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Modeling the Impact of Clustered Septic Tank Systems on Groundwater Quality

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ABSTRACT

Contamination of groundwater from onsite disposal of septic tank effluent is an increasing concern. In this study, HYDRUS-2D simulated the impact of clustered disposal systems on NO_3^- and fecal coliforms in groundwater in a rural community near Christchurch, New Zealand. The model included nine disposal boulder pits, embedded 4 m below the surface in alluvial gravel media, in a domain of 3.3 km by 30 m (including both unsaturated and saturated zones). Water movement between the ground surface and the disposal pits was simulated using HYDRUS-1D. The performance of the two-dimensional model was evaluated using monitoring data obtained from a 1977 study. Applying the daily climate data for 1974 to 1977, the simulated NO_3^- and bacteria concentrations in the groundwater were similar to those observed. Both observed and simulated results showed that clustered disposal systems have a significant cumulative impact on NO_3^- concentrations in groundwater, but the impact of fecal coliforms from individual systems is localized. This study further demonstrates that groundwater has a limited ability to dilute NO_3^-, requiring at least 2.9 km for NO_3^- to be reduced to near background levels. Therefore, disposal systems must treat effluent efficiently and water wells downgradient of closely clustered disposal systems must be deep enough to avoid the adverse health effects of NO_3^-. Sensitivity analysis suggests that the model results are most sensitive to changes in hydraulic conductivity, effluent concentrations and discharge rate, and the removal rate of bacteria in the unsaturated zone. Therefore, the accurate estimation of these parameters is a fundamental requirement for the model to produce realistic results.

Onsite wastewater treatment systems are used worldwide in many rural and urban fringe areas. These onsite systems are often developed as clusters, with a number of homes adjacent to one another, all on separate onsite sewage disposal systems. This is particularly true in many parts of New Zealand, where there is a trend toward subdividing rural land to form "lifestyle blocks." Dwellings within these blocks receive neither reticulated water nor sewerage services, and often the groundwater used for human consumption and hygiene is drawn from the same areas that are used for the disposal of sewage effluent. The potential threat to public health associated with this practice is an increasing concern as wastewater can contain many potentially harmful contaminants, such as disease-causing microorganisms and NO_3^-. Where a dwelling in a lifestyle block is isolated, and its water supply well is located upgradient of the disposal system, contamination of the drinking water by the disposal system can be avoided. Where a number of dwellings are clustered, however, some water supply wells in the community could be affected by a neighboring disposal system, or they might experience the cumulative effect of several upgradient disposal systems. For the purpose of making policy and overseeing rural development, there has been a greater requirement from government authorities to evaluate the cumulative effects of clustered systems on groundwater quality.

Numerical models are useful tools for the quantitative assessment of the impact of onsite systems on groundwater quality and the elucidation of the importance of factors that control contaminant concentrations in receiving waters. Studies on the development and applications of numerical models for evaluating the impact of onsite wastewater systems on environmental quality are sparse (Shutter et al., 1994; MacQuarrie and Sudicky, 2001; Beach and McCray, 2003). Using a variably saturated flow and first-order transport model, Shutter et al. (1994) simulated the movement of Na and a surfactant from septic drain field. MacQuarrie and Sudicky (2001) developed a multicomponent flow and reactive transport model, which couples the most relevant physical, geochemical, and biochemical processes involved in wastewater plume evolution in sandy aquifers. They experimentally evaluated their model by simulating wastewater migration in a 1-m-long unsaturated column. Beach and McCray (2003) applied HYDRUS-2D to simulate the influence of soil clogging of wastewater soil absorption systems on flow regimes. McCray et al. (2005) presented a critical review of model-input parameters to simulate the transport of onsite wastewater pollutants. Most studies reported in the literature, however, have focused on evaluating the performance of onsite systems or specific processes that are associated with onsite systems (e.g., nitrification and denitrification), and little has been reported on modeling the cumulative impact of onsite systems on groundwater quality. The need to quantitatively predict potential cumulative effects of onsite systems on groundwater quality has been indicated in the review by McCray et al. (2005). They commented that understanding the cumulative effects of onsite systems is critical for determining the adverse effects on nearby drinking water supply wells.

The impact of clustered septic tank systems on groundwater quality in New Zealand has previously been investigated in two field studies performed at a site near Christchurch, in 1977 (Sinton, 1982) and in 1986 (Close et al., 1989). Elevated NO_3^- levels and locally high fecal coliform levels in groundwater were found in both surveys. The 1977 survey (Sinton, 1982) also found a clear trend of increasing NO_3^- concentrations in the downgradient groundwater as the number of upgradient septic tank systems increased, but this trend was not found for

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fecal coliforms. The aim of our study was to establish a contaminant transport model that can predict the effects of clustered disposal systems on groundwater quality, using the same field site as Sinton (1982) as an example. The capability of the contaminant transport model was then evaluated using historic monitoring data. The transport of NO_3^- and fecal coliforms in the unsaturated zone and groundwater was simulated, and simulated concentrations were compared with those monitored in groundwater. We also examined the relative importance of the transport parameters to the model results. These results will help with the design of future experiments so that more accurate parameter values can be obtained for further model refinement.

MATERIALS AND METHODS

Study Site and Characteristics of Subsurface Media

The site selected for modeling was a 3.3-km length of Buchanans Road in the Yaldhurst area of Christchurch, New Zealand. The uppermost layer (1.9 m) of soil is Waimakariri deep silt loam on sand (interbeds of silt loam, sandy loam, and sand), and the substrata below are alluvial gravels with a sand matrix. No soil information was directly available from this site. Water retention data were therefore obtained from a nearby reference site with similar soil profiles. Table 1 lists the hydraulic properties of the soil layers, determined from water retention data using the RETC program (van Genuchten et al., 1991).

In the 1970s and 1980s, this area of Christchurch was a semirural unsewered community. Each property was served by an individual septic tank system and a drinking water well. Onsite disposal systems have been used in this area since the 1950s, and the number of systems has increased over the years. As a common practice in the past on the Canterbury Plains, effluent was not disposed of through any soil layers but via a boulder pit (about 4 m deep and 1 m diameter), which was in direct contact with underlying alluvial gravels (Sinton, 1982). The rate of effluent discharge into the each pit varied between 200 and 600 L d^{-1}.

Although direct discharge of septic tank effluent into gravel was very effective for the rapid disposal of unwanted effluent, unfortunately it was ineffective in the removal of contaminants. In a baseline groundwater quality survey in the Yaldhurst area performed in 1976, 33% of 120 household water wells contained fecal coliforms or streptococci (Sinton, 1982). Septic tank systems were considered to be the major source of fecal contamination. Subsequently, 25 wells were selected in 1977 for monitoring of fecal coliforms and NO_3^- (Sinton, 1982). These 25 wells had indicator bacteria present and high levels of electrical conductivity in the 1976 survey. The results of the 1977 survey showed that the drinking water wells in this study site had elevated levels of NO_3^- (up to 25.8 mg L^{-1}) and fecal coliforms [up to 74 cfu (100 mL)$^{-1}$]. The NO_3^- level in the background groundwater, immediately upgradient of the study area, was approximately 6 mg L^{-1} and it had zero fecal coliforms. This background NO_3^- concentration was estimated from the average of five samples taken in 1977 from two wells used by Environment Canterbury (a regional authority) to monitor groundwater quality. The depth of the groundwater table was about 12 to 16 m below ground level, with a hydraulic gradient of approximately 0.0012 to 0.0014 estimated from water levels. The groundwater flow direction in this area was approximately parallel to the Buchanans Road so any cumulative effects resulting from onsite effluent disposal systems should become apparent in the wells along the road. Figure 1 shows the locations and depths of disposal pits and drinking water wells along the Buchanans Road. Although there were many other septic tanks and wells in the same area at a distance from the Buchanans Road, the cross-section of the two-dimensional model does not embody them.

Model Construction and Inputs

HYDRUS-2D was used to simulate contaminant transport through the unsaturated zone and groundwater system. HYDRUS-2D is a Microsoft Windows-based finite-element model used to simulate water flow and contaminant transport in variably saturated porous media (Šimůnek et al., 1999). The program solves the Richards equation for saturated–unsaturated water flow, uses a Fickian-based advection–dispersion equation for contaminant transport, and includes provisions for linear equilibrium adsorption, zero-order production, and first-order reduction. The governing equations are solved using a Galerkin-type linear finite-element scheme.

To reduce the complexity of the problem and the simulation time, a two-step approach was used in the modeling. The model conceptualization is illustrated in Fig. 1. As the effluent was generally discharged at a depth of 4 m (Sinton, 1982), a one-dimensional model using HYDRUS-1D (Šimůnek et al., 1998) was first constructed for the unsaturated soil layers above the 4-m depth. Nine soil layers were included in this model (Table 1). The top boundary of this one-dimensional model received daily rainfall and potential evapotranspiration using local climate data measured from 1974 to 1977. A free-drainage boundary was assigned at its bottom. Only flow movement was simulated in the one-dimensional model. Hydraulic conductivities of the soil layers considered in the one-dimensional model were assumed not to be affected by

Table 1. Soil hydraulic properties used in the HYDRUS models.†

Model dimensions	Depth	Lithology	ρ_b	K_{sat}	θ_r	θ_s	α	n
	m		g cm^{-3}	m d^{-1}	m^3 m^{-3}	m^3 m^{-3}	m^{-1}	
1	0.27	silt loam	1.38	1.798	0.000	0.524	3.583	1.237
1	0.38	fine sandy loam	1.38	0.533	0.020	0.493	9.495	1.278
1	0.54	loamy fine sand	1.38	0.621	0.024	0.517	5.656	1.381
1	1.13	fine sand	1.38	0.415	0.000	0.492	6.527	1.417
1	1.24	coarse sand	1.38	1.798	0.000	0.524	3.583	1.237
1	1.60	silt loam	1.38	1.798	0.000	0.524	3.583	1.237
1	1.80	fine sandy loam	1.38	0.533	0.020	0.493	9.495	1.278
1	1.90	loamy sand	1.38	0.533	0.020	0.493	9.495	1.278
1	4.00	gravels with sand matrix	1.38	25.92	0.059	0.507	6.700	2.267
2	12 to 18	unsaturated gravels with sand matrix	2.12	600.0	0.059	0.400	6.700	2.267
2	30.0	saturated gravels with sand matrix	2.12	600.0	0.059	0.300	6.700	2.267

† ρ_b, bulk density; K_{sat}, saturated hydraulic conductivity; θ_r, residual water content; θ_s, saturated water content; a, coefficient in the soil water retention function; n, exponent in the soil water retention function.

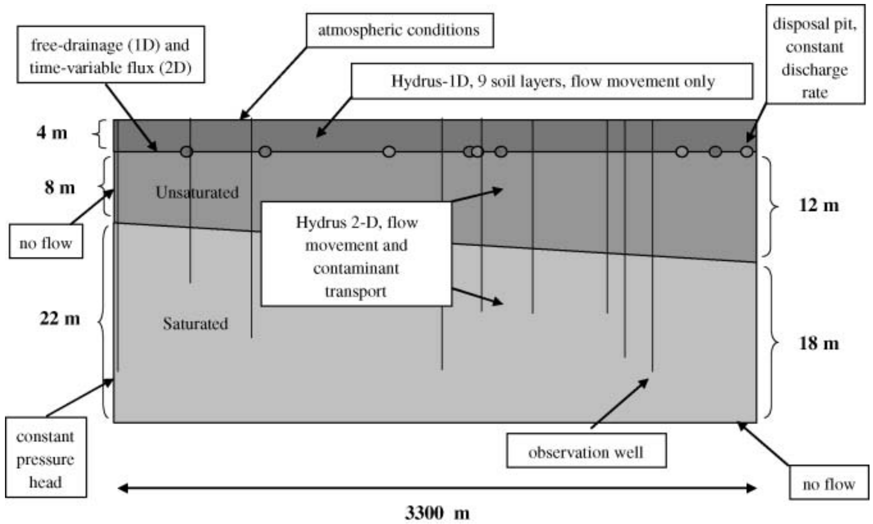

Fig. 1. Schematic of the transport domain and conceptual model, including defined boundary conditions.

operation of effluent disposal (e.g., clogging) as disposal of effluent occurred below these layers.

The second model, using HYDRUS-2D, simulated both flow movement and contaminant transport. The model domain was 3.3 km wide and 30 m deep, and included an 8- to 12-m unsaturated zone and an 18- to 22-m saturated zone. The grid consisted of a total of 6232 nodes and 12062 finite elements. Two material layers (the unsaturated zone and saturated zone) were distinguished (Table 1). Nine onsite sewage disposal systems, located along the Buchanans Road downgradient from each other, were included in the model domain, with effluent discharge rates between 200 and 600 L d^{-1}. As the site was not surveyed, locations of each disposal system were estimated from site visits. The grid was spaced vertically at 0.1 to 0.5 m in the top 5 m (denser at the top) and 1 m downward, and horizontally at 0.5 m for the effluent pit and its adjacent nodes and then gradually became sparser by a factor of two for its up- and downgradient areas.

The top boundary of the two-dimensional model received the daily soil water drainage from the bottom boundary of the one-dimensional model for the areas without effluent disposal pits, and constant effluent discharges for the areas with the effluent pits. The left and right boundaries were assumed to have constant pressure heads in the saturated zone, calculated from the hydraulic gradient of the groundwater, and no flow for the unsaturated zone. No flow was assumed at the bottom boundary. Conceptualized boundary conditions are shown in Fig. 1. No surface ponding of effluent and rainwater was allowed at the highly permeable gravel media. The permeability of coarse gravel media was much greater than the intensity of rainfall and effluent discharge. First- and third-type solute transport boundary conditions were applied for the effluent pits and other boundaries, respectively, excluding the no-flow boundary. Drinking water wells (Fig. 1) were treated as observation wells for groundwater quality and the effect of pumping was not considered in the model.

The simulation period for both one- and two-dimensional models started from winter 1974, about 3 yr before the 1977 monitoring data were obtained, to allow equilibration of the system. Winter was chosen as the starting point to allow a relatively wet soil profile to be initially assigned to the model to facilitate the stabilization of the simulated systems. To further stabilize the system, the models were first run for a 20-d simulation and its final pressure heads were imported as the initial pressure heads of the models that were run for a 3-yr simulation. Time discretizations were assigned as follows: initial time step 1×10^{-3} d, minimum allowed time step 1×10^{-4} d, and maximum allowed time step 4×10^{-3} d.

A longitudinal dispersivity value of 0.4 m was assigned for Material Layer 1 (depth 4 m) as dispersivity is about one order of magnitude smaller than the transport distance (Gelhar et al., 1992). The longitudinal dispersivity value for Material Layer 2 was highly uncertain. In a review by Gelhar et al. (1992), longitudinal dispersivities for a scale of 1 to 3.5 km varied between 6 and 170 m. We were, however, careful about using a very large dispersivity value, as the largest scale that has highly reliable dispersivity data reported in the Gelhar et al. (1992) survey was only about 250 m. Considering multiple inputs of contaminants, a longitudinal dispersivity value of 10 m was assigned for Material Layer 2. The rationale for the choice of this value will be further discussed below. Transverse dispersivity values were assumed to be one-tenth of their relevant longitudinal dispersivity values.

Choosing a suitable value of the hydraulic conductivity for heterogeneous coarse gravel media was also difficult. Hydraulic conductivities determined from pumping tests varied between 0.86 and 864 m d^{-1} for clean sand and gravels on the Canterbury Plains (North Canterbury Catchment Board and Regional Water Board, 1983), while those determined from a tracer experiment performed at the nearby site at Burnham varied from 5064 to 14112 m d^{-1} (Pang et al., 1998). Different results obtained by the two methods were explained by different segments of aquifers being measured and by the tracer study probably being affected by preferential flow. While a pumping test deals with the entire pore network within its zone of influence, a tracer experiment measures only the domain between sampling wells and has the tendency to overestimate water fluxes since the tracer may move through preferential flow pathways. Considering that the model domain is fairly large and probably involves both preferential and matrix flow, we considered that the use of the K (hydraulic conductivity) value of 600 m d^{-1} in our model was appropriate. This value proved to be the most reasonable value in the sensitivity analysis.

We have assumed that the hydraulic conductivity of the subsurface media did not change with time due to processes such as clogging. No experimental data supporting the hypothesis of hydraulic conductivity changes due to operation of effluent disposal were collected at the site, nor is relevant data available in the literature for coarse gravel media. The impact of effluent disposal on hydraulic conductivity would be much less significant in highly permeable coarse gravel media of the site than in fine porous media (such as sands and silts). Alluvial gravels on the Canterbury Plains are typically very permeable and coarse, with an average of 10% finer than 0.90 (0.20–17) mm and 50% finer than 18.3 (12.9–34) mm, and with cobbles up to 150 mm in size. Although the substrata also contain some silt and clay, our field experiments suggested that solute transport in heterogeneous coarse gravels is dominantly in preferential flow paths (Pang et al., 1998).

In sewage effluent, N is almost entirely in the form of dissolved NH_4^+ and NO_3^- concentrations are insignificant (Wilhelm et al., 1994). Therefore, NH_4^+ and fecal coliforms were introduced to the model domain through the flux and concentration of the sewage discharge. As the actual concentrations of NH_4^+ and fecal coliforms in sewage effluent were not available for the study site, they were obtained from the mean concentrations reported by Auckland Regional Council (2004), and values of 64 mg NH_4^+ L^{-1} and 10^6 fecal coliforms per 100 mL were used in the model (Table 2). These concentration values are typical for sewage effluent. In a review of model-input parameters for transport of onsite wastewater treatment by McCray et al. (2005), the median NH_4^+ concentration was summarized as 75 mg L^{-1} ($n = 37$, Table 2). In a summary of data from a number of studies, Pang et al. (2003) reported that sewage effluent typically contains 10^6 fecal coliforms per 100 mL. Considering the relatively dry climate in the Canterbury Plains, a mean effluent discharge rate of 200 L person^{-1} d^{-1} was used. This assumes that all water used by a household is discharged through the wastewater disposal system. This discharge rate is slightly lower than the rate of 260 L person^{-1} d^{-1} summarized in a review by McCray et al. (2005).

When dissolved NH_4^+ leaves the anaerobic environment of the septic tank, it is transformed to NO_3^- by autotrophic bacteria in the presence of O_2 (MacQuarrie and Sudicky, 2001). Ammonium is nitrified to NO_3^- via a three-species nitrification chain reaction ($NH_4^+ \rightarrow NO_2^- \rightarrow NO_3^-$) as follows:

$$NH_4^+ + 1.5O_2 \rightarrow NO_2^- + H_2O + 2H^+ \quad [1]$$

$$NO_2^- + 0.5O_2 \rightarrow NO_3^- \quad [2]$$

Under anoxic conditions and in the presence of an organic C source, the reverse process occurs and NO_3^- is reduced to N_2 gas. The denitrification chain reaction is

$$NO_3^- + 2CH_3OH \rightarrow 6NO_2^- + 2CO_2 + 4H_2O \quad [3]$$

$$6NO_2^- + 3CH_3OH \rightarrow 3N_2 + 3CO_2 + 3H_2O + 6\ OH^- \quad [4]$$

These processes can be considered in the HYDRUS model. Although an intermediate product, NO_2^-, is involved in a three-species chain reaction, the conversion between NH_4^+ and NO_2^- is so rapid (McCray et al., 2005) that the reaction can be simplified to two single steps ($NH_4^+ \rightarrow NO_3^-$ or $NO_3^- \rightarrow N_2$ gas). The coupled first-order reaction rates for nitrification and denitrification are

$$\frac{\partial[NH_4^+]}{\partial t} = -\lambda[NH_4^+] \quad [5]$$

$$\frac{\partial[NO_3^-]}{\partial t} = \lambda[NH_4^+] - \mu[NO_3^-] \quad [6]$$

where λ and μ are the nitrification and denitrification rate coefficients [T^{-1}], respectively, and t is the time [T].

Denitrification was not considered in our study. Although groundwater NH_4^+ was not analyzed in the 1977 survey, it was examined later in a survey in 1986 (Close et al., 1989). The NH_4^+ to NO_3^- ratio was estimated from the 1986 data of Close et al. (1989) to be usually <2% (i.e., NH_4^+–N/NO_3^-–N ratio ≤1%). This indicates that almost all inorganic N was converted to the oxidized form. Since groundwater was at fairly oxidized conditions, denitrification was thus negligible for the system investigated here. We believe that oxidation also predominated over reduction in the overlying permeable unsaturated gravels. Groundwater samples taken from a similar field site 15 km nearby contained dissolved O_2 at a concentration of 11.6 mg L^{-1} (Pang and Close, 1999), and an organic C content of 0.04% (Pang et al., 2005). These findings further support our inference of an unfavorable environment for denitrification in coarse gravel aquifers.

Reference values for the first-order nitrification rate (λ) and linear adsorption coefficients of NH_4^+ (K_d) are reported in

Table 2. Parameters identified to be sensitive to the model results and their reasonable values.

Parameter	Reasonable range	Reference
Aquifer properties		
Hydraulic conductivity for coarse gravels, m d^{-1}	0.86–864	North Canterbury Catchment Board & Regional Water Board (1983)†
	5,064–14,112 (n = 15)	Pang et al. (1998)‡
Longitudinal dispersivity, m, for a scale of 1–3.5 km	6–170	Gelhar et al. (1992)
Nitrogen		
NH_4^+ concentration in septic tank effluent, mg L^{-1}	22–230 (n = 37, mean 75)	McCray et al. (2005)
	51–77 (poor operation, mean 64)	Auckland Regional Council (2004)
	26–39 (good operation, mean 33)	Auckland Regional Council (2004)
Nitrification rate, d^{-1}	0.08–211 (n = 19, mean 2.9)	McCray et al. (2005)
	0.02–0.5 (arable soils)	Lotse et al. (1992)
	0.23–0.43 (soils)	Selim and Iskandar (1981)
	0.15–0.25 (unsaturated soils)	Ling and Al-Kadi (1998)
	<0.15 (coarse gravels)	Considered in this study
NH_4^+ adsorption coefficient, L kg^{-1}	3–4 (arable soils)	Lotse et al. (1992)
	1.5 (soils)	Selim and Iskandar (1981)
	3.5 (unsaturated soils)	Ling and Al-Kadi (1998)
	<1.5 (coarse gravels)	Considered in this study
Fecal coliform		
Removal rate in saturated gravels (d^{-1})	1.14	Sinton et al. (1997)§
Removal rate in unsaturated gravels, d^{-1})	>1.14	Considered in this study
Concentration in septic tank effluent, cfu (100 mL)$^{-1}$	$4 \times 10^5 - 8.4 \times 10^6$	Pang et al. (2003)

† Pumping test for coarse clean sand and gravel on Canterbury Plains.
‡ Tracer experiment at a nearby site 15 km away.
§ Determined from a tracer experiment in sewage-contaminated gravel aquifer at a nearby site.

some studies (Table 2); however, most of these reported values are derived from natural soils, not onsite wastewater systems specifically (McCray et al., 2005). In addition, reported values are predominately derived from cropped soils within the root zones, where microbial activity, O_2 levels, and the presence of organic matter favor nitrification and adsorption of NH_4^+ onto soil media. In contrast, in our study septic tank effluent was discharged at a depth of 4 m, where O_2 levels, microbial activity, and levels of organic matter in the coarse gravels are relatively low. In addition, coarse gravel media are highly permeable and heterogeneous. Taking all these into account, values of $\lambda = 0.12$ d^{-1} and $K_d = 0.01$ L kg^{-1} were chosen in our study. While these values are lower than most values reported in the literature, we consider them to be reasonable for the system investigated. The impact of these parameter values on model results will be further discussed below. The same transformation rate was applied to both the liquid and solid phases. Although an application of effluent would introduce microbes and organic matter into the system, thus affecting nitrification rates, since no experiments and monitoring were performed for the disposal systems in the study area, we assumed that these parameter values were constant.

The attenuation and transport of indicator bacteria in the groundwater of alluvial gravels have been studied at two nearby field sites, Templeton (Sinton et al., 1997) and Burnham (Pang et al., 1998; Sinton et al., 2000; Pang et al., 2005). These studies showed that bacteria are not retarded in the groundwater of coarse gravel aquifers. Therefore, the adsorption coefficient for the bacteria was set to zero in the model. A removal rate of 1.14 d^{-1} for fecal coliforms was adopted from Sinton et al. (1997) for the sewage-contaminated aquifer. This rate encompasses the effects of all irreversible processes (e.g., die-off, filtration, and irreversible sorption). Pang et al. (2005) demonstrated that the removal rate of microbes is generally one order of magnitude lower in sewage-contaminated aquifers than it is in clean aquifers. The value of the removal rate for the unsaturated zone was the most uncertain, compared with all the other model parameters, and it was expected to be greater than that for the saturated zone. Since there were no reference values available in the literature for bacterial removal rates in unsaturated coarse gravel media, the removal rate was manually calibrated by trial and error to obtain a reasonable result from the simulations. A removal rate of 3 d^{-1} was found to be the optimal value for fecal coliforms, and this was demonstrated in the sensitivity analysis. An input concentration of 10^6 fecal coliforms per 100 mL was used.

Model Sensitivity Analysis

As the large two-dimensional model required a long CPU (central processing unit) time (>2 d on a 3-GHz computer) and thus inverse modeling was practically impossible; only direct modeling could be completed. A sensitivity analysis was performed since the values for a number of input parameters were highly uncertain due to the wide range of possible values (Table 2). The following parameters with the most uncertain values in Table 2 were examined in the sensitivity analysis: hydraulic conductivity, the adsorption coefficient for NH_4^+, the rate coefficient for the transformation of NH_4^+ to NO_3^-, longitudinal dispersivity, and NH_4^+ input concentration. In addition, the discharge rate of the effluent was also included in the sensitivity analysis. Model simulations were performed within the possible ranges of parameter values (Table 2) to examine their influence on the model's output. One parameter was tested at a time while all other parameters were fixed. The best manually calibrated values were used as the baseline for comparison of the model results, together with the change in maximum NO_3^- concentration.

The sensitivity analysis not only assists in understanding where weaknesses in the modeling results may lie, but it also assists with data interpretations, and provides guidance in identifying which parameter values must be refined by future work, using field measurements or laboratory experiments. Model runs were also performed to demonstrate the effect of the density of disposal systems on groundwater quality and to determine the separation distance between two disposal systems in which NO_3^- concentrations in downgradient groundwater can be reduced to a near natural level.

Since neither the observed nor model-simulated data showed a significant cumulative impact of fecal coliform contamination in groundwater under the influence of clustered disposal systems (see below), the focus of the sensitivity analysis was mainly on simulations of NO_3^-. Sensitivity analysis for the bacteria was only performed for the removal rate in the unsaturated gravels, as that value was the most uncertain compared with all other model parameters, as mentioned above.

RESULTS AND DISCUSSIONS

Impact of Clustered Septic Tank Systems on Groundwater Quality

Nitrate

Figure 2 shows simulated NO_3^- plumes in the vertical plane at 1000 d, which suggests a clear cumulative impact of clustered septic tank systems on the NO_3^- level in groundwater. This cumulative effect can also be seen clearly from the NO_3^- concentrations at selected depths across the model domain (Fig. 3). Figure 3 shows that, as the number of disposal systems increases, the NO_3^- concentrations in downgradient groundwater increases cumulatively. At 1 m below the water table, NO_3^- concentrations rise sharply in association with each disposal system, followed by a sharp decrease with distance due to the dilution effect, before the next discharge point. As the depth increases, however, this sharp change in NO_3^- concentrations becomes much less apparent. At 10 m below the water table, NO_3^- concentrations show a steep increase in association with a disposal system, but remain relatively constant between disposal systems.

The density impact of disposal systems on NO_3^- accumulation in groundwater is further demonstrated in two hypothetical cases given in Fig. 4. Figure 4 shows that when there are three upgradient disposal systems, NO_3^- concentrations in downgradient groundwater at the same distance (e.g., 3.3 km) are higher than when there are only two upgradient disposal systems. Therefore, we can conclude that, with an increase in the density of clustered onsite systems, a greater drinking water well depth is required to obtain less contaminated groundwater. We need to note that this conclusion assumes the same discharge rate for all systems. With fewer systems in place, a greater well depth is also required when loading of the system is increased. We should also point out that obtaining less contaminated groundwater by increasing the well depth is a passive approach. The best solution is to increase the efficiency of effluent treatment within septic tanks before the effluent is discharged into disposal pits.

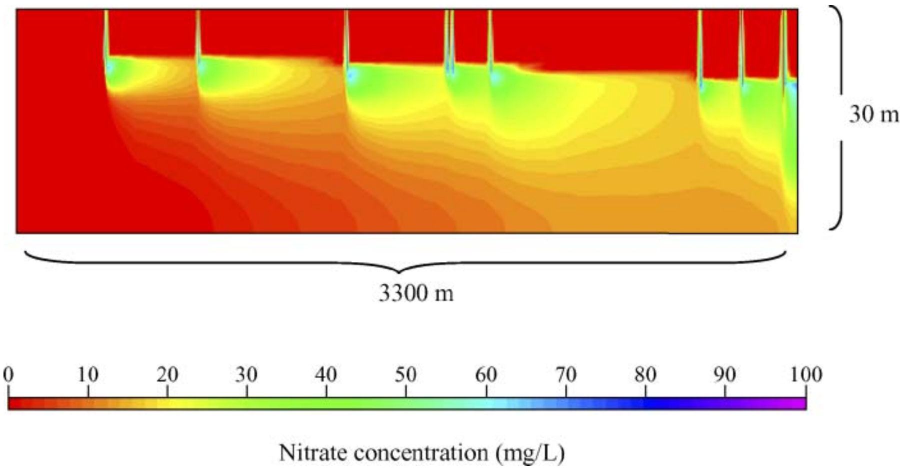

Fig. 2. Hydrus-2D simulated NO_3^- plumes developed under the impact of clustered septic tank systems (results at 1000 d).

An explanation for NO_3^- accumulation in groundwater is that NO_3^- is a conservative solute and its concentrations can only be reduced by dilution when denitrification is negligible in the investigated system; however, the capacity of groundwater to dilute NO_3^- concentrations is very limited, and a very large distance is needed for NO_3^- levels to reduce to the background level. This is illustrated in Fig. 5, which shows that only when the distance is greater than 2.9 km can NO_3^- in downgradient groundwater be reduced to near background levels. Such a large separation distance is not feasible, and thus it is critical for the disposal systems to treat effluent efficiently. We note here that since we used a two-dimensional model that cannot account for dilution in the transverse direction (the direction that is perpendicular to the transport domain), our results are more conservative than they would be in reality. A review by McCray et al. (2005) also discussed the limited dilution of NO_3^- in groundwater for onsite wastewater systems. They indicated that although dilution may play a part in reducing N concentrations in groundwater in the short term, it is not a practical long-term solution because as the area within which unsewered development increases along with the density of onsite disposal systems, so the dilution capacity of the aquifer media is diminished.

Model-simulated NO_3^- concentrations in groundwater, using the best manually calibrated parameter values, compared reasonably well with those observed in the 1977 study (Sinton, 1982), as shown in Fig. 6. Both observed and simulated datasets show a trend of increasing NO_3^- concentrations with distance. It should be noted that to make an unbiased comparison, the data presented in Fig. 6 were only for the nine wells that had the same sampling dates (1 July and 1 Aug. 1977), and the same depths were matched when comparing observed with predicted data. Although the model was only manually calibrated, the good agreement between predicted and observed NO_3^- concentrations suggests that the model we have constructed can simulate the problem under investigation and that the input parameter values are reasonable. Figure 6 shows that simulated results using $K = 600$ and 1000 m d^{-1} match equally well with observed results (with the same $r^2 = 0.72$ for the 1 July 1977 data and $r^2 = 0.57$ for the 1 Aug. 1977 data for linear regression relationships between simulated and predicted values). It should be noted that, as mentioned above, our two-dimensional model cannot account for dilution (although relatively small) in the direction perpendicular to the transport domain. Therefore, the good agreement between concentrations predicted with the two-dimensional model and those observed in the three-dimensional field means that the hydraulic conductivity chosen for the two-dimensional model is probably higher than that in the field.

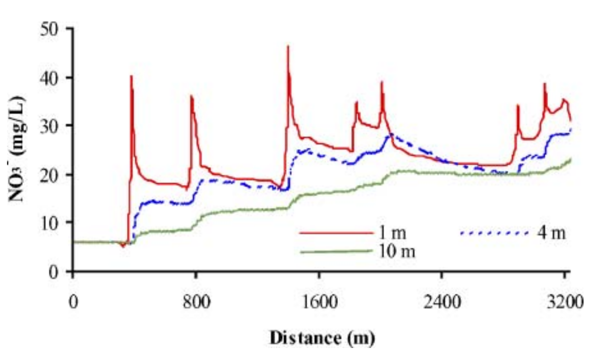

Fig. 3. Simulated NO_3^- concentrations in groundwater 1, 4, and 10 m below the water table across the model domain (results at 1000 d).

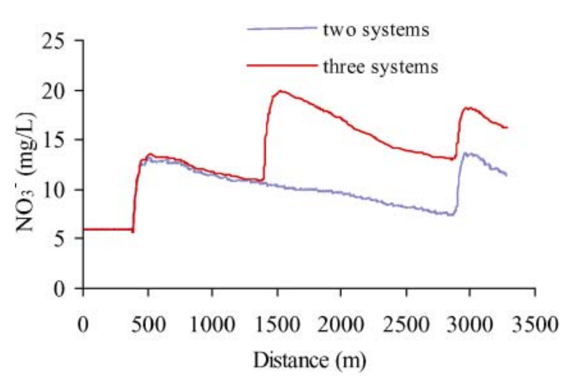

Fig. 4. Comparison of simulated NO_3^- concentrations in groundwater 4 m below the water table in the presence of two and three upgradient disposal systems (results at 1000 d).

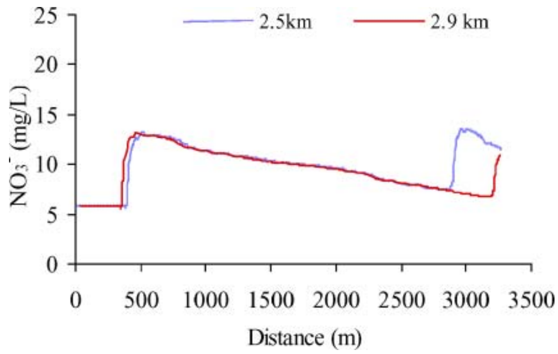

Fig. 5. The effect of separation distance between two adjacent disposal systems on NO_3^- accumulation in groundwater (results at 1000 d, 4 m below the water table).

Although the cumulative impact of clustered disposal systems is shown by both observed and model-simulated data, the NO_3^- concentrations in groundwater are still below the World Health Organization drinking water guideline of 50 mg L^{-1} (WHO, 2004), even where a high input effluent NH_4^+ concentration was used. The WHO guideline of 50 mg L^{-1} for NO_3^- in drinking water is also the maximum acceptable value allowed in New Zealand (Ministry of Health, 2000). When drinking wells are 10 m below the water table, the NO_3^- concentrations in groundwater can be reduced to a level of <23 mg L^{-1} (Fig. 3), and therefore the adverse health effect of NO_3^- in drinking water can be significantly reduced.

Fecal Coliforms

In contrast to the results of NO_3^-, model predictions do not show cumulative effects in relation to the concentrations of fecal coliforms in groundwater as a result of clustered disposal systems—there is a localized impact on bacterial concentrations (Fig. 7). This is consistent with Sinton's (1982) finding that there was no correlation between microbial contamination and distance in the direction of groundwater flow. This can be explained by the efficiency of bacterial removal processes (e.g., straining, attachment, and die-off) in the subsurface media,

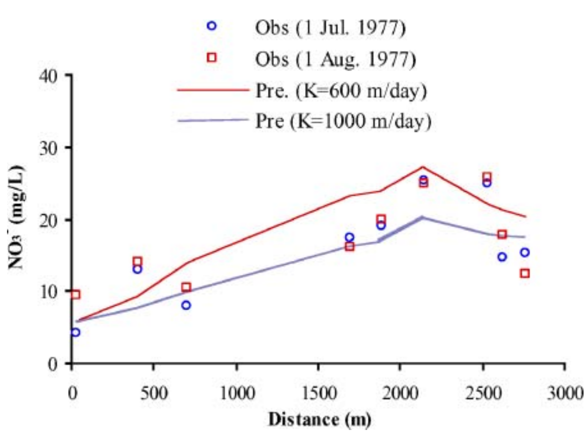

Fig. 6. Comparison of NO_3^- concentrations in groundwater simulated in this study (at hydraulic conductivity K values of 600 and 1000 m d^{-1}) and observed (Obs) in 1977 by Sinton (1982) for the nine wells that were sampled on the same dates (1 July and 1 Aug. 1977).

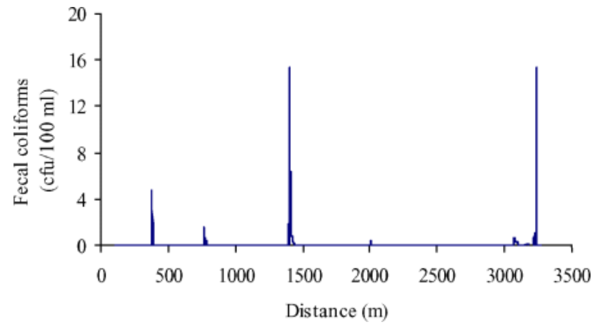

Fig. 7. Simulated concentrations of fecal coliforms in groundwater 1 m below the water table (results at 1000 d).

preventing the bacteria from being carried far downstream and reducing the likelihood of cumulative effects becoming apparent.

Simulated bacteria concentrations decrease with depth; however, to avoid the diagram being cluttered, only results at 1 m below the water table are shown in Fig. 7. Simulated fecal coliform concentrations in groundwater are mostly <16 cfu (100 mL)$^{-1}$. These results agree with those observed in 1977 [mean 2.9 cfu (100mL)$^{-1}$, ranging 0–74 cfu (100mL)$^{-1}$] by Sinton (1982). Unfortunately, fecal coliforms at individual sampling wells measured in the 1977 survey were not given in Sinton (1982) and, thus, we were unable to make any further comparison between predictions and observations. Nevertheless, the good match in the magnitude of observed and predicted bacteria concentrations suggests that the model we have constructed and its inputs describe reasonably well the transport of fecal coliforms in the gravel media investigated.

Sensitivity Analysis

Considering NO_3^- concentrations at a specific location (horizontal distance 1907 m, 4 m below water table) and time (1000 d), the sensitivity of the model results to the input parameters within the range of their reasonable values is shown in Table 3 and Fig. 8. It is very clear that K, the NH_4^+ input concentration (C_o), and the discharge rate (Q) have the most significant influence on the model results. In contrast, the nitrification rate coefficient (λ), longitudinal dispersivity (α_x), and adsorption coefficient for NH_4^+ (K_d) have much less influence on the model results. Figure 8 illustrates that the simulated NO_3^- concentrations are best described with a power function for the impact of K, K_d, α_x, and λ (with a relative importance of $K > \alpha_x > K_d > \lambda$ and with a linear function for the impact of C_o (and probably also Q). The simulated NO_3^- concentrations are positively related to C_o, Q, and λ but inversely related to K, K_d, and α_x. The relative importance of parameters $C_o > Q > K > \alpha_x > K_d > \lambda$ is also shown in Table 3, and is consistent with the results shown in Fig. 8.

The impact of input parameters on model results is further demonstrated in Fig. 9, which compares simulated NO_3^- concentrations 4 m below the water table across the model domain. Again, K, C_o, and Q are shown to be the most sensitive parameters, and λ, α_x,

Table 3. Parameter values used in the sensitivity analysis and a change in the maximum concentration (C_{max}).

Parameter	Value	Factor changed	C_{max}	Change in C_{max}
			mg L^{-1}	%
Hydraulic conductivity, m d^{-1}	100	0.17	56.5	68.2
	300	0.50	45.1	34.2
	600	*1.00*	*33.6*	*0.00*
	1000	1.67	24.0	−28.6
	5000	8.33	1.89	−94.4
	10000	16.67	1.89	−94.4
NH_4^+ concentration, mg L^{-1}	22	0.34	15.2	−54.6
	32‡	0.50	19.5	−42.0
	64§	*1.00*	*33.6*	*0.00*
	75	1.17	37.6	11.90
	230	3.59	97.9	191
Discharge rate Q, m^3 d^{-1}	1/2 Q	0.50	20.7	−38.4
	Q¶	*1.00*	*33.6*	*0.00*
Nitrification rate, d^{-1}	0.06	0.50	32.8	−2.38
	0.08	0.67	33.1	−1.5
	0.12	*1.00*	*33.6*	*0.00*
	2.9	24.2	33.6	0.00
	211	1758	34.2	1.79
NH_4^+ adsorption coefficient, L kg^{-1}	0.005	0.50	33.6	0.00
	0.01	*1.00*	*33.6*	*0.00*
	0.1	10.0	33.0	−1.79
	1.5	150	27.5	−18.2
	4	400	27.5	−18.2
Longitudinal dispersivity, m	5	0.50	33.0	−1.79
	10	*1.00*	*33.6*	*0.00*
	50	5.00	22.6	−32.7
	100	10.0	20.4	−39.3

† Italic numbers are the best values used in the model.
‡ Good operation (Auckland Regional Council, 2004): NH_4^+ = 25.72–38.57 mg L^{-1}.
§ Poor operation (Auckland Regional Council, 2004): NH_4^+ = 51.43–77.14 mg L^{-1}.
¶ Q = no. of residents × 200 L $person^{-1}$ d^{-1}.

and K_d have little impact on the model results. Of the three most sensitive parameters, possible K values for heterogeneous gravel media have the widest range. Figure 9a shows that K values between 600 and 1000 m d^{-1} give the most reasonable predication of NO_3^- concentrations. Therefore our choice of K = 600 m d^{-1} as the most likely K input value is reasonable. Similar results were obtained with K = 5000 and 10000 m d^{-1}, with predicted NO_3^- concentrations close to the background level. As simulated NO_3^- concentrations obtained with K = 100 m d^{-1} were unrealistically high (up to 70 mg L^{-1}) and their inclusion would have cluttered the plots, we have excluded them from Fig. 9a. The impact of the K value on model results is reflected in both the magnitude and shape of the NO_3^- concentration profile as K is directly proportional to the groundwater flux. As K (and thus flux) increases, NO_3^- concentrations in groundwater decrease, and the peak concentrations associated with the disposal systems become less pronounced. In contrast, at low K (low flux), NO_3^- from the leached effluent is not rapidly flushed away, which results in a substantial increase in the NO_3^- concentration.

The K_d coefficient for NH_4^+ does not affect the model predictions no matter which value is selected from a relatively large interval of tested values. Therefore our choice of K_d = 0.01 L kg^{-1} as the best input K_d value does not significantly affect model results, or the conclusions based on them. The insensitivity of the model results to the K_d value is a consequence of a rapid transformation of NH_4^+ to NO_3^-. The transformation of NH_4^+ to NO_3^- is a relatively fast process compared with other processes occurring in the system as all tested λ values (0.06–211 d^{-1}) correspond with a half-life of <12 d. It can thus be expected that the transformation is mostly completed within the unsaturated zone. For the same reason, the model results are also insensitive to λ, even though the range of tested λ values is reasonably wide. When $\lambda < 2.9$ d^{-1}, the model results are very similar no matter which value is selected. Therefore our choice of λ = 0.12 d^{-1} for the model input would also have no major effect on the model results.

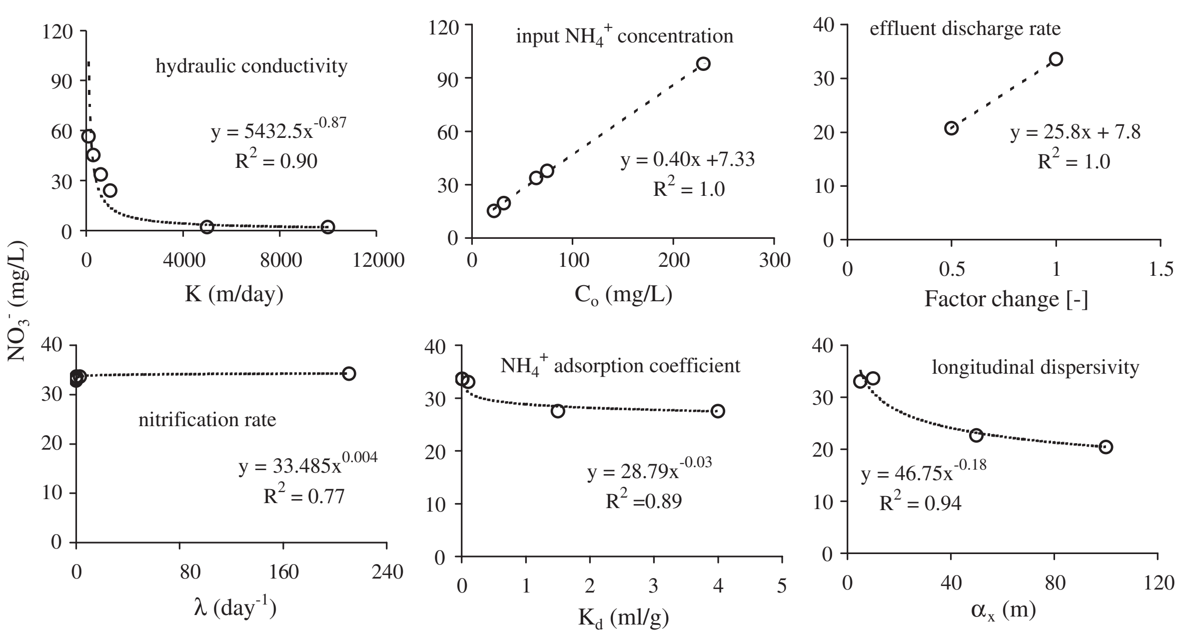

Fig. 8. Sensitivity of NO_3^- concentrations simulated using the range of parameter values for a specific location (distance 1907 m, 4 m below water table) and time (1000 d).

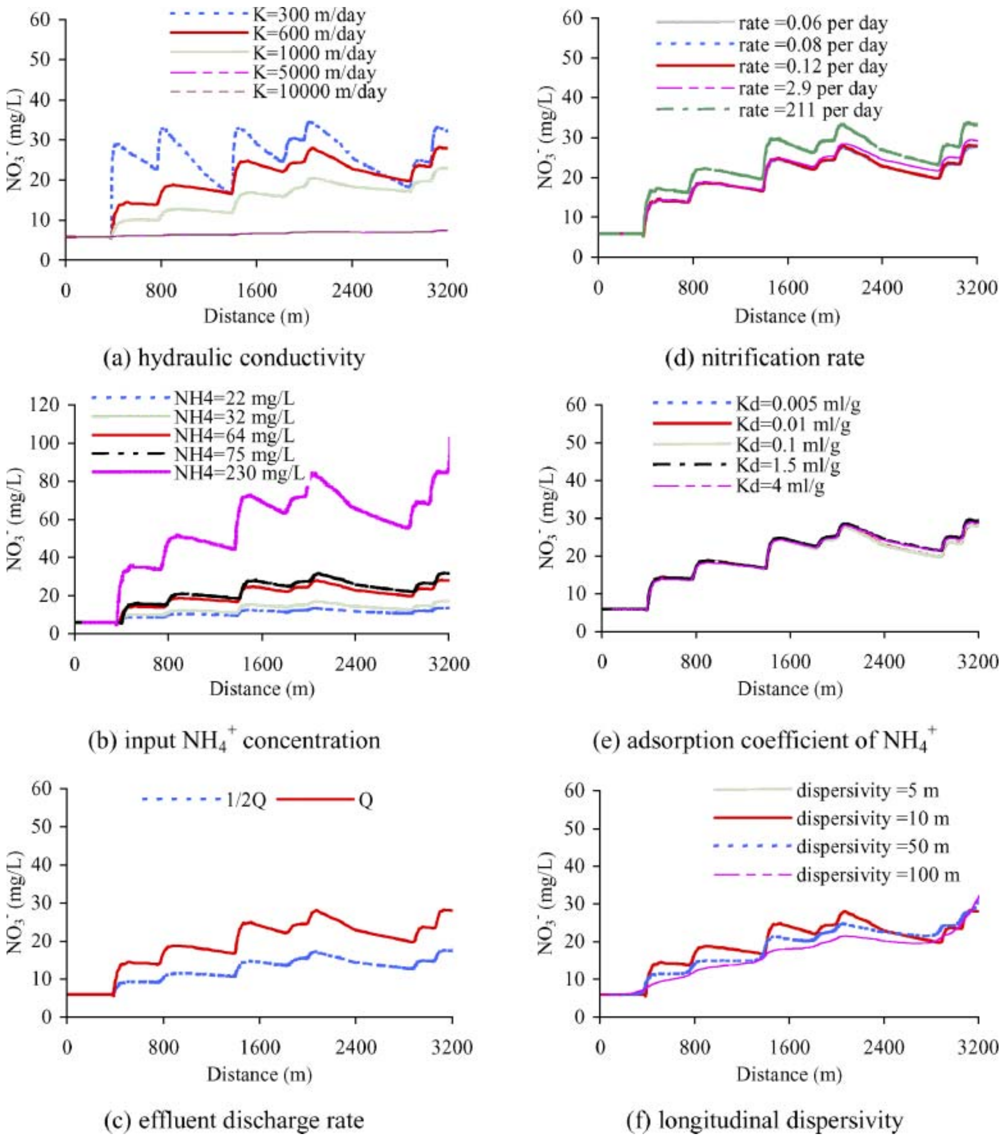

Fig. 9. Sensitivity of NO_3^- concentrations simulated using a range of parameter values for the cross-section 4 m below the water table at 1000 d.

Similarly, model results change only slightly for different values of the longitudinal dispersivity (α_x = 5–100 m were considered). Our choice of α_x = 10 m would thus have not impacted the model results in any significant way. We believe that the insensitivity of the model results to the α_x parameter arises because the physical dispersion is overshadowed by the numerical dispersion involved in large-scale models where many horizontal grid spaces are much greater than the dispersivity values used in the sensitivity analysis. Therefore, the difference in the simulated NO_3^- concentrations when using different dispersivity values might well be within the error caused by numerical dispersion. This error caused by numerical dispersion was inevitable for the large model domain that we constructed. If the horizontal discretization was finer than the dispersivity value of 10 m, the model simulation time would have dramatically increased.

As mentioned above, sensitivity analysis for the bacteria predictions was only performed for the removal rate in the unsaturated zone. As expected, the level of fecal coliforms in groundwater is very sensitive to the removal rate in the unsaturated zone (Fig. 10). This emphasizes the important role that the unsaturated zone plays in reducing bacterial contamination in groundwater. Of the removal rate coefficients examined (1.5–11.4 d^{-1}), a removal rate of 3 d^{-1} for the unsaturated zone seemed to yield the most satisfactory results in comparison with observed results.

CONCLUSIONS AND IMPLICATIONS

The large-scale transport model that was constructed based on HYDRUS-2D was satisfactorily evaluated using the historic groundwater monitoring data. Both model simulations and field observations have demonstrated that, while the clustered septic tank systems have a cumulative impact on NO_3^- concentrations in groundwater, they have a localized impact on fecal coliform concentrations in groundwater. These contrasting re-

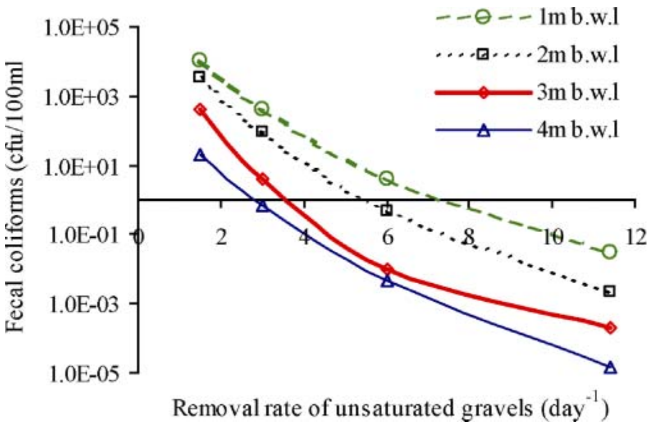

Fig. 10. Simulated concentrations of fecal coliforms (on a logarithmic scale) at 1, 2, 3, and 4 m below the water table (mb.w.1) at a distance of 2005 m. The sensitivity analysis was performed with different removal rates in the unsaturated zone (results at 1000 d).

sults for NO_3^- and fecal coliforms reflect the fact that NO_3^- is a conservative solute that can only be attenuated by dilution (as denitrification is negligible), while bacteria are effectively removed in subsurface media (especially through the unsaturated zone) by processes such as filtration and die-off. The predicted levels of NO_3^- and fecal coliforms in groundwater also agreed well with those observed, suggesting that the constructed model and its inputs are reasonable for describing nitrification and bacteria transport in alluvial gravel media under the influence of clustered septic tank systems.

The results of the model simulations suggest that NO_3^- accumulates in groundwater as the density of upgradient clustered onsite systems increases, and that reduction of NO_3^- in groundwater by dilution is limited. For NO_3^- levels to reduce to a near background level, the separation distance required between two adjacent disposal systems would have to be at least 2.9 km in the coarse gravel aquifers investigated. It is impractical to use such a large distance for regulatory purposes. Therefore, to minimize contamination of groundwater, it is essential for the waste disposal systems to treat effluent efficiently. In addition, drinking water wells downgradient of the clustered waste disposal systems should be at a sufficient depth, ideally >10 m below the water table, so that NO_3^- concentrations can be reduced to the level that has a minimal health impact. In spite of the elevated NO_3^- levels in groundwater within the study area, the NO_3^- concentrations observed and simulated were still below the WHO drinking water guidelines of 50 mg L^{-1}, even when the NH_4^+ concentration in the effluent was high.

Simulated NO_3^- results were most sensitive to the input values for the hydraulic conductivity, effluent concentrations, and the discharge rate. This indicates that both the aquifer properties and the operation of the waste treatment and disposal systems are critical to the potential impact of disposal systems on groundwater quality. Therefore, an accurate estimation of these parameters is the fundamental requirement to enable the model to make reliable predictions. For the ranges of selected parameter values, the model results are insensitive to the NH_4^+ adsorption coefficient, nitrification rate, and longitudinal dispersivity. The relative importance of model parameters to model outputs provides useful information for the design of our future experiments and subsequent model refinement.

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