COMPUTATIONAL MODEL FOR ANALYSIS SPREAD IN FLOOD CHANNELS URBAN DRAINAGE

MODELO COMPUTACIONAL PARA EL ANÁLISIS DE LA PROPAGACIÓN DE LA INUNDACIÓN EN LOS CANALES DE DRENAJE URBANO

Stênio de Sousa Venâncio*
Luiz Fernando Resende dos Santos Anjo**

ABSTRACT

The scenario analysis for knowledge of the transitional system of urban macro drainage work is necessary for the planning of structural and nonstructural measurements. To this end, a computational model 1D is presented to study the flood propagation in urban drainage channels. This work investigates the unsteady flow in the Cocó River estuary, located in the city of Fortaleza-CE. It is one of the cases studied in the first author's graduate program, which proposes a computation model to simulate unsteady flows of open channels with many purposes (such as hydroelectric power, water supply, irrigation, etc.) and contributes to automation of their operational control systems . The determination of maximum water level achieved along the estuary is the aim of this study, having practical application on the definition of elevation of streets, avenues and new constructions to be executed on the border or inside the flood areas. The complete 1D hydrodynamic equations of Saint-Venant are approximated by a completely implicit method of finite differences and conveniently discretized for the model, which was developed in FORTRAN language. The flow given by the entrance hydrograph of the analyzed estuary (upstream boundary condition) was obtained for a return period of 50 years. The water depth is the boundary condition downstream of the problem, and its variation, obtained by measuring the tide in a 24 hour period, was approached by a mathematical function. This function was obtained for the purpose of measuring the maximum water level that occurs in the estuary. Was still considered to two lateral hydrographs and an inflow distributed along the estuary. The unsteady flow analysis is based on the temporal results of water level and flow at several cross sections of the estuary.
Keywords: urban flooding, macrodrainage, computational model.

RESUMEN

El análisis de escenarios para el conocimiento del sistema transitorio de obra de drenaje urbano macro son necesaria para la planificación de medidas estructurales y no estructurales. Con este fin, se presenta un modelo computacional 1D para estudiar la propagación de inundación en los canales de drenaje urbano. Este trabajo investiga el flujo transitorio en la desembocadura del río Cocó, ubicado en la ciudad de Fortaleza-CE. Es uno de los casos estudiados en el programa de posgrado del primer autor, que propone un modelo de computación para simular flujos inestables de canales abiertos con muchos propósitos (por ejemplo, energía hidroeléctrica, abastecimiento de agua, riego, etc.) y contribuye a la automatización de sus sistemas de control operacional. La determinación del nivel de agua máximo alcanzado a lo largo del estuario es el objetivo de este estudio, teniendo aplicación práctica en la definición de la elevación de las calles, avenidas y nuevas construcciones para ser ejecutado en la frontera de o dentro de las áreas de inundación. Las ecuaciones hidrodinámicas completa de 1D de Saint-Venant son aproximadas por un método totalmente implícito de diferencias finitas y discretizar convenientemente para el modelo, que fue desarrollado en lenguaje FORTRAN. El flujo dado por el hidrograma de entrada de la ría analizado (condición de frontera aguas arriba) se obtuvo para un periodo de retorno de 50 años. La profundidad del agua es la condición de frontera aguas abajo del problema, y su variación, obtenidos mediante la medición de la marea en un período de 24 horas, fue abordado por una función matemática. Esta función se obtuvo con el propósito de medir el nivel máximo de agua que se produce en el estuario. Todavía era considerado a dos hidrogramas laterales y una afluencia distribuidos a lo largo del estuario. El análisis de flujo transitorio se basa en los resultados temporales de flujo en varias secciones transversales de la ría y el nivel del agua.
Palabras clave: las inundaciones urbanas, macrodrenaje, modelo computacional.

* Prof. Adjunto II - Departamento de Engenharia Civil, Instituto de Ciências Tecnológicas e Exatas - ICTE, Universidade Federal do Triângulo Mineiro - UFTM

** Professor Adjunto III da Universidade Federal do Triângulo Mineiro (UFTM) e Diretor do Instituto de Ciências Tecnológicas e Exatas (ICTE) da UFTM, Departamento das Engenharias, Universidade Federal do Triângulo Mineiro

Recibido: 10/7/2012
Aceptado: 31/08/2013

1. INTRODUCTION

The growth of cities was not accompanied by governing instruments for the soil use and occupation. As stated by CRUZ & TUCCI (2008):

> "According to IBGE (2000), considering 5.507 Brazilian municipalities, only 841 have Urban Master Plans (PDU) (15.3%), and of these, only 489 in the latest version of the plan are later than 1990 (8.9%). When analyzing only those cities with population exceeding 20,000 inhabitants, 485 have PDU from a total of 1483 municipalities (32.7%). Even where there are PDUs, most of these plans only deal with architectural aspects, without considering environmental effects and especially on **the drainage infrastructure**. Within the urban practices that have spread across the country, the use of valleys avenues associated with the plumbing of urban streams was observed. This sort of urbanization amplifies the impacts and changes the environment in an inappropriate way. Solutions of this type generally have a cost much higher than a sustainable solution and increased losses due to flooding, erosion and water quality. The sum of technical ignorance of an important part of professionals working on drainage, population and policy makers have kept this scene."

Changing this setting requires changing the strategic pattern of the city integrated planning and it involves: urban planning and soil use, sanitation, solid waste and urban drainage. All these elements have strong interference with each other and require integrated solutions.

All this dynamic and the increased frequency of floods in recent decades have become increasingly important to incorporate tools to help the decision making process in the management of urban drainage.

The change in rainfall patterns, widely treated by the media on the issue of global climate change, is an aggravating factor in the secure management of urban infrastructure in terms of flooding, with additional impact on the coastal cities where the tide cycle is decisive in determining timing of water levels in the drainage channel.

According to Mark et al. (2004), with the current advances in computer technology, many cities in developed countries use computer simulations to solve their local problems of flooding. The practice involves building models of drainage systems using applications such as MOUSE (Abbott et al., 1982, Lindberg & Jørgensen, 1986), SOBEK-Urban (Heeringen et al., 2002), InfoWorks CS (Chan & Vass, 2002) and SWMM (Huber & Dickinson, 1992). Based on simulation results, mitigation measures can be evaluated and the optimal solution can be implemented. The main caveat with regard to commercial software packages is in the restriction of access to source code, which complicates the understanding of certain types of errors, besides many of them do not include important parameters in the analysis process, or are not used properly by users. One of these parameters is sediment transport, whose effect impact in reducing flow capacity of the urban drainage channels. Some conclusions about the hydrosedimentological simulation are presented at the end, from the literature review by Venâncio (2009). The description of the item 2.2 (Numerical Model) is presented in Venâncio, Sousa and; Villela (2005).

2. METHODOLOGY.

2.1 Study Area.

The Rio Coco is part of the basin of the rivers in the east coast of Ceará, with its catchment area of approximately 485 km ², with a total length of the main river about 50 km (SEMACE, 2012). The river runs through the municipalities of Pacatuba, Maracanaú and Fortaleza, and its source is situated on the eastern slope of the Sierra Aratanha. Its mouth is on the edge of the beaches of Hunting and Fishing and Sabiaguaba, thus emptying into the Atlantic Ocean. The Coco River Basin is considered the largest river in the city of Fortaleza, it has 25 km of its length in the capital.

The area chosen for analysis in question is the entire area that is related to influence Coco River estuary, and comprises approximately 15 km of the length of the river from its mouth upstream, within the limits of the city of Fortaleza (Figure 1). The portion of the river, which is the state capital of Ceará, has mangrove ecosystem associated with fluvial-marine plain, which occupies the excerpts from the river located on BR-116 to its mouth, where it forms an estuary.

These areas due to favorable conditions such as mixing of saline water with fresh water, soil type, low coefficients of oxygen in the soil (ie, reducing environment) and tidal regimes, the dominant species are Rhizophora mangle L, Avicenia Schaveriana Stapf and Leech, and Laguncularia racemosa (SEMACE, 2012).

Natural environments are subject Cocó intensely human activity that intervenes significantly through constructions of households, urban roads, public facilities and commercial developments. These urban facilities directly interfere in sedimentary processes, morphological, ecological and oceanographic region. Historically, since 1933 the Estuary Coco concentrated low-income housing, and currently has a number of environmental and social problems arising from lack of urban planning, mainly due to population growth and inadequate implementation of structural designs, such as sanitation basic fluvial drainage works, construction of affordable housing and other forms of occupation (RGP, 2009).

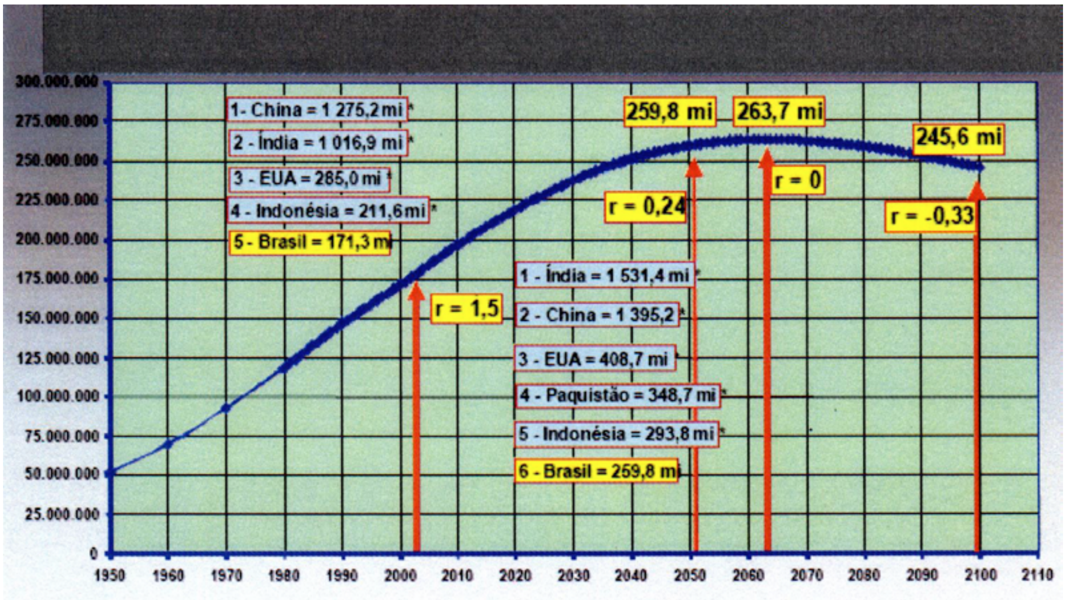

Figure 1. Location of the study area in Fortaleza/CE.

Even being inserted in the semi-arid climate, its location change this reality by being among nearby hills causing summer rains occur more often in the city and surrounding area than in the rest of the state. The average annual temperature is 26° C. The average rainfall is about 1600 mm. Without the seasons well defined, there is only the rainy season, from January to July and the dry from August to December. With most of the sandy soil agriculture becomes of little economic significance, and since the 1990 the entire length of the city was considered urban area (WIKIPEDIA, 2013).

The average flow in the estuary, in steady state, is 10 m^3/s, obtained by measuring station river. The monitoring of historical rainfall data and qualitative and quantitative water resources of the Basin estuarine Coco are managed by the COGERH (Company Water Resources Management) from the state government of Ceará.

2.2 Numerical Model

The mathematical model conceptual employed to study the free flow in the transient regime with bidirectional flow, is set out in the literature, for validations generated in various applications. It is composed of the hydrodynamic equations completes, Continuity and amount movement, known as the Saint-Venant equations.

2.2.1 Governing Equations

The continuity equation follows the way presented by Henderson (1966):

$$\frac{\partial y}{\partial t} + \frac{1}{B}\frac{\partial Q}{\partial x} \pm \frac{q_{LAT}}{B} = 0 \qquad (1)$$

where $\partial y/\partial t$ is the rate of temporal variation of water height considering the bottom of the channel; $\partial Q/\partial x$ rate of spatial variation of flow, B the width of the free surface, and q_{LAT} the intake of side flow in meters from the banks. From the relation $\partial y/\partial t = \partial h/\partial t - \partial z/\partial t$, where $\partial h/\partial t$ is the variation of water height relative to a horizontal reference plane at the considered time and $\partial z/\partial t = 0$ (once the bottom slope does not vary with time), the general equation can be rewritten as:

$$\frac{\partial h}{\partial t} + \frac{1}{B}\frac{\partial Q}{\partial x} \pm \frac{q_{LAT}}{B} = 0 \qquad (2)$$

The equation of movement amount is given as follows:

$$S_f - S_0 + \frac{\partial y}{\partial x} + \frac{V}{g}\frac{\partial V}{\partial x} + \frac{1}{g}\frac{\partial V}{\partial t} = 0 \qquad (3)$$

where S_f is the slope of the power line; S_0 to the channel bottom slope; $\partial y/\partial x$ the rate of spatial variation of water depth; $\partial V/\partial x$ rate of spatial variation of mean flow velocity; $\partial V/\partial t$ the rate of temporal variation of mean velocity and g is the acceleration of gravity. Substituting S_0 by $\pm \partial z/\partial x$ and S_f by $V^2/C_H^2.R_H = Q^2/A^2.C_H^2.R_H$ in Eq. (3), it can be written as:

$$\frac{Q^2}{A^2 C_H^2 R_H} \pm \frac{\partial z}{\partial x} + \frac{\partial y}{\partial x} + \frac{V}{g}\frac{\partial V}{\partial x} + \frac{1}{g}\frac{\partial V}{\partial t} = 0 \qquad (4)$$

where A is the cross-sectional area, and R_H is the hydraulic radius of the section, and C_H the Chezzy coefficient, a function of hydraulic radius and Manning roughness coefficient (C_H=R_H1/6/n) determined for each step of time of the established discretization. Introducing the relation ± ∂z/∂t = ∂h/∂x - ∂y/∂x in Eq. (4) and multiplying it by g.A², it becomes:

$$\frac{\partial Q}{\partial t}A + Q\frac{\partial Q}{\partial x} + \frac{g}{C_H^2 R_H}Q|Q| + gA^2\frac{\partial h}{\partial x} = 0 \qquad (5)$$

2.2.2 Discretization

One of the most used schemes for the variable flow analysis in channels is the implicit finite difference scheme by Preissmann (apud LIGGET and CUNGE 1975), given by:

$$\frac{\partial f}{\partial t} = \frac{1}{\Delta t}\left[\phi\left(f_{i+1}^{k+1} - f_{i+1}^{k}\right) + (1-\phi)\left(f_i^{k+1} - f_i^{k}\right)\right]$$

$$\frac{\partial f}{\partial x} = \frac{1}{\Delta x}\left[\theta\left(f_{i+1}^{k+1} - f_{i}^{k+1}\right) + (1-\theta)\left(f_{i+1}^{k} - f_i^{k}\right)\right] \qquad (6)$$

θ and ϕ are weighting factors which, for ϕ = 0,5 e θ = 1 a fully implicit scheme considered by Preissmann is presented as follows:

$$\frac{\partial f}{\partial t} = \frac{1}{2}\left[\frac{\left(f_{i+1}^{K+1} - f_{i+1}^{K}\right)}{\Delta t} + \frac{\left(f_i^{K+1} - f_i^{K}\right)}{\Delta t}\right]$$

and $$\frac{\partial f}{\partial x} = \frac{f_{i+1}^{K+1} - f_i^{K+1}}{\Delta x} \qquad (7)$$

where the average of the variable f is calculated by

$\bar{f} = \dfrac{f_{i+1}^{K} + f_i^{K}}{2}$, which i would represent the sections,

k the calculation time and f the representative value of any variable of the problem where, for the presented case is given by Q (m³ / s) and h (m) .

Discretizing the continuity equation Eq. (2) for this scheme, it follows

$$\frac{1}{2}\left[\frac{h_{i+1}^{k+1} - h_{i+1}^{k}}{\Delta t} + \frac{h_i^{k+1} - h_i^{k}}{\Delta t}\right] + \frac{1}{\bar{B}}\left[\frac{Q_{i+1}^{k+1} - Q_i^{k+1}}{\Delta x}\right] \pm \frac{q_{LAT}}{\bar{B}} = 0 \qquad (8)$$

which is multiplied by 2Δ*t* and it is

$$h_{i+1}^{k+1} - h_{i+1}^{k} + h_i^{k+1} - h_i^{k} + \frac{2\Delta t}{\Delta x}\frac{1}{\bar{B}}\left(Q_{i+1}^{k+1} - Q_i^{k+1}\right) \pm 2\Delta t\frac{q_{LAT}}{\bar{B}} = 0 \qquad (9)$$

Defining $\dfrac{2\Delta t}{\Delta x}$ as α and rearranging Eq. (9) in terms of K and K +1, you can write it as

$$h_{i+1}^{k+1} + h_i^{k+1} + \frac{\alpha}{\bar{B}}Q_{i+1}^{k+1} - \frac{\alpha}{\bar{B}}Q_i^{k+1} = h_i^{k} + h_{i+1}^{k} \pm 2\Delta t\frac{q_{LAT}}{\bar{B}} \qquad (10)$$

As $h_{i+1}^{k} + h_i^{k} = 2\bar{h}$ the Eq. (10) is

$$-\frac{\alpha}{\bar{B}}Q_i^{k+1} + h_i^{k+1} + \frac{\alpha}{\bar{B}}Q_{i+1}^{k+1} + h_{i+1}^{k+1} = 2\bar{h} \pm 2\Delta t\frac{q_{LAT}}{\bar{B}} \qquad (11)$$

Therefore

$$A_J = -\frac{\alpha}{\bar{B}};\ B_J = 1;\ C_J = \frac{\alpha}{\bar{B}};\ D_J = 1;\ E_J = 2\bar{h} + 2\Delta t\frac{q_{LAT}}{\bar{B}}$$

(for side flow input), and $E_J = 2\bar{h} - 2\Delta t\dfrac{q_{LAT}}{\bar{B}}$

(for side flow output), reminding that if there is no entry and exit of side flow, the term E_J becomes $E_J = 2\bar{h}$ the discrete equation of continuity is the following

$$A_J V_i^{k+1} + B_J h_i^{k+1} + C_J V_{i+1}^{k+1} + D_J h_{i+1}^{k+1} = E_J \qquad (12)$$

Applying the approximation scheme for the movement amount equation Eq. (5) and following the same steps taken earlier, it is

$$\left(1 - \alpha\frac{\bar{Q}}{\bar{A}} + \frac{g\Delta t|\bar{Q}|}{\bar{A}\bar{C}_H^2\bar{R}_H}\right)Q_i^{k+1} - \alpha g\bar{A}h_i^{k+1} +$$

$$\left(1 + \alpha\frac{\bar{Q}}{\bar{A}} + \frac{g\Delta t|\bar{Q}|}{\bar{A}\bar{C}_H^2\bar{R}_H}\right)Q_{i+1}^{k+1} + \alpha g\bar{A}h_{i+1}^{k+1} = 2\bar{Q} \qquad (13)$$

with $\bar{Q} = \dfrac{Q_{i+1}^{k} + Q_i^{k}}{2}$ and $Q_{i+1}^{k} + Q_i^{k} = 2\bar{Q}$.

Reducing the terms of the equation for

$$A_{JL}=\left(1-\alpha\frac{\bar{Q}}{\bar{A}}+\frac{g\Delta t\left|\bar{Q}\right|}{\bar{A}\bar{C}_H^2\,\bar{R}_H}\right);\ B_{JL}=-\alpha g\,\bar{A};\ C_{JL}=\left(1+\alpha\frac{\bar{Q}}{\bar{A}}+\frac{g\Delta t\left|\bar{Q}\right|}{\bar{A}\bar{C}_H^2\,\bar{R}_H}\right);\ D_{JL}=\alpha g\,\bar{A}\ \text{e}\ E_{JL}=2\,\bar{Q}$$

the equation of movement is treated as

$$A_{JL}Q_i^{k+1}+B_{JL}h_i^{k+1}+C_{JL}Q_{i+1}^{k+1}+D_{JL}h_{i+1}^{k+1}=E_{JL}\quad(14)$$

3. APLICATION

The studied estuary was spatially discretized into 32 sections (N_Z = 32 sections), listed from upstream to downstream, with a total length L = 15.500m. The boundary conditions of the problem are the upstream input hydrograph and downstream tide equation. Two side hydrographs are still considered. The outline of the problem is shown in Figure 1 below.

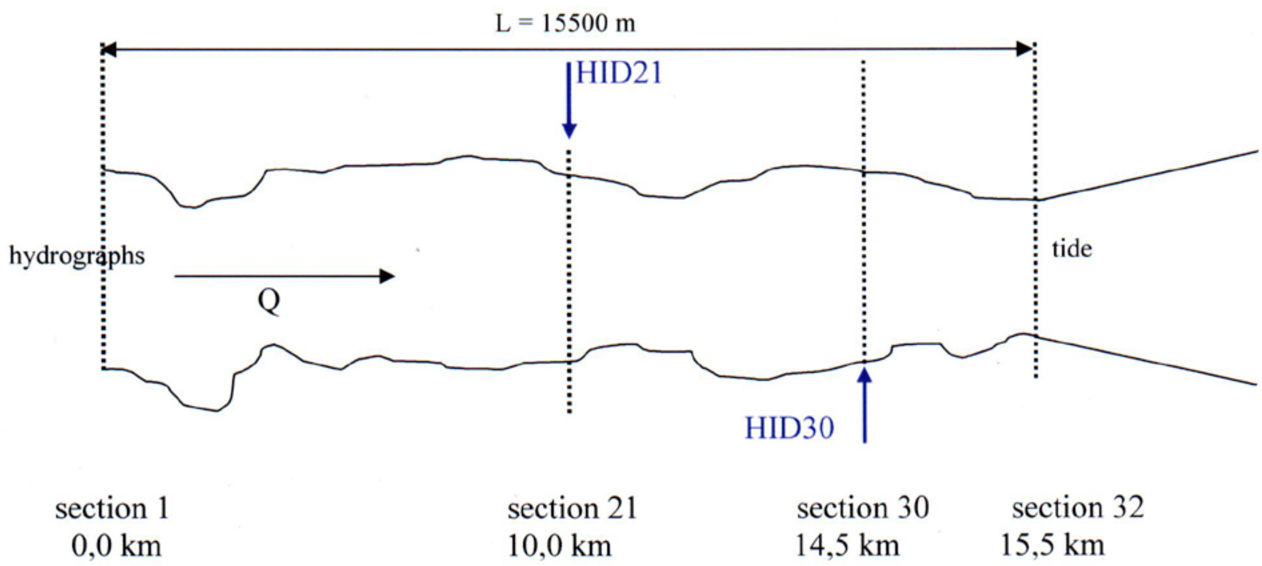

Figure 2 - The Cocó River estuary Scheme to the application of the model, with HID21 and HID30 representing the two side hydrographs.

For the discretized mathematical equations above, the following linear system occurs:

$$\begin{aligned}
&A_{J1}Q_1^{K+1}+B_{J1}h_1^{K+1}+C_{J1}Q_2^{K+1}+D_{J1}h_2^{K+1}=E_{J1}\\
&A_{JL1}Q_1^{K+1}+B_{JL1}h_1^{K+1}+C_{JL1}Q_2^{K+1}+D_{JL1}h_2^{K+1}=E_{JL1}\\
&A_{J2}Q_2^{K+1}+B_{J2}h_2^{K+1}+C_{J2}Q_3^{K+1}+D_{J2}h_3^{K+1}=E_{J2}\\
&A_{JL2}Q_2^{K+1}+B_{JL2}h_2^{K+1}+C_{JL2}Q_3^{K+1}+D_{JL2}h_3^{K+1}=E_{JL2}\\
&\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\\
&\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\\
&A_{J31}Q_{31}^{K+1}+B_{J31}h_{31}^{K+1}+C_{J31}Q_{32}^{K+1}+D_{J31}h_{32}^{K+1}=E_{J31}\\
&A_{JL31}Q_{31}^{K+1}+B_{JL31}h_{31}^{K+1}+C_{JL31}Q_{32}^{K+1}+D_{JL31}h_{32}^{K+1}=E_{JL31}\quad(15)
\end{aligned}$$

where the overwritten index of Q and h represent the time of calculation, and the subscribed index represent the considered sections. The values A, B, C, D and E are determined by expressions previously developed in an explicit way. Thus we can conclude that for N_Z = n sections, the system consisted of 2.(*n*-1) = *equations* and 2.*n*= *unknowns* what in this case represents a number of 62 equations and 64 unknowns. Introducing the boundary system, this becomes:

$$
\begin{aligned}
&h_1 = F_1\\
&A_{J1}Q_1^{K+1} + B_{J1}h_1^{K+1} + C_{J1}Q_2^{K+1} + D_{J1}h_2^{K+1} = E_{J1}\\
&A_{JL1}Q_1^{K+1} + B_{JL1}h_1^{K+1} + C_{JL1}Q_2^{K+1} + D_{JL1}h_2^{K+1} = E_{JL1}\\
&A_{J2}Q_2^{K+1} + B_{J2}h_2^{K+1} + C_{J2}Q_3^{K+1} + D_{J2}h_3^{K+1} = E_{J2}\\
&A_{JL2}Q_2^{K+1} + B_{JL2}h_2^{K+1} + C_{JL2}Q_3^{K+1} + D_{JL2}h_3^{K+1} = E_{JL2}\\
&\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\\
&\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\\
&A_{J31}Q_{31}^{K+1} + B_{J31}h_{31}^{K+1} + C_{J31}Q_{32}^{K+1} + D_{J31}h_{32}^{K+1} = E_{J31}\\
&A_{JL31}Q_{31}^{K+1} + B_{JL31}h_{31}^{K+1} + C_{JL31}Q_{32}^{K+1} + D_{JL31}h_{32}^{K+1} = E_{JL31}\\
&Q_{32} = F_{32}
\end{aligned}
\tag{16}
$$

and therefore a system with 64 equations and 64 unknowns. $F_1 = h_1$ is obtained by downstream tide equation as follows:

$$F_1 = h_1 = h_{MARÉ} = y_{inic} + A\,sen\left[\left(\frac{2\Pi}{T}\right)t\right] + z_1 \tag{17}$$

Where:

yinic = 1.00 m = initial height of the tide in section 1 (when.t = 0h); A = 1.60 m = height of the tide; Π = 3.141592654; T = 12 H = period of the tide; z_1 = share of the channel bottom in section 1; t = time of computation in hours; $F_{32} = Q_{32}$, is the upstream boundary condition attributed directly to the characteristic hydrograph of the estuary.

To solve the system, are given further: Qi = 10.00 m3 / s = initial flow in the channel; g = 9.81 m/s2 = acceleration of gravity; z = 500 m = spacing between sections; Dt = 3600 s = time interval of calculation; Nt = 6 = number of time intervals for calculating; q_{LAT} = 0.0001 m3/sm = lateral contribution; n = 0.035 = Manning coefficient; Nz = 32 = number of discrete sections; HID21 = side hydrograph in section 21; HID30 = side hydrograph in section 30; ALT = initial height of water, obtained by the energy equation via Step Method; QUOTA = bottom of the channel quota obtained topographically.

The system of equations is then prepared in a matrix, according to Fortuna (2000), expressed as in Figure 3, to be solved numerically. The Matrix A and vector B are explicitly calculated in time K and the vector U obtained implicitly by solving the system at time K+1.

3.1 Computational Model

With the aim of implementing the simulation of transient flow in open channels, with discretization of the Saint-Venant 1D equations through the scheme implicit by Preissmann, a computer model in FORTRAN language was developed. The model applied in this study can be extended to other cases making adjustments in geometry, boundary conditions, temporal and spatial discretization. In cases of natural channels, the approach must be careful, because the geometry of the sections is approximated by mathematical functions that may exhibit divergence in the limits of maximum and minimum water depth. The block diagram of the model is shown in figure 4.

$$
\underset{\textbf{Matrix A}}{\begin{bmatrix}
1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0\\
B_{J1} & A_{J1} & D_{J1} & C_{J1} & 0 & 0 & 0 & 0 & 0 & 0\\
B_{JL1} & A_{JL1} & D_{JL1} & C_{JL1} & 0 & 0 & 0 & 0 & 0 & 0\\
0 & 0 & B_{J2} & A_{J2} & D_{J2} & C_{J2} & 0 & 0 & 0 & 0\\
0 & 0 & B_{JL2} & A_{JL2} & D_{JL2} & C_{JL2} & 0 & 0 & 0 & 0\\
- & - & - & - & - & - & - & - & - & -\\
- & - & - & - & - & - & - & - & - & -\\
0 & 0 & 0 & 0 & 0 & 0 & B_{J31} & A_{J31} & D_{J31} & C_{J31}\\
0 & 0 & 0 & 0 & 0 & 0 & B_{JL31} & A_{JL31} & D_{JL31} & C_{JL31}\\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1
\end{bmatrix}}
*
\underset{\textbf{Vetor U}}{\begin{bmatrix}
h_1\\ Q_1\\ h_2\\ Q_2\\ h_3\\ -\\ -\\ Q_{31}\\ h_{32}\\ Q_{32}
\end{bmatrix}}
=
\underset{\textbf{Vetor B}}{\begin{bmatrix}
F_1\\ E_{J1}\\ E_{JL1}\\ E_{J2}\\ E_{JL2}\\ -\\ -\\ E_{J31}\\ E_{JL32}\\ F_{32}
\end{bmatrix}}
$$

Figure 3 – Matrix of the equations used for the numeric solution.

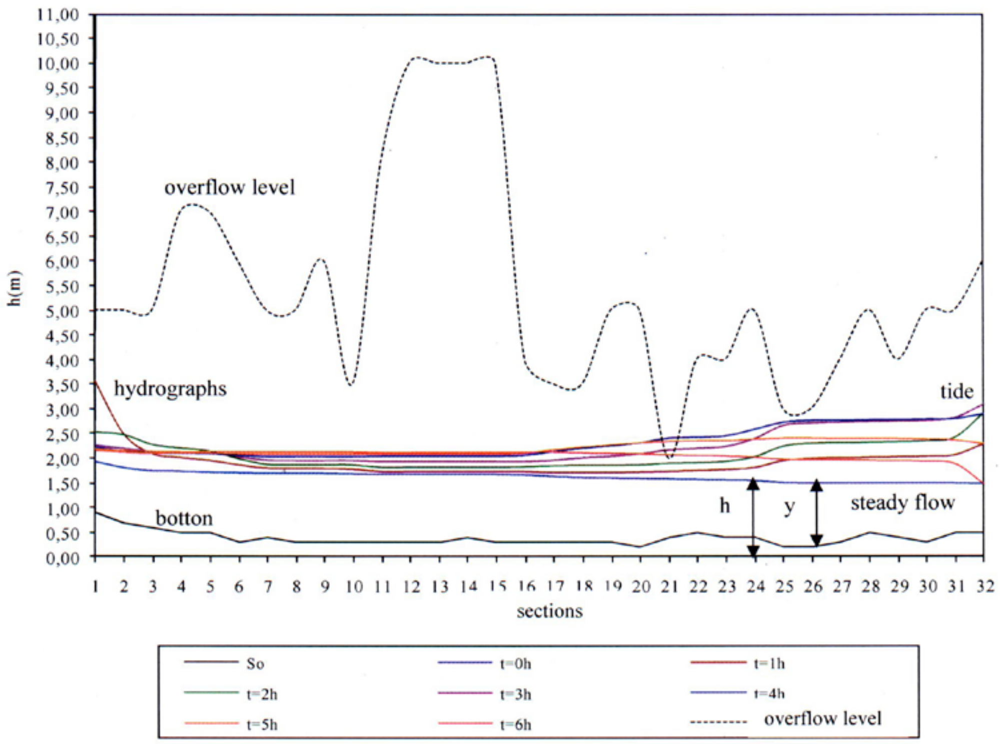

Figure 4 – Block Diagram of the Model.

4. RESULTS

The height of water data, flow and speed in function of time are summarized in Figures 5, 6 and 7 for the discretized sections of the estuary. The initial time t = 0 h was considered when the estuary is at steady state. The time steps are in a period of 1min in a simulated total period of 6 hours. Figure 5 shows the evolution of water height with time along the estuary, with a predominance of the tide on the amount of flow between sections 16 and 32. The maximum height occurs within the first 2 hours for the sections between 1 and 5 due to the influence of the hydrograph peak. For other sections the maximum is reached in the third hour, depending on the maximum height of the tide. The overflow observed in section 21 is an important detail of the simulation, because it corresponds to a characteristic point of flooding.

Figure 6 shows the propagation of the downstream flow (characterized by the rising tide into the estuary) the time of 3h, when its height is maximum . After a period of three hours the tide begins to go down, freeing the flow of the accumulated amount flow in the period. The flow then tends to steady state, with positive values along the estuary, i.e., with the predominance of the amount flow.

The graph in Figure 7 shows the peak level reached in the various sections of the estuary in the simulated period, expressing clearly the sector where flooding occurs (section 21).

Hydraulic changes occurring in the estuary, during periods of high and low tide, are equivalent to the results of work carried out for the same purpose (variation of water levels in the estuary), where the main bottleneck is the unavailability of field data to feed calibration of the model. As an example of equivalent studies can be cited: Como exemplo de estudos equivalentes pode ser citado: Pinho (2005); Stoschek e Zimmermann (2006); Kwnow, Maa e Lee (2007); Ganju e Schoellhamer (2009) and Hu et al. (2009).

6. CONCLUSIONS

This computer model simulation performed for the conditions of the drainage channel, estuarine stretch of the River Coco within a period of 6 hours, with the determination of maximum levels in the estuary. These levels served as parameters for interventions in the areas adjacent to the estuary for urbanization purposes. Therefore, the main objective was achieved.

For the application of the computational model presented in this study in other cases, even with no estuary, there must be an adaptation to the specific geometric and hydraulic situation addressed, as well as boundary conditions of the particular problem. It is also important to note that the computational model implemented here was motivated by the need to serve a practical purpose and, therefore, it is limited to only simulate the occurrence of peak levels.

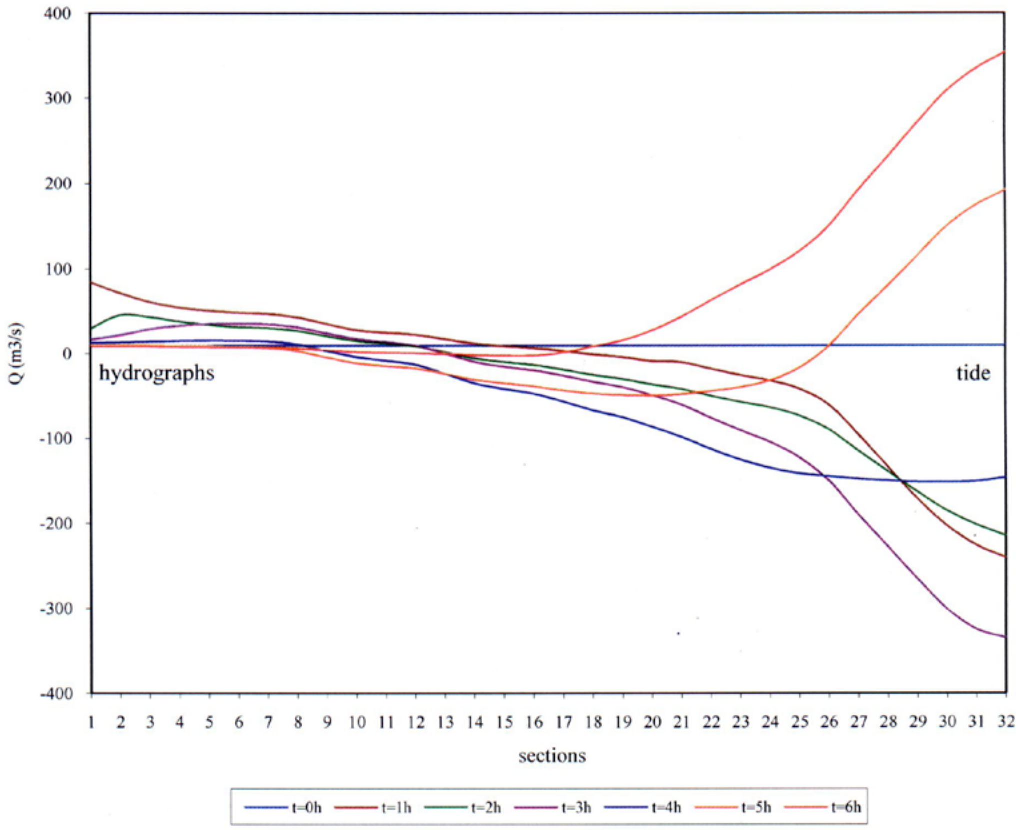

Figura 5 - Evolution of water height in time.

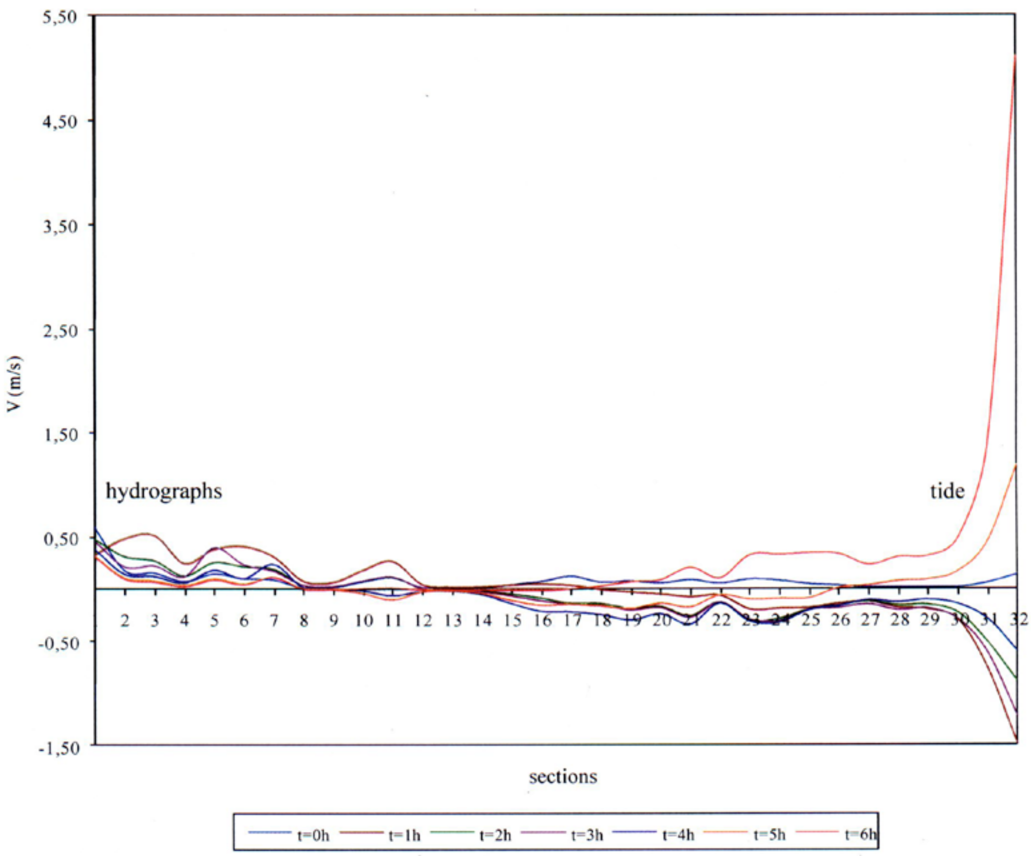

Figura 6 - Evolution of the flow in time.

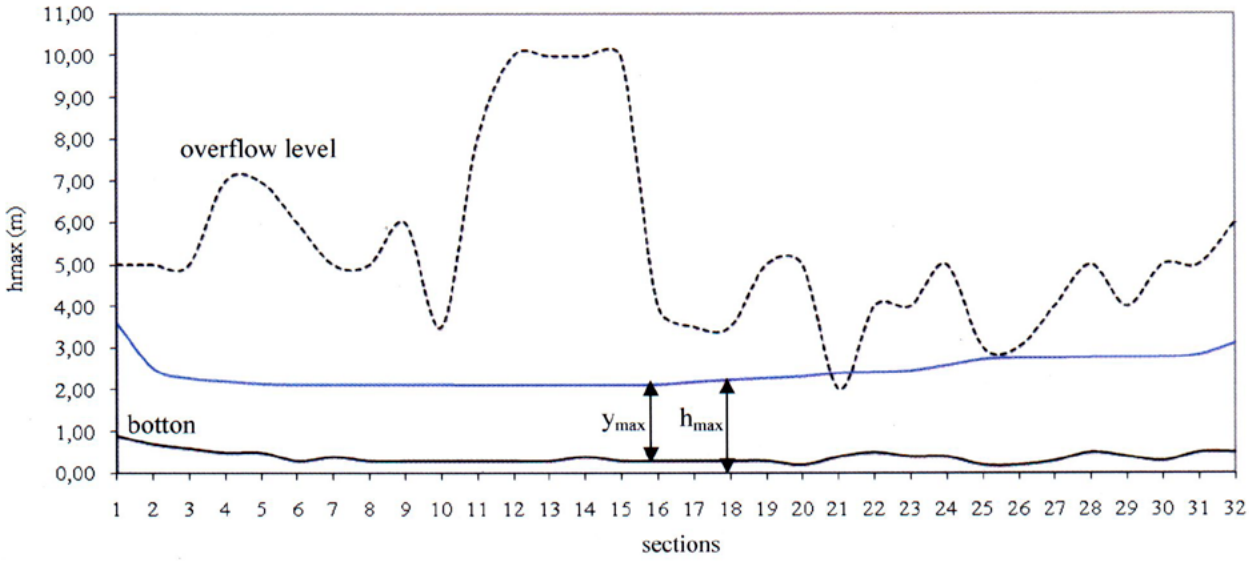

Figura 7 - Maximum level of water in the estuary.

This limitation is imposed by the tidal equation which does not reproduce the minimum heights.

Despite the actual phenomenon is reproduced in the same order of magnitude (of space and time) the validation of this simulation is subject to making measurements of water height in the channel during the rise of the tide. These field data, not available to carry out this simulation, would allow the calibration of the model and check its efficiency.

In the case of natural channels (not prismatic sections), it is worth noting that care must be taken as the geometric aspect. As the geometric configuration of the sections are approximated by mathematical functions (A = f (y)) and P = f (y)), the correlation obtained should be analyzed carefully for the maximum and minimum heights reached in the simulation of water, avoiding to numerical instabilities. The next step of this work is the validation with the consequent inclusion of sediment transport module. The major deadlock in this type of modeling, based on previous work is the unavailability of field data, no contemplation of important parameters for numerical models (winds, turbulence, temperature, etc.) and empirical mathematical modeling with results showing significant errors. Not less important is the necessary qualification for the collection and interpretation of data and appropriate use of model for each situation. All this reality justifies the continuation of studies in this direction, pointing to computational programs with open source codes, which facilitate the interpretation of errors in the results and relevant adjustments, which do not happen with commercial software because of the so-called “black boxes”. Without intending to model the nature accurately, the computational tool provides more efficient and integrated actions from the quantification of transient phenomenon in the drainage channels.

Finally, the availability of data for field calibration and validation extensive computational time for the adjustment and sensitivity analysis of the data and also the limitations of templates to cover certain data types, or to consider them as appropriate, make process of numerical simulation results that hard to not always represent accuracy.

6. REFERENCES.

ABBOTT, M.B.; HAVNO, K.; HOFF-CLAUSSEN, N.E.; KEJ. A., A modeling system for the design and operation of storm sewer networks. Engineering Aplications of computacional hydraulics. Editors: ABBOTT, M.B.; CUNGE, J.A. Pitman. v.1, ch.2, 1982, p.11-39. London

CHAN, A.; VASS, A., Modeling a Pressurized Wastewater System, a Case Study. Ninth International Conference on Urban Storm Drainage (ICUD), 2002. CD-ROM. Portland.

CRUZ, M. A. S. e TUCCI, C. E. M., Avaliação dos Cenários de Planejamento na Drenagem Urbana, Revisa Brasileira de Recursos Hídricos – RBRH, v.3 n.3, set-2008, p.59-71.

GANJU, N. K.; SCHOELLHAMER, D. H. *Calibration of an estuarine sediment transport model to sediment fluxes as an intermediate step for simulation of geomorphic evolution.* Continental Shelf Research 29, 2009. p.148-158.

HEERINGEN, K.; VERWEY, A.; MELGER, E. Dutch Approach to High Speed Urban Drainage Modeling with SOBEK.,2002. In.: Ninth International Conference on Urban Storm Drainage (ICUD). CD-ROM. Portland.

HENDERSON, F. M., Open Channel Flow, Macmillan Publishing, Inc, New York, 1966, 522p.

HU, K.; DING, P.; WANG, Z.; YANG, S. *A 2D/3D hydrodynamic and sediment transport model for the Yangtze Estuary, China.* Journal of Marine Systems, 2009. 23p.

HUBER, W.C.; DICKINSON, R.E., Storm Water Management Model, version 4: user's manual. EPA/600/3-88/001a (NTIS PB88-236641/AS), 1992, U.S. Environmental Protection Agency. Athens, Georgia.

KWON, J. I.; MAA, J. P. Y.; LEE, D. Y. *A preliminary implication of the constant erosion rate model to simulate turbidity maximums in the York River, Virginia, USA.* Estuarine and Coastal Fine Sediments Dynamics, 2007. p.331-354.

LIGGETT, J.A. e CUNGE, J.A., Unsteady Flow in Open Channels – Cap 4 – Volume I – editado por K.Mahmood e V.Yevjevich, 1975, 484p.

LINDBERG, S.; JØRGENSEN, T.W., Modelling of Urban Storm Sewer Systems. In: Proceedings of the International Symposium on Comparison of Urban Drainage Models with Real Catchment Data. UDM '86. Dubrovnik, 1986, Yugoslavia.

MARK, O.; WEESAKUL, S.; APIRUMANEKUL, C.; BOONYAAROONNET, S.; DJORDJEVIC, S., Potential and limitations of 1D modelling of urban flooding. Journal of Hydrology. Volume 299, Issues 3-4, December 2004, Pages 284-299.

PINHO, J.L.S., Modelação da hidrodinâmica e dinâmica sedimentar no estuário do rio Cávado. Revista da Universidade do Minho, Portugal – Dep. Engenharia Civil - ISSN 0873-1152. 24 (2005) 5-16.

RGP – Revista Geografia e Pesquisa, Carttografia Aplicada à Análise Geoambiental: Um Estudo de Caso com Fotografias Aéreas de Pequeno Formato no Lagamar do Rio Cocó – Fortaleza – Ceará, UNESP/Ourinhos, v.3, n.1, jan-jun (2008), ISSN 1982-9760.

SEMACE – Secretaria do Meio Ambiente do Estado do Ceará. Parque Ecológico do Rio Cocó. Disponível em: <http://www.semace.ce.gov.br/2010/12/paque-ecologico-do-rio-coco/>. Acesso em: 12 out. 2012.

STOSCHEK, O.; ZIMMERMANN, C. *Water Exchange and sedimentation in na estuarine tidal harbor using three-dimensional simulation.* Journal of Waterway, Port, Coastal, and Ocean Engineering, ASCE, 2006. p.410-414.

WIKIPÉDIA – Disponível em:< http://pt.wikipedia.org/wiki/Geografia_de_Fortaleza>. Acesso em: 17 jul. 2013.

VENANCIO, S. S., Simulação Numérica Aplicada ao Assoreamento do Reservatório Represa Velha, tese doutorado EESC/USP, 2009, 168p.

VENANCIO, S. S.; SOUSA, L. B. S. de; VILLELA, S. M., Modelo Computacional para Análise de Transiente Hidráulico em Canais – o caso do estuário do Rio Cocó. XVI Simpósio Brasileiro de Recursos Hídricos – SBRH, Nov/2005 – João Pessoa-PB- pg. 1-11.