Hydrogeology Journal

DOI 10.1007/s100400000086

Original paper

Hydrodynamic model of cells for designing systems of urban groundwater drainage

E.D. Zimmermann( ) · G.A. Riccardi

E.D. Zimmermann · G.A. Riccardi
Center of Hydroenvironmental Research, Rosario State University, Riobamba 245 bis, 2000 Rosario, Argentina

E-mail: erikz@fceia.unr.edu.ar
Fax: +54-341-4808541

Received: 15 September 1998 / **Accepted:** 15 May 2000 / **Published online:** [redacted]

Abstract. An improved mathematical hydrodynamic quasi-two-dimensional model of cells, CELSUB3, is presented for simulating drainage systems that consist of pumping well fields or subsurface drains. The CELSUB3 model is composed of an assemblage of algorithms that have been developed and tested previously and that simulate saturated flow in porous media, closed conduit flow, and flow through pumping stations. A new type of link between aquifer cells and drainage conduits is proposed. This link is verified in simple problems with well-known analytical solutions. The correlation between results from analytical and mathematical solutions was considered satisfactory in all cases. To simulate more complex situations, the new proposed version, CELSUB3, was applied in a project designed to control the water-table level within a sewer system in Chañar Ladeado Town, Santa Fe Province, Argentina. Alternative drainage designs, which were evaluated under conditions of dynamic recharge caused by rainfall in a critical year (wettest year for the period of record) and a typical year, are briefly described. After analyzing ten alternative designs, the best technical-economic solution is a subsurface drainage system of closed conduits with pumping stations and evacuation channels.

Résumé. Un modèle hydrodynamique perfectionné de cellules en quasi 2D, CELSUB3, est présenté dans le but de simuler des systèmes de drainage qui consistent en des champs de puits de pompage ou de drains souterrains. Le modèle CELSUB3 est composé d'un assemblage d'algorithmes développés et testés précédemment et qui simulent des écoulements en milieu poreux saturé, en conduites et dans des stations de pompage. Un nouveau type de lien entre des cellules d'aquifères et des drains est proposé. Ce lien est vérifié dans des problèmes simples dont les solutions analytiques sont bien connues. La corrélation entre les résultats des solutions analytiques et des solutions mathématiques a été considérée comme satisfaisante dans tous les cas. Afin de simuler des situations plus complexes, la nouvelle version proposée, CELSUB3, a été mise en œuvre dans un projet destiné à contrôler le niveau de la nappe à l'intérieur d'un système d'égouts, dans la ville de Chaar Ladeado (province de Santa Fe, Argentine). Différentes organisations du projet de drainage, qui ont été testées pour des conditions de recharge dynamique causées par la pluie au cours d'une année critique (la plus humide de la chronique disponible) et une année typique, sont brièvement décrites. Après analyse de dix organisations différentes, la meilleure solution technico-économique retenue est un système de drainage souterrain de conduites avec des stations de pompage et des canaux d'évacuation.

Resumen. Se presenta un modelo matemático hidrodinámico cuasi-bidimensional de celdas, CELSUB3, apto para la simulación integral de sistemas de drenaje subterráneo basados en campos de bombeo o drenes subsuperficiales. El modelo de simulación presenta un ensamble de algoritmos, previamente desarrollados y testeados, que representan al escurrimiento a través del medio poroso saturado, escurrimiento en conducciones cerradas, estaciones de bombeo, etc. En la estructura del modelo se propone un nuevo tipo de vinculación entre celdas acuíferas y conductos de drenaje, la cual es verificada en problemas simples con solución analítica conocida arrojando, en todos los casos, resultados satisfactorios. Abordando situaciones más complejas, la nueva versión propuesta fue aplicada en un proyecto de control de niveles freáticos que acompaña un sistema de conductos cloacales, en la localidad de Chañar Ladeado, Santa Fe, Argentina. Se describen las alternativas de drenaje consideradas las cuales fueron evaluadas bajo recargas dinámicas provocadas por años críticamente lluviosos y en situaciones típicas. Los resultados derivados permitieron definir, tras analizar una decena de proyectos alternativos, la mejor solución técnico-económica consistente en un sistema de drenes subterráneos, estaciones de bombeo y canales de evacuación.

Key words. numerical modeling - groundwater management - urban groundwater - groundwater/surface-water relations - unconsolidated sediments

Introduction

Flatland areas are characterized hydrogeologically by the vertical flow of moisture. Rainy years result in considerable rises in phreatic levels, which may reach ground level in areas of shallow aquifers. This situation, common in the wetland region of Argentina, causes a significant sanitary risk in cities lacking a sewer system. In order to eliminate serious health risks, composite sewer systems providing both wastewater collection and groundwater drainage to control the water-table level, should be designed and installed. Groundwater drainage is achieved by pumping well systems or subsurface systems (trenches or pipe drains). Usually, due to the low topographic gradient in this region, the latter system requires pumping stations and channels to discharge the drained water.

Equations for steady state and transient conditions developed for parallel drains (Hooghoudt *1940*; Ernst *1956*; Glover and Dumm *1960*) are useful tools for designing groundwater drainage systems. Also, a method based on an electric analogy was proposed for open drains of different sizes and with different water levels, under steady-state conditions (Ritzema *1994*). In view of the highly heterogeneous nature of the components of the drainage systems, however, the preceding methodologies are not adequate to analyze the behavior of the systems for the desired design scenarios.

The modular three-dimensional groundwater model MODFLOW has a drain package that calculates flow rates and volumes for water moving from the groundwater flow system to drains (Harbaugh and McDonald *1996*). In order to make this calculation, water elevations in the drains would have to be specified as a boundary condition. In view of the fact that MODFLOW was designed as a porous-media model, however, it cannot simulate flows in conduits or even the water-level fluctuations in pumping stations.

In this paper, a model capable of simulating the latest complex drainage problems was developed, verified in well-known situations, and applied to a design project in Chañar Ladeado town, Santa Fe, Argentina. Numerical models were adopted based on specific cell-model characteristics that allow the assembling of different components of the drainage systems. These cell characteristics are based on a continuity equation in the cell together with a momentum equation that characterizes the type of connection established between cells (Cunge *1975*). The use of cell characteristics schemes was applied in surface-water flow models that simulate rivers and floodplains (Riccardi and Zimmermann *1993*), conduits (Riccardi et al. *1995*), and rainfall-runoff processes (Zimmermann and Riccardi *1995*).

In order to simulate groundwater flow, a first version of a quasi-two-dimensional model of cells, CELSUBS1, was developed. The model was compared with the two-dimensional groundwater flow model GW8 (Karanjac and Braticevic *1989*) and with situations involving known analytic solutions, such as flow to parallel channels subject to recharge and transient flow toward pumped wells. Results indicated a good correlation between groundwater levels and flow simulated by means of CELSUBS1 and the other methodologies (Riccardi and Zimmermann *1999*).

In this project, a new model, CELSUBS3, is proposed; it includes the groundwater flow links of the CELSUBS1 model, the buried conduits, head loss, and pumping-station links (Riccardi *1997a*, *1997b*), and, in addition, a new type of link between aquifer cells and pipe drains (Zimmermann *1997*).

Model of Groundwater Cells

Continuity Equation

It is supposed that the whole of cell *i* corresponds to a characteristic water level z_i, which is assumed to be in the cell center. Also, it is assumed that the water surface is horizontal between the borders of the cell. Two hypotheses are assumed:

- The water volume V_i, stored in cell *i*, is directly related to its water level z_i as $V_i = V(z_i)$.
- The discharge $Q_{i,k}$ between two adjacent cells *i* and *k* at a given time is a function of the water levels in the connected cells: $Q_{i,k}(z_i, z_k)$.

The continuity equation can be expressed in the differential form used by the model of Cunge (*1975*):

$$A_{Si}(z_i)\frac{dz_i}{dt} = P_i(t) + \sum_{k=1}^{j} Q_{i,k}(t) \tag{1}$$

where A_{Si} is the surface area of the cell *i*; the term $P_i(t)$ allows the addition or extraction of external flow at each cell, usually to represent effluents, overflows, extraction for irrigation, recharge, pumping, evapotranspiration processes, etc.; and *j* is the number of cells connected with cell *i*.

Laws of Discharge Between Cells with Surface Flows

Conduit link

This type of link is used for connections between cells representing closed conduits (e.g., drain pipes). The discharge equation is a discretization of the momentum equation neglecting inertia and using the Strickler-Manning resistance formula. Cunge (*1975*) introduced this expression originally to simulate river links. Then it was adapted to closed conduit flow (Riccardi et al. *1995*). In flow under surcharge, the surface area in Eq. (1) is calculated as a minimum value corresponding to the surface of the Preissmann slot along the conduit. This element allows the simulation of flow under surcharge as an equivalent free-surface flow (Cunge and Wegner *1964*). The slot width is calculated as $B_s = gA_{cll}/a^2$, where g is the acceleration of gravity, A_{cll} is the full cross section of the closed conduit, and a is the velocity of elastic-wave propagation. The discharge equation can be given by:

$$Q_{i,k} = \pm(z_i - z_k)\frac{Rh_{i,k}^{2/3}At_{i,k}}{\eta\sqrt{\Delta x_{i,k}}}\sqrt{|z_i - z_k|} \tag{2}$$

where $Q_{i,k}$ is the discharge between cells k and i, $Dx_{i,k}$ is the distance between the centers of cells i and k, $Rh_{i,k}$ is the hydraulic radius, $At_{i,k}$ is the cross section of the wetted area linking cells i and k, and h is the Manning resistance coefficient in $m^{-1/3}$ s.

Head loss link

This link is suitable for flow singularities with head loss due to abrupt changes in the cross section. These are usually present in collector mouths, junction boxes, and expansions and contractions, which are considered as control sections. Two flow conditions are possible: free and submerged (Riccardi *1997a*):

$$Q_{i,k} = \sqrt{2g}\sqrt{(z_i - z_{cri})/(Cd_f^{-2}A_{cri}^{-2} - At_i^{-2})}\text{(free)} \tag{3a}$$

$$Q_{i,k} = \sqrt{2g}\sqrt{(z_i - z_k)/(Cd_s^{-2}A_{Sc}^{-2} - At_i^{-2})}\text{(submerged)} \tag{3b}$$

where g is the acceleration of gravity; z_{cri} is the critical level in the control section; Cd_f and Cd_s are the discharge coefficients in the control section for free and submerged-condition flow, respectively; A_{Sc} and A_{cri} are the wetted areas in the control section for subcritical and critical flow, respectively; and At_i is the cross section of the wetted area of the cell i.

Pumping link

This link is used to represent elements of flow elevation due to the addition of external energy, such as a single pump or pumping stations (Riccardi *1997b*). It is necessary to specify the pumping sequence as a function of time:

$$t_1 - Q_{k,i1};\ t_2 - Q_{k,i2};\ t_3 - Q_{k,i3};\ t_4 - Q_{k,i4};\ldots t_m - Q_{k,i\,m} \quad (4a)$$

or as a function of the level at the upstream cell:

$$z_{k1} - Q_{k,i1};\ z_{k2} - Q_{k,i2};\ z_{k3} - Q_{k,i3};\ z_{k4} - Q_{k,i4};\ldots z_{km} - Q_{k,im} \quad (4b)$$

Laws of Discharge Between Cells with Groundwater Flow

Free aquifer link

The model evaluates the flow exchange between cells with the Darcy equation for uniform flow in a porous medium. The Dupuit-Forchheimer assumptions are assumed to apply, which represent an empirical approximation to the actual flow field. In this case, the effective superficial area A_{Sei} should be used instead of A_{Si} in Eq. (1); A_{Sei} is the product of the surface area of the cell and its storage coefficient ($A_{Sei}=A_{Si}\ S$). The link equation between aquifer cells, according to Riccardi and Zimmermann (*1999*), is:

$$Q_{i,k} = \frac{b_{i,k} K_{i,k}}{\Delta x_{i,k}} h_{i,k} \Delta z_{i,k} \quad (5)$$

where $h_{i,k}$ is the average hydraulic height between the cells *i* and *k*, $K_{i,k}$ is the average hydraulic conductivity between cells in m s^{-1}, $b_{i,k}$ is the width of the link between cells, $\Delta z_{i,k} = z_i - z_k$ is the difference between water levels in the cells, and $\Delta x_{i,k}$ was defined in Eq. (2).

Numerical Formulation

For numerical formulation, the discharge functions between cells are explicit; then they are introduced in Eq. (1). An implicit method of finite difference for the numerical resolution is used (Cunge *1975*):

$$A_{S_i} \frac{\Delta z_i}{\Delta t} = P_i + \sum_{k=1}^{j} Q_{i,k}^n + \sum_{k=1}^{j} \frac{\partial Q_{i,k}^n}{\partial z_i} \Delta z_i + \sum_{k=1}^{j} \frac{\partial Q_{i,k}^n}{\partial z_k} \Delta z_k \quad (6)$$

where A_{Si}, P_i and $Q_{i,k}$ are known at time $t=n\Delta t$ and the increments Δz_i and Δz_k are unknowns. There are as many equations (6) as there are cells in the model. For *N* cells, the system of *N* ordinary differential equations is established for *N* unknown functions z_i of the independent variable *t.* The solution exists and is unique according to Cunge (*1975*) if the initial conditions, $z_i(t)$, are prescribed. An algorithm based on the Gauss-Seidel matricial method (Riccardi and Zimmermann *1999*) carries out the numerical resolution.

The coefficients of the different links are parameters of the model and the geometric characteristics of the cells identify the drainage system. Values of recharge, because of the infiltration in aquifer cells and external pumping extractions, are the inputs of the model, which depend upon time. Water levels in each cell and flows between interconnected cells are the outputs of the model, which also depend upon time.

Boundary and Initial Conditions and External Discharge Exchange

A necessary and sufficient condition that should be specified is data on water levels as a function of time, z(t), in the geographic boundaries of the area being modelled. In practice, this is not always possible or desirable, so that in actual situations three types of conditions can appear: (1) water level as a function of time, z(t); (2) discharge as a function of time, Q(t); and (3) water level-discharge relationship, Q=f(z). The model requires values for the water level in all the cells at time zero.

Simulation of Link Between Aquifer Cell and Drain Cell

In order to describe the flow of groundwater toward parallel pipe drains, the Dupuit-Forchheimer theory is generally used. Relationships can be derived between the drain properties (diameter, depth, and spacing), soil characteristics (stratigraphic profile and hydraulic conductivity), depth of the water table, and discharge. Available theories were analyzed to develop the link equation in the CELSUBS3 model.

Steady-State Conditions

Under steady-state conditions (typical for areas with a humid climate) the Hooghoudt equation (Hooghoudt *1940*) can be used under the assumption that the rate of aquifer recharge is uniform and steady and that it equals the discharge rate through the drainage system (Ritzema *1994*). The drain discharge per unit cross-sectional area can be given by:

$$q = \frac{8KDH + 4KH^2}{L^2} \tag{7}$$

where q is the drain discharge per unit cross-sectional area, K is the hydraulic conductivity of the soil, L is the drain spacing, H is the water-table elevation above the water level in the drain, and D is the water-table elevation in the drain relative to an ideal impervious layer placed below the drain. The first term in Eq. (7) represents the flow below the drain level and the second represents the flow above the drain level. For the situation where the impervious layer is far below the drain level ($D>L/4$), Hooghoudt (*1940*) introduced the equivalent depth concept to maintain the assumption of horizontal flow. In this case, D must be replaced by the equivalent depth (d), as given by the equation of Van der Moolen and Wesseling (*1991*):

$$d = \frac{\frac{\pi L}{8}}{\ln \frac{L}{\pi r_0} + F(x)} \tag{8}$$

where $r_0 = u/\pi$ is the drain radius, u is the wetted perimeter of the pipe drain, and $x = 2\pi D/L$. Hooghoudt (*1940*) based his theory on two assumptions: the drains are running half full and there is no entrance resistance. $F(x)$ can be obtained by:

$$F(x) = 4\sum_{n=1}^{\infty} \frac{e^{-2nx}}{n(1-e^{-2nx})}, n = 1, 3, 5 \ldots\ldots \qquad (9a)$$

which converges rapidly for *x> 1*. For *x<1*, the following highly accurate approach can be used:

$$F(x) = \frac{\pi^2}{4x} + \ln\frac{x}{2\pi} \qquad (9b)$$

Unsteady-State Conditions

The equations for unsteady state assume that recharge varies with time. The equation of Glover and Dumm (*1960*) is used to describe a declining water table after its sudden rise due to an instantaneous recharge event, a typical situation in irrigated areas. Applying the theory of Glover and Dumm (*1960*), the following equation can be obtained:

$$H_t = 1.16 H_0 e^{-\alpha t} \qquad (10)$$

where H_t is the water-table elevation above the drain level as a function of time, H_o is the initial water-table elevation, and α (the reaction factor) = $\pi^2 Kd/\mu L^2$, where μ is the drainable pore space (expressed as a fraction), and the remaining variables are defined above. It can be proved that the discharge per unit cross-sectional area at any time is given by:

$$q_t = \frac{2\pi Kd}{L^2} H_t \qquad (11)$$

This is similar to the Hooghoudt equation (Hooghoudt *1940*) for describing the flow below the drain level, except that the factor *8* is now replaced by 2π .

The equation of De Zeeuw and Hellinga, cited by Ritzema (*1994*), is used to describe a fluctuating water table. In this approach, a non-uniform recharge event is divided into shorter time periods in which the recharge can be assumed to be constant. This is the typical situation for humid areas with periods of high-intensity rainfall that occur as a series of discrete storms. In this case, the water-table elevation is given by:

$$H_t = H_{t-1} e^{-\alpha \Delta t} + \frac{R}{0.8\mu\alpha}(1 - e^{-\alpha \Delta t}) \qquad (12)$$

where *R* is the recharge per time period in m s^{-1}.

Equation Adopted in the CELSUBS3 Model

In the extensive, flat wetland regions in Argentina, where a very deep impervious layer occurs, the dominant contributions of water are from below the drain system. For a typical set of parameter values, of application, e.g., $L \approx$ 150 m, $d \approx$ 10 m, and $H \approx$ 2 m, the relative contributions of flow from below and above the drain level can be analyzed by Eq. (7), as follows: Flow from below drain/Flow from above drain=2dH/$H^2 \approx$ 10. In this case, the flow contribution from below is an order of magnitude greater than that from above, and the second term in Eq. (7) can be neglected. Also, an equivalence has been noted between the equation of Glover and Dumm (*1960*) for unsteady-state conditions and the simplified version of the Hooghoudt equation (Hooghoudt *1940*) for steady-state conditions. Consequently, the latter was selected as the general expression of the aquifer-drain link, as follows:

$$q_t = \frac{8Kd}{L^2} H_t \tag{13}$$

The water-table elevation and discharge rate were considered as unsteady variables in the study area. To assemble Eq. (13) in the CELSUBS3 model, H_t had to be replaced by the differences in level (Δz) between cells; then, according to Zimmermann (*1997*), Eq. (13) can be given as:

$$Q_{i,k} = 2 \frac{b_{i,k} K_{i,k} d_{i,k}^2}{\Delta x_{i,k}^2} \Delta z_{i,k} \tag{14}$$

where $Q_{i,k}$ is the discharge rate, $\Delta x_{i,k}$ is the average distance between the centers of cells *i* and *k* , $b_{i,k}$ is the average width of the link between cells *i* and *k*, and $d_{i,k}$ is the average equivalent depth of cells *i* and *k*.

Validation Tests

In order to test the validity of the new link, the CELSUBS3 model was applied to simple steady-state and unsteady-state problems, contrasting it with the Hooghoudt Eq. (7), the Glover-Dumm Eq. (10), and the De Zeeuw-Hellinga Eq. (12). A symmetrical scheme of parallel drains, shown in Fig. 1, with a 120-m spacing between drains, was used in the simulations. In order to approach two-dimensional flow conditions, a 12,000-m drain length was assumed. The spatial discretization consisted of five cells: an aquifer, two drains, and two cells for defining boundary conditions. The assumed value for the hydraulic conductivity of the soil was 8.64 m/d and its storage coefficient was 0.05.

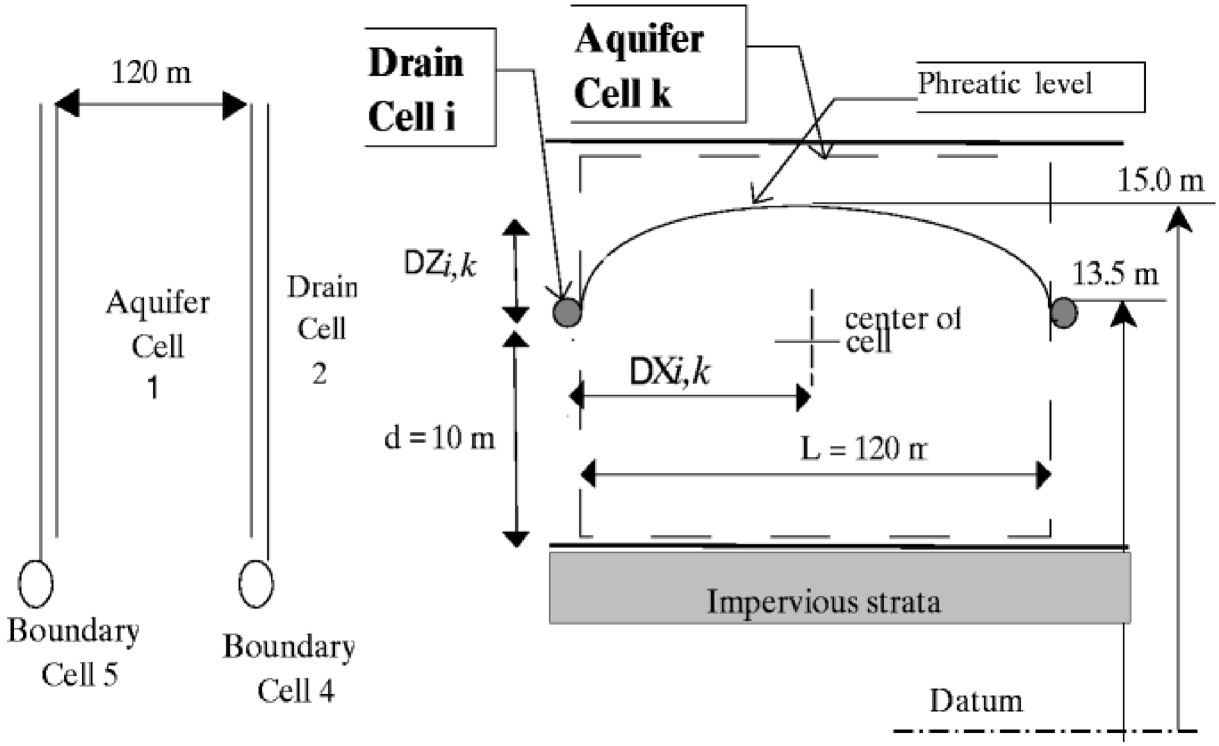

Fig. 1. Scheme of cells for validation purposes

For steady state, a 10-mm/d recharge rate was assumed. The Hooghoudt equation predicted a 0.2083-m water-table elevation *H* in dynamic equilibrium with the recharge, whereas the CELSUBS3 model predicted a 0.2087-m elevation with a 9.9999-mm/d drain-discharge rate. The first unsteady-state simulation consisted of (1) the assumption of an initial water-level elevation of 15 m for the aquifer cell and (2) verification of the necessary time to reach a certain elevation (13.5 m) without any recharge to the system. The model layout is shown in Fig. 1. Figure 2 shows a satisfactory correlation between the application of the Glover-Dumm equation and that of the CELSUBS3 model. The standard error (SE) of the simulated levels is 0.0135 m, calculated according to:

$$SE = \sqrt{\frac{\sum (HS - HT)^2}{N}} \tag{15}$$

where *HS* is the simulated level by applying CELSUBS3, *HT* is the theoretical level by applying analytical equations, and *N* is the number of calculated points.

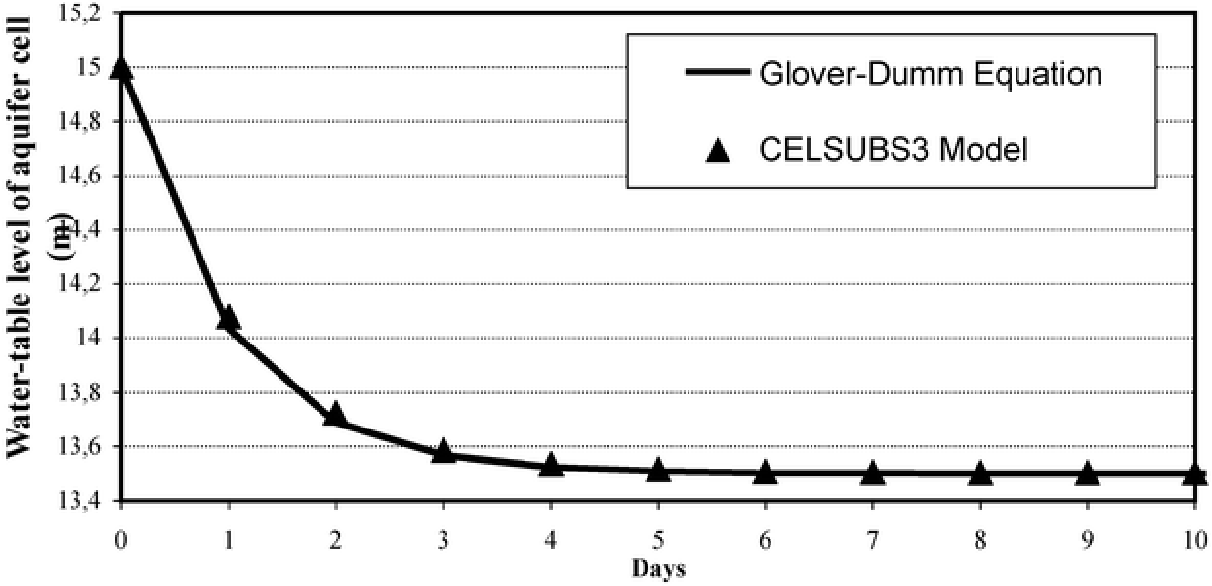

Fig. 2. Comparison of simulated decline of water table with the Glover-Dumm equation and CELSUBS3 model

---

A second unsteady-state simulation consisted of varying the recharge rate with time in such a way that the equation of De Zeeuw-Hellinga can be used to calculate the response of the water table to this variation. Figure 3 shows a satisfactory correlation with the CELSUB3 model simulation. The standard error of the simulated water-level elevations is 0.0431 m.

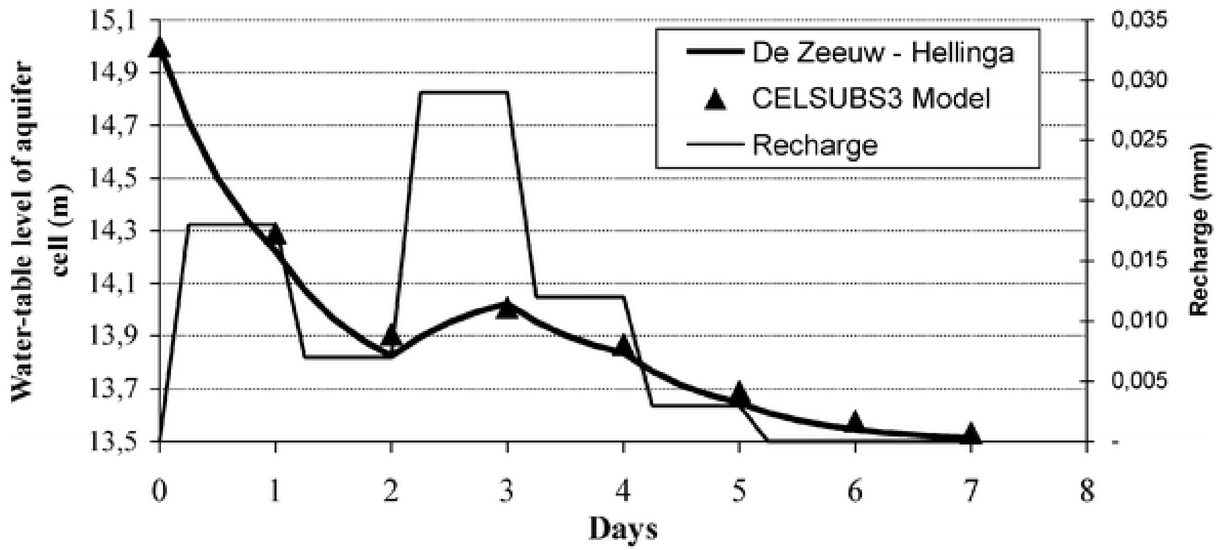

Fig. 3. Comparison of simulated water-table fluctuations in response to recharge by De Zeeuw-Hellinga equation and the CELSUBS3 model

---

An initial conclusion, therefore, is that the CELSUBS3 model, incorporating a link between an aquifer cell and a drain cell, adequately models simple steady-state and transient problems and gives results similar to those of analytical solutions. Potentially, the mathematical model allows the simulation of more complex conditions that are not amenable to analytical solutions.

Application of CELSUBS3 to Subsurface Urban Drainage

Chañar Ladeado Drainage Project

The model was applied to the design of a drainage system for Chañar Ladeado town, Santa Fe, Argentina; locations are shown in Fig. 4. The study area is in a flatland region with low slopes of about of 0.4 m/km and areas with natural surface storage and poor surface drainage. Groundwater is recharged mainly from precipitation (about 1000 mm/year) that infiltrates through unconsolidated Quaternary loess. Chañar Ladeado town has 5200 inhabitants and a municipal water supply system but no sewer system. As a result of rainy years early in the 1990s, the water table reached the ground level, cellars were flooded, and septic tanks were disabled. A project was started to design sewer and drainage systems in order to alleviate this situation (Zimmermann and Portapila *1997*). The design consisted of drain pipes parallel to sewer conduits in the same excavation. The drainage conduits are in a gravel trench and consist of perforated PVC pipes with an envelope of geotextile. The main collectors have a 0.3-m diameter and field drains have a 0.15-m diameter; both are located below the sewer collectors. The aim of the design is to always keep groundwater levels more than 1 m below ground level in the populated areas of the town.

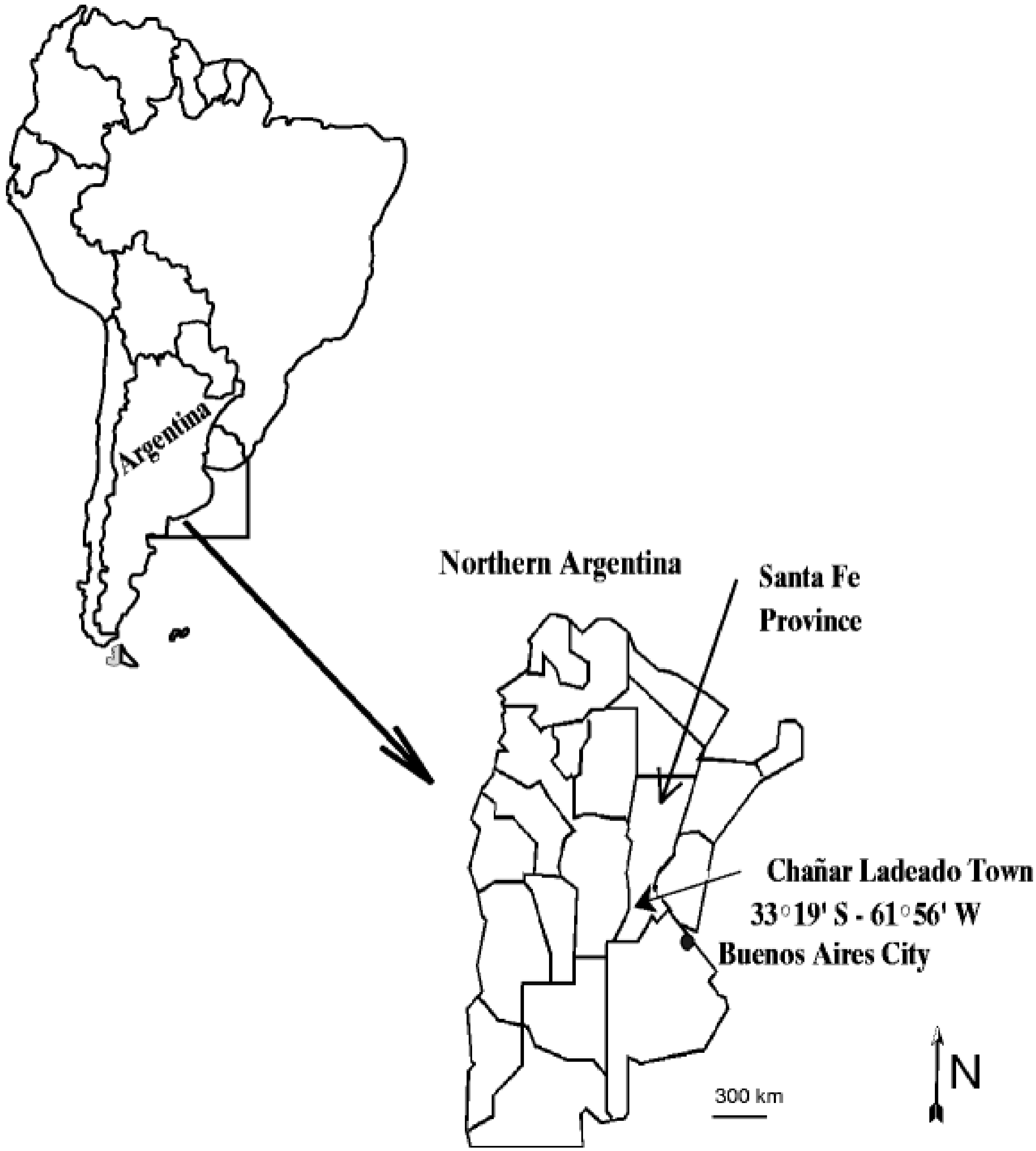

Fig. 4. Location of study area

Recharge Analysis

Temporary variations in the rate of recharge were considered by simulating the variation for annual time periods. In this way it was possible to study the mode of operation of the pumping station, water-level fluctuations in drain pipes, the equilibration time of the whole system with regard to its response to rainy periods, and its recovery in dry periods.

Infiltration was estimated as the difference between rainfall and runoff volumes. An infiltration coefficient as a function of the cover type and its corresponding runoff coefficient affected rainfall data as follows:

$$Cf_k = \frac{(1 - Ce_p)\, A_{pk} + (1 - Ce_i)\, A_{ik}}{A_{tk}} \qquad (16)$$

where Cf_k is the infiltration coefficient of aquifer cell k; Cr_p and A_{pk} are the runoff coefficient and the area of pervious zones of aquifer cell k, respectively; Cr_i and A_{ik} are defined as above but for the impervious areas; and A_{tk} is the total area of cell k. Pervious and impervious areas of each urban block were computed from aerial photographs. As a result, it was possible to estimate that 63% of the whole urban area is impervious. For pervious areas, a runoff coefficient of 50% (Cr_p=0.50) was estimated, and that of the impervious areas was 90% (Cr_i=0.90) (Jacobsen and Harremoes *1981*). Infiltration coefficients were calculated for each urban block using Eq. (16), as shown in Fig. 5. The average value of the infiltration coefficient is 0.28 (areally weighted average of Cf_k for each urban block).

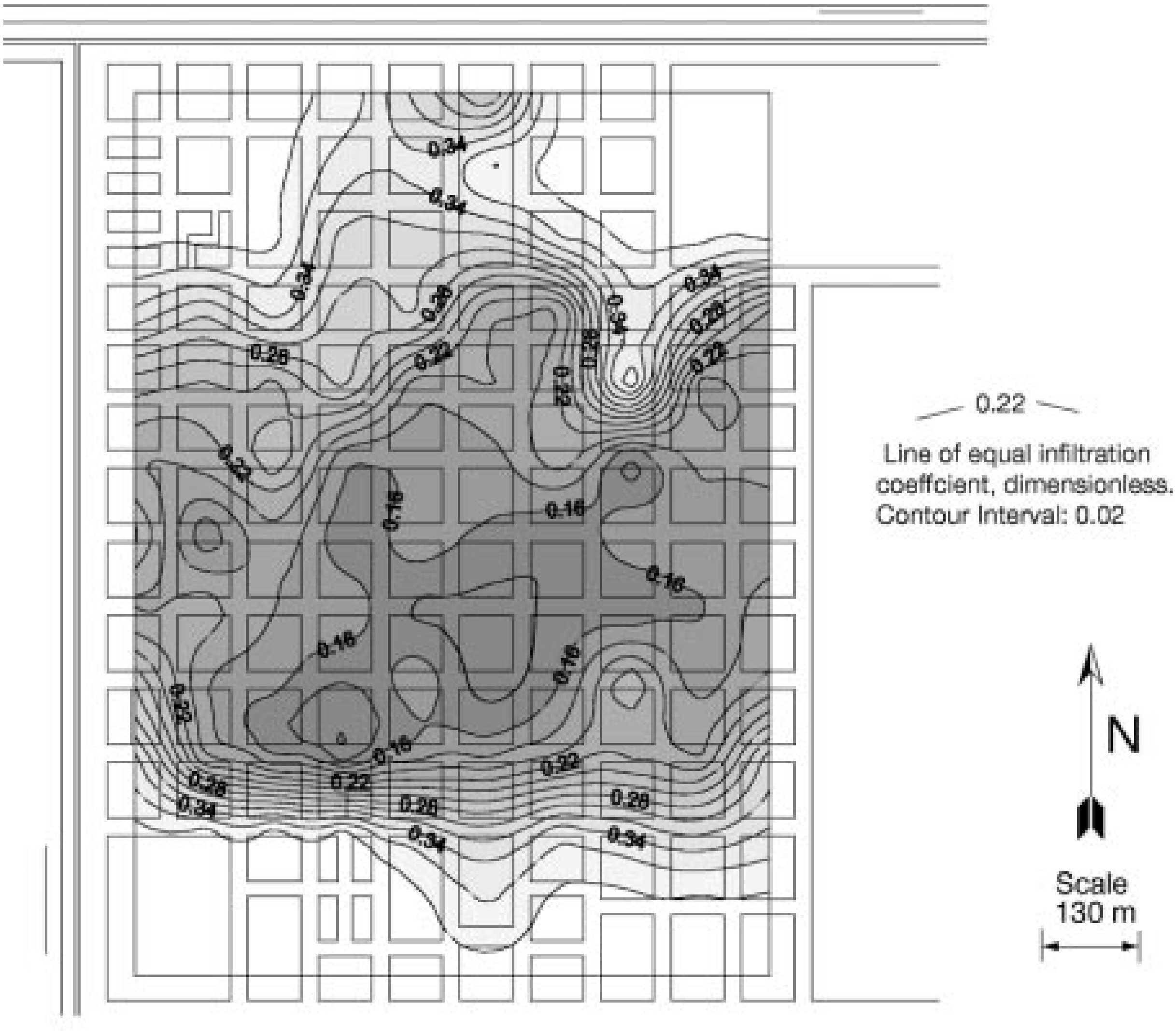

Fig. 5. Modeled area showing distribution of infiltration coefficient

Rainfall data recorded in Chañar Ladeado town for a 64-year period (1928-1992) were analyzed. "Typical" and "critical" years were selected from the observed data. The typical year is the one in which the rainfall amount deviated the least from the average annual amount for the period recorded. A critical year is the one with the largest rainfall amount for the period recorded. The data are shown in Table 1.

Table 1. Monthly rainfall (in mm) for typical and critical years at Chañar Ladeado Gauge Station

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec	Annual
Typical year, 1964	81	92	339	96	22	3	0	27	59	29	97	122	967
Critical year, 1973	117	269	233	122	24	125	85	0	4	143	90	119	1331

Model Configuration

In applying the CELSUBS3 model, urban blocks were considered as 130 × 130-m aquifer cells.

Perforated pipes, enveloped with gravel, drain the aquifer cells. As boundary conditions, fixed water-table elevations were assumed in the peripheral channels. Drain cells were considered to have a length of 130 m. Equivalent depths in the links between aquifer and drain cells were calculated with Eqs. (8) and (9), which resulted in a value of about 10 m. Conduit links between drain pipes were used with Manning's coefficient of 0.011 (Ritzema *1994*).

The aquifer properties were estimated from information concerning the storage coefficients and hydraulic conductivities in preliminary studies in the area. By means of a pumping test, hydraulic-conductivity values of 2.3-5.1 m/d and a storage coefficient of 0.055 were computed (Moloeznik *1970*; Orsolini et al. *1992*).

In addition, four measurements of hydraulic conductivity by means of the inverse auger-hole method (also known as the Porchet method) were carried out. This method consists of boring a hole into the soil and filling this hole with water until the soil below and around the hole is practically saturated. For this situation, the hydraulic conductivity can be assumed as the infiltration rates, which can be measured. The hydraulic conductivity values obtained range from 1.42 to 8.54 m/d. Hydraulic conductivity values were interpolated by statistical methods (kriging), assigning discrete values to each aquifer cell, as shown in Fig. 6.

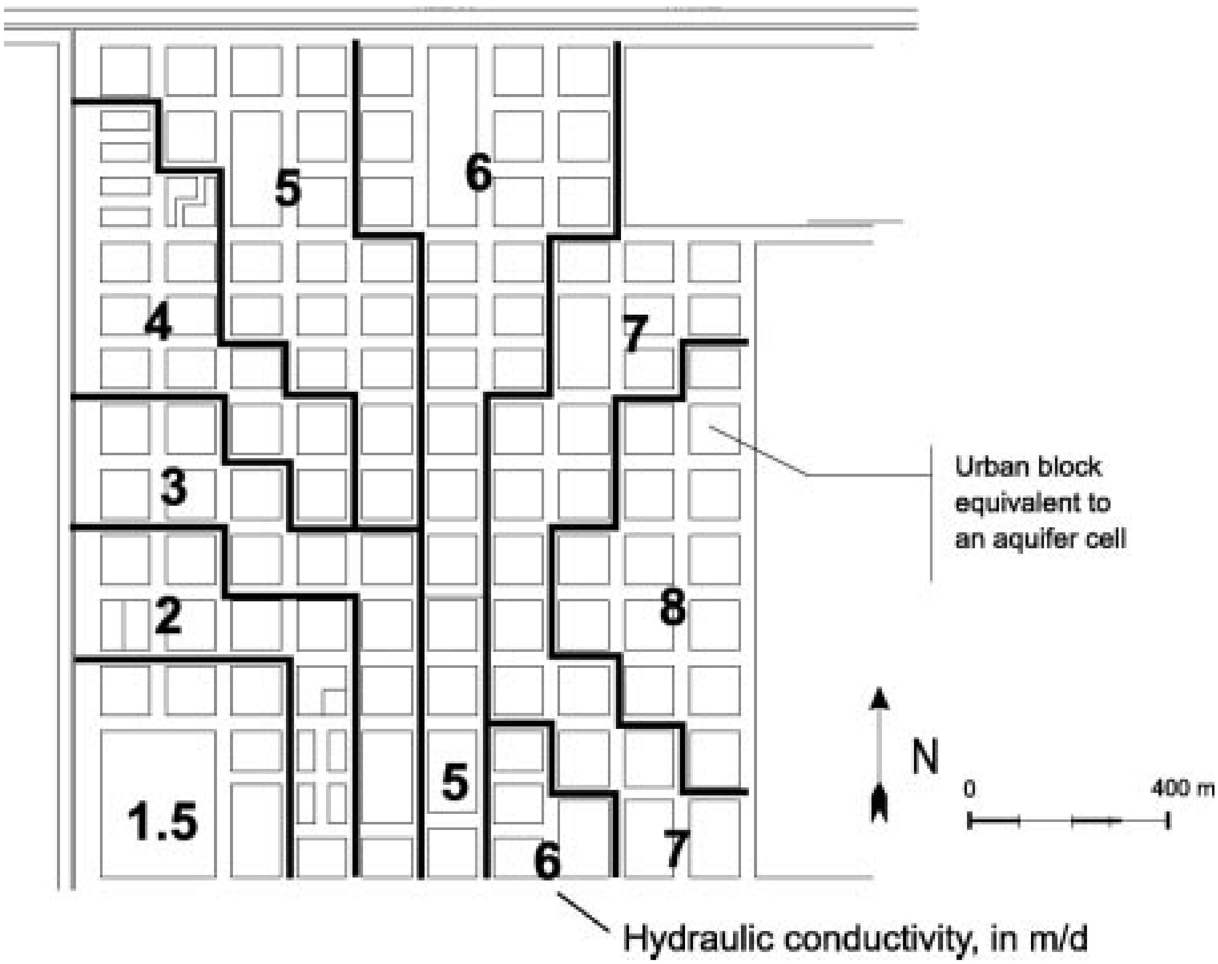

Fig. 6. Modeled area showing interpolated distribution of hydraulic conductivity

From all variants analyzed for the drain-pipe systems, the one that best achieved the technical goal previously mentioned is the one shown schematically in Fig. 7 (the cells do not need to be numbered sequentially). The topology used in the simulations included 130 aquifer cells, 46 peripheral channel cells, 10 internal channel cells, two pumping station cells, two external cells that capture the effluents of the pumping station, and 37 drain-pipe cells, for a total of 227 cells. There were 376 links between cells. For all the simulations, the maximum recorded water-table levels (autumn of 1990) were considered as the initial condition.

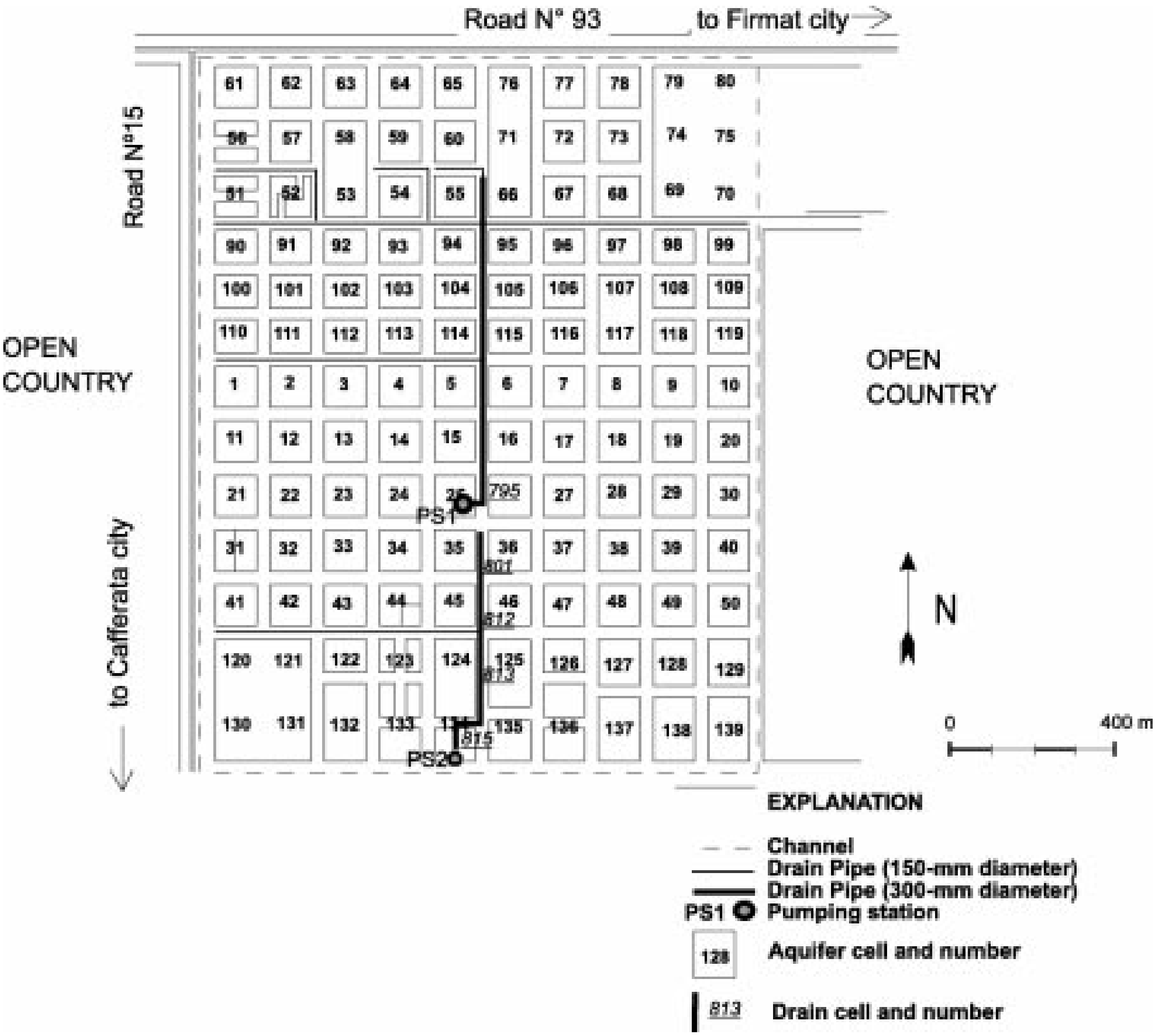

Fig. 7. Modeled area showing layouts of drainage system

Results and Discussion

The typical rainfall year was considered in the evaluation as representing the mean behavior of the system for its design life. The critical rainfall year was used to verify whether the drainage system would provide a complete control of phreatic levels. Additionally, the simulations were carried out with the drain pipes operating at 60% of their hydraulic capacity. This reduction is justified because of the probable existence of obstructions in the conduits, such as sediment deposition or the penetration of tree roots.

Fluctuations of the water table in a critical rainfall year for representative cells are shown in Figs. 8 and 9. Water-table elevations reached their maximum 66 d after the start of the simulation period (Fig. 8). The water levels in the main collector conduit (represented by cells 795-815 in Fig. 9) reach a stationary state 20 d after the start of the simulation. The cell corresponding to the pumping station (PS2) shows erratic changes in its water level, the result of intermittent pumping (see Fig. 7 for locations of cells and pumping stations).

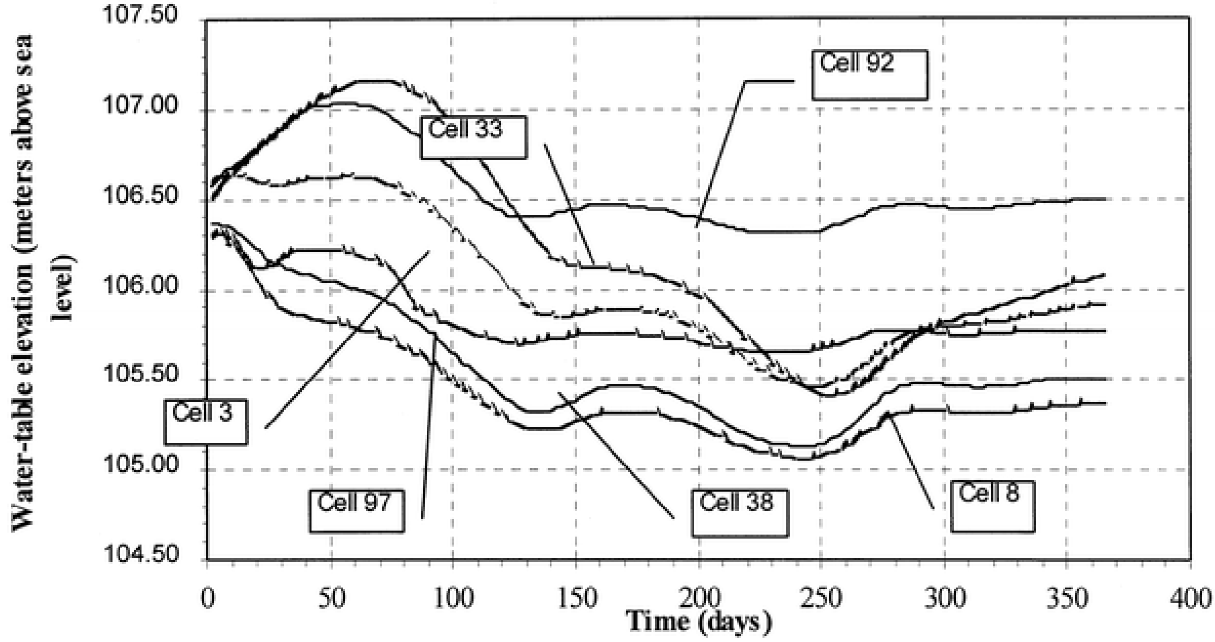

Fig. 8. Hydrographs of water table in representative aquifer cells in critical rainfall year

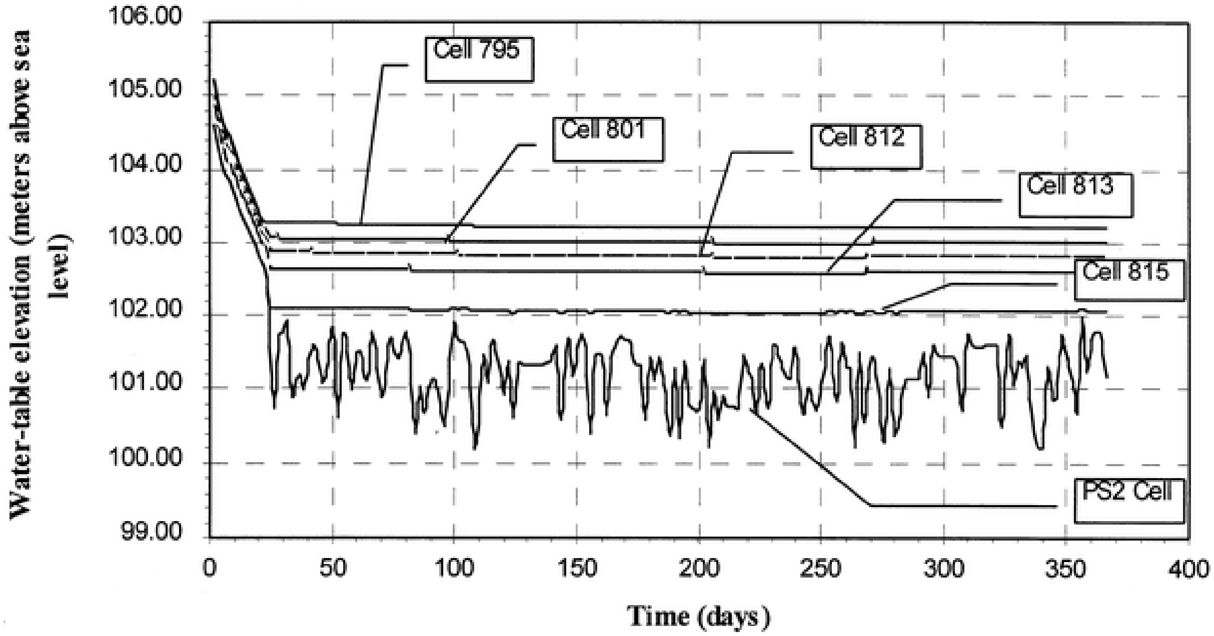

Fig. 9. Hydrographs of water table in representative aquifer cells and in pumping-station cell (*PS2*) in critical rainfall year

---

Contours of equal depth to water table below ground level at 66 d from the start of the simulation are shown in Fig. 10. In the northern part of the town, the depths to the water table are less than 1 m at 20 cells, but the area is practically uninhabited. In a critical rainfall year, the results of the simulation show that the field drains and main collector conduits operated with depth/diameter ratios of 79 and 87%, respectively. A safety margin exists even in the worst situation, since it could be observed that the drain pipes do not work under surcharge conditions.

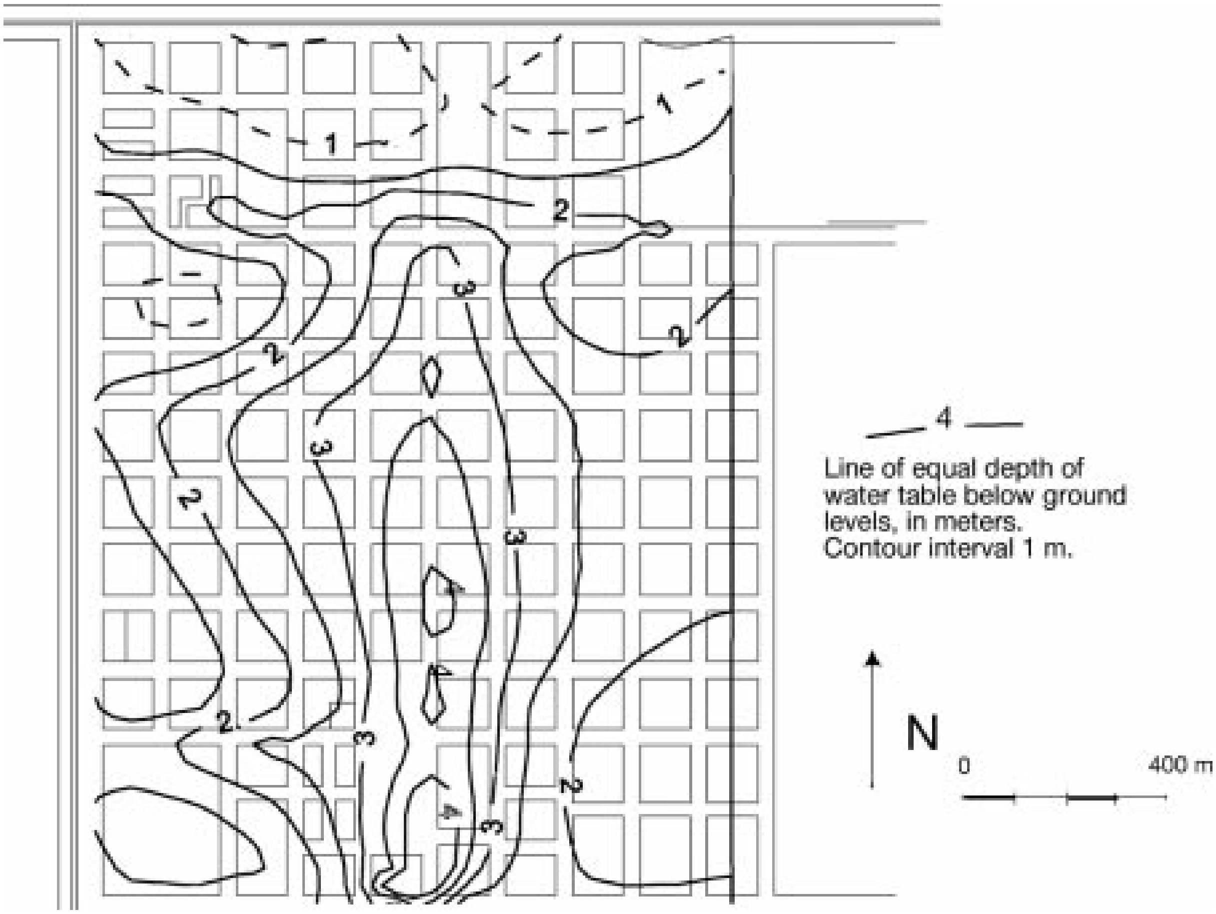

Fig. 10. Modeled area showing distribution of depth to water table for critical rainfall year

Summary and Conclusions

A quasi-two-dimensional model of cells, CELSUBS3, was developed in order to simulate the flow of groundwater drainage systems. This version of the model allows the incorporation and combination of different elements, including aquifer cells, drain-pipe cells, and pumping-station cells using the same scheme of equations and numeric resolution. The model satisfactorily reproduces solutions to simple problems of drainage by well-known analytical methods for steady and transient states. It was applied to the design of a drainage system projected for Chañar Ladeado town, where peripheral channels, drain pipes, and pumping stations compose the system. Temporary variations in annual recharge rates were considered and typical and critical rainfall years were selected from the rainfall records in carrying out the simulations. An infiltration coefficient was proposed as a function of size of the pervious and impervious areas and the runoff coefficients of each urban block. Spatial variations in hydraulic conductivity were interpolated from in-situ measurements made by the inverse auger-hole method.

Simulations with the model CELSUBS3 resulted in calculations of water-level fluctuations and discharges in all components of the drainage system for different design scenarios. These results could not have been obtained with conventional methods, such as the use of analytical equations or analog models, due to the heterogeneity of the components of the system. The outputs of the model made it possible to select the best technical-economic solution from diverse drainage-system designs. It is concluded that the drainage-system design fulfills the objectives and that the proposed CELSUBS3 model constitutes a powerful tool for analyzing and evaluating drainage projects.

References

Cunge J (1975) Two dimensional modeling of flood plains. In: Mahmood K, Yevjevich V (eds) Unsteady flow in open channels. Water Resources Publications, Fort Collins, Colorado, pp 705-762

Cunge J, Wegner M (1964) Integration numérique des équation d'ecoulement de Barré de Saint Venant par un schemé implicité de differences finies. Aplicattion au caso d'une galerie tantôt en charge tantôt à surface libre. Houille Blanche 1:33-39

De Zeeuw J, Hellinga F (1958) Neerslag en afvoer (in Dutch with English Summary). Landbouwk Tijdschr 70:405-422

Ernst LF (1956) Calculation of the steady flow of groundwater in vertical cross-sections. Neth J Agric Sci 4:126-131

Glover F, Dumm L (1960) Validity and use of the transient flow concept in subsurface drainage. Proc American Society of Agricultural Engineers Meeting, Memphis, pp 4-7

Harbaugh AW, McDonald MG (1996) Programmer's documentation for MODFLOW-96, an update to the US Geological Survey modular finite-difference ground-water flow model. US Geological Survey open-file Rep 96-486, pp 131-142

Hooghoudt SB (1940) Algemeene beschouwing van het probleem van de detailontwatering en de infiltratie door middel van parallel loopende drains, greppels, slooten, en kanalen. Versl Landbouwk Onderz 46(14)B

Jacobsen P, Harremoes P (1981) Significance of semi-pervious surfaces in urban hydrology. In: Proc 2nd Int Conf on Urban Storm Drainage, Urbana, Illinois

Karanjac J, Braticevic D (1989) Groundwater software. User's manual. Water Resources Branch, Natural Resources and Energy Division, Department of Technical Co-operation for Development, United Nations, New York

Moloeznik P (1970) Estudio hidrogeológico del area de Chañar Ladeado [Hydrogeologic study for Chañar Ladeado area]. Informe técnico. Cooperativa de agua potable de la Comuna de Chañar Ladeado

Orsolini H, Magnaghi E, Zimmermann E, Bosio M, Fernandez G, Rodriguez J (1992) Proyecto de drenaje subterráneo. Localidad de Chañar Ladeado [Groundwater-drainage project. Chañar Ladeado town]. Informe final. Departamento de Hidráulica, Facultad de Ciencias Exactas, Ingeniería y Agrimensura, Universidad Nacional de Rosario

Riccardi G (1997a) The mathematical modeling of flood propagation for the delimitation of inundation risk zones. In: Rosbjerg D (ed) Sustainability of water resources under increasing uncertainty. IAHS Publ 240, pp 355-364

Riccardi G (1997b) El mapeo de riesgo de inundación por medio de la modelación matemática hidrodinámica [Flood risk mapping by means of hydrodynamic mathematical modelling]. Rev Ing Agua 4:45-56

Riccardi G, Zimmermann E (1993) Mapas de riesgo de inundaciones aguas abajo de la presa de retención de crecidas del arroyo Ludueña [Flood risk maps downwater of Ludueña River dam]. In: Proc 1er Simp Sobre Recursos Hídricos del Cono Sur. Asociación Brasileña de Recursos Hídricos, Parana, pp 535-544

Riccardi G, Zimmermann E (1999) La modelación matemática bidimensional del escurrimiento subterráneo mediante esquemas de celdas [Two-dimensional mathematical modelling of groundwater flow by means of cell schemes]. In: Proc 2nd Congr Argentino de Hidrogeología, Santa Fe

Riccardi G, Zimmermann E, Maurig R (1995). Mathematical modeling of flood for the delimitation of rural, semi-urbanized and urbanized zones with inundation risk. In: Runoff computations for water projects. IHP-IV Project M-1-4. UNESCO, Paris, pp 127-138

Ritzema HP (1994) Drainage principles and applications. Publ 16, 2nd edn. International Institute for Land Reclamation and Improvement, Wageningen

Van der Moolen W, Wesseling J (1991) A solution in closed form and a series solution to replace the tables for the thickness of the equivalent layers in Hooghoudt’s drains spacing formula. Agric Water Manage 19:1-16

Zimmermann E (1997) Informe Anual de la Carrera de Investigador [Annual report for the Research Council]. CONICET, Centro Universitario Rosario de Investigaciones Hidroambientales, Facultad de Ciencias Exactas, Ingeniería y Agrimensura, Universidad Nacional de Rosario

Zimmermann E, Portapila M (1997) Actualización proyecto de obra cloacal y drenaje subterráneo de la Comuna de Chañar Ladeado [Bringing up to date of sewer and groundwater-drainage projects for Chañar Ladeado town]. Informe final. Departamento de Hidráulica, Facultad de Ciencias Exactas, Ingeniería y Agrimensura, Universidad Nacional de Rosario

Zimmermann E, Riccardi G (1995) A preliminary model for the hydrological simulation in flatlands areas. In: Proc Int Symp on Runoff Computations for Water Projects, International Association for Hydrological Sciences, St. Petersburg