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Incorporating Hydraulic Structures in an Open-Channel Model

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Abstract

The open-channel flow model, BRANCH, is a routinely used numerical tool for modeling rivers, canals, and waterway networks. Although a simplified hydraulic structure representation is included in the model, no universal subroutine that can represent hydraulic structures by their rating curve equations has been included in the BRANCH model.

Accordingly, a subroutine has been developed that specifies flow through a structure by a stage-discharge relation. The structure flow equation is used to generate coefficients in the solution matrix that represent the structure in the same computational format as the open-channel flow equations. Field applications have shown that this new subroutine properly represents the effects of hydraulic structures in the open-channel flow regime.

Introduction

Numerical models in open-channel flow simulations are useful tools in analysis and prediction of flow regimes. The U.S. Geological Survey (USGS) model, BRANCH, is a widely used simulator of unsteady flow in networks of open channels (Schaffranek & others 1981). However, no generally accepted feature has previously been included in BRANCH to allow the model to simulate effects of hydraulic structures such as flood/tide gates, culverts, weirs, and pumps.

Purpose and Scope

This paper documents a subroutine in BRANCH that represents the effects of hydraulic structures in the open-channel flow equations. The different types of structures represented are described and a comparison with field data is included.

Background

Effects of hydraulic structures have been incorporated into the solution scheme in open-channel flow models that are simpler than BRANCH (Fread 1978; Hydrologic Engineering Center 1982). It is useful to introduce structures into the BRANCH model due to

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its widespread use and the recent coupling of BRANCH with the ground-water model, MODFLOW (Swain & Wexler 1991).

The open-channel flow model, BRANCH (Schaffranek & others 1981), solves the unsteady, nonuniform equations of momentum and mass continuity for one-dimensional channel flow. The effects of tide gates have been incorporated into BRANCH (Goodwin 1991) by allowing an internal junction to change between two boundary conditions: (1) the stage on each side of the junction being equal, and (2) no flow allowed through the junction. The first condition indicated an open gate with no head drop across the gate, and the second condition indicated a closed gate. This method does not allow intermediate gate opening, gates located elsewhere than a junction, or structures other than a gate. By incorporating the actual flow equations for hydraulic structures, these limitations can be overcome.

Methodology

Basic Equations--The equations for flow in a channel segment are solved in BRANCH in the form (Schaffranek & others 1981):

$$\zeta Q_{i+1}^{j+1} + Z_{i+1}^{j+1} + \omega Q_i^{j+1} - Z_i^{j+1} = \epsilon \qquad (1)$$

$$Q_{i+1}^{j+1} + \Gamma Z_{i+1}^{j+1} - Q_i^{j+1} + \Gamma Z_i^{j+1} = \delta \qquad (2)$$

where: Q is discharge, Z is stage, subscripts are location, superscripts are time level, and ζ, ω, ϵ, Γ, and δ are coefficients derived from the finite difference form of the unsteady flow equations.

Instead of representing flow in a channel segment, equations 1 and 2 can represent flow across a hydraulic structure by choosing appropriate values of the coefficients. Thus, once the values of ζ, ω, ϵ, Γ, and δ are determined, the structure flow equations are of the same form as the channel flow equations.

Assuming that there are no storage effects at the structure, equation 2 can express mass continuity across the structure by setting Γ and δ to zero, yielding:

$$Q_{i+1}^{j+1} - Q_i^{j+1} = 0 \qquad (3)$$

The discharge is the same upstream and downstream of the structure. This form is used for all the structures presently represented in the subroutine.

Values of the coefficients in equation 1 depend on type of structure represented. Two options are briefly described below.

Gated Spillways--The gated spillways in south Florida are automated to follow specific operating rules (South Florida Water Management District, written commun., 1991). When the upstream stage exceeds a specified maximum, the gate opens at a given rate until the stage drops to a design level. When the upstream stage drops below a specified minimum, the gate closes until the design level is reached. When the gate is submerged, the flow is expressed by:

$$Q_i = C\sqrt{Z_i - Z_{i+1}} \qquad (4)$$

where: $C=C_g Bh\sqrt{2G}$, C_g is submerged orifice coefficient, B is width of the gate, h is the vertical distance from the gate bottom to the gate sill, and G is gravitational acceleration. When minimum or maximum stage is reached and the gate is opened or closed, the value of h in equation 4 is changed at the appropriate rate and C is recalculated. With:

$$\zeta = 0,\ \omega = |Q_i^{j+1}|/C^2,\ \text{and}\ \epsilon = 0,$$

equation 1 becomes:

$$|Q_i^{j+1}|Q_i^{j+1} = C^2\left(Z_i^{j+1} - Z_{i+1}^{j+1}\right), \qquad (5)$$

the equation for orifice flow through the submerged gate.

The discharge is checked for each BRANCH time step to ensure that the gate is not allowing more flow to pass than would be possible if the gate were out of the water. If so, it is assumed that the gate is out of the water and flow is calculated by the weir equation:

$$Q_i^{j+1} = C_w\left(C_{w1} - C_{w2}\left[\frac{Z_{i+1}^{j+1} - Z_{sill}}{Z_i^{j+1} - Z_{sill}}\right]\right) B\left(Z_i^{j+1} - Z_{sill}\right)^{1.5} \qquad (6)$$

where: C_w is the weir coefficient, C_{w1} and C_{w2} are submergence ratio coefficients, and Z_{sill} is elevation of the gate sill. When the gate is out of the water, the coefficients are set:

$$\zeta = 0,\ \omega = 1/\left(C_w C_{w2} B\sqrt{Z_i^{j+1} - Z_{sill}}\right)$$

and

$$\epsilon = (C_{w1}/C_{w2} - 1)\left(Z_i^{j+1} - Z_{sill}\right)$$

With these values, equation 1 reduces to equation 6.

Culverts--The subroutine also allows for the representation of fully submerged culverts with or without gates. For the submerged culvert, the head-loss factors for entrance, exit, or friction losses are lumped into a coefficient C_c giving:

$$Z_i^{j+1} - Z_{i+1}^{j+1} = C_c\frac{Q^2}{2G\,A^2} \qquad (7)$$

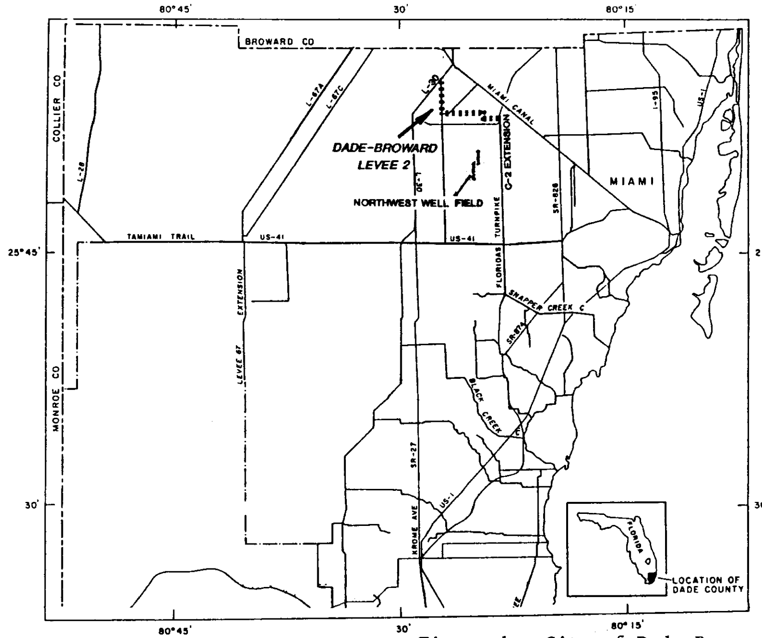
where: A is cross-sectional area of the culvert(s). To put equation 1 into the form of equation 7, the coefficients are set:

$$\zeta = 0,\ \omega = C_c\,|Q_i^{j+1}|/(A^2 2G),\ \text{and}\ \epsilon = 0.$$

To simulate the gated culverts, additional head loss for the gate is added as in equation 5. Algorithms also exist in the subroutine to represent pumps/structures with rating curve equations.

Test Problem

Model Setup--To validate the hydraulic structures subroutine a BRANCH model of the Dade-Broward Levee 2 Canal in Dade County was modeled (fig. 1). This canal stretches 7-1/4 miles (11.7 kilometers), and its proximity to the Northwest Well Field makes it an important water conveyance system for the area. The canal flows through a culvert 3-1/3 miles (5.4 kilometers) from the



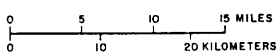


Figure 1.--Site of Dade-Broward Levee 2 Canal.

junction with L-30. Discharge measurements, made by ultrasonic velocity meters, and stage measurements are recorded at 15-minute intervals at the junction with L-30 Canal, just downstream of the culvert, and at the junction with C-2 Extension Canal (fig. 1). Using the junctions with L-30 and C-2 Extension as boundaries, the canal is modeled including the culvert in the canal.

The cross-sectional data and longitudinal dimensions of the canal were obtained from Florida Department of Environmental Regulation design memoranda. A Mannings n factor of 0.040 was selected, although the model is not very sensitive to this factor. The culvert's dimensions, diameter 14 feet (4.27 meters) and length 38 feet (11.58 meters), with a Mannings n of 0.0303, yield values for equation 7 of $C_c = 1.893$ and A = 153.9 square feet (14.32 square meters). The discharge measurements at the beginning and end of the modeled area are used as boundary conditions with a model time interval of 15 minutes. As in most south Florida canals, leakage to or from the aquifer is an important factor, so aquifer loss and gain were accounted for with the MODBRANCH package (Swain & Wexler 1991). The 48-hour period from 0:15 on May 2, 1991, to 0:15 on May 4, 1991, was simulated.

Results

The stage and discharge values, both measured and computed by BRANCH directly downstream of the culvert, are shown in figures 2 and 3, respectively. The sharp drop in both stage and discharge at 15:00 on May 2, 1991, is caused by the shutting of the gate above the upstream boundary. The close correlation of field measurements and model computed values indicates that the structures subroutine accurately represents the culvert in the BRANCH model.

Conclusions

To properly simulate the flow conditions in south Florida canals, it was necessary to incorporate a subroutine into the BRANCH unsteady flow model that would allow the model to represent flow through hydraulic structures. This subroutine creates coefficients for BRANCH's solution matrix that are derived from the flow equations for gates, pumps, culverts, or rating curves. A test run involving a culvert yielded results comparable with field measurements and indicated that the subroutine is functioning properly in the case of a simple submerged culvert. Further testing of other types of structures is needed.

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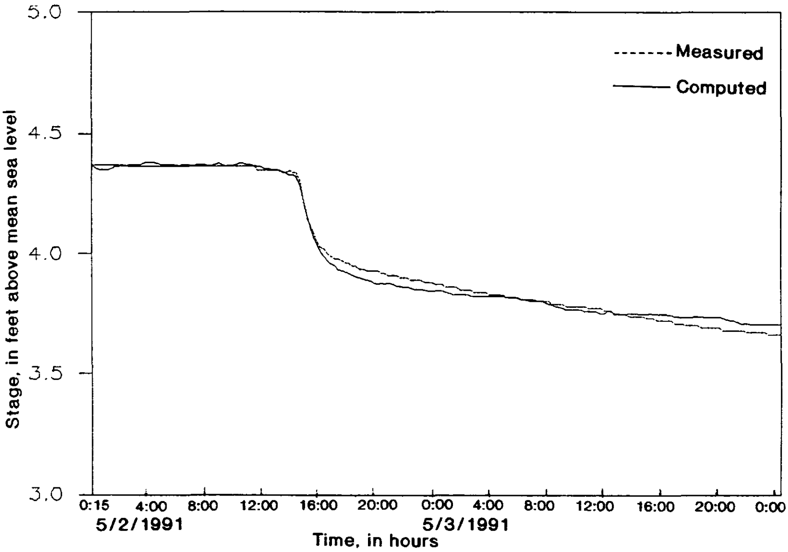

Figure 2. -- Measured and computed stage downstream of culvert.

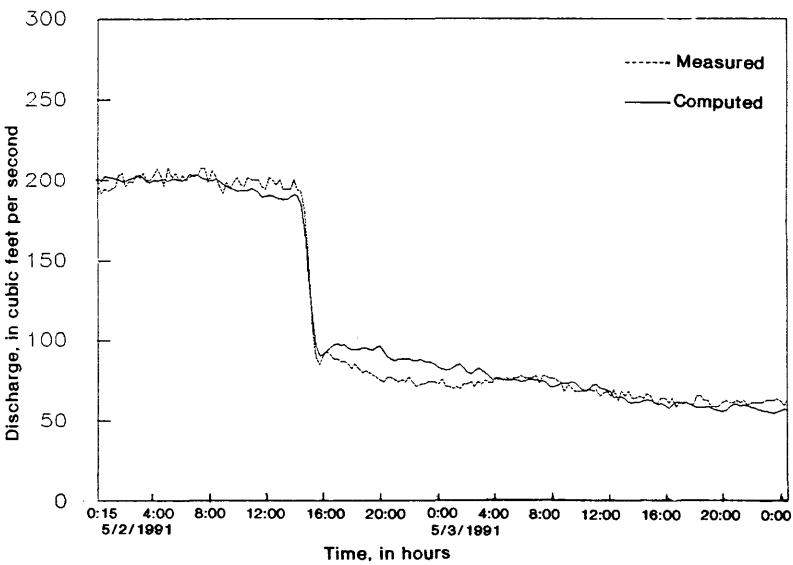

Figure 3. -- Measured and computed discharge downstream of culvert.