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Development and application of an analytical model of stream/aquifer interaction

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Abstract

A mathematical model to simulate stream/aquifer interactions in an unconfined aquifer subjected to time varying river stage was developed from the linearized Boussinesq equation using the principle of superposition and the concept of semigroups. The mathematical model requires an estimate of three parameters to simulate ground-water elevations; transmissivity, specific yield, and recharge. The solution has physical significance and includes terms for the steady-state water level, the steady-state water level as influenced by a change in river stage, a transient redistribution of water levels in the aquifer from the previous day, and a transient change in water level caused by a change in river stage. The mathematical model was tested using observed water table elevations at three locations across a 2-km-wide alluvial valley aquifer. The average absolute deviation between observed and simulated daily water levels was 0.09 m. The difference in river stage over the test year was 4.9 m. © 1997 Elsevier Science B.V. © 1997 Elsevier Science B.V.

Keywords: Mathematical model; Unconfirmed aquifer; Boussinesq equation; Water levels

1. Introduction

An important problem in the study of alluvial valley aquifers is the quantification of stream/aquifer hydraulics. Ground-water levels in alluvial valley aquifers fluctuate with changes in stage levels in the associated surface stream (Tabidian et al., 1992 and

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Govindaraju and Koelliker, 1994). If the river stage is increased over a short time period, a flow reversal may occur in the aquifer as a result of the change in gradient between the stream and the aquifer. Essentially, a flood wave is propagated into the unconfined aquifer (Sophocleous, 1991). The distance of influence of the flood wave is dependent on the transmissivity and porosity of the aquifer, the change in stream stage during the flood wave, and the length of time that conditions for a flow reversal exist.

The propagation of a river flood wave into the aquifer raises other important questions when the water quality of the river is different from that of the aquifer. The spread of contaminants from the river into the aquifer, or from the aquifer into the river, is a problem intimately related to the hydraulics of the river–aquifer system.

The hydraulics of the stream–aquifer system could be studied via the solution of the Laplace's equation subject to a non-linear free-surface boundary condition, and time-dependent river boundary conditions. In most applications this represents an alternative for which analytical solutions are difficult. Even after certain simplifying assumptions have been imposed, such as linearization of the free-surface boundary condition, the solution to the Laplace's equation could be elaborate (Polubarinova-Kochina, 1962; Kirkham, 1966). Recently van de Giesen et al. (1994) obtained a solution to the Laplace's equation subject to a linearized free-surface boundary condition and compared the solution to the linearized Boussinesq equation for the case of a sudden drawdown in the river levels, the case of uniform rainfall, and the case of non-uniform rainfall.

When the Dupuit assumptions of mild hydraulic gradients and essentially horizontal flow are valid, the linearized Boussinesq equation appears to be a viable alternative to the use of Laplace's equation. With the Boussinesq equation, the vertical coordinate is eliminated, and the free-surface boundary condition is not needed. The result is a simplified model where the effect of time-dependent river boundary conditions can be easily incorporated in the analysis. Serrano (1995) derived new analytical solutions of the Laplace's equation subject to a steady non-linear free-surface boundary condition and of the non-linear Boussinesq equation. These solutions were compared with solutions to the linearized Boussinesq equation. It was concluded, that under mild regional gradients, the linearized Boussinesq equation offered reasonably accurate results. Govindaraju and Koelliker (1994) studied the effects of arbitrarily shaped hydrographs on bank storage using a numerical solution to the nonlinear form of the Boussinesq equation and an analytical solution to the linearized equation. They found that errors associated with using the simplified linear model were less than errors related to inadequate knowledge of aquifer parameters.

In the present paper, a simplified model based on the linearized Boussinesq equation is developed with the purpose of predicting ground-water heads across an aquifer subject to a highly fluctuating river boundary condition. With a simplified model, the derivation of hydraulic gradients, velocity vectors, and other flow parameters of interest in contaminant dispersion problems is facilitated. The principle of superposition is used in conjunction with the concept of analytic semigroups, which has proven useful in the analysis of partial differential equations subject to variable parameters, forcing functions, and boundary conditions (Serrano and Unny, 1987b). The model is verified with respect to observed ground-water levels in an alluvial valley aquifer subjected to time varying river stage with favorable results.

2. Development of the mathematical model

The simplest model describing transient changes in regional ground-water flow is one whose governing differential equation is the linearized Boussinesq equation with Dupuit assumptions, subject to time-dependent boundary conditions (Fig. 1). For conditions when the right boundary remains constant, the governing differential equation and boundary conditions are

$$\frac{\partial h}{\partial t}-\frac{T}{S}\frac{\partial^2 h}{\partial x^2}=\frac{I}{S}, \quad 0\le x\le L_x, \quad 0<t, \quad h(0,t)=h_1(t), \quad h(L_x,t)=h_2, \quad h(x,0)=h_0(x) \tag{1}$$

where $h(x, t)$ is the hydraulic head (L); T is the average aquifer transmissivity ($L^2\ t^{-1}$); S is the aquifer specific yield; I is the mean recharge to the aquifer ($L\ t^{-1}$); x represents the space coordinate (L); t is the time coordinate; L_x is the horizontal dimension of the aquifer (L); $h_1(t)$ represents the time-dependent left boundary condition (L); h_2 represents the right boundary condition, assumed constant over time (L); and $h_0(x)$ is the initial condition (L).

Due to the linearity of Eq. (1), the solution may be split into different components: the steady state component, the transient component, and the component due to the initial condition. For the transient component, the differential equation subject to a time-dependent boundary condition will be transformed into one with the time-dependent boundary information expressed as an input function and a set of homogeneous boundary conditions (Carslaw and Jaeger, 1971). The procedure will result in a more manageable, simpler solution. Thus the solution to Eq. (1) may be expressed as the summation of two main components (Powers, 1979):

$$h(x,t)=V(x)+u(x,t) \tag{2}$$

where $V(x)$ represents the steady state component and $u(x, t)$ represents the transient component. If an average constant recharge is assumed, the steady state function satisfies

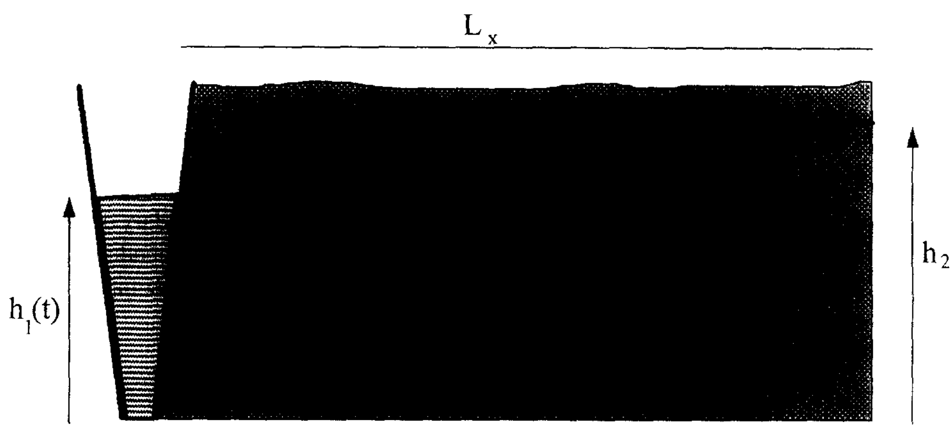

Fig. 1. Sketch of the physical conditions to be modeled in an alluvial valley aquifer subject to fluctuations in river stage.

$$\frac{d^2V}{dx^2} = -\frac{I}{T}, \quad 0 \le x \le L_x, \quad V(0) = h_1(0), \quad V(L_x) = h_2 \tag{3}$$

where the left boundary condition for the steady problem has been taken as the river level at $t = 0$. The solution to Eq. (3) is simply

$$V(x) = -\frac{Ix^2}{2T} + Ax + h_1(0), \quad A = \frac{h_2 - h_1(0)}{L_x} + \frac{IL_x}{2T} \tag{4}$$

The transient component, $u(x, t)$, in Eq. (2) satisfies the following boundary-value problem:

$$\frac{\partial u}{\partial t} - \frac{T}{S}\frac{\partial^2 u}{\partial x^2} = 0, \quad 0 \le x \le L_x, \quad 0 < t, \quad u(0,t) = h_1(t) - h_1(0),$$
$$u(L_x, t) = 0, \quad u(x,0) = h_0 - V(x) \tag{5}$$

In order to transform the above system into one with homogeneous boundary conditions, the transient component, $u(x, t)$, must be expressed as the summation of two components:

$$u(x,t) = M(x,t) + W(x,t) \tag{6}$$

Substituting Eq. (6) into Eq. (5),

$$\frac{\partial W}{\partial t} - \frac{T}{S}\frac{\partial^2 W}{\partial x^2} = -\left(\frac{\partial M}{\partial t} - \frac{T}{S}\frac{\partial^2 M}{\partial x^2}\right), \quad W(0,t) = h_1(t) - h_1(0) - M(0,t),$$
$$W(L_x, t) = -M(L_x, t), \quad W(x,0) = h_0(x) - V(x) - M(x,0) \tag{7}$$

A smooth function, $M(x, t)$, is arbitrarily chosen such that the boundary conditions in Eq. (7) become zero at all times. For instance, let M be

$$M(x,t) = (h_1(t) - h_1(0))\left(\frac{L_x - x}{L_x}\right) \tag{8}$$

Substituting Eq. (8) into Eq. (7),

$$\frac{\partial W}{\partial t} - \frac{T}{S}\frac{\partial^2 W}{\partial x^2} = -\frac{dh_1(t)}{dt}\left(\frac{L_x - x}{L_x}\right), \quad W(0,t) = W(L_x, t) = 0, \quad W(x,0) = h_0(x) - V(x) \tag{9}$$

Thus, the boundary-value problem in terms of u with time-dependent boundary conditions has been transformed into an equivalent one in terms of W with a time-dependent forcing function and homogeneous boundary conditions. The right side of Eq. (9) has the time derivative of the left boundary condition as input.

The solution to Eq. (9) is given by (Serrano and Unny, 1987a, b)

$$W(x,t) = J_t(h_0(x) - V(x)) - \int_0^t J_{t-t'}\left(\frac{dh_1(t')}{dt'} \cdot \frac{L_x - x}{L_x}\right) dt' \tag{10}$$

where the operator $J_t()$ is the strongly continuous semigroup associated with Eq. (9), which may be deduced via traditional Fourier series as (Serrano et al., 1985)

$$J_t(g(t)) = \sum_{n=0}^{\infty}\left[\frac{2}{L_x}\int_0^{L_x} g(t)\sin(\lambda_n\xi)\mathrm{d}\xi\right]\sin(\lambda_n x)\mathrm{e}^{-\frac{\lambda_n^2 Tt}{S}},\quad \lambda_n = \frac{n\pi}{L_x} \tag{11}$$

where ξ is a dummy variable of integration. The semigroup may be viewed as an operator acting on a function. The effect of this operator is to dissipate in space and time the given function. Substituting Eq. (11) into Eq. (9)

$$W(x,t) = \sum_{n=0}^{\infty}\left[\frac{2}{L_x}\int_0^{L_x}(h_0(\xi) - V(\xi))\sin(\lambda_n\xi)\mathrm{d}\xi\right]\sin(\lambda_n x)e^{-\frac{\lambda_n^2 Tt}{S}}$$
$$-\int_0^{t}\sum_{n=0}^{\infty}\left[\frac{2}{L_x}\int_0^{L_x}\left\{\frac{\mathrm{d}h_1(t')}{\mathrm{d}t'}\left(\frac{L_x-\xi}{L_x}\right)\right\}\sin(\lambda_n\xi)\mathrm{d}\xi\right]\sin(\lambda_n x)\mathrm{e}^{-\frac{\lambda_n^2(t-t')}{S}}\mathrm{d}t' \tag{12}$$

At this point, modeling approximations are necessary due to the fact that the left boundary condition, or its time derivative, is not available in analytic form. If the left boundary condition is reported on an incremental basis Δt, then a simulation interval of Δt is appropriate. Thus if the time coordinate is set to take on integer values ($t = 1, 2, \ldots, i - 1, i, i + 1, \ldots$), and the time derivative of the left boundary is approximated as

$$\frac{\mathrm{d}h_1(t=i)}{\mathrm{d}t} \approx \frac{h_1(i) - h_1(i-1)}{\Delta t} \tag{13}$$

simulations can be conducted on a sequential basis (Govindaraju and Koelliker, 1994). The simulation problem reduces to: given the head in the left boundary at time $i - 1$, $h_1(i - 1)$, the level at the end of the current time period i, $h_1(i)$, and the ground-water head at the end of the time period $i - 1$, that is $h_0(x) = h(x, i - 1)$, calculate $h(x, i)$ at the end of increment i. Next, the new head becomes the initial condition for the following time period, and the process is repeated.

Under the above conception, the function V could be interpreted as the steady state obtained after the left boundary head at the end of the previous day $i - 1$, $h_1(i - 1)$, has completely propagated across the aquifer. Substituting $h_1(0) = h_1(i - 1)$ in Eq. (3), then Eq. (4) becomes

$$V(x,i) = -\frac{Ix^2}{2T} + Ax + h_1(i-1),\quad A = \frac{h_2 - h_1(i-1)}{L_x} + \frac{IL_x}{2T} \tag{14}$$

Similarly, substituting $h_1(0) = h_1(i-1)$, and $h_0(x) = h(x, i - 1)$ in Eq. (7), then Eq. (8) becomes

$$M(x,i) = (h_1(i) - h_1(i-1))\left(\frac{L_x - x}{L_x}\right) = \Delta h_1(i)\left(\frac{L_x - x}{L_x}\right) \tag{15}$$

where $\Delta h_1(i) = h_1(i) - h_1(i - 1)$ is an approximation of the change in the left boundary condition over the time increment Δt. Thus for a simulation interval of Δt, setting the initial condition $h_0(x) = h(x, i - 1)$, and approximating the time derivative of the left boundary as above, Eq. (12) becomes

$$W(x,i)=W_1(x,i)+W_2(x,i),$$

$$W_1(x,i)=\sum_{n=0}^{\infty}\left[\frac{2}{L_x}\int_0^{L_x}(h(\xi,i-1)-V(\xi,i))\sin(\lambda_n\xi)\mathrm{d}\xi\right]\sin(\lambda_n x)\mathrm{e}^{-\frac{\lambda_n^2 T}{S}},$$

$$W_2(x,i)=\sum_{n=1}^{\infty}\frac{2SL_x^2\Delta h_1(i)}{Tn^3\pi^3}\sin\left(\frac{n\pi x}{L_x}\right)\left(\mathrm{e}^{-\frac{n^2\pi^2 T}{L_x^2 S}}-1\right) \tag{16}$$

Note that W_1 requires a space numerical integration of the difference between the ground-water head and the steady state at $t=i-1$.

The final result is obtained as

$$h(x,i)=V(x,i)+M(x,i)+W_1(x,i)+W_2(x,i) \tag{17}$$

where $V(x,i)$ is given by Eq. (14), $M(x,i)$ is given by Eq. (15), and $W_1(x,i)$ and $W_2(x,i)$ are given by Eq. (16).

Each of the components in the solution Eq. (17) has physical significance. The head at the end of time period i, $h(x,i)$, is made of an 'eventual' steady function V when the previous-time boundary head, $h_1(i-1)$, has settled in the aquifer, a function M depicting the 'eventual' steady state when the increase in the boundary head $\Delta h_1(i)$ has settled in the aquifer, a transient W_1 because of the 'unsettled' head from the previous time thus (i.e. a 'correction' on V), and a transient W_2 caused by the new increase in the boundary head $\Delta h_1(i)$ (i.e. a 'correction' on M).

3. Application of the model

The mathematical model developed above was evaluated with data obtained during a study of stream/aquifer interactions in an alluvial valley in south-central Ohio (Fig. 2). An extensive data set of daily ground-water levels and river elevations was collected at the Ohio Management Systems Evaluation Area (OMSEA) (Workman et al., 1991; Ward et al., 1994; Jagucki et al., 1995) over the period of August 1991 to December 1995. A portion of the data, October 1991 to September 1992 was discussed in a report by Jagucki et al. (1995). These data showed rapid movement of the unconfined aquifer coincident with changes in stage of the Scioto River and will be used as a test case for the mathematical model.

The hydrogeology of the OMSEA site and the areas adjacent to the site have been extensively studied (Norris and Fidler, 1969; Norris, 1983a, b; Nortz et al., 1994; Jagucki et al., 1995). These studies found the hydraulic conductivity of the aquifer to range from 122 to 152 m day^{-1} with a mean value of 142 m day^{-1}. The specific yield of the site was estimated to be 0.18 to 0.22 with a mean of 0.2. At the OMSEA site, the unconfined aquifer was approximately 18–20 m thick and extended across the width of the valley

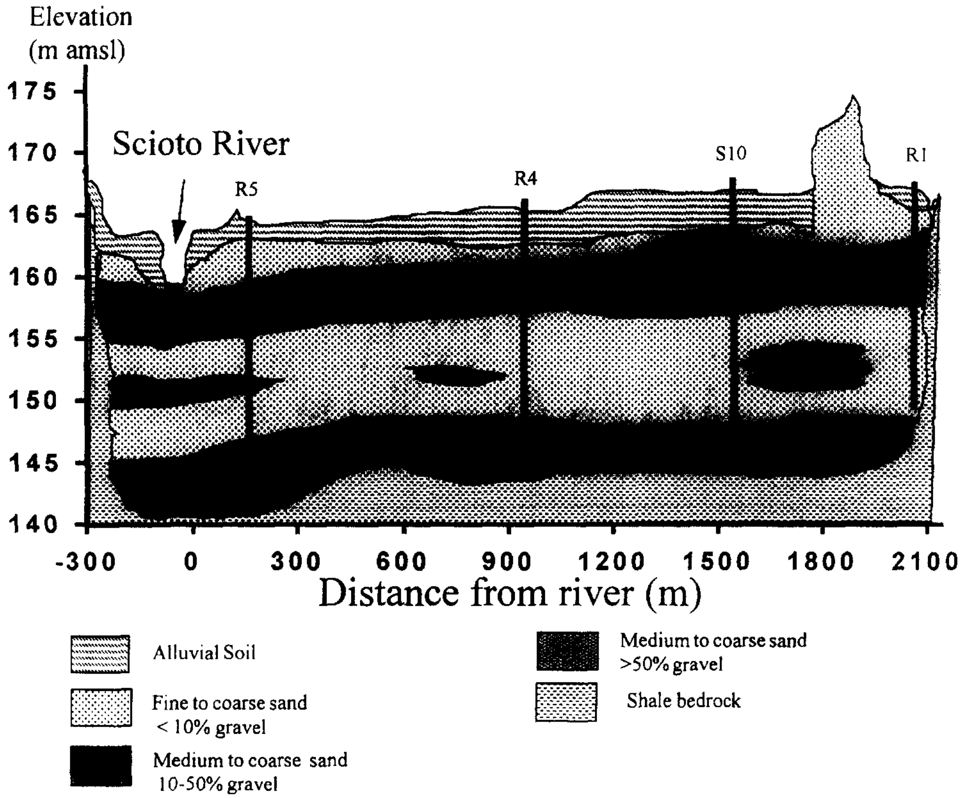

Fig. 2. Cross-section of the Scioto River alluvial valley aquifer. The average hydraulic conductivity of the aquifer materials is 142 m day^{-1}.

(approximately 2 km) (Fig. 2). The vertical scale of Fig. 2 was greatly exaggerated to depict the cross-section of the aquifer.

The left boundary of the problem was taken to be the daily recorded elevation of the Scioto River. The transient nature of flow in the river can be seen in Fig. 3. The Scioto River drains much of central Ohio. A stream gage at Higby, Ohio (approximately 21 km upstream from the OMSEA) has monitored flow in the 13 290 km^2 watershed for 60 years (Nortz et al., 1994). Over the 60-year period, the mean flow rate was 130 m^3 s^{-1} with a minimum of 6.9 m^3 s^{-1} and a maximum flow of 5012 m^3 s^{-1}. The change in river stage between maximum and minimum flows was 7.4 m. The National Weather Service (NWS) operates a wire-weight gage at Piketon, Ohio. Scioto River elevations adjacent to the OMSEA site were used to develop a gradient correction factor of 1.52 m between the NWS gage site and OMSEA site (Jagucki et al., 1995). The total change in stage recorded during the period of October 1991 to September 1992 was approximately 4.9 m (Fig. 3).

A well (R1) located at the right boundary of the flow domain shows a gradual rise in elevation over the water year (Fig. 3). Big Beaver Creek, located at the right boundary of the flow domain, drains an area composed predominantly of the poorly permeable bedrock

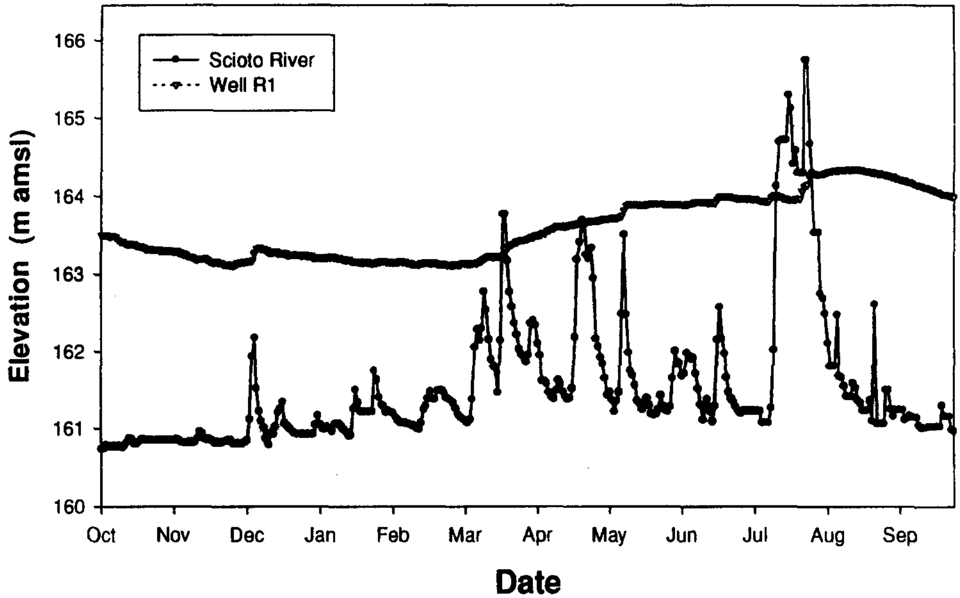

Fig. 3. Daily elevation of the Scioto River located at the left boundary of the OMSEA and well R1 located at the right boundary of the OMSEA from October 1991 to September 1992.

uplands from which runoff is rapid and bank storage is minimal (Jagucki et al., 1995). The conditions produce rapid stage fluctuations in the creek during storm events but only intermittent flow during the dry summer months.

Eleven water-table wells were constructed over a 260-ha area surrounding the OMSEA site. The wells were constructed with 152-mm-diameter PVC casing. A 6.1-m-long, 2.54-mm slotted, PVC screen was positioned to bracket the highest and lowest expected water-table elevations in each well (Jagucki et al., 1995). All of the water-table wells were instrumented with shaft encoders and electronic dataloggers that recorded hourly water levels. The three wells (R5, R4, and S10) shown in Fig. 2 lie on a flow path from the eastern edge of the OMSEA site to the Scioto River (Jagucki et al., 1995). These wells are located 215, 975, and 1525 m from the Scioto River, respectively.

The assumptions specified for the development of the mathematical model were met with the aquifer underlying the OMSEA site; the regional ground-water flow in the Scioto River alluvial valley aquifer was predominantly in the horizontal direction; the aquifer mean thickness was small compared with its horizontal dimension; the water table elevation exhibited mild slopes; the seasonal fluctuation in the water level at the right boundary appeared to be relatively small with respect to time (well R1, Fig. 3); the seasonal fluctuation in the water level of the Scioto River was important and appeared to directly affect the ground-water levels in areas near the river (Fig. 3); and except for the top-soil sediments, the aquifer was composed of various types of sand and gravel with high transmissivity values.

For purposes of testing the model, measured values from the previous characterization of the aquifer were used for each of the parameters in the mathematical model. The hydraulic conductivity was 142 m day^{-1}, the aquifer thickness was 18.3 m, and the specific yield was 0.2. No recharge was simulated in the aquifer for purposes of determining the direct influence of the river on ground-water levels. The aquifer was assumed to be at steady state at the beginning of the simulation. Initial water levels across the aquifer were computed from Eq. (14).

An estimate of the ability of the model to simulate the observed water levels was determined by computing an average absolute deviation (α):

$$\alpha = \frac{\sum_{i=1}^{n} \left| P_i - O_i \right|}{n} \tag{18}$$

where P is the predicted water elevation, O is the observed water elevation, and n is the number of days simulated.

4. Results and discussion

The analytical model required a minimal amount of input for simulating day-to-day fluctuations in ground-water elevations. Fig. 4(a)–(c) shows the simulated and observed water levels in three wells located 215, 975, and 1525 m from the Scioto River for the water year from October 1991 to September 1992. The right boundary condition was held constant at 163.7 m above mean sea level (a.m.s.l.).

There was excellent agreement between observed and simulated water levels in well R5, located approximately 215 m from the river (Fig. 4(a)). The average absolute deviation between simulated and observed values was 0.091 m. The summer of 1991 was one of the driest on record in south-central Ohio. The Scioto River was under base-flow conditions with very little deviation in the hydrograph at the beginning of the study (Fig. 3). Rainfall events in the Scioto River watershed upstream of the OMSEA site from March to August 1992 caused fluctuations in the hydrograph of the river at the OMSEA site that were simulated well with the mathematical model. The water level in the Scioto River never exceeded the carrying capacity of the channel and overbank flow did not occur.

A large flow event occurring at the end of July 1992 was modeled well with Eq. (17) for locations near the river. An increase of 3.2 m over a 16-day period was observed compared with a simulated rise of 2.9 m in the observation well R5 (Fig. 4(a)). The flow event caused a rise (0.65 m) in the aquifer 1525 m from the river (S10); however, the simulated rise was not as large (0.27 m) nor did it occur as rapidly as the observed data (delay of 5 days) (Fig. 4(c)).

Agreement between observed and predicted water levels diminished somewhat as the distance from the river was increased. At a distance of 975 m from the river (well R4, Fig. 4(b)), the average absolute deviation was 0.19 m. The average deviation at a distance of 1525 m (well S10) was 0.29 m. The river certainly had an effect on the water table elevations as far as 1525 m from the river since no recharge or movement of the right boundary was simulated. A comparison of the gradual change in elevation of the right hand

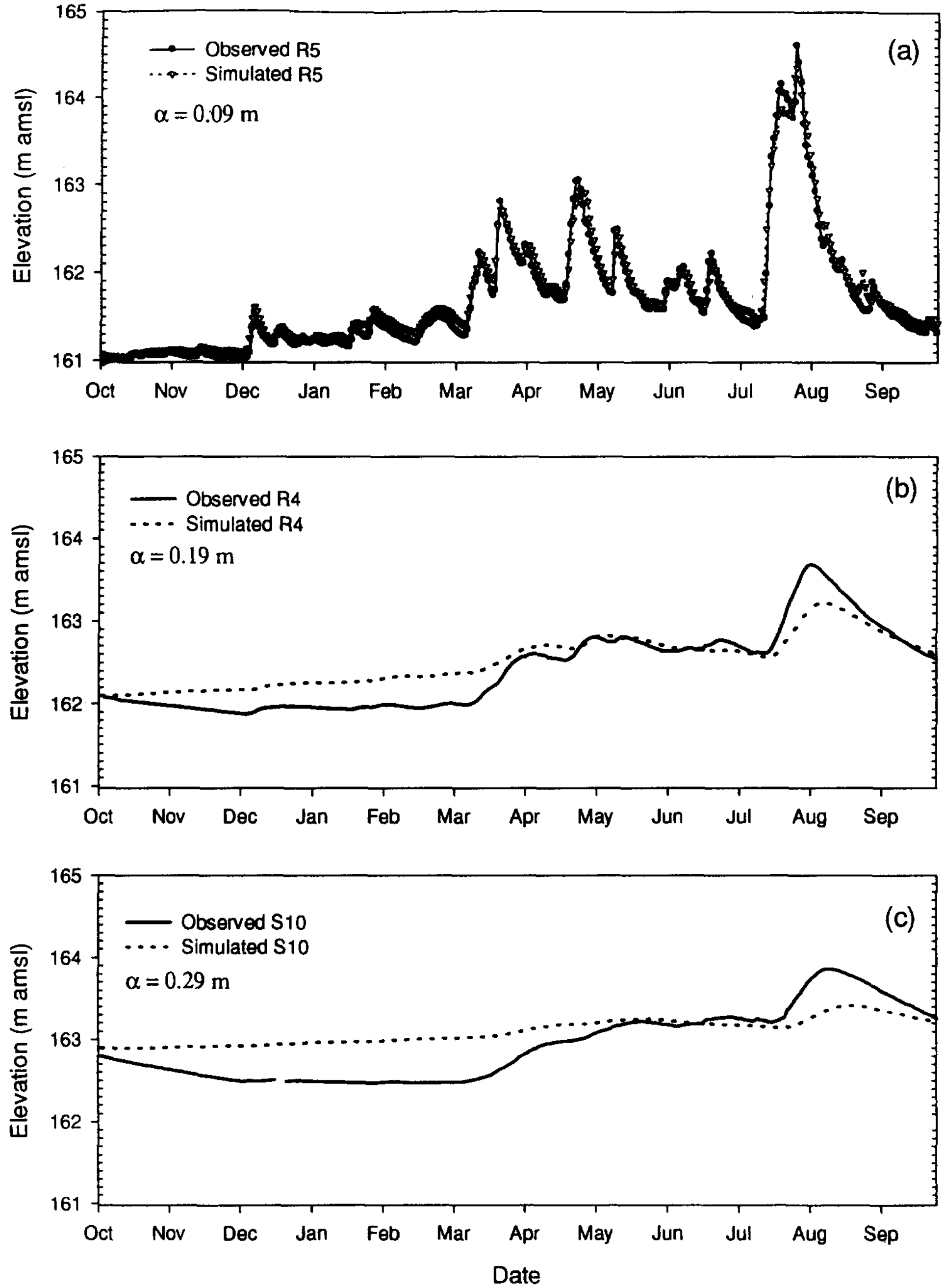

Fig. 4. Observed and simulated water table elevations at locations of 215 m (a), 975 m (b), and 1525 m (c) from the Scioto River. The right boundary was simulated to remain constant at an elevation of 163.7 m a.m.s.l.

boundary of the flow domain (well R1 in Fig. 3) and Fig. 4(c) indicates that the right hand boundary influenced the observed water levels in well S10.

A close inspection of Eqs. (14)–(17) indicates that if the change in the right boundary is gradual, an approximation to the case of having both boundaries time dependent can be made by allowing h_2 to vary in Eq. (14). The time dependent term associated with the right boundary (similar to Eq. (15)) would be quite small and would not affect well S10.

A simulation was conducted allowing h_2 to be described by the hydrograph of well R1 (Fig. 3) located at the right hand boundary of the flow domain (Fig. 5(a)–(c)). Including the gradual fluctuation in h_2 resulted in excellent agreement between observed and simulated water levels at all locations in the aquifer. The average absolute deviation was 0.08, 0.10, and 0.09 m at well locations R5, R4, and S10, respectively. Allowing the right boundary condition, h_2, to fluctuate only slightly increased the agreement in the model simulations at the well location R5 (average deviation from 0.09 to 0.08 m). The overall increased accuracy of the simulations with varied boundary conditions on each side was the result of a better simulation of the low flow conditions from November 1991 to April 1992.

The solution technique includes terms for the steady-state changes in aquifer response (V) and river elevations (M) and transient changes in these two terms (W_1 and W_2). A simulation was conducted to determine the importance of the transient terms in predicting water levels (Fig. 6(a)–(c)). The right hand boundary was held constant at a value of 163.7 m a.m.s.l. to allow comparisons with Fig. 4(a)–(c). The steady-state terms allowed the model to simulate the fluctuations in water table elevation associated with changes in river stage for well R5 located closest to the river with good agreement (α of 0.22 m). The steady-state terms (V and M), however, overpredicted both the rise and fall of the water table in response to changes in river stage. Inclusion of the transient terms allowed the model to dampen the response to stage changes in the river (Fig. 4(a)–(c)). The benefit of the transient terms was more evident as the distance from the river was increased. The gradual changes in water levels simulated with the inclusion of the transient terms (Fig. 4(b), (c)) through the period from April to August were much closer to the observed data than the rapid fluctuations in water level that would be simulated with steady state equations (Fig. 6(b), (c)).

The transient terms require the solution of two infinite series and a space integration. The combination of transmissivity, specific yield, and aquifer length determine the number of terms required to evaluate the summations. Approximately ten terms were required to evaluate the summation for the aquifer in this study.

5. Summary and conclusions

A mathematical model has been developed to simulate stream/aquifer interactions in an alluvial valley aquifer. The mathematical model requires relatively few parameters to simulate ground-water elevations; transmissivity, specific yield, and recharge. The model is physically based and allows the user to simulate each of the processes in a flood wave propagation into an unconfined aquifer. These processes include both steady state changes in water levels and transient changes caused by the redistribution of water.

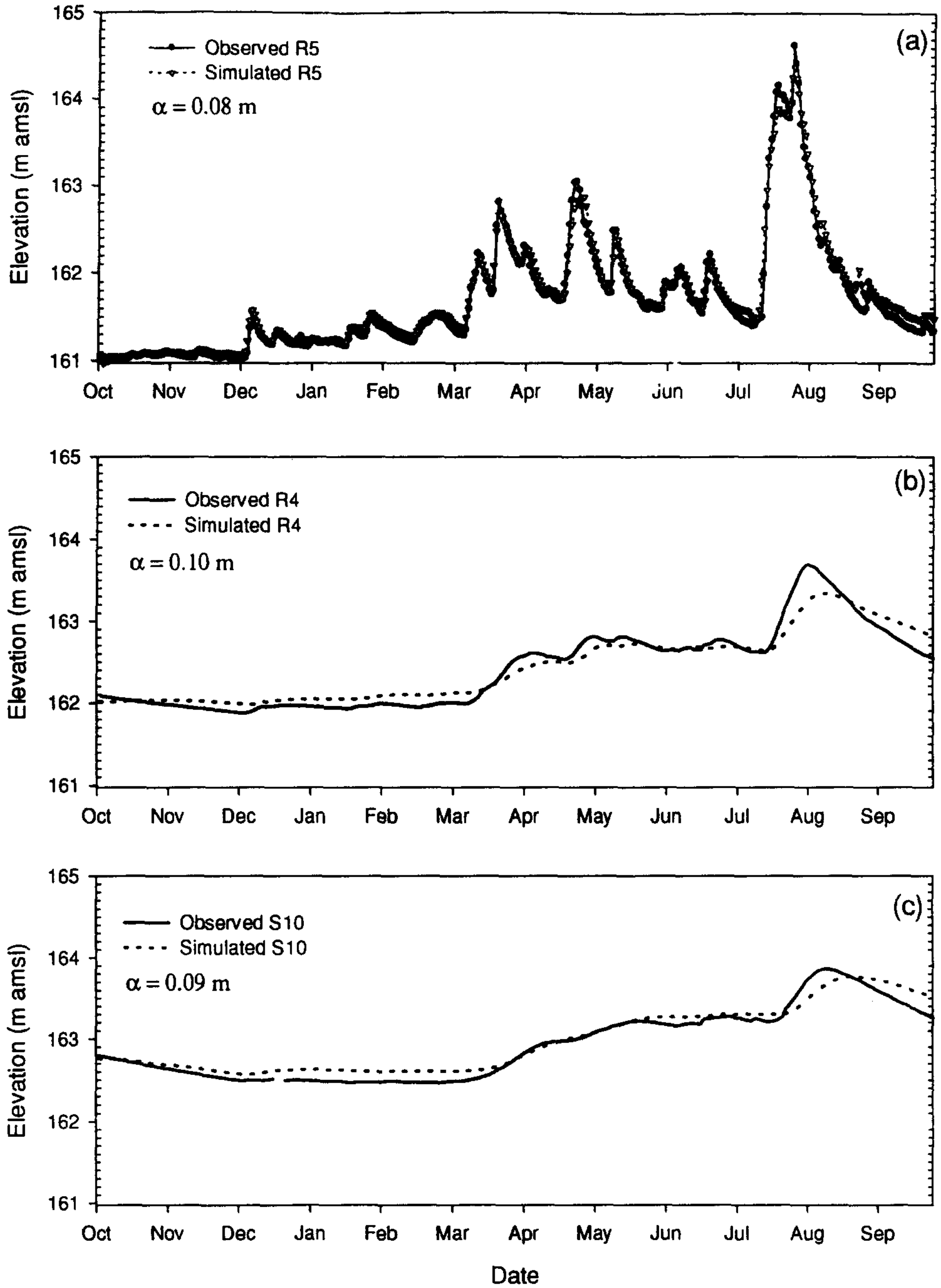

Fig. 5. Observed and simulated water table elevations at locations of 215 m (a), 975 m (b), and 1525 m (c) from the Scioto River. The right boundary was described with the measured water levels in well R1 (Fig. 3).

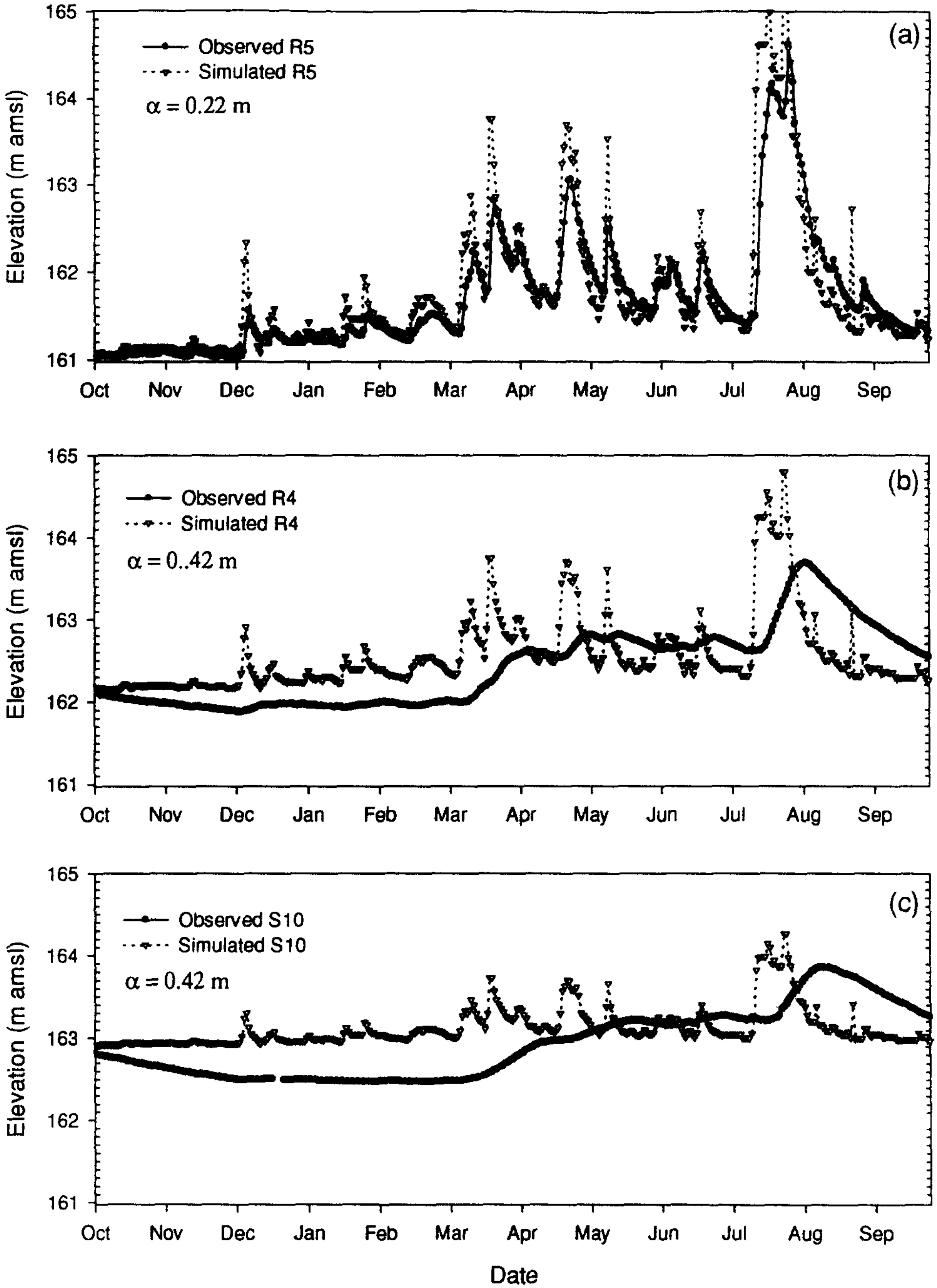

Fig. 6. Observed and simulated water table elevations at locations of 215 m (a), 975 m (b), and 1525 m (c) from the Scioto River neglecting the transient capabilities of the stream/aquifer model. The right boundary was simulated to remain constant at an elevation of 163.7 m a.m.s.l.

The model was tested using observed water table elevations at three locations across a 2-km-wide alluvial valley aquifer. The average daily deviation between observed and simulated water table elevations was 0.09 m at a location 215 m from the river and 0.29 m at a location 1525 m from the river. Allowing the boundary conditions to vary on both boundaries increased the model accuracy; the largest average daily deviation in the three wells was 0.10 m. The transient redistribution of water in the aquifer was simulated well with the model.

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