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FLDWAV: A Generalized Flood Routing Model

by

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Abstract. A new unsteady flow simulation model, FLDWAV, has been developed by the National Weather Service for application on either micro-, mini-, or mainframe computers. It is intended to replace the popular DAMBRK and DWOPER models since it will allow the utilization of their combined capabilities, as well as provide new hydraulic simulation features within an improved user-friendly model structure. FLDWAV is based on an implicit finite-difference solution of the complete one-dimensional Saint-Venant equations of unsteady flow coupled with an assortment of internal boundary conditions for simulating unsteady flows controlled by a wide spectrum of hydraulic structures. The flow may occur in a single waterway or a system of inter-connected waterways in which sinuosity effects are considered. The flow which can range from Newtonian (water) to non-Newtonian (mud/debris, mine tailings) may freely change with time and location from subcritical to supercritical or vice versa, and from free-surface to pressurized flow. Special modelling features include time-dependent dam breaches, levee overtopping and crevasse, time-dependent gate controlled flows, assorted spillway flows, bridge/embankments, tidal flap gates, off-line detention basins and/or pumping basins including individual pump specifications, and floodplain compartments with free/submerged weir flow connecting with the waterway or adjacent compartments. FLDWAV can be automatically calibrated for a single channel or dendritic system of channels; calibration is achieved through an efficient automatic adjustment of the Manning coefficient that varies with location and flow depth. The model has automatic selection of the critical computational time and distance steps. Data input is through either a batch or an interactive mode and output is tabular and graphic. Input/output may be in English or metric units. FLDWAV can be used for a wide range of unsteady flow applications including real-time flood forecasting in a dendritic system of rivers, dam-breach analysis for sunny-day piping or overtopping associated with a PMF flood, design of waterway improvements, floodplain mapping for flood insurance studies, irrigation system design, debris flow inundation mapping, and storm sewer design.

INTRODUCTION

Flood routing or unsteady flow simulation is an essential tool for flood forecasting and engineering design/analysis of hydraulic structures. A new generalized flood routing model (FLDWAV) has been developed by the National Weather Service (NWS). It is intended to replace two widely used NWS models, DWOPER (Fread, 1978) and DAMBRK (Fread, 1984), since it will utilize their combined unsteady flow simulation capabilities, as well as, provide new hydraulic simulation features and improved user-friendly data input. The

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FLDWAV model can be used on either micro-, mini-, or mainframe computers. It is planned that future NWS research and development in river mechanics will be integrated within the FLDWAV model frame-work. The model can be used by hydrologists/engineers for a wide range of unsteady flow applications including real-time flood forecasting in a dendritic system of waterways subject to backwater effects; dam-breach analysis and inundation mapping for sunny-day piping failures or overtopping failures due to PMF reservoir inflows including the complexities associated with failure of two or more dams sequentially located along a watercourse; design of waterway improvement structures such as levees, off-channel detention ponds, etc.; floodplain mapping for flood insurance studies; analysis of irrigation systems with gate controlled flows; analysis of storm sewer systems having a combination of free surface and/or pressurized unsteady flows; mud/debris flow inundation mapping; and unsteady flows due to hydro-power operations.

GOVERNING EQUATIONS

The governing equations of the FLDWAV model are: (1) an expanded form of the one-dimensional equations of unsteady flow originally derived by Saint-Venant (1871); (2) an assortment of internal boundary equations representing flow through one or more hydraulic flow control structures located sequentially along the main-stem river and/or its tributaries (distributaries); and (3) external boundary equations describing known upstream/downstream discharges or water elevations which vary either with time or each other.

Expanded Saint-Venant Equations. The Saint-Venant equations of conservation of mass and momentum may be expressed in an expanded form to account for some effects omitted in their original derivation. These effects are: (1) lateral inflows/outflows, (2) nonuniform velocity distribution across the flow section, (3) expansion/contraction losses, (4) off-channel (dead) storage, (5) flow-path differences between a sinuos channel and its floodplain, (6) surface wind resistance, and (7) internal viscous dissipation of non-newtonian (mud/debris) flows. The conservation of mass (continuity) equation is:

$$\partial Q/\partial x + \partial s(A+A_o)/\partial t - q = 0 \qquad (1)$$

in which Q is discharge (flow), A is wetted active cross-sectional area, A_o is wetted inactive off-channel (dead) storage area associated with topographical embayments or tributaries, s is a depth-dependent sinuosity coefficient (DeLong, 1986) that accounts for channel meander, q is lateral flow (inflow is positive, outflow is negative), t is time, and x is distance measured along the mean flow-path of the floodplain or along the channel if there is minimal channel meander. The conservation of momentum equation is:

$$\partial(sQ)/\partial t + \partial(\beta Q^2/A)/\partial x + gA(\partial h/\partial x + S_f + S_e + S_i) + L + W_f B = 0 \qquad (2)$$

in which g is the gravity acceleration constant; h is the water surface elevation; B is the wetted cross-sectional active topwidth; L is the momentum effect (Strelkoff, 1969) of lateral flows ($L = -qv_x$ for lateral inflow, where v_x is the lateral inflow velocity in the x-direction; $L = -q\,Q/(2A)$ for seepage lateral outflows; $L = -q\,Q/A$ for bulk lateral outflows); W_f is the wind factor (Fread, 1985), i.e., $W_f = C_w/V_r/V_r$ in which C_w is the wind resistance coefficient, and $V_r = Q/A - v_w \cos\omega$, where v_w is the wind velocity and ω is the acute angle between the wind direction and x-axis; S_f is the boundary friction slope, i.e., $S_f = |Q|Q/K^2$ in which K is the total conveyance determined by summing conveyances of the left/right floodplains and channel in which the channel conveyance is modified by the factor $(1/\sqrt{s})$ and all

conveyances are determined automatically from the data input of top-width/Manning n versus elevation tables for cross sections of the channel and left/right floodplains; S_e is the expansion/contraction slope, i.e., $S_e = k_e/(2g)\ \partial(Q/A)^2/\partial x$ in which k_e is the expansion/contraction loss coefficient; β is the momentum coefficient for non-uniform velocity distribution and is internally computed from the conveyances and areas associated with flow in the channel and left/right floodplains and S_i is the non-Newtonian internal viscous dissipation slope (Fread, 1987), i.e.,

$$S_i = \kappa/\gamma[(b+2)Q/(AD^{b+1}) + (b+2)/(2D^b)\ (\tau_o/\kappa)^b]^{1/b} \quad \text{.....(3)}$$

in which D=A/B; κ is the apparent fluid viscosity; γ is the fluid's unit weight; τ_o is the initial shear strength of the fluid; and b = 1/m where m is the exponent of a power function that represents the fluid's stress (τ_s)-rate of strain (dv/dy) relation, i.e., $\tau_s = \tau_o + \kappa(dv/dy)^m$ in which v and y are the flow velocity and depth, respectively.

Internal Boundary Equations. There may be various locations (internal boundaries) along the main-stem and/or tributaries where the flow is rapidly varied in space and Eqs. (1-2) are not applicable, e.g. dams, bridges/road-embankments, waterfalls, short steep rapids, weirs, etc. The following equations are used in lieu of Eqs (1-2) at internal boundaries:

$$Q_i - Q_{i+1} = 0 \quad \text{.....(4)}$$

$$Q_i = f(h_i,\ h_{i+1},\ \text{properties of control structure}) \quad \text{.....(5)}$$

in which the subscripts i and i+1 represent sequential cross sections located just upstream and just downstream of the structure, respectively. If the structure is a bridge, then Eq. (5) assumes the following form:

$$Q=\sqrt{2g}\ C_b A_b\ (h_i - h_{i+1} + v^2/2g - \Delta h_f)^{1/2} + C_e L_e K_e (h_i - h_e)^{3/2} \quad \text{.....(6)}$$

in which C_b is the coefficient of flow through the bridge, A_b is the wetted cross-sectional area of the bridge opening, v = Q/A, Δh_f is the head loss through the bridge, C_e is the coefficient of discharge for flow over the embankment, L_e is the length of the road embankment, h_e is the elevation of the embankment crest, and K_e is a broad-crested weir submergence correction, i.e., $K_e = 1-23.8[(h_{i+1} - h_e)/(h_i - h_e) - 0.67]$. If the flow structure is a dam, then Eq. (5) assumes the following form:

$$Q=K_s C_s L_s (h_i-h_s)^{3/2} + \sqrt{2g}\ C_g A_g (h_i-h_g)^{1/2} + K_d C_d L_d (h_i-h_d)^{3/2} + Q_t + Q_{br} = 0 \quad \text{...(7)}$$

in which K_s, C_s, L_s, h_s are the uncontrolled spillway's submergence correction factor, coefficient of discharge, length of spillway, and crest elevation, respectively; K_d, C_d, L_d, h_d are similar properties of the crest of the dam; C_g, A_g, h_g are the coefficient of discharge, area, and height of opening of a fixed or time-dependent moveable gate spillway; Q_t is a constant or time-dependent turbine discharge; and Q_{br} is the flow through a time-dependent breach of the dam (Fread, 1977) given by the following:

$$Q_{br} = C_v K_b [3.1\ b_i (h_i - h_b)^{3/2} + 2.45\ Z\ (h - h_b)^{5/2}] \quad \text{.....(8)}$$

in which b_i is the known time-dependent bottom width of the breach, h_i is the known time-dependent bottom elevation of the breach, Z is the side slope of the breach (1: vertical to 2: horizontal), C_v is a velocity of approach correction factor, and K_b is a broad-crested weir submergence correction factor similar to K_e in Eq. (6). If the structure is a natural rapids or waterfall, then a simple critical flow equation can be used for Eq. (5).

Also, empirical rating curves of Q versus h may be used in lieu of Eq. (5) or in place of any or all of the first three terms in Eq. (7).

External Boundary Equations. External boundary equations at the upstream or downstream extremities of the waterway must be specified to obtain solutions to the Saint-Venant equations. In fact, in many applications, the unsteady disturbance is introduced to the waterway at one or more of the external boundaries via a specified time series of discharge (a discharge hydrograph) or water elevation as in the case of a lake level or estuarial tidal fluctuation. At the downstream extremity, the boundary equation could be Eq. (7), an empirical rating of h and Q, or a channel control, loop-rating based on the Manning equation in which S (the dynamic energy slope) is approximated by:

$$S = (h_{N-1} - h_N)/\Delta x - (Q^{t+\Delta t} - Q)/(gA\ \Delta t) - [(Q^2/A)_N - (Q^2/A)_{N-1}]/(gA\ \Delta x) \quad (9)$$

in which Δx represents the reach length between the last two cross sections at the downstream extremity of the waterway.

Solution Technique. The Saint-Venant Eqs. (1-2) cannot be solved directly; however they can be solved by finite-difference approximations. The FLDWAV model utilizes a weighted four-point nonlinear implicit finite-difference solution technique as described by (Fread, 1985). Substitution of appropriate simple algebraic approximations for the derivative and non-derivative terms in Eqs. (1-2) result in two nonlinear algebraic equations for each Δx reach between specified cross sections which, when combined with the external boundary equations and any necessary internal boundary equations, may be solved by an iterative quadratic solution technique (Newton-Raphson) along with an efficient, compact, quad-diagonal Gaussian elemination matrix solution technique. Initial conditions are also required at t = 0 to start the solution technique. These are automatically obtained within FLDWAV via a steady flow back-water solution or they may be specified as data input for unsteady flows occurring at t = 0. A river system consisting of a main-stem river and one or more principal tributaries is efficiently solved using an iterative relaxation method (Fread, 1973) in which the flow at the confluence of the main-stem and tributary is treated as the lateral inflow/outflow term (q) in Eqs. (1-2). If the river consists of bifurcations such as islands and/or complex dendritic systems with tributaries connected to tributaries, etc., a network solution technique is used (Fread, 1985), wherein three internal boundary equations which conserve mass and momentum at each confluence. This system of algebraic equations requires another special compact Gaussian elemination matrix technique to achieve an efficient solution.

SPECIAL FEATURES

The FLDWAV model has several special features including: (1) a subcritical/supercritical mixed-flow solution algorithm, levee overtopping/floodplain interactions, automatic calibration, combined free surface/pressurized flow capabilities, and automatic selection of computational time and distance steps. Some of these are described herein.

Subcritical/Supercritical Algorithm. This optional algorithm (Fread, 1985) automatically subdivides the total routing reach into sub-reaches in which only subcritical (Sub) or supercritical (Sup) flow occurs. The transition locations where the flow changes from Sub to Sup or vice versa are treated as external boundary conditions. This avoids the application of the Saint-Venant equations to the critical flow transitions. At each time step, the

solution commences with the most upstream sub-reach and proceeds sub-reach by sub-reach in the downstream direction. The upstream boundary (UB) and downstream boundary (DB) are automatically selected as follows: (1) when the most upstream sub-reach is Sub, the UB is the specified discharge hydrograph and the DB is the critical flow equation since the next downstream sub-reach is Sup; (2) when the most upstream sub-reach is Sup, the UB is the specified hydrograph and a loop-rating quite similar to that previously described as an external boundary condition, and no DB is required since flow disturbances created downstream of the Sup reach cannot propagate upstream into this sub-reach; (3) when an inner sub-reach is Sup, its two UB conditions are the discharge just computed at the DB of the adjacent upstream sub-reach and the computed critical water surface elevation at the same DB; (4) when an inner sub-reach is Sub, its UB is the discharge just computed at the most downstream section of the adjacent upstream Sup sub-reach and the DB is the critical flow equation. Hydraulic jumps are allowed to move either upstream or downstream prior to advancing to another computational time step; this is accomplished by comparing computed sequent elevations (h_s) with computed backwater elevations (h) at each section in the vicinity of the hydraulic jump. The jump is moved section by section upstream until $h_s > h$ or moved downstream until $h > h_s$. The Froude number ($Fr = \sqrt{gQ/(AD)}$) is used to determine if the flow at a particular section is Sub or Sup, i.e., if $Fr<1$ the flow is Sub and if $Fr \geq 1$ the flow is Sup. The Sub/Sup algorithm increases the computational requirements by approximately 20 percent.

Levee Overtopping. Flows which overtop levees located along either or both sides of a main-stem river and or its principal tributaries may be simulated within FLDWAV. The overtopping flow is considered lateral outflow (-q) in Eqs. (1-2), and is computed as broad-crested weir flow. Three options exist for simulating the interaction of the overtopping flow with the receiving floodplain area. The first option simply ignores the presence of the floodplain. The second option treats the receiving floodplain as a storage or ponding area having a user-specified storage-elevation relationship. The floodplain water surface elevations are computed via the simple storage (level-pool) routing equation. The overtopping broad-crested weir flow is corrected for submergence effects if the floodplain water elevation exceeds sufficiently the levee crest elevation. In fact, the overtopping flow may reverse its direction if the floodplain elevation exceeds the river surface water elevation. In the third option the floodplain is treated as a tributary of the river and the Saint-Venant equations are used to determine its flow and water surface elevations; the overtopping levee flow is considered as lateral inflow (q) in Eqs. (1-2). In each option the levee may also crevasse (breach) along a user-specified portion of its length. If the receiving floodplain area is divided into separate conpartments by additional levees or road-embankments located perpendicular to the river and its levees, the flow transfer from a compartment to an adjacent upstream or downstream compartment is simulated via broad-crested weir flow with submergence correction; flow reversals can occur when dictated by the water surface elevations within adjacent compartments, which are computed by the storage equation.

Automatic Calibration. An option within FLDWAV allows the automatic determination of the Manning n such that the difference between computed water surface elevations (stage hydrographs) and observed hydrographs is minimized. The Manning n can vary with either flow or water elevation and with sub-reaches separated by water level recorders. The algorithm (Fread and Smith, 1978) for efficiently accomplishing this is applicable to a single

multiple-reach river or a main-stem river and its prinicpal tributaries. FLDWAV also provides an option to conveniently utilize a methodology (Fread and Lewis, 1986) for determining optimal n values which may for some applications eliminate the need for time-consuming preparation of detailed cross-sectional data. Approximate cross sections represented by separate 2-parameter power functions for the channel and the floodplain are used. Optimized n values can be constrained to fall within user-specified min-max ranges. Also, specific cross-sectional properties at key sections (bridges, natural constrictions, etc.) can be utilized wherever considered appropriate.

PROGRAM STRUCTURE

The FLDWAV model is FORTRAN IV coded with over 80 subroutines which provide the desired modularity for future expansions to enable the simulation of other hydraulic phenomena. Arrays are coded with a variable dimensioning technique which utilizes a single, large array as the only fixed array size. At each execution of the model, the large array is automatically partitioned into individual variable arrays required for a particular hydraulic application. The size of each array is automatically determined by user-specified data which describes the hydraulic application. This program structure allows maximum utilization of storage space since arrays not used during a particular simulation require no storage space.

The input/output is in either English or metric units. The input is either batch or interactive (question/answer). The output is in numerical tabular and/or graphical form. It is anticipated that the FLDWAV model will be available for trial use by the engineering community by the latter part of this year.

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