Model-Prototype Correlation of Hydraulic Structures

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Unsteady Canal Flow - Comparison of Simulations with Field Data

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Abstract

The accuracy of numerical simulation of unsteady flows in water conveyance and irrigation canals is a direct function of the time and distance increments used in the scheme for solving the equations of unsteady open channel flow (St. Venant equations). However, simply decreasing either the distance step or the time step alone does not necessarily increase the accuracy of the computations. This is shown by the comparison of results obtained from simulations using three different solution schemes.

The best way to evaluate the ability of a simulation scheme to represent unsteady canal flow processes is through comparison of the simulations with prototype data. In this study, field data from a large multiple-reach canal are used to evaluate canal simulation model results. The results of these comparisons indicate that the use of appropriate boundary conditions and an appropriate time step to distance step ratio are important in assuring good computational accuracy.

Two sets of field data which represent conditions in a single canal reach were used. The first is for a case with rapid canal shut down for a moderate canal flow. The second data set is for a large flow change in the same canal.

Introduction

Numerical simulation of canal flows can be made with good accuracy using computer models based on the one-dimensional equations of unsteady open channel flow (St. Venant equations). A number of canal system models are currently available (DeVries 1987); if properly formulated and applied most of the models using the full unsteady flow equations can give good results.

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Flow in irrigation and water supply canals is usually regulated by "checks" or "cross regulators" positioned at intervals of several miles along the length of the canal. One of the main factors in determining check spacing is canal slope. Usually canals are constructed so that the maximum change in water level from full flow to no canal flow is about 3 ft.

When canals are operated to provide steady flows with only minor, slow changes in discharge, transient effects can be ignored. When rapid flow changes are made transient effects (surges) can be larger. For example, in the California Aqueduct, flow changes of 6000 cfs are made daily, and fluctuations in the canal water level on the order of 1 ft above and below steady-state levels occur as a result of these flow changes.

The tests described in the paper were made using an implicit four-point finite difference form of the St. Venant equations. The model is described by Hromadka, Durbin and DeVries (1985). The procedure used to solve the unsteady flow equations is similar to other implicit schemes in that time derivatives are based on finite differences made using a time step Δt, and spatial derivatives are approximated by a difference operator positioned between time values according to a weighting factor (θ). Thus spatial derivatives depend on both Δx and θ.

When the weighting factor θ is between 0.5 and 1.0 the implicit four-point scheme is unconditionally stable (Cunge, Holly and Verwey 1980; Fread 1974). Fread (1974) has performed a number of numerical experiments using an implicit four-point difference scheme model. His results show that as θ becomes progressively greater than 0.5, accuracy decreases. Because a computationally-based instability can arise for the case of θ=0.5, Fread recommends using θ=0.55.

Fread (1974) also gives an error analysis (based on a linearized form of the equations) which shows that the computational error (truncation error) depends on the sum of an error term involving Δx and an error term involving Δt.

Field Data

Field data on flow, water levels and gate openings for the North San Joaquin Division of the California Aqueduct was supplied by the California Department of Water Resources. The division is comprised of twelve pools starting at the Delta Pumping Plant through which all flow enters the aqueduct to the O'Neill Forebay which is an intermediate storage reservoir on the aqueduct. There are only minor turnouts along this section of the canal. The pools are separated by radial gate check structures. In the first pool, there is Bethany Reservoir which is for short-term in-stream storage. Check 1 is at the outlet of Bethany Reservoir. Following the time when the pumps are started, there may be a delay before the check structure gates are opened and water starts to flow along the canal. Flow entering the first pool can be stored in Bethany Reservoir until flow is established in the pools downstream. Flows downstream may be higher than the flow into the first pool because of the storage available from this reservoir.

The length of the pools range from 4.5 to 6.3 miles, most of the pools are about 6 miles long.. Check structures between pools have four 20 ft wide radial gates. The canal is concrete lined and has a bed width of 40 ft, a bed slope of 0.000045 and side slopes of 1:1.5. The canal has been designed for a peak flow of 10,000 cfs, but the present installed pump capacity is 6300 cfs.

Water levels upstream and downstream of each check structure are measured by float gages. Data from these gages and the height of each gate is opening are recorded approximately every 60 sec. Flow through each gate is calculated using the following equation:

$$Q = a^h * b * h * (H^{0.5}) \qquad (1)$$

where
- Q = discharge through on gate (cfs)
- H = difference in water level across the structure (ft)
- h = gate opening (ft)
- a = 1.0367
- b = 115.35

Data on the water levels, gate openings, and flows are displayed on video display units (VDU) at the control center and updated approximately one per minute intervals. The first data set was obtained by writing down the water surface elevations as they were displayed. These data represent a canal shut down under manual operation of the system. The flow was reduced from 1700 cfs to zero in less than 3 min at the upstream check structure by moving the gates at their fastest speed. The operator then drove to the check structure at the downstream end of the canal reach (about 6 mi away) and reduced the flow to zero there also. Fig. 1 shows the recorded transients for this case.

Large Flow Change

The canal operation for the case of the second data set was much more complex. Data for a 19-hour period were recorded on magnetic tape for an initial flow change at the pumping plant of 0 cfs to 6300 cfs and a later flow change from 6300 cfs to 0 cfs. The timing of the pumping plant flow changes were as follows:

Time (min)	Flow (cfs)
0	0
25	6300
598	6300
621	0
1123	0

The flow in each of the individual pools differs from pool to pool even after 550 min of inflow from the pumps due to flow coming from

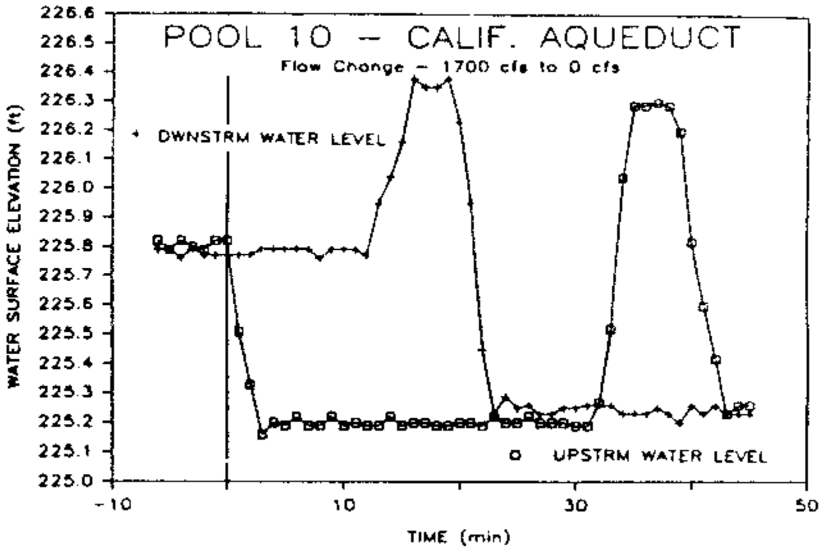

Figure 1. Observed Transients - Moderate Flow Change.

Bethany Reservoir plus water from storage in the various pools. Fig. 2 shows the recorded water levels as a function of time for Pool 4.

Gate Operation

Gates on the check structures are operated remotely and they are moved at a rate of about 0.6 ft/min. In the operation described here gate movements at all structures are started simultaneously. For example, Gate Number 1 at all checks will start moving at the same clock time. When Gate Number 1 on all structures has come to its final opening, a period of time elapses before Gate Number 2 is moved. This allows the total change in discharge to occur over a long enough period of time to minimize transient effects. Gate 3 and 4 in each are moved in similar fashion. Gate openings are calculated for the required flow after the change. At the head of the canal flow changes are made by altering the discharge at the Delta Pumping Plant. Check structure gates are not usually moved between flow changes unless water levels differ from desired levels, in which case individual gates are adjusted as required. This mode of operation regulates the canal transient effects and keeps the wave heights to about one ft. If the flow change from 6300 cfs to zero flow were made in a very short time the wave heights would be about 2 ft.

Analysis of Data for Moderate Flow Change

The flow change from 1700 cfs to zero flow was modeled using a θ value of 0.55, a Δx of about 1000 ft, and a Δt value of 60 seconds. These Δx and Δt values give a ratio between the actual wave speed and the computational wave speed ($\Delta x/\Delta t$) of nearly unity. The results of the simulation are shown in Fig. 3.

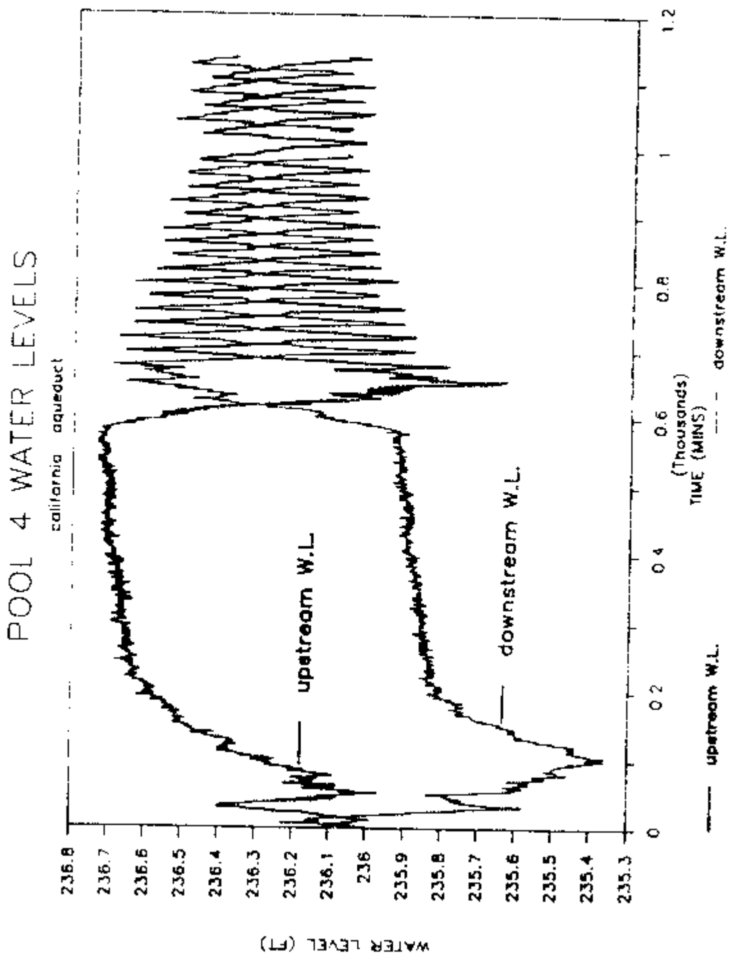

Figure 2. Observed Transients - Large Flow Change -- Upstream and Downstream Ends of Pool 4, California Aqueduct.

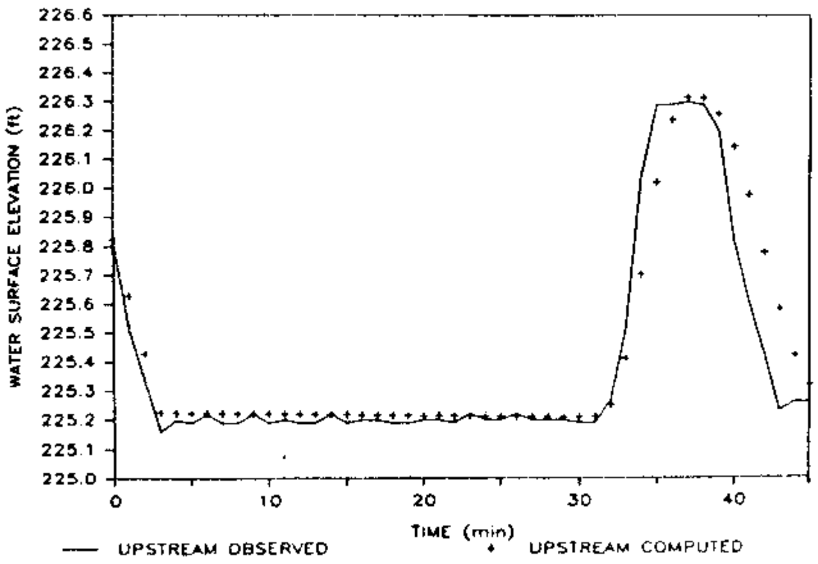

Figure 3. Computed Transients - Moderate Flow Change.

Commonly used error analysis procedures would lead one to believe that decreasing either the distance step or the time increment would improve the computational accuracy. However, when Δt was decreased without changing the other calculation parameters, the fit between the observed and computed data became poorer. This is indicated in Fig. 4, which shows that when the time increment is reduced to 10 sec, the transient heights are overpredicted.

Because implicit schemes are inherently stable, it is sometimes recommended that large time steps be used to cut down on the number of computations that must be made. For rapid changes in flow such as the ones reported here using large time steps can lead to inaccurate results. Fig. 5 shows results of computations based on a $\Delta t = 120$ sec. The transient wave heights are underpredicted when large time steps are used and Δx is kept constant. If Δx is increased along with an increase in Δt to keep the $\Delta x/\Delta t$ ratio approximately equal to the actual wave speed, the results are quite similar to those in Fig. 3.

Analysis of Data for Large Flow Change

Water levels and flow changes in Pool 4 (between Check Structure 3 and Check Structure 4) were studied. The observed water level changes at the upstream and downstream ends of the pool are shown in Fig. 2. Analysis was concentrated on modeling the flow changes resulting from

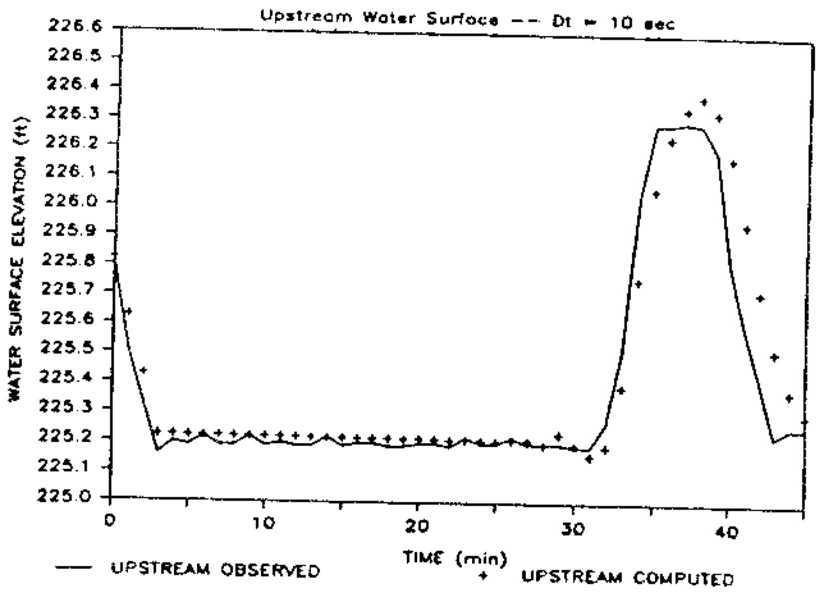

Figure 4. Computed Transients for Δt = 10 sec - Moderate Flow Change.

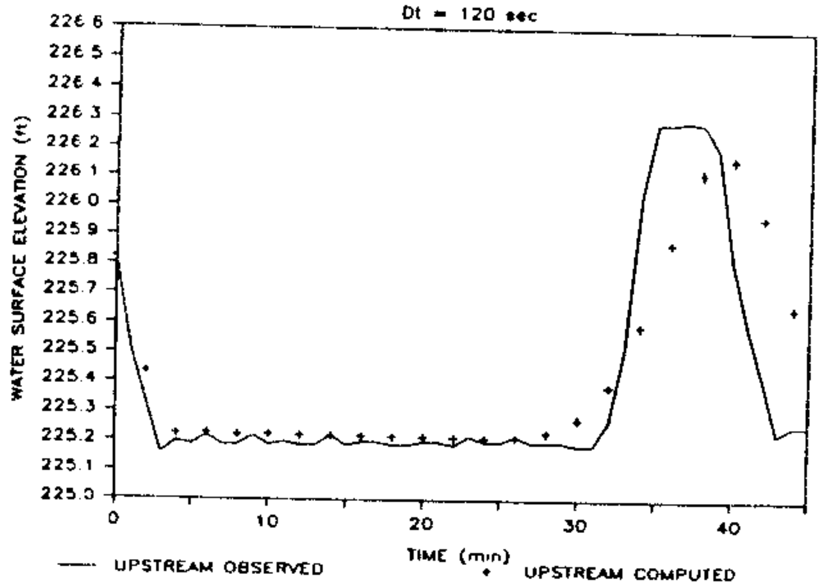

Figure 5. Computed Transients for Δt = 120 sec - Moderate Flow Change.

gate closure because the transients caused by closure are very pronounced. The simulations started at a time corresponding to 500 minutes on Fig. 2. Nearly steady flow existed at this time, and the flow into and out of the pool was 7000 cfs. During the flow closure period the flows through the check structures were calculated using observed water levels and Eq. 1. Figs. 6 and 7 give the simulation results for the best fitting of the data.

The unsteady flow simulation is quite sensitive to the specified boundary conditions, because the boundary conditions dictate the magnitude of the initial transient and the associated momentum of the water in the canal. Boundary conditions used in the simulations are upstream and downstream flows at specified times. The initial condition is a steady flow profile based on a specified initial downstream water level. The Manning's n used in the simulations were adjusted so that the calculated water levels fitted the observed water levels, because the latter are a direct measurement of conditions in the canal.

Water levels calculated during the period of gate closure are not influenced significantly by the variables Δx, Δt and θ. However, these parameters influence the magnitude and time of the subsequent waves. The effect of each of these parameters on the simulation was evaluated separately:

1) Weighting Factor θ:

The effects of using various values of θ are shown in Table 1 where water levels for individual peaks and troughs associated with the water level fluctuations are given. For the initial peak (Peak 1), the results were nearly identical for all values of θ. With increasing time (higher peak numbers) the damping of the computed transients was much greater for higher values of θ.

The some damping effect was observed for the troughs. It was not possible to match the observed water elevations in the troughs, al though some improvement was noted when the boundary conditions were adjusted. The results were found to be quite sensitive to variations in the boundary values (for these runs the boundary conditions were specified flows as a function of time). The complex nature of this large flow change made it much more difficult to simulate than the moderate flow change case.

2) Time and Distance Increments:

The effect of Δt on the computed results are given in Table 2. As for the moderate flow change case, increasing Δt produced damping of the computed transients, and this effect was present at the first peak already.

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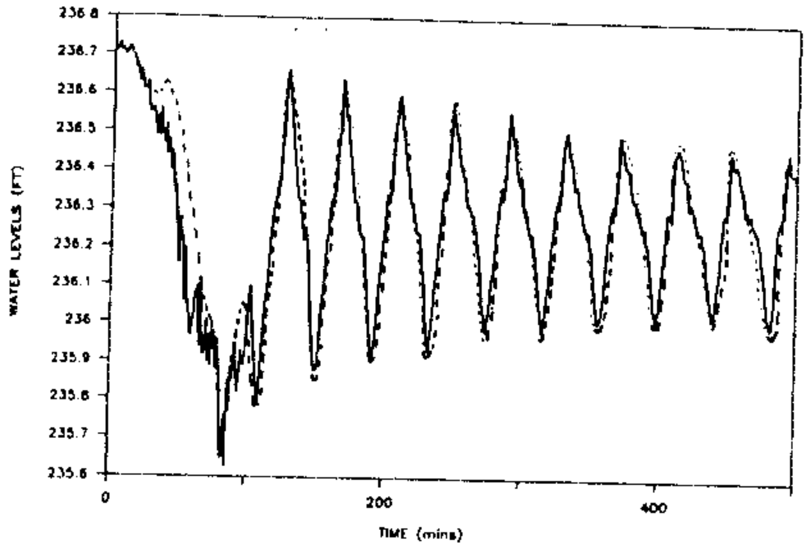

Figure 6. Computed Transients for Large Flow Change -- Upstream End of Pool 4, California Aqueduct.

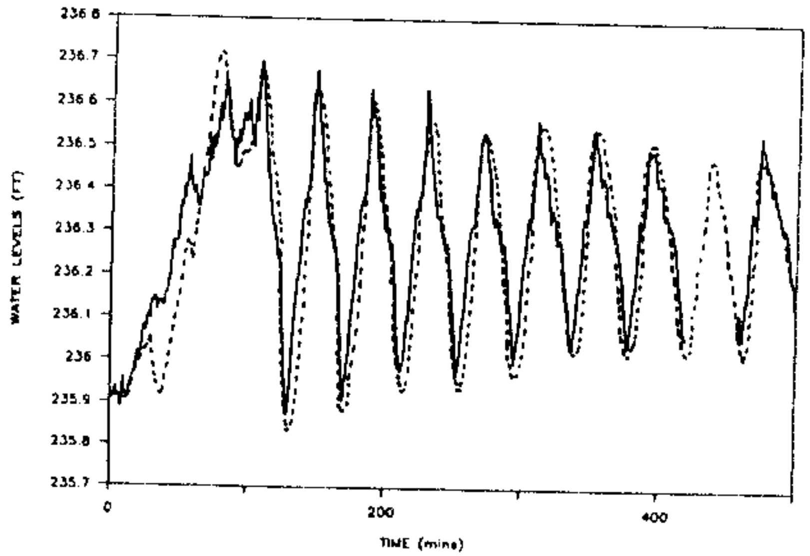

Figure 7. Computed Transients for Large Flow Change -- Downstream End of Pool 4, California Aqueduct.

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Table 1. Water Levels for Different θ Values at Downstream End - Pool 4.

	PEAK NUMBER (AFTER GATE CLOSURE) 1	3	5	7
OBSERVED	236.67	236.61	236.55	236.49
θ = 0.50	236.67	236.61	236.46	236.42
θ = 0.55	236.66	235.57	236.52	236.47
θ = 0.60	236.65	236.54	335.57	236.50
θ = 0.70	236.65	236.48	236.39	236.33

	TROUGH NUMBER (AFTER GATE CLOSURE) 1	3	5	7
OBSERVED	235.88	235.98	236.00	236.06
θ = 0.50	235.74	235.80	235.86	235.86
θ = 0.55	235.74	235.88	236.93	236.97
θ = 0.60	235.82	236.92	236.00	236.06
θ = 0.70	235.85	236.98	336.08	236.13

Note: Similar changes in water level were found for the calculated water levels upstream.

Table 2. Water Levels for Different Δt Values at Downstream End - Pool 4.

	PEAK NUMBER (AFTER GATE CLOSURE) 1	3	5	7
OBSERVED	236.67	236.61	236.55	236.49
Δt = 60 sec	236.64	236.57	336.52	236.47
Δt = 120 sec	236.55	236.50	236.45	236.41
Δt = 240 sec	236.53	236.46	236.38	236.32

Notes: 1. Below Δt = 60 sec only small changes in computed water levels occured until after the seventh peak.
2. For larger Δt transients dampen rapidly.