particular empirical relation until very late in the derivations. Therefore, the overall structure of the computation is independent of particular empirical expressions used to evaluate auxiliary relations, and thus they can be exchanged rather easily.

The hydrodynamic (depth-averaged Reynolds) equations are solved numerically using a split-operator procedure (momentum advection and diffusion steps had not yet been implemented in the code at the time of this project), and the resulting system of linear algebraic equations is solved by the alternative direction implicit method.

The sediment equations (including bed load for each size class and bed evolution) are solved simultaneously for each computational point using the Newton-Raphson method. Some of the important features of MOBED2 include the following:

The global set of sediment equations for all size classes, taken as a whole and solved simultaneously, describes the behavior of a mixture, including natural phenomena such as differential settling, armoring, and hydraulic sorting.

Sediment particles can move either in suspension or as bed load, depending on local flow conditions. Criteria for distinguishing between bed-load and suspended-sediment transport, as well as mechanisms defining exchange between the two, are incorporated into the code.

Sediment mixtures in natural watercourses are represented through a suitable number of discrete size classes.

Both the hydrodynamic and sediment equations are solved in a curvilinear coordinate system, which implies transformation of the governing equations in the real coordinates $X-Y$ of the so-called physical plane into the computational $\xi-\eta 1$ plane.

The goals of this study were to demonstrate the ability of MOBED2 to simulate unsteady water-sediment flow for the three Iowa reservoirs (Coralville, Saylorville, and Red Rock) and to provide a preliminary calibration of the data sets preliminary to transfer of the code and data sets to the sponsoring user, the Rock Island District of the U.S. Army Corps of Engineers.

15.11.4.2 Data Sources and Model Construction Topographical data came from two sources:

1. 1:24,000 U.S. Geological Survey topographic maps
2. U.S. Army Corps of Engineers Sedimentation Survey Reports

Because the computational-grid spacing was much denser than the spacing of the sedimentation survey sediment ranges (SR), an interpolation procedure, performed by University of Iowa GIS specialists, was used to obtain the computational-grid topology and topography. Numerous manual modifications of the data sets were performed in an iterative process, using the preliminary computation runs, to ensure correct numerical solution of the governing equations. This manual grid adjustment and refinement were a preliminary calibration of the model data sets.

The hydrologic data were provided by the Rock Island District of the U.S. Army Corps of Engineers. For the purpose of preliminary calibration the data were used to set inflow discharges and suspended sediment concentrations for the test runs. The most important data—regarding suspended-sediment size and bed-load size and distribution—were not available, and were thus assumed from Spasojevic (1988).

Specification of the two-dimensional plan-view grid is relatively simple for rectangular (or nearly rectangular) channels, and/or if the expected variations of free-surface elevations are small. However, in natural watercourses, any significant change of the free-surface level may notably change the plan-view contour of the flow domain. One approach to resolving this problem is to define the maximum model-domain contour based on the maximum expected free-surface level, and to treat the periodically dry areas of the model by a special procedure if the water level lowers significantly, so that the flow domain shrinks. However, at the time of this study, MOBED2 was not designed to cope easily with frequent and large changes in wetted and dry areas within the model domain. For this particular study, in which the Old River channels were permanently submerged below the dam-maintained reservoir elevation and the reservoir banks were relatively steep, it was possible to simulate extended periods of time with a single computational grid.

For the Iowa reservoir models, the downstream boundary was the dam itself, the impermeable side boundaries were determined by the maximum water levels, and the upstream boundaries were selected in consultation with Rock Island District engineers so that the major part of the sediment entrapped in the reservoir lay within the computational domain. The computational grid was specified to provide sufficiently detailed information on the studied reservoirs, yet not so detailed as to unnecessarily encumber the already time-consuming computations.

15.11.4.3 General Boundary and Initial Conditions For initial conditions, MOBED2 requires known values or hydrodynamic and sediment quantities appropriate to the beginning of the simulation period: water-surface elevations and two-directional velocity fields for the hydrodynamic equations, and suspended-sediment concentration and distribution of the bed material for each size class and the initial bed elevation for the sediment equations.

Both inflow (upstream) and (outflow) downstream boundary conditions are required for the hydrodynamic computation. The outflow boundary condition can be a rating curve or a given discharge or free-surface elevation hydrograph, whereas the inflow boundary condition can be a discharge or free-surface elevation hydrograph only. For the sediment equations, boundary conditions are required only at inflow (hydrodynamic) boundaries, with prescribed evolution of suspended-sediment concentrations, bed-material distribution, and the bed elevation for each computational point across the inflow boundary.

The test cases for the preliminary calibration were selected to demonstrate the ability of the model to simulate two-dimensional unsteady water-sediment flow in the three Iowa reservoirs, that is, to show that the code can provide long-term simulations without numerical problems.

Accordingly, the test cases presented here were selected to treat a hypothetical hydrological situation, i.e., not to follow the strict details of a particular hydrological time-series. Moreover, even in a less hypothetical application of the models, it is suggested that only major flood events be simulated, i.e., those in which the majority of the sediment inflow occurs. (For example, one may simulate the important sedimentation features of a 50-year period by running only 100–200 months.) In addition to significant savings of CPU time, this helps to alleviate potential dry-bed problems (as explained earlier), because the large flood-flow discharges tend to correspond to the higher pool elevations for which the computational grids were laid out.

Initial data for the hydrodynamic computations required the initial distribution of both components of the depth-averaged velocity and the free-surface elevations. A zero-flow initial condition was assumed, implying a horizontal water level and a zero velocity field for the entire computational domain.

The hydrodynamic boundary condition along the upstream inflow boundaries was the distribution of unit discharge across the boundary. Because measured data for the flow distributions was not available, a reasonable estimate was obtained by distributing the total discharge across the upstream boundary in accordance with the cross-sectional area distribution. Imposition of the free-surface elevation along the dam cross section seemed to be an appropriate boundary condition at the downstream boundary, given the small velocities in the vicinity of the dam.

The sediment computations require representation of the natural sediment mixture in the reservoir by an appropriate number of size classes and their distribution. Measurements and analyses of size distributions for natural sediment mixtures in the Iowa reservoirs are extremely scarce, especially for the bed material. Therefore, the values from Spasojevic (1988) were used as a reasonable assumption for all three reservoirs. Only two size classes were chosen to simulate the natural sediment mixture. Size class 1 represents fine sediment capable of moving in suspension, whereas size class 2 represents coarser sediment mainly confined to the bed. A characteristic diameter of $D = 0.0025$ mm, taken from the size-distribution curve for suspended sediment at the Marengo gauging station (Spasojevic 1988), was used as an equivalent diameter for size class 1, whereas a diameter of $D = 0.6$ mm was used as the equivalent diameter for size class 2. It was assumed that, immediately after the dam was built, the bed consisted predominantly of coarser sediment (size class 2); thus, the initial active-layer size fractions were assigned to be zero for size class 1 and unity for size class 2. Initial bed elevations (as well as the entire geometry of the model domain) were defined based on the original reservoir survey data.

The dam section was treated as an outflow boundary with zero bed-load flux during sediment computations. Imposed suspended-sediment concentrations (obtained from the data provided by the Rock Island District) defined the inflow boundary condition for the suspended sediment; a zero bed-load influx and constant bed elevations, were assumed to be appropriate upstream assumptions for the bed-load boundary conditions, given the lack of meaningful field data.

Ten-year periods were simulated for each of the reservoirs. The first year represented a schematic annual hydrological cycle to demonstrate that the code can perform under unsteady-flow conditions (see Fig. 15-18). The upstream hydrodynamic boundary condition was a schematic discharge hydrograph with a base of Q_{min} and peak of Q_{max}, whereas the similar schematic pool-elevation hydrograph determined the downstream boundary condition; the suspended-sediment concentration variations were assumed to correspond to the inflow hydrograph variations (Fig.15-19). The remaining portion of the 10-year period was simulated with a constant discharge at the representative flood peak Q_{max}, the maximum pool elevation Z_{max}, and the maximum suspended sediment concentration C_{max}, for each of the three reservoirs.

15.11.4.4 Coralville Reservoir The Coralville Reservoir is a flood-control impoundment located on the Iowa River near Iowa City, Iowa. The Coralville reservoir model represents the part of the reservoir from the Coralville Dam up to Sediment Range (SR) No. 21. To define the computational domain of the Coralville model, a flood situation with free-surface elevation around 217 m (roughly 712 ft) was adopted. For this condition the reservoir can be thought of as consisting of two parts with distinctly different hydraulic characteristics, as seen in Fig. 15-20. The part between Coralville Dam and the Curtis Bridge is relatively narrow, with the majority of the cross sections being either roughly trapezoidal or triangular in shape. The part between the Curtis Bridge and the upstream boundary is primarily a broad valley with dominant flood plains.

Fig. 15-20 shows the two-dimensional (plan-view) contour of the model domain, together with the orthogonal curvilinear computational ($\xi-\eta$) grid constructed to fit the model domain. The total number of computational points was 2,937, with $I = 267$ points in the ξ-direction (which is roughly the direction of the flow) and $J = 11$ points in the η-direction (which is roughly the direction perpendicular to the flow).

As described earlier, a zero flow state, with horizontal free-surface elevations and zero flow field, was used for the hydrodynamic initial condition. The initial suspended-sediment concentration for size class 1 (fine sediment) was set to 100 ppm over the entire domain; for size class 2 (coarse sediment), a global zero concentration was assigned as an initial condition.

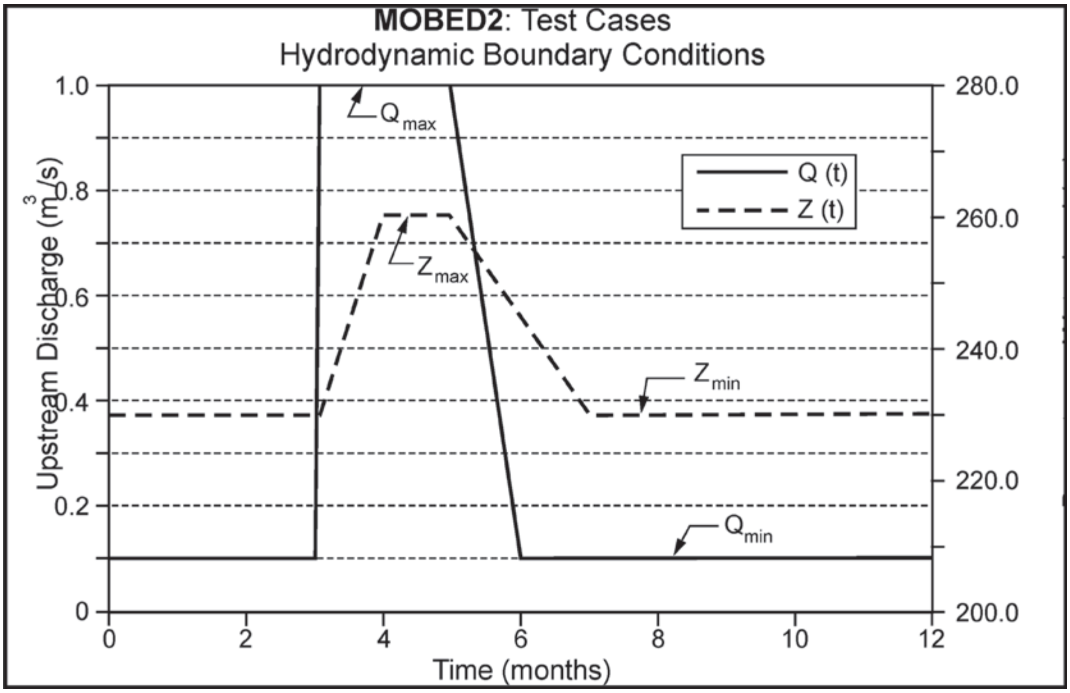

Fig. 15-18. Hydrodynamic boundary conditions for the test cases.

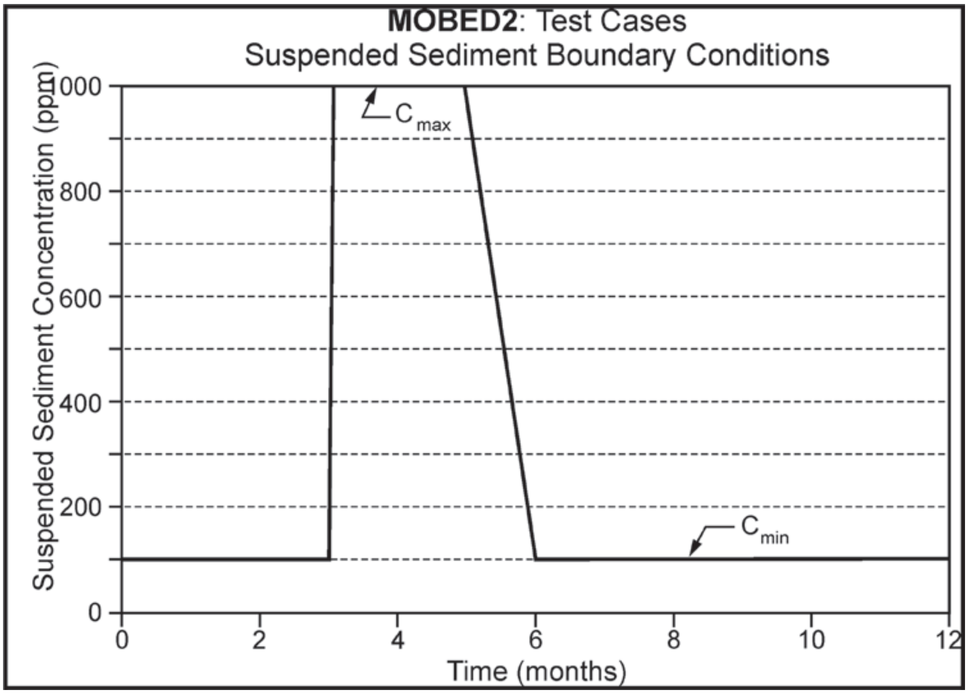

Fig. 15-19. Suspended-sediment boundary conditions for the test cases.

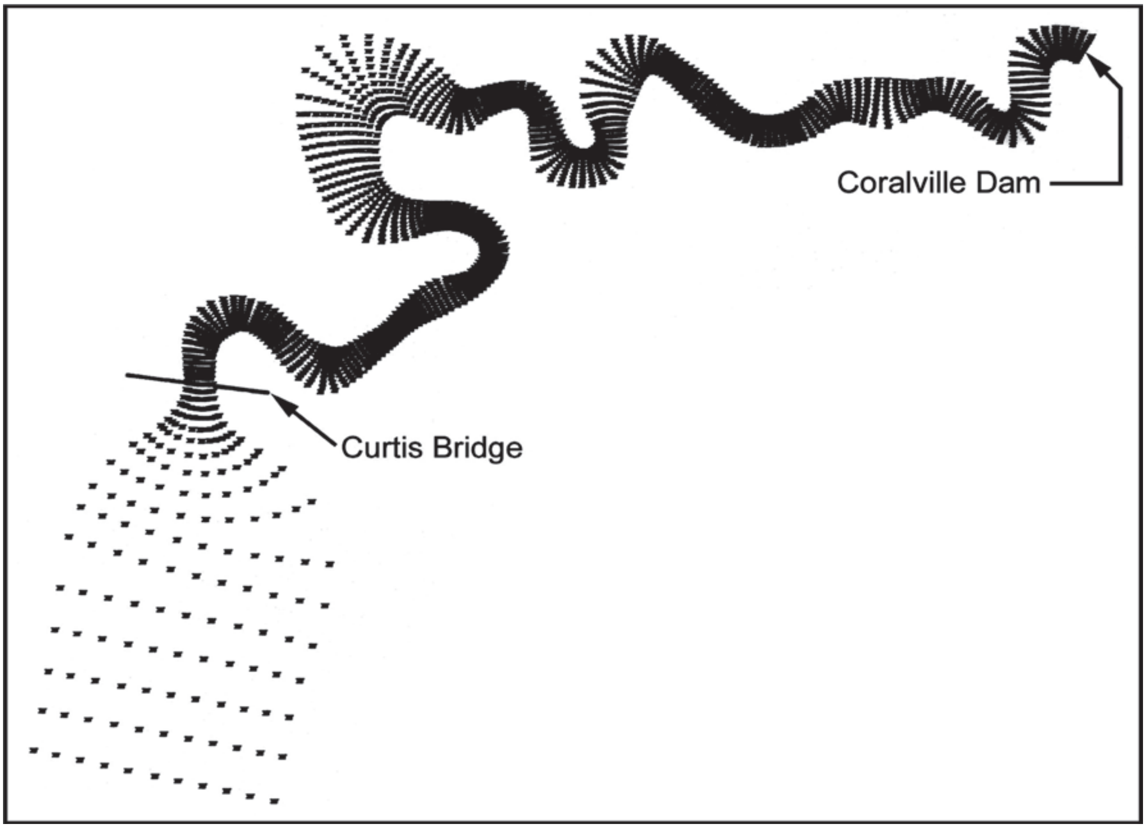

Fig. 15-20. Bed-elevation changes for Section $I = 228$ of Coralville Reservoir for simulation times of $t = 8.2$ and 11 years.

The maximum and minimum discharges of $Q_{max} = 300\ m^3/s$ (10,600 cfs) and $Q_{min} = 50\ m^3/s$ (1,765 cfs) for the upstream boundary condition (see Fig. 15-18) were selected in accordance with the historical hydrologic data. For the downstream boundary condition, free-surface elevations of $Z_{max} = 217$ m (712 ft) and $Z_{min} = 213$ m (698 ft) were selected, thus obviating possible dry-bed conditions, but still leaving the possibility to simulate pool-management operations during flood periods. The suspended sediment concentrations of $C_{max} = 1{,}000$ ppm and $C_{min} = 100$ ppm (Fig. 15-19) were considered to be a reasonable approximation for the purpose of preliminary calibration.

Two characteristic cross sections were chosen to present flow and sediment variables for this test simulation. Figures 15-21 to 15-24 show selected flow/sediment variables at cross section $I = 228$ (corresponding approximately to sediment range SR-5), whereas Figs. 15-25 to 15-28 show cross section $I = 7$ (close to sediment range SR-20). Cross section $I = 228$ (i.e., range SR-5) is located in the narrow part of the reservoir, whereas cross section $I = 7$ (SR-20) is in the wide inundation area upstream of Curtis Bridge.

The distribution of the unit discharge component in the flow direction (U_{st} discharge) across the section $I = 228$ is presented in Fig. 15-19. As expected, larger discharges occur in the zones of larger depth, and the suspended-sediment concentration distribution (for size class 1, i.e., fine sediment) roughly follows the U_{st} discharge pattern (Fig. 15-20). The bed deposition (shown in Fig. 15-21) reflects closely the suspended-sediment concentration distribution, because the deposition component of the suspended-sediment source term (which is the dominant source of bed deposition) is mainly governed by the depth-averaged concentrations and the flow field.

The picture is somewhat different for the section $I = 7$, where the wide cross section produced the velocity field less dominated by ξ-direction velocities, and where the influence of the upstream boundary was felt more strongly. The result was a more evenly distributed bed-deposition (Fig. 15-28), which is in general agreement with the observed field data.

Due to an effective Courant-number limitation, the hydrodynamic computational time step had to be limited to 1 h. This relatively small time step is impractical for simulation of slowly varying sediment movement. Sediment variables changed very little during 1 h; moreover, the sediment computations are extremely time-consuming, and a sediment time step of 1 h would have enormously increased the CPU time. For a slowly varying flow field the problem is circumvented by choosing a "global" time step (for sediment computations) to be much longer than the hydrodynamic one. Hence, within a single global time step, water computations are performed for several short "water" time steps, only the first and latest computed flow fields being used in sediment

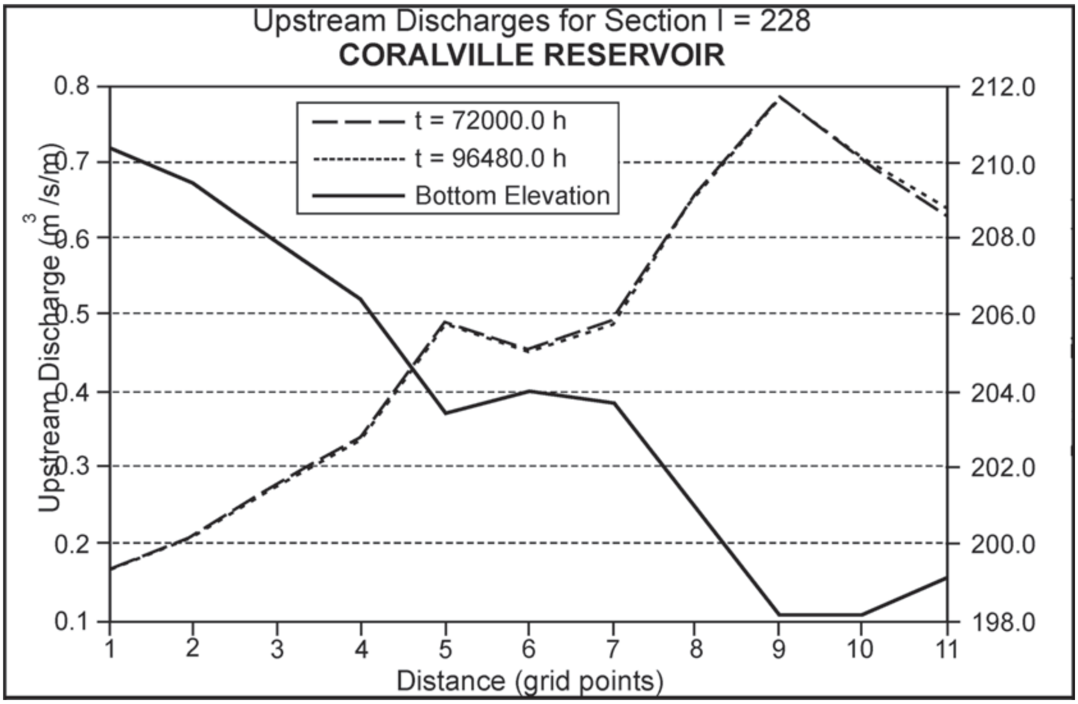

Fig. 15-21. Numerical grid for Coralville Reservoir.

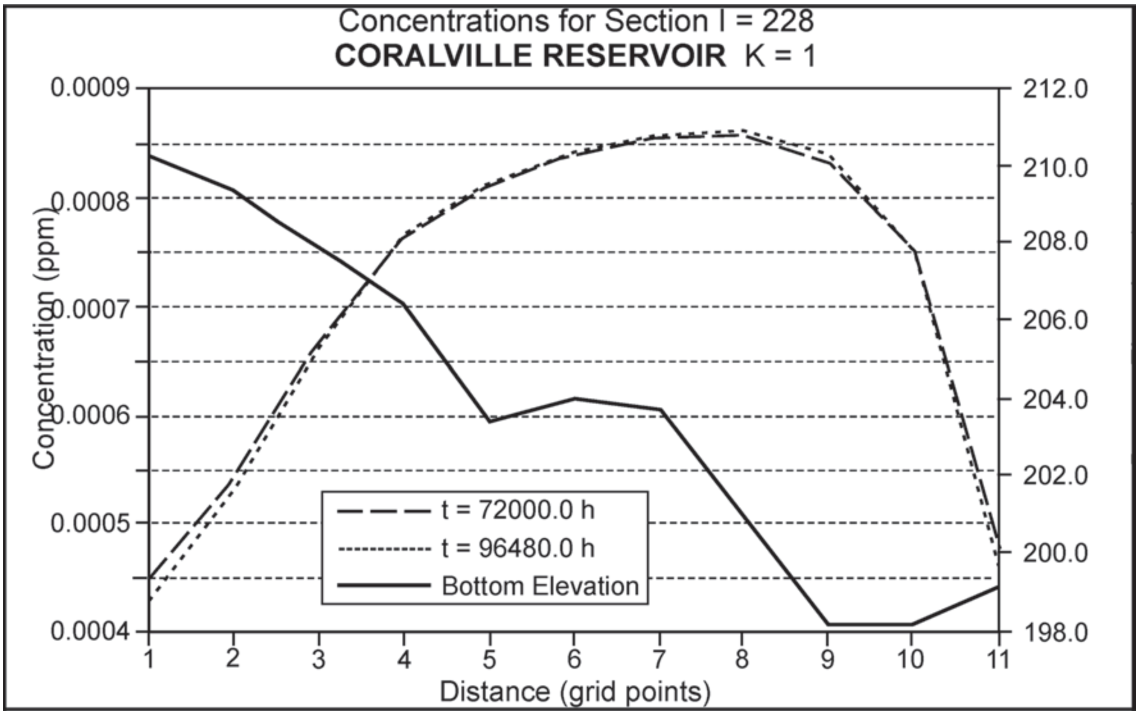

Fig. 15-22. Unit longitudinal staggered discharges for Section $I = 228$ of Coralville Reservoir for simulation times of $t = 8.2$ and 11 years.

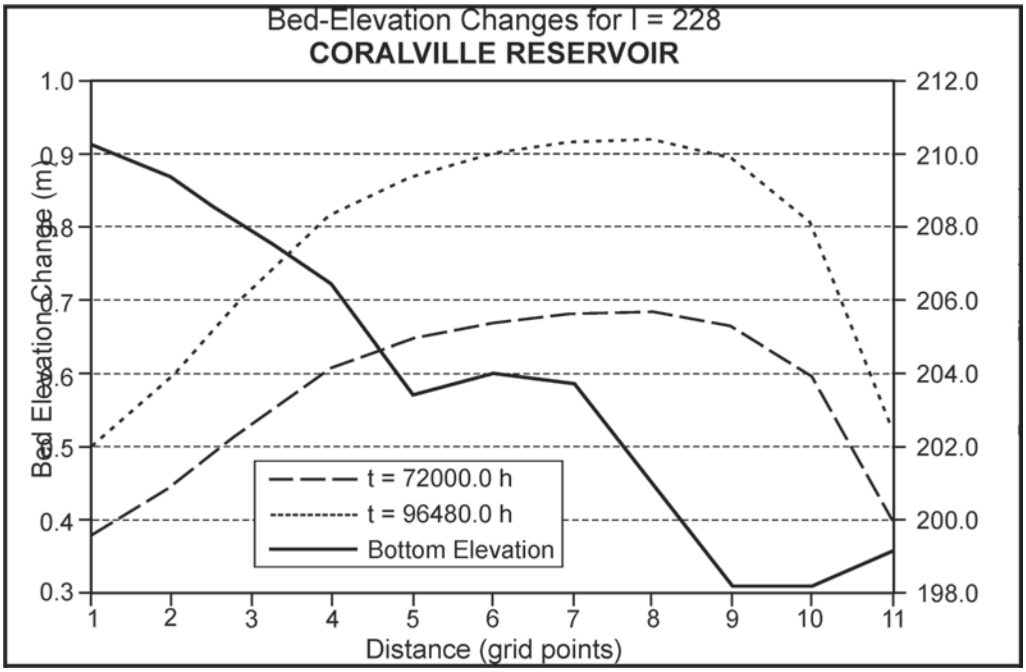

Fig. 15-23. Suspended-sediment concentrations for Section $I = 228$ of Coralville Reservoir for simulation times of $t = 8.2$ and 11 years.

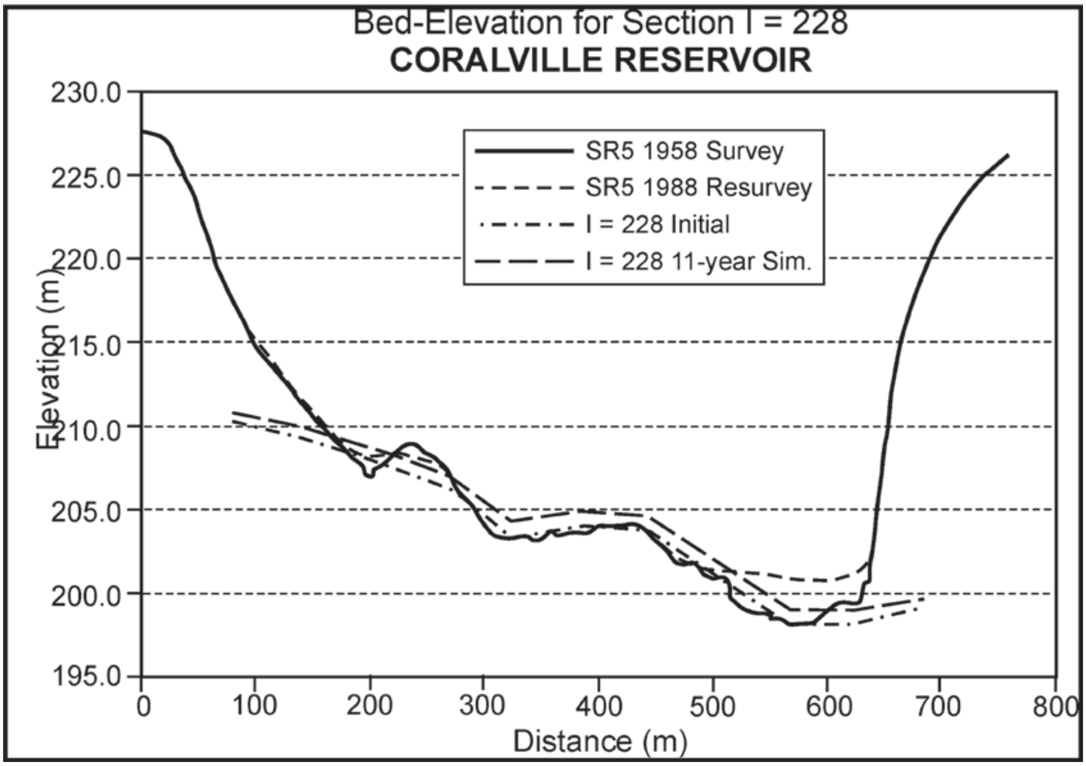

Fig. 15-24. Bed elevations for the Coralville-model Section $I = 228$ at the beginning and end of the 11-year simulation, compared to Sediment Range SR-5 (1958 and 1988) surveys.

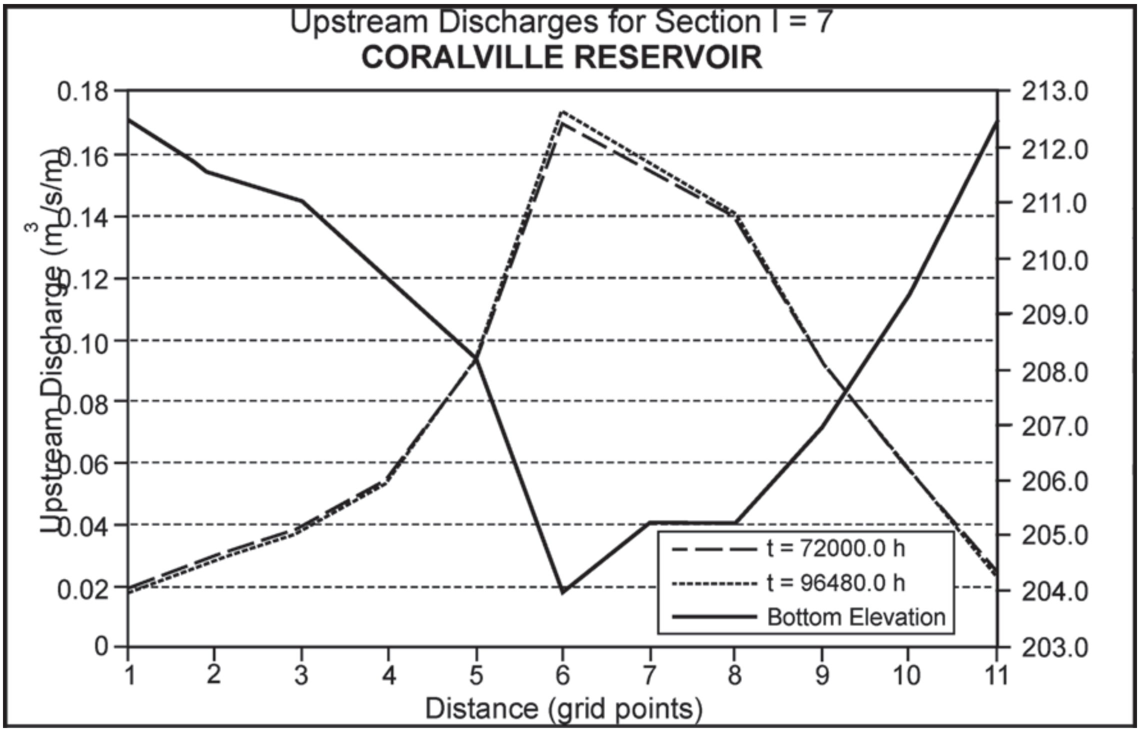

Fig. 15-25. Unit longitudinal staggered discharges for section $I = 7$ of Coralville Reservoir for simulation times of $t = 8.2$ and 11 years.

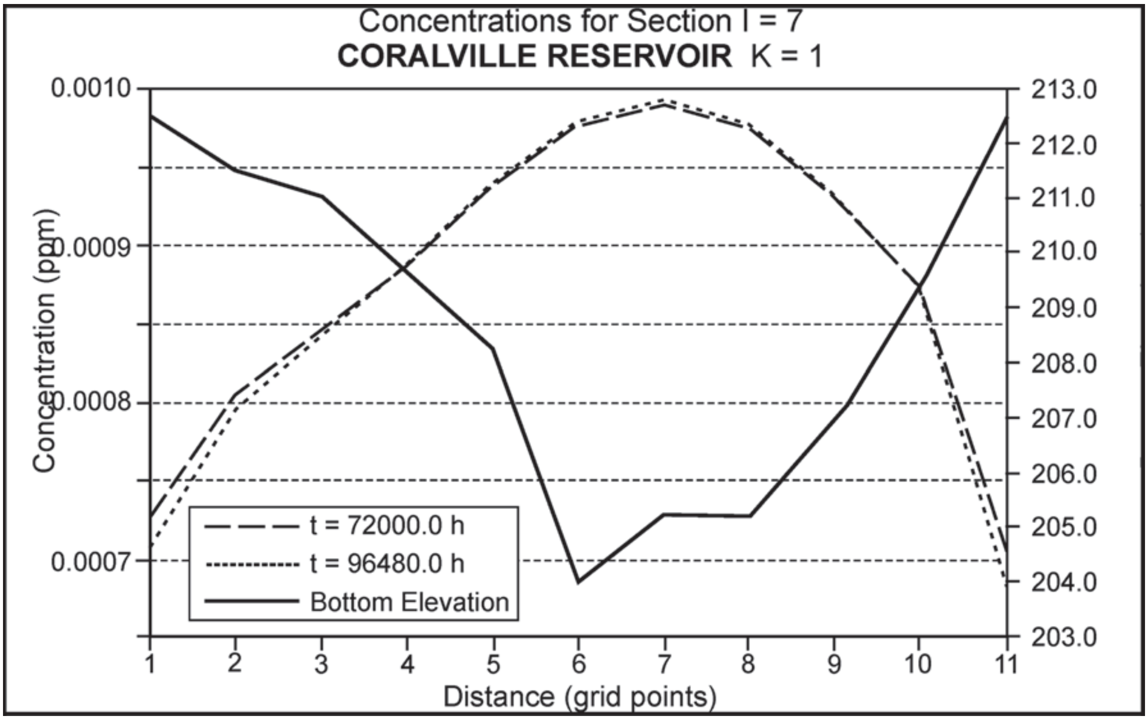

Fig. 15-26. Suspended-sediment concentrations for section $I = 7$ of Coralville Reservoir for simulation times of $t = 8.2$ and 11 years.

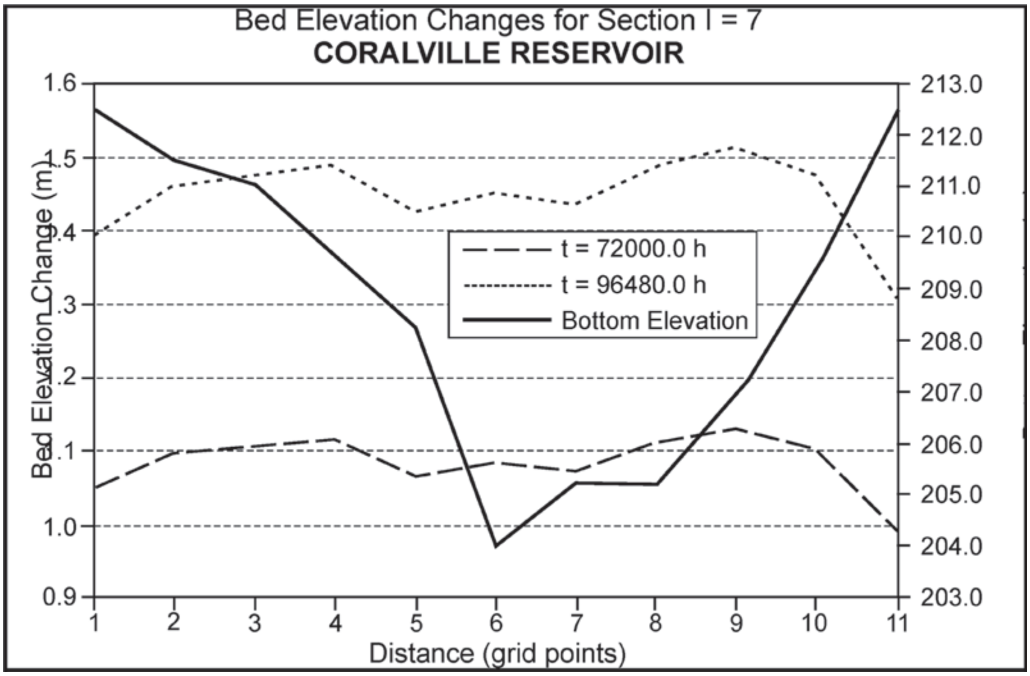

Fig. 15-27. Bed-elevation changes for Section $I = 7$ of Coralville Reservoir for simulation times of $t = 8.2$ and 11 years.

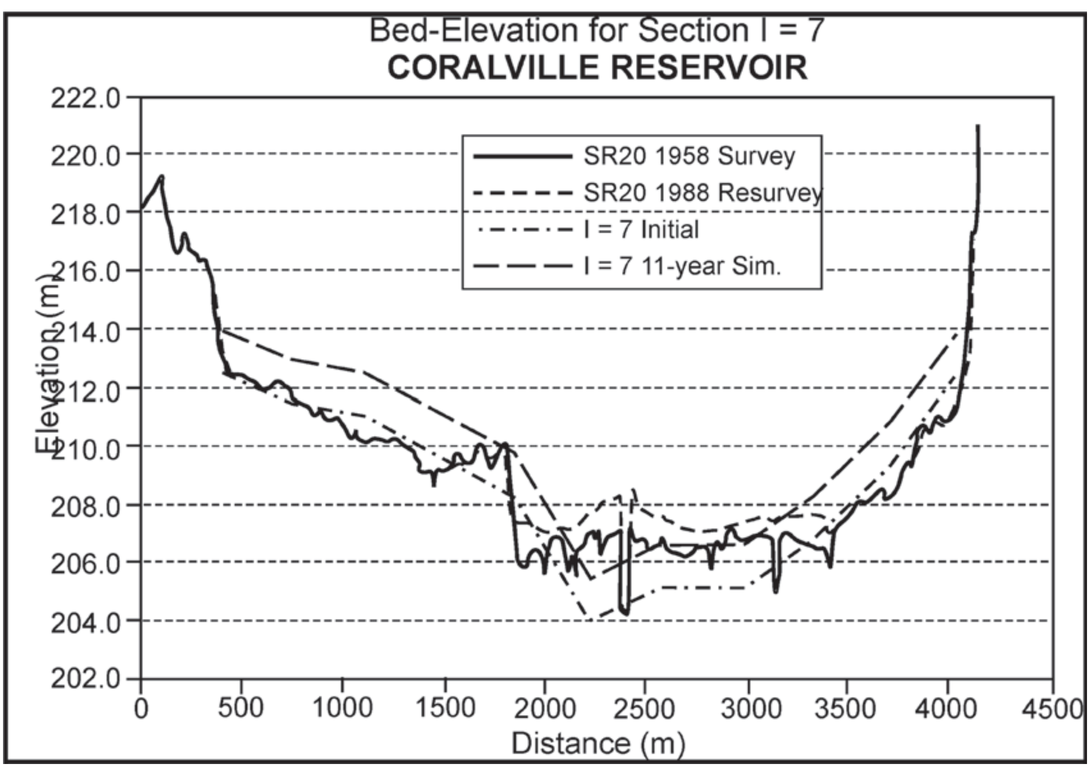

Fig. 15-28. Bed elevations for the Coralville-model Section $I = 7$ at the beginning and end of the 11-year simulation, compared to Sediment Range SR-20 (1958 and 1988) surveys.

computations. A global time step of 24 h was found to be an optimum value for the Coralville Reservoir model.

The CPU time required for the described 11-year simulation was around 200 h on a 486/33 MHz personal computer, using the Lahey 32-bit compiler; one would expect the same run to have taken only about 6 h on a state-of-the-art personal computer. (More iterations, and accordingly, more CPU time were needed for the unsteady part of the computations, i.e., for the first year of the simulation.) The storage memory requirements, beyond the 500K required for the program load module, were 1,650K for the Coralville model.

15.11.4.5 Saylorville and Red Rock Reservoirs The Saylorville and Red Rock reservoir model construction and operation followed the same general pattern as for the Coralville reservoir. Therefore in this section only brief descriptions of the physical situation and model grids are given.

The Saylorville Reservoir is located on the Des Moines River upstream of Des Moines, Iowa. The Saylorville Reservoir model represents the part of the reservoir from the Saylorville Dam up to Sediment Range (SR) No. 15. The computational domain of the model is defined for a flood situation, with pool elevation at 271.3 m (890 ft). Cross-section sediment range 1 is immediately upstream of the dam site, whereas Sediment Range SR-15 is close to the upstream boundary of the model domain. Fig. 15-29 shows the two-dimensional contour of the model domain and the computational grid. The total number of computational points was 1,144, with $I = 104$ points in the ξ-direction (the direction of the flow) and $J = 11$ points in the η-direction.

The Red Rock Reservoir is located on the Des Moines River, downstream of Des Moines, Iowa. The model represents the part of the reservoir from the dam up to Sediment Range (SR) No. 19. The computational domain of the model is defined for a flood situation, with the pool elevation at 237.75 m (780 ft). Cross-section Sediment Range 1 is upstream of the dam site, whereas Sediment Range SR-19 is close to the upstream boundary of the domain. Fig. 15-30 shows the contour of the model domain and the computational grid. The total number of computational points was 781, with $I = 71$ points in the ξ-direction (the direction of the flow) and $J = 11$ points in the η-direction.

15.11.4.6 Summary This two-dimensional example has been included primarily to point out the possibility—even in 1993, when this study was done—of making multiyear simulations to detect sedimentation trends subject to a succession of real or schematic hydrographs. As of this writing, it is not possible to envision such long-term simulations with three-dimensional models, even those based on the hydrostatic pressure assumption. As long as vertical accelerations and secondary flows are relatively unimportant to the problem under study, two-dimensional modeling offers a great deal of power at relatively low computational cost, and therefore is a viable tool within its known constraints.

15.12 CRITICAL ASSESSMENT OF STATE OF THE ART AND FUTURE PERSPECTIVES

As of this writing, two-dimensional (depth-averaged) fixed-bed modeling has reached a certain maturity and seen moderate use. But after a promising beginning, development of two-dimensional (depth-averaged) mobile-bed modeling has taken a back seat to three-dimensional. Meanwhile, three-dimensional fixed-bed modeling is rapidly becoming an effective engineering tool, and its mobile-bed counterpart is receiving considerable developmental attention and enjoying some success in practical engineering use.

It is unfortunate that development and application of two-dimensional (depth-averaged) mobile-bed modeling has become somewhat of an orphan in the rush to develop three-dimensional tools. Two-dimensional modeling, although unable to resolve mobile-bed responses closely related to secondary flow, detailed water and sediment dynamics around structures, and other three-dimensional effects, still offers the possibility of making truly long-term simulations in a way that is currently unthinkable with three-dimensional models. To exploit this potential fully, two-dimensional models need to be based on unstructured or nonorthogonal curvilinear structured grids, have robust wetting and drying capability for application to multiyear hydrologic series, and include both bed-load and suspended-load transport mechanisms in a nonuniform sediment environment.

In both two- and three-dimensional modeling, there is the issue of structured versus unstructured grids. Structured grids (usually nonorthogonal curvilinear and associated with finite-difference methods) are not well suited to grid refinement around local areas of interest or adjacent to hydraulic structures, but are generally attractive for their minimization of computational time (and thus their enabling of longer-term simulations and/or finer resolution of nonuniform sediment). Structured grids (e.g., finite-element or finite-volume, usually associated with flux-based methods) offer great flexibility in grid refinement around structures and local features of interest and lend themselves well to dynamic adaptive refinement, at the cost of relatively high demands on computer resources. Although it is tempting to believe that continuing rapid increases in computer processor speed and parallel systems will eventually make the speed advantages of structured grids irrelevant, experience has shown that this is unlikely to be the case. Indeed, it is always desirable to use a finer grid resolution, adopt more sediment size classes, run for longer periods, or test a greater number of cases, i.e., to push the limits of practical CPU time with whatever numerical tool is being used. It is likely that there will continue to be partisans of, and real needs for, both structured and unstructured modeling systems into the foreseeable future.

Another issue of importance as of this writing is that of fully three-dimensional versus quasi-three-dimensional

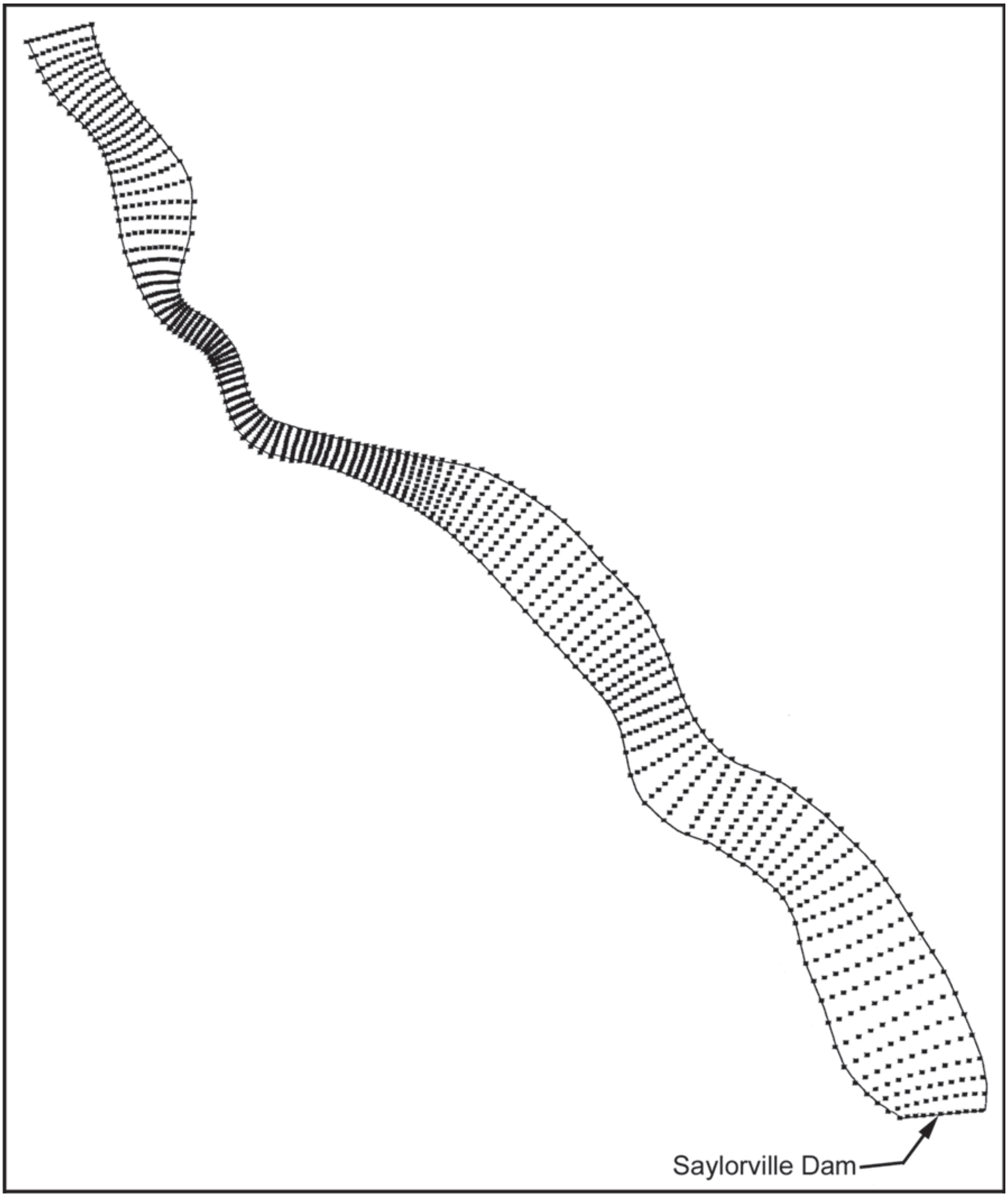

Fig. 15-29. Numerical grid for Saylorville Reservoir.

(i.e., hydrostatic) hydrodynamic modeling as a framework for mobile-bed models. Experience has shown that water and sediment movement in the immediate vicinity of structures (e.g. submerged dikes and bridge piers) can be correctly represented only if vertical acceleration components are explicitly taken into account, i.e., only if the model explicitly includes the vertical momentum equation. Quasi-three-dimensional models, in which the vertical momentum equation is replaced by the hydrostatic pressure assumption, offer the considerable advantage of orders of magnitude decreases in computational time (the solution essentially comprises a two-dimensional one followed by application of the three-dimensional water continuity equation to recover vertical velocities). At the present time, truly unsteady simulations of any significant duration cannot be performed using full three-dimensional models, whereas they are becoming feasible with models based on the hydrostatic pressure assumption, as described in the examples of the previous section. In time, increases of computing speed