving to a well due to the abnormally large capture zone (caused by lawn watering), may be anywhere from one to three months.

Because pumping is performed for a certain number of hours per day, the pump discharge rate is not constant [i.e., $Q = Q(t)$]. To simplify the analysis, however, the discharge rate was assumed to be steady-state. Harmsen (1989) has shown that the particle trajectories are essentially identical under intermittent, daily cyclic pumping and steady-state pumping, when the total volume of water pumped per day is the same in both cases. Assuming a steady daily discharge rate, Q_d, the computer times was reduced considerably and made possible the use of a reverse pathline approach (Shafer, 1987) for determining $\mathrm{SYSD_W}$ and $\mathrm{SWD_W}$.

Based on the conditions described above for the Monte-Carlo simulation, the pumping duration, t_{pd}, was varied from 0 to 100 days and the daily discharge rate, Q_d, was varied from 0 to 45 $m^3 d^{-1}$. The maximum value used for Q_d is equivalent to pumping 11.4 $m^3 h^{-1}$ (50 $g\,min^{-1}$), 4 hours per day.

The well screen length used for all simulations was 1.15 m. This value was selected as an average between the two lengths used by homeowners in the Central Wisconsin sand plain subdivisions (0.91 m and 1.22 m). In the simulations, the top of the well screen was placed 3 m below the static water tabe. A homeowner survey conducted in June of 1987, in the two subdivisions (Central Wisconsin Groundwater Center, unpublished data, 1987), revealed the average well depth to be approximately 10 m with the average depth to water at approximately 7.5 m (i.e., 2.5 m depth of penetration). Several local well installers indicated that the driven point wells were typically driven as shallow as possible, with as little as 1.5 m of water from the bottom of the screen to the static water table.

Figures 1 and 2 show the plan view and cross section, respectively, for the drainfield analysis. For the plan view (Fig. 1), the particle representing the contaminant was released on the water table surface 7.5 m up gradient of the drainfield, slightly offset from the drainfield center-line a distance of 0.3 m in the negative y direction. In some cases, the particle passed beneath the projected area of the drainfield, owing to the small hydraulic mound. When this situation occurred, the simulation was temporarily stopped, the particle was moved out to the edge of the drainfield (in the negative y direction) and the simulation was then continued. The simulation was terminated when the particle had passed beyond the drainfield in the positive x direction and movement in the y direction became negligible. The width of contamination, or the safe Y separation distance for the drainfield, $\mathrm{SYSD_{df}}$, was calculated by subtracting the final particle y coordinate from the y coordinate associated with the edge of the drainfield.

For the cross sectional simulation (Fig. 2), the particle was released 7.5 m up gradient from the drainfield along the drainfield center-line at an elevation corresponding to that of the static water table minus 0.03 m. After the particle was released, the hydraulic mound produced by the drainfield forced the particle downward until it moved far enough down gradient from the drainfield so that its path became parallel with the horizontal regional flow, at which time the simulation was terminated. The safe Z separation distance associated with the drainfield, $\mathrm{SZSD_{df}}$, was then calculated by subtracting the final particle z coordinate from the static water table elevation.

For the pumping well capture zone simulation, particles were released successively from points along the perimeter of a 1.5-m radius circle surrounding the well (Fig. 1), at the elevation of the top of the well screen. The elevation of the top of the well screen generally corresponded with the elevation associated with the capture zone's maximum width (Harmsen, 1989). A negative daily discharge rate ($-Q_d$) and a negative regional hydraulic gradient ($-i_{r,x}$) were used to produce reverse flow. After the particle was released (at $t = 0$) it was allowed to travel until $t = t_{pd}$, the specified pumping duration in days.

The first particle was released at $r = 1.5$ m, $a = 0$ (Fig. 1), where r is the radius and a is the angle between the x-axis and the line intersecting the well and the particle starting position. Due to symmetry the particle remained on the x-axis. After increasing the angle, a, by a small amount, the second particle was released and allowed to travel until $t = t_{pd}$. The maximum particle y coordinate, associated with $t = t_{pd}$, was recorded. By successively releasing particles along the perimeter for increasing values of a, the overall maximum y value, Y_{max}, was determined by a trial and error method. By definition Y_{max} is the safe Y separation distance due to the well (SYSD_W). The overall safe Y separation distance was then calculated from the relation, $\text{SYSD} = \text{SYSD}_W + \text{SYSD}_{df}$.

To determine the safe well depth (SWD), a reverse pathline analysis was also used. Figure 2 shows the cross section illustrating the method for determining the SWD. The particle was released at the elevation of the top of the well screen at $y = 0$ and $x = 0.6$ m, and the top of the well screen was placed at one-half the aquifer thickness. The particle was allowed to move through the flow field until $t = t_{pd}$ or until the particle entered the contaminated zone ($z > D - \text{SZSD}_{df}$). If the particle entered the contaminated zone and t was less than t_{pd}, the top of the well screen was lowered to 0.25 D. If, however, the particle did not enter the contaminated zone during the time period from $t = 0$ to $t = t_{pd}$, the top of the well screen was raised to 0.75 D. This interval halving was continued until the SWD was determined to within 0.3 m.

RESULTS AND DISCUSSION

Figures 4–9 show the variation of the SYSD, SXSD and SWD means and standard deviations with pumping duration (t_{pd}) and the daily volume of water pumped (V_{daily}). The response surfaces shown in Figs. 4–9 were constructed by linear interpolation of discrete response values obtained throughout the region of interest (each discrete value of the response was obtained from an individual Monte-Carlo simulation). Several characteristics are common to the contour diagrams: (1) maximum response sensitivity occurs near the ordinate and abscissa. The mean response surfaces (Figs. 4,

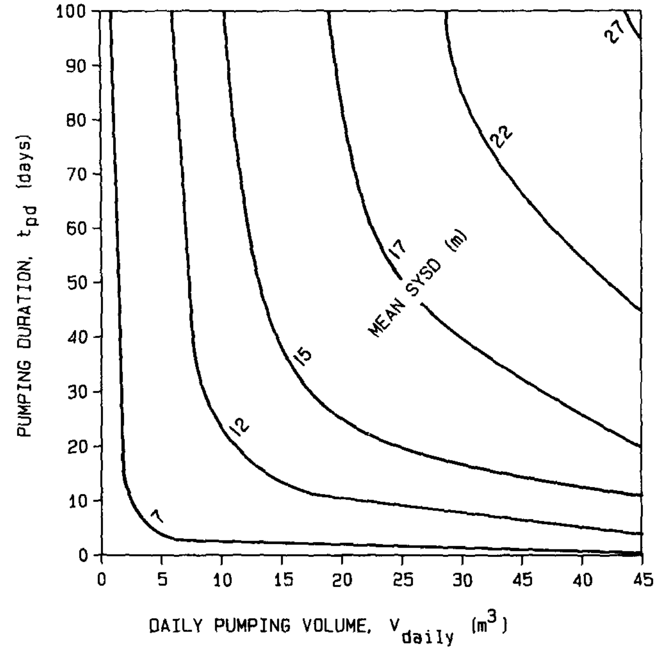

Fig. 4. Mean safe Y separation distance, $\text{SYSD}_{\text{mean}}$, with t_{pd} and V_{daily}.

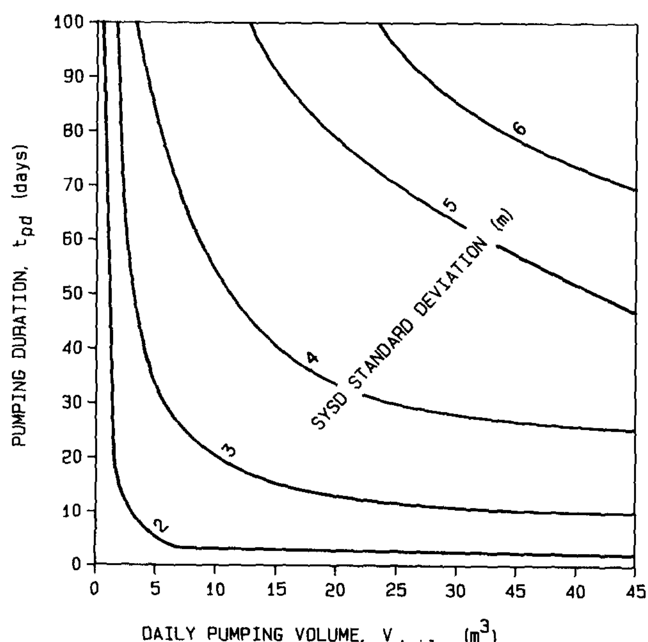

Fig. 5. Standard deviation of the safe Y separation distance, SYSD_{sd}, with t_{pd} and V_{daily}.

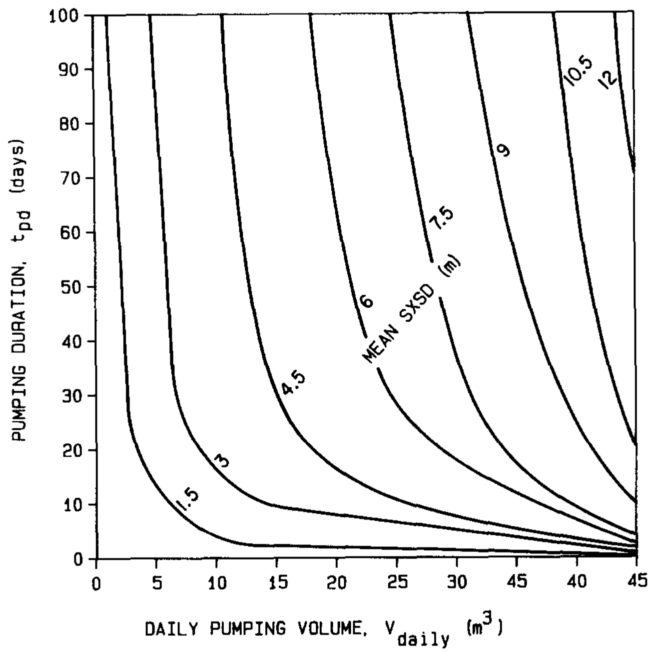

Fig. 6. Mean safe X separation distance, SXSD_{mean}, with t_{pd} and V_{daily}.

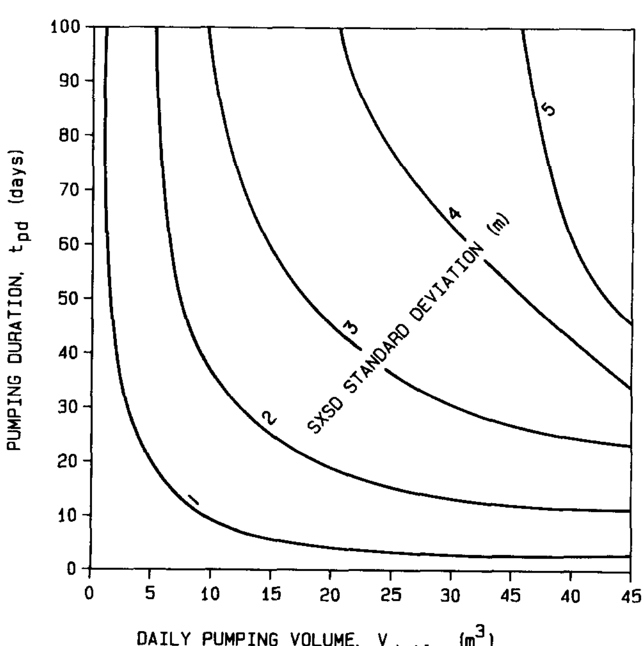

Fig. 7. Standard deviation of the safe X separation distance, SXSD_{sd}, with t_{pd} and V_{daily}.

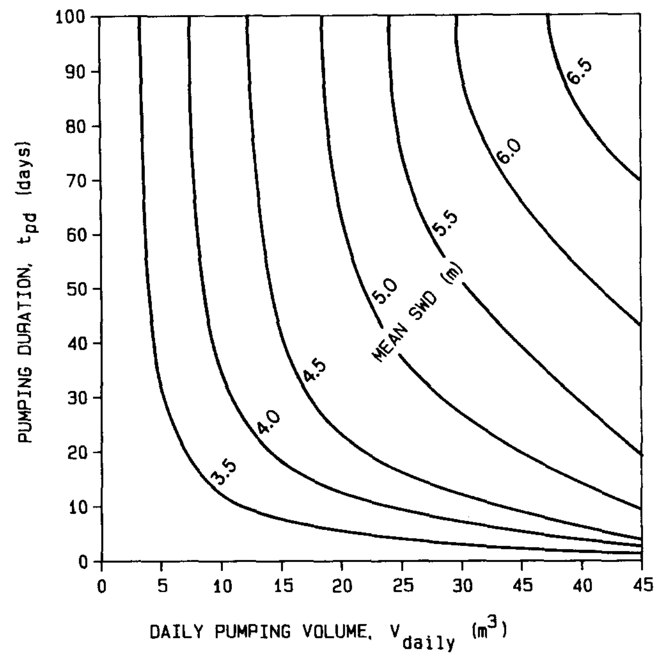

Fig. 8. Mean safe well depth, SWD_{mean}, with t_{pd} and V_{daily}.

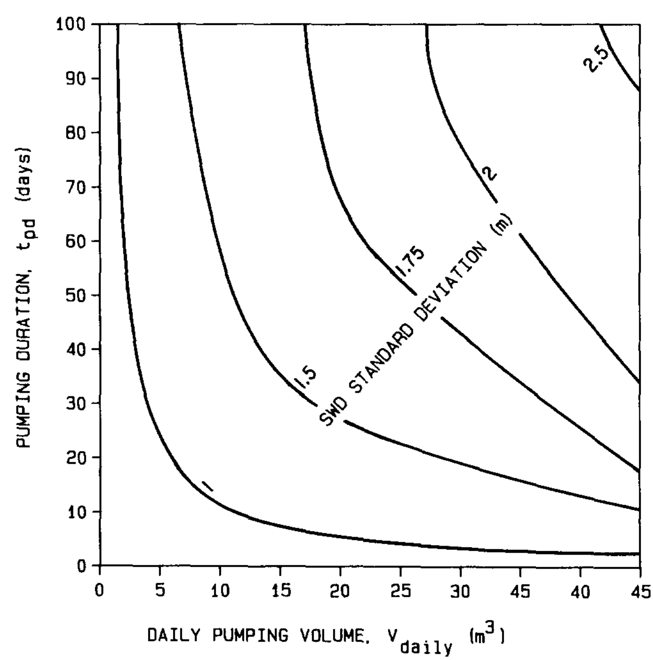

Fig. 9. Standard deviation of the safe well depth SWD_{sd}, with t_{pd} and V_{daily}.

6 and 8) were especially sensitive at high daily pumping volumes and low pumping durations; (2) relatively large response values (i.e., magnitudes) occurred close to the origin; and (3) for large daily volumes and pumping durations the response surface was generally less sensitive (i.e., response surface was flatter).

The number of runs required to achieve convergence in a Monte-Carlo simulation is not known a priori. Use of the term convergence here refers to the minimum number of runs (or minimum sample size) required to obtain a sufficiently accurate estimate of the response probability distribution function. In this study, a sample size of 200 was considered sufficient to achieve convergence. Considering the number of randomized variables in this study (eight), a sample size of 200 is consistent with what others have found necessary for convergence. Persaud et al. (1985) modeling noninteracting, steady-state solute transport movement in heterogeneous soil, using two randomized variables, found 200 to be the minimum sample size to achieve convergence. Carsel et al. (1988b), modeling transient interacting solute movement with a model using sixteen randomized variables, found a sample size of 500 necessary to achieve convergence. Figure 10 shows the mean SYSD, SXSD, SWD, $SYSD_{df}$ and $SZSD_{df}$, for selected Monte-Carlo simulations as a function of sample size. The relatively good convergence, even for a sample size of 100, can be attributed to the use of the latin hypercube sampling technique and to the insensitivity of the model to several of the randomized variables.

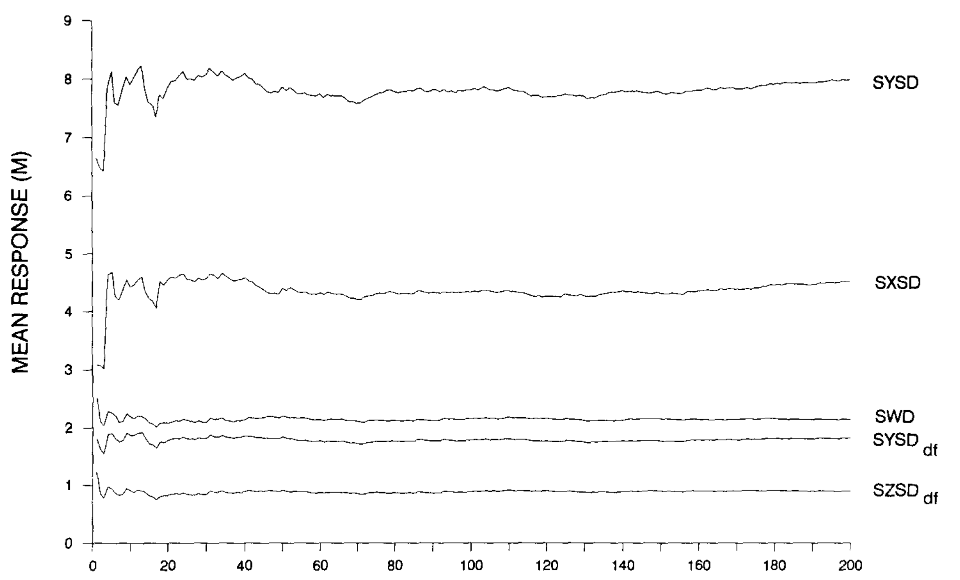

Fig. 10. Calculated mean SYSD, SXSD, SWD, $SYSD_{df}$ and $SZSD_{df}$ for various sample sizes.

The histograms of the response variable distributions indicated normality to weak lognormality for most cases. The calculated means and standard deviations of the response variables were based on the assumption of normality except in the case of the SXSD for $V_{\text{daily}} < 10\,\text{m}^3$. In this region the SXSD exhibited strong lognormality, and hence, the calculated SXSD means and standard deviations were based on the assumption of lognormality. Plots of the individual randomized variables with SYSD, SXSD and SWD, generally showed a large degree of scatter, with the horizontal hydraulic conductivity plots showing the greatest degree of correlation.

The responses SYSD_{df} and SZSD_{df} represent the maximum distance the contaminant moves outward from the edge of the drainfield and the maximum depth of contamination, respectively. The mean, standard deviation and maximum SYSD_{df} were 3.6, 1.5 m and 9.2 m, respectively. The maximum SYSD_{df} indicates that under certain Central Wisconsin sand plain conditions the effect of lateral contamination movement may be significant relative to the dimensions of a typical subdivision lot (e.g., 30 m × 30 m). The mean, standard deviation and maximum SZSD_{df} were 1.0, 0.5 and 3.1 m, respectively. The mean of 1.0 m is consistent with nitrate-N plume thicknesses observed at two unsewered subdivisions in the Central Wisconsin sand plain (Harmsen, 1989).

A sensitivity analysis was performed on the results and the randomized variables were ranked according to their importance. Ranking was achieved by the following procedure: linear regression equations of the general form:

$$\text{response}_i = c_{1,i}[\ln(K_h)] + c_{2,i}(k') + c_{3,i}(1/n) + c_{4,i}[\ln(i_{r,x})] + c_{5,i}(D) + c_{6,i}(R) + c_{7,i}(W) + c_{8,i}(L)$$

were determined, where $c_{k,i}$ is the regression equation variable coefficient associated with the kth randomized variable ($k = 1$–8) and the ith response variable ($i = 1$–5). From the five regression equations, the student T-ratios for each of the regression equation coefficients were calculated and ranked according to their absolute value. The results of the ranking are shown in Table 2. Slight improvement in the correlation between the response variables and the randomized variables was achieved by transforming K_h to $\ln(K_h)$. The SYSD, SXSD and SWD results shown in the Table 2 were based on $t_{\text{pd}} = 5$ days and $V_{\text{daily}} = 5\,\text{m}^3$.

The T-ratio represents the ratio of the regression equation variable coefficient to the standard deviation of the estimated coefficient. If the absolute value of the T-ratio is greater than about 2.5, the coefficient is considered to be significantly larger than the error associated with the estimate (Box et al., 1978) and consequently, the variable associated with the coefficient helps to predict the response. Because of the likelihood of nonlinearities in the true response, the intent of using the linear model was only to assist in ranking the variables. In fact, the diagnostic information provided with the regressions

TABLE 2

The randomized variable coefficient T-ratios for SYSD, SXSD, SWD, $SYSD_{df}$ and $SZSD_{df}$.

Response variable	Randomized variable	Coefficient T-ratio
SYSD	ln (K_h)	−48.60
	K'	42.22
	ln ($i_{r,x}$)	−29.20
	R	12.80
	W	−5.22
	$1/n$	4.76
	L	1.57
	D	−0.56
SXSD	ln (K_h)	−45.52
	K'	29.20
	ln ($i_{r,x}$)	−27.37
	$1/n$	3.42
	L	1.11
	R	−0.71
	D	−0.36
	W	−0.07
SWD	ln (K_h)	−24.36
	K'	−14.99
	ln ($i_{r,x}$)	−14.09
	R	−14.09
	W	−3.33
	L	−2.72
	$1/n$	1.66
	D	−0.79
$SYSD_{df}$	ln (K_h)	−36.78
	K'	24.16
	ln ($i_{r,x}$)	−21.95
	R	11.36
	W	−4.53
	$1/n$	−1.39
	L	1.17
	D	−0.45
$SZSD_{df}$	ln (K_h)	−25.41
	ln ($i_{r,x}$)	−14.83
	K'	−12.88
	R	8.58
	L	−4.05
	W	−0.84
	D	0.57
	$1/n$	0.17

indicated, in general, that a linear model was not sufficient for predicting the response surfaces. Therefore, when ranking the variables in Table 2, if two of the randomized variable T-ratios were similar, the one with the higher ratio was ranked higher only for convenience, and the influence of the variables on the response should be considered to be about the same. Furthermore, for this analysis, the randomized variables should probably not be considered significant unless the T-ratio is around 10 or greater, since for T-ratios less than this, no visible trend could be discerned in the data when plotting the response variable against the randomized variable.

Several conclusions can be drawn from Table 2. For all of the response variables, the three most important randomized variables were ln (K_h) (horizontal hydraulic conductivity), K' (anisotropy ratio) and ln $(i_{r,x})$ (regional hydraulic gradient). For all of the response variables except SXSD, the drainfield recharge rate, R, ranked fourth in importance but was only marginally significant. The lack of influence of R on SXSD was expected, since the SXSD calculation did not include the effects of the drainfield. Four of the randomized variables (n, D, L, and W) had little or no influence on the model response variables.

For the simulation with $D = 7.6$ m, the smallest D value in the sample population, a safe well depth could not be determined owing to the small value of D. Even though the well screen was lowered to the impermeable layer, the reverse particle trajectory still contacted the contaminated zone. This suggests that in thin aquifers there may be no safe well depth. (In this case the well could still be protected by using the SYSD.) Thus, despite the results of the sensitivity analysis, D can become a critically important parameter at small values.

The sign of the T-ratio is the same as that of the predictor coefficient and therefore can be used to determine the direction the variable moves the response surface. For example, the T-ratio for the ln (K_h) was negative indicating that with increasing ln (K_h) the response decreases. It is interesting to note that the sign of the T-ratios for several of the variables changed with response variable. For example, the T-ratio for K' was positive for SYSD and SXSD, meaning that SYSD and SXSD increased with increasing K'; but for SWD and $\mathrm{SZSD_{df}}$ the sign of the T-ratio was negative, meaning that SWD and $\mathrm{SZSD_{df}}$ decreased with increasing K'. The T-ratios were calculated for SYSD, SXSD and SWD, for the case where t_{pd} and V_{daily} equalled 100 days and 27 m^3, respectively. The results were essentially the same as those shown in Table 2.

Pratical application

As an example of the use of results given in Figs. 4–9, suppose that a planning agency in the Central Wisconsin sand plain wished to implement well protection measures for a new subdivision development. As a first step the agency would need to determine a realistic daily volume (V_{daily}) and pumping duration (t_{pd}) for the homes in the subdivision. Suppose that a design pumping rate, frequency and pumping duration of $2\,\text{m}^3\,\text{h}^{-1}$, 4 hours per day and 50 days, respectively, are selected. Therefore, $V_{\text{daily}} = 8\,\text{m}^3$ and $t_{\text{pd}} = 50$ days. For purposes of discussion assume that the planning agency has little site specific information on the hydrogeology and, therefore, wish their setback distances and minimum well depth to be conservative. By using the mean plus two standard deviations the resulting separation distances and well depth necessary to provide well protection can be determined for the worst case conditions (with 95% confidence) expected in the Central Wisconsin sand plain (i.e., for the range of conditions covered in Table 1). From Figs. 4–9 the estimates of the design safe X and Y separation distances and the design safe well depth are:

$$\text{SYSD}_{\text{design}} = \text{SYSD}_{\text{mean}} + 2(\text{SYSD}_{\text{sd}}) = 12 + 2(4) = 20\,\text{m}$$

$$\text{SXSD}_{\text{design}} = \text{SXSD}_{\text{mean}} + 2(\text{SXSD}_{\text{sd}}) = 4 + 2(2) = 8\,\text{m}$$

$$\text{SWD}_{\text{design}} = \text{SWD}_{\text{mean}} + 2(\text{SWD}_{\text{sd}}) = 4 + 2(1.5) = 7\,\text{m}$$

The design safe separation distances and safe well depth can be used to avoid direct contamination of a well by a nearby septic tank-drainfield in the Central Wisconsin sand plain and at other locations, provided the conditions are similar to those covered in Table 1.

However, if certain conditions exist which violate the model assumptions, the design estimates may be inadequate and their use may not prevent well-water contamination. Knowledge of the groundwater flow direction beneath the subdivision is essential for properly placing a water-supply well with respect to a septic tank-drainfield. In many cases, available water table elevation maps are inadequate at the subdivision scale. If the assumed flow direction is incorrect, then the application of the model results cannot be expected to provide supply-well protection. Therefore, a number of water table piezometers should be installed around the subdivision to determine the direction(s) of the local flow system. The exact number of piezometers required will depend on the complexity of the groundwater flow field.

An attempt should be made to determine the subdivision position within the regional groundwater flow system. Estimation of the design safe well depth is based on the assumption of regional horizontal flow. If the subdivision is in a recharge area, vertical flow may render the design safe well depth inadequate. In some cases it is difficult to determine whether a particular site is in a recharge, discharge or transitional area. Regional and local topographical features may give an indication of the vertical flow regime, but nested piezometers may provide more direct information on the vertical flow occurring at point locations. Where downward vertical flow occurs near the water table it may be possible to use the design safe well depth, but add to it the distance over which the downward gradients occur.

The stratigraphy at the subdivision should be determined. Geophysical techniques (e.g., ground-penetrating radar) may be an economical way to determine the gross variations in aquifer stratigraphy. Estimation of the safe separation distances and safe well depth are based on the assumption that the aquifer is homogeneous. If significant variations are observed (e.g., layers of contrasting soil texture), the design safe separation distances and well depth should be used with caution. If, for example, a highly permeable gravel layer exists at some depth below the water table, the septic tank-drainfield plume may move preferentially downward and into the gravel layer. This condition would violate the assumptions upon which the safe separation distances and safe well depth are based.

The background concentration in groundwater at the up gradient end of the subdivision should be determined. If the subdivision is located in the lower portion of a groundwater basin and there is significant nonpoint groundwater pollution occurring up gradient (e.g., agricultural activity), then the background contaminant (e.g., nitrate-N) concentrations will probably be elevated. In this case the subdivision may contribute relatively dilute water and concentrations of nitrate-N may increase with depth below the subdivision (Harsen, 1989) and a safe well depth may not exist.

It should be noted that the analysis discussed in this paper does not consider aquifer recharge by rainfall and, consequently, the method does not address the prevention of well-water contamination by nonpoint pollutants such as lawn fertilizers and pesticides.

Septic tank-drainfields in subdivision developments are, in effect, randomly scatter throughout the subdivision. In practice, therefore, it will be difficult to prevent septic tank-drainfields on up gradient lots from falling within the rectangular well protection area (RWPA) of a down gradient lot. Even if the RWPA itself is maintained free of contaminant sources, over long distances the effects of contaminant dispersion may cause contaminants from up gradient lots to enter a specific RWPA.

SUMMARY AND CONCLUSIONS

A Monte-Carlo analysis using a three-dimensional groundwater-contaminant particle tracking model was described. The mean and standard deviations of the theoretical safe well/septic tank-drainfield separation distances and safe well depth for a range of conditions typical of the Central Wisconsin sand plain were determined. A sensitivity analysis using a regression method demonstrated that the most important input variables were the horizontal hydraulic conductivity, permeability anisotropy ratio and regional hydraulic gradient. The curves generated in the analysis were used to determine the design safe separation distances and well depth for a hypothetical subdivision in the Central Wisconsin sand plain. Because a relatively wide range of conditions were considered in this study, these results may be valid for other similar sand aquifers provided the basic model assumptions can be met.

ACKNOWLEDGEMENT

This research was supported by the Small Scale Waste Management Project, School of Natural Resources, College of Agriculture and Life Sciences, University of Wisconsin-Madison.

REFERENCES

Box, G., W. Hunter and J. Hunter, 1978. Statistics for Experimenters: An Introduction to Design, Data Analysis, and Model Building. Wiley, New York, NY, pp. 287–298.

Carsel, R.F., R.L. Jones, J.C. Hanson, R. Lamb and M.P. Anderson, 1988a. A simulation procedure for groundwater quality assessment of pesticides. J. Contam. Hydrol., 2: 125–138.

Carsel, R.F., R.S. Parrish, R.L. Jones, J.L. Hansen and R.L. Lamb., 1988b. Characterizing the uncertainty of pesticide leaching in agricultural soil. J. Contam. Hydrol., 2: 111–124.

Central Wisconsin Groundwater Center, 1987. Unpublished data. Univ. Wis.-Stevens Point, Stevens Point, WI.

Converse, J.C. and E.J. Tyler, 1987. On-site wastewater treatment using Wisconsin mounds on difficult sites. Am. Soc. Agric. Eng., 30(2) 362–368.

Dagan, G., 1967. A method of determining the permeability and effective porosity of unconfined anisotropic aquifers. Water Resour. Res., 3(4): 1059–1071.

Harmsen, E.W., 1989. Siting and depth recommendations for water supply wells in relation to on-site domestic waste disposal systems. Ph.D. thesis, Univ. Wis., Madison, WI.

Harmsen, E.W., J.C., Converse, M.P. Anderson and J.A. Hoopes, 1991. A model for evaluating the three-dimensional groundwater dividing pathline between a contaminant source and a partially penetrating water-supply well. J. Contam. hydrol., 7.

Holt, C.L.R., Jr., 1965. Geology and Water Resources of Portage County Wisconsin. U.S. Geol. Surv., Water-Supply Pap. 1796, 77 pp.

Iman, R.L. and M.J. Shortencarier, 1984. A FORTRAN 77 Program and User's Guide for the Generation of Latin Hypercube and Random Samples for Use With Computer Models. Prepared by Sandia National Laboratories, Albuquerque, NM 87185 and Livermore, CA 94550, for the U.S. Dept. of Energy under Contract DE-AC04-76D00789, 50 pp.

Mckay, M.D., R.J. Beckman and W.J. Conover, 1979. A comparison of three methods for sampling values of input variables in the analysis of output from a computer. Technometrics, 21(2): 239–245.

Persaud, N., J.V. Giraldez, and A.C. Chang, 1985. Monte-Carlo simulation of non-interacting solute transport in a spatially heterogeneous soil. Soil Sci. Am. J., 49: 562–568.

Portage County, 1988. Unpublished data. Portage County Office, Stevens Point, WI.

Rothchild, E.R., 1982. Hydrogeology and Contaminant Transport Modeling of the Central Wisconsin Sand Plain, Wisconsin, M.S. thesis, Univ. Wis., Madison.

Rothchild, E.R., R.J. Manser, and M.P. Anderson, 1982. Investigation of Aldicarb in ground

water in selected areas of the Central Sand Plain of Wisconsin. Ground Water, 20(4): 437–445.
Shafer, J.M., 1987. Reverse pathline calculation of time-related capture zones in nonuniform flow. Ground Water, 25(3): 283–289.
Weeks, E.P., 1964. Field methods for determining vertical permeability and aquifer anisotropy. U.S. Geol. Surv., Prof. Pap., 501-D; D193–D198.
Wisconsin Department of Labor and Human Relations. Private Sewage Systems. ILHR 83.10. Wisconsin Administrative Code. Register, June 1983, No. 330.
Yates, M.V. and Yates, S.R., 1989. Septic tank distances: a way to minimize virus contamination of drinking water. Ground Water, 27(2) 202–208.