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A DIFFUSION HYDRODYNAMIC MODEL OF A SHALLOW ESTUARY: VERIFICATION

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Abstract

Many of the hydrodynamic models used for estuary tidal flows and storm surges are based on two two-dimensional hydrodynamic equations which are obtained form the full three-dimensional equations by averaging the vertical coordinates. Various numerical techniques, such as finite difference, finite element, and the method of characteristics have been used to solve these models. The Diffusion Hydrodynamic Model (DHM) has been developed to simulate two-dimensional surface water flows. In a prior study, the DHM has been applied to a deep water estuary with results comparable to those obtained using the method of characteristics. In this paper, the DHM is applied to a shallow water estuary located in southern California. The main objective is to determine local flow velocities and circulation patterns in the shallow estuary caused by the incoming and outgoing tide. Verification of the DHM is provided by comparison of tidal gauge measurements and computed flow depths.

Introduction

The Diffusion Hydrodynamic Model or DHM (Guymon and Hromadka, 1986; Hromadka and Yen, 1986) has been applied to several applications (Hromadka et al, 1989). In this paper, the DHM is extended to irregularly-shaped finite element four-sided polygons. The extended

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DHM is then applied to a shallow estuary in order to simulate water surfaces and flow velocities. Tidal gauge measurements are collected for comparison to the computed DHM results, providing for both calibration and verification of the DHM approach.

Description of Study Region

Newport Bay is a shallow estuary with a mean water surface area of about 1.9 square miles and about 23 miles of shoreline. The bay is subject to a shallow but complex tidal cycle. Two low and two high tides occur diurnal and are particularly pronounced during the highest tides during the new moon phase of the lunar cycle. Tides are propagated rapidly throughout the Bay system.

Mathematical Model Development

Because of the estuary's shallowness relative to its aerial extent, the water column is considered as vertically integrated. Hydrodynamic equations of fluid motion are developed by considering conservation of mass and momentum. Conservation of momentum is expressed in terms of the "St. Venant equations" (Hromadka et al, 1989). Discharge, Q, may be linked to friction slope, S_f, by Manning's equation.

$$Q_i = \frac{1.486}{n} A_i R_i^{2/3} S_{f_i}^{2/3}; \; i = x,y \tag{1}$$

where n is Manning's roughness coefficient; and R is the hydraulic radius. For wide shallow channels $R \sim h$ (depth). For relatively shallow flows where velocities are relatively small, the local acceleration and inertial terms of the flow equations may be neglected without undue loss of precision. Consequently, the St. Venant equations reduce to (H, water surface elevation)

$$S_{f_i} = \frac{\partial H}{\partial X_i}; \; i = x,y \tag{2}$$

Manning's equation may be rearranged as follows:

$$Q_i = -K_i \frac{\partial H}{\partial X_i}, \tag{3}$$

where (b_i, flow basewidth in i-direction)

$$K_i = \frac{1.486}{n} b_i h_i^{5/3} / \left| \frac{\partial H}{\partial X_i} \right|^{1/2}; \; i = x,y \tag{4}$$

This result may be substituted into the continuity equation to yield the so-called "diffusion hydrodynamic equation" in two-dimensions,

$$\frac{\partial(K_x \frac{\partial H}{\partial x})}{\partial x} + \frac{\partial(K_y \frac{\partial H}{\partial y})}{\partial y} = A_{x,y}\frac{\partial h}{\partial t} \tag{5}$$

where $A_{x,y}$ is the aerial area of the cell. This equation is a parabolic equation (diffusion equation) that is highly nonlinear. A finite difference approximation of (5) may be obtained by writing in terms of total discharge Q,

$$\frac{\partial Q_x}{\partial x}\Delta x + \frac{\partial Q_y}{\partial y}\Delta y = A_{x,y}\frac{\partial h}{\partial t} \tag{6}$$

Integrating the partial-differential components,

$$\Delta Q_x + \Delta Q_y = A_{x,y}\frac{\Delta h}{\Delta t} \tag{7}$$

or, in an explicit solution scheme for any shaped cell,

$$h^{J+1} = h^J + \frac{\Delta t}{A_{x,y}}\sum_i Q_i \tag{8}$$

where J is the time step number, and Q_i is discharge across a perpendicular cell face, being positive of inflow and negative if outflow, and Q_i and velocities are determined from (3), where K_i is determined from (7). The simulation is commenced with a flat bay water surface at MLLW. Two types of boundary conditions are considered. One tidal boundary condition is specified at the jetty-ocean mouth based upon the National Oceanic and Atmospheric Administration (NOAA) tidal gauge located at the Harbor Master headquarters compound about one quarter of a mile inside the bay. The second kind of boundary condition consists of inflow hydrographs from 25 different watersheds.

To solve the numerical hydrodynamic equation analog, the Bay is discretized into quadrilateral finite elements. The discretization scheme selected is shown in Figure 1. There are 395 cells and 549 nodes in the finite element system.

The verification approach was to measure a tidal sequence in several discrete portions of the Bay and compare these data to simulated tides. Because the Manning's n-factor for bottom roughness is assumed, this approach is actually a partial verification and calibration effort combined. Partial verification is achieved in the sense that n-factors for each cell must be "tuned" to give good results and we must use judgment to determine if such tuning is realistic and the simulated tides and currents are reasonable. Water surface elevations were measured at six locations (Figure 2) in the Bay over a 24-hour tidal diurnal cycle during 24-

25 May 1990. This period, 5 AM on the 24th to 6 AM on the 26th, corresponded to maximum new-moon tides during the lunar cycle. Measured tidal elevations at the NOAA tide gauge were used as a boundary condition at the jetty mouth. Tidal measuring stations were selected on the basis on proximity to Orange County bench marks. A bench mark was surveyed for each measurement point and water levels were measured.

Results are shown in Figures 3 and 4 which compare measured tides to simulated tides at five locations. The actual tidal measuring sequence began at 5 AM. To simulate measured tides a pseudo-tide was simulated for the previous five hours to remove the assumed Bay water surface elevations initial conditions effects. As can be seen, good results were achieved using initially assumed n-factors.

Conclusions

A Diffusion Hydrodynamic Model (DHM) of a shallow estuary has been developed using an extension of the USGS DHM computer model. The DHM analog includes several hundred irregular polygon control volumes and nodal points. Simulation of estuary flow depths by the DHM analog were compared to measured tidal gauge flow depths, with good agreement. Because the shallow estuary flow characteristics are two dimensional, a verification of the DHM is provided by the comparisons between tidal gauge measurements and flow depths.

Acknowledgment

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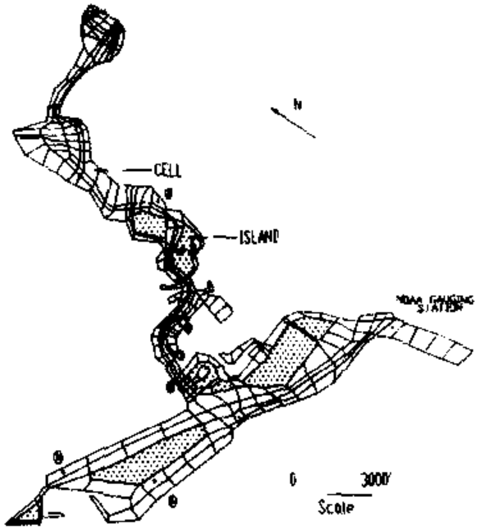

Fig. 1. Newport Bay, Mathematical Model Grid.

Fig. 2. Location of Stations for Water Surface *Elevation Measurements in Newport Bay.*

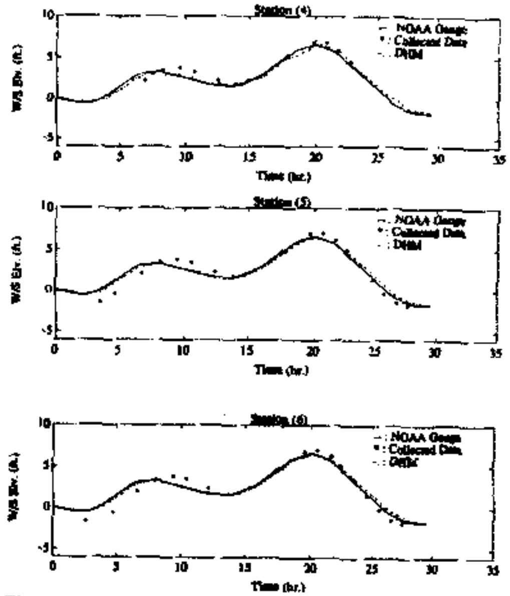

Fig. 3. Lower Newport Bay, Comparison of *Simulated Tides to Measured Tides.*

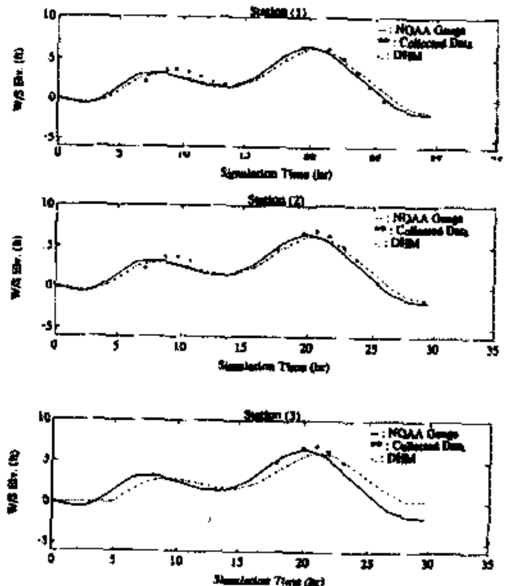

Fig. 4. Upper Newport Bay, Comparison of Simulated Tides to Measured Tides.