CHAPTER 14

Computational Modeling of Sedimentation Processes

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14.1 INTRODUCTION

The notion that a river channel is stable is often accompanied by the mental image of a river channel whose bed profile, cross sections, and channel pattern do not change over time. However, dynamic equilibrium is a more appropriate concept for describing a stable alluvial channel. Dynamic equilibrium is the process by which an alluvial river transports its water-sediment mixture. Typical responses of a channel that is in dynamic equilibrium are deposition of sediment on the bed and erosion from it, channel widening and channel narrowing, bank failure and bank migration, smoother banks and rougher banks, the growth and removal of bank vegetation, and changes in the channel planform. Seldom do these processes occur singly. They are closely interrelated, and they seem to be delicately balanced to maintain a dynamic state of equilibrium. Experience has shown that changing or limiting one of these responses can impact the others (see Chapter 18).

The study of how a river develops is called river morphology by Leopold (1994) and Rosgen (1996) and fluvial geomorphology by Schumm (1971). River morphology studies correlate the dimensions, planform, and movement of a river channel with the historical loads imposed on it (see Chapter 6). The river can be described in terms of six variables:

- channel width;
- channel depth;
- channel slope;
- hydraulic roughness;
- bank line migration;
- channel pattern.

For example, the historical channel width is correlated with the historical water discharges and the type of materials that formed the banks of the channel. The channel depth and longitudinal slope are correlated with water discharge and the size of sediment particles. The meander pattern and changes in channel planform are correlated with channel width, slope, and water discharge. In each case these variables are correlated with the load imposed on the river. That load is composed of the water-sediment mixture conveyed by the river and the base-level energy control.

However, the correlations are empirical and do not describe the physics of the processes. Without physical theories one is not able to calculate the reaction of a channel to changes in the loads imposed upon it. Therefore, river morphology studies alone are not adequate for project design, but they do make valuable contributions to river engineering. First, they identify the variables that river engineers must analyze and change in the design of a new project or in the restoration of an existing river to a historical condition. Second, river morphology studies recognize that those variables are interrelated. Third, the variables are identified as the dependent variables in a river system and not the independent variables. Fourth, the river morphology approach recognizes that the materials through which a natural river flows are extremely diverse, and it allows nature to aggregate the microdistributions of force and resistance into average values for the six variables listed above. Finally, river morphology studies provide a framework for identifying and organizing the data that are essential for the computational modeling of river systems.

This chapter presents a systematic procedure for applying one-dimensional computational sedimentation models to the study of alluvial rivers. A computational sedimentation model includes the five basic processes of sedimentation: erosion, entrainment, transportation, and deposition of mixtures of sediment particles, and compaction of sediment deposits. Of paramount importance is the fact that computational sedimentation models may include only some of the equations that are needed to predict the morphology of a river channel. Therefore, the river morphology equations that are included in one-dimensional computational sedimentation models need to be identified, and the model should then be used in combination with river morphology principles to perform the desired sediment study.

The water-sediment mixture conveyed by a channel and the base-level control go together to determine the load on the river system. The load is the independent variable in the correlations discussed above. The six variables that are listed are the dependent variables. The significance of classifying these variables as either dependent or independent has to do with project stability. A design can change the value of an independent variable, but if a dependent variable is changed it will not remain changed. For example, a project in which the channel width is increased will not function as designed without continual maintenance because channel width is a dependent variable. Two- and three-dimensional models are discussed in Chapter 15.

14.2 LOCAL SCOUR AND DEPOSITION

This chapter does not address local scour or deposition. Local scour, as compared to channel degradation, refers to the scour hole that forms around a bridge pier, downstream from a hydraulic structure, along the outside of a bend, etc. The process involves fluid forces beyond local boundary shear. Such forces come from three-dimensional flow accelerations, pressure fluctuations, and gravity forces on the sediment particles. Three-dimensional computational models that make such calculations are in various stages of development, but at present the complexity of local scour processes relegates analysis to empirical equations or physical model studies.

Local deposition refers to deposits over a relatively small space, as opposed to channel aggradation, which raises the bed profile of the river over a substantial distance. Local deposition can be predicted with one-dimensional equations provided that adequate attention is given to the rate of expansion of the flow, both horizontally and vertically.

14.3 GENERAL EQUATIONS FOR FLOW IN MOBILE BOUNDARY CHANNELS

14.3.1 Energy and Continuity Equations

The one-dimensional differential equations of gradually varied unsteady flow in movable bed channels are extensions of the Saint-Venant equations for rigid boundary channels. They are the equation of continuity for sediment, the equation of continuity for water, and the equation of motion for the water-sediment mixture. The forms developed by Chen (1973) are as follows:

$$\frac{\partial(\rho Q)}{\partial t} + \frac{\partial(\rho QU)}{\partial x} + gA\frac{\partial(\rho y)}{\partial x} = \rho gA\,(S_o - S_f + D_l) \tag{14-1}$$

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} + \frac{\partial A_d}{\partial t} q_w = 0 \tag{14-2}$$

$$\frac{\partial G_s}{\partial x} + (1-P)\frac{\partial A_d}{\partial t} + \frac{\partial A_s}{\partial t} - g_s = 0 \tag{14-3}$$

where

A = end area of channel cross section;
A_d = volume of sediment deposited on the bed per unit length of channel;
A_s = volume of sediment suspended in the water column per unit length of channel;
D_l = momentum loss due to lateral inflow;
g = acceleration of gravity;
G_s = sediment discharge;
g_s = lateral sediment inflow per unit length of channel, outflow (−), inflow (+);
P = porosity of the bed deposit (volume of voids divided by the total volume of sample);
Q = water discharge;
q_w = lateral water inflow per unit length of channel, outflow (−), inflow (+);
S_f = friction slope;
S_o = slope of channel bottom;
t = time;
U = flow velocity;
x = horizontal distance along the channel;
y = depth of flow;
ρ = density of the water.

The following assumptions were cited in deriving these equations:

1. The channel is sufficiently straight and uniform in the reach so that the flow characteristics may be physically represented by a one-dimensional mode.
2. The velocity is uniformly distributed over the cross section.
3. Hydrostatic pressure prevails at every point in the channel.
4. The water surface slope is small.
5. The density of the sediment-laden water is constant over the cross section.
6. The unsteady-flow resistance coefficient is assumed to be the same as for steady flow in alluvial channels and is approximated from resistance equations applicable to alluvial channels or from field survey.

14.3.2 Sediment Transport Equations

Sediment transport equations are so numerous and varied that only the most general functional form is selected to demonstrate the significant parameters,

$$G_s = f(\bar{U}, r, S_f, b, d_e, s, \mathrm{SF}, d_{si}, P_i, s_l, T, C_{fm}) \tag{14-4}$$

where

$\bar{U}$ = mean velocity at vertical;
r = hydraulic radius;

S_f = slope of energy gradient;
b = width;
d_e = effective grain size of the bed material mixture;
s_s = specific gravity of the particles;
SF = shape factor of the particles $_i$ in the bed mixture;
D_{si} = diameter of each size class, $_i$ in the bed mixture;
P_i = fraction of each size class, $_i$ in the bed mixture;
s_f = specific gravity of the fluid;
T = water temperature;
C_{fm} = concentration of fine sediment in the water column.

The variables U, r, S_f, and b are the hydraulic parameters. Sediment grain parameters are d_e, s_s, SF, D_{si}, and P_i. Fluid parameters are S_pG_f, T, and C_{fm}.

Computational modeling requires that sediment transport be calculated by size class. Therefore, if the transport function is a single-grain-size representation, the computational model must provide a separate bed-sorting algorithm to account for hiding and armoring processes. Even the multiple-grain-size functions require additional, sophisticated bed-sorting algorithms to accommodate the nonequilibrium conditions in the entrainment, transportation, and deposition processes being modeled (Copeland 1993).

To date most researchers in sedimentation have dealt with sand-bed streams (see Chapter 2). Less is known about gravel transport (see Chapter 3). Even less research has been conducted on cobble/boulder transport than has been conducted for gravels. Cohesive sediment transport is not understood as well as noncohesive sedimentation (see Chapter 4). The processes include electrochemical forces, and the presence of the sediment particles can change the properties of the water-sediment mixture. Transport capacity does not obey the equilibrium principle, which states that the number of particles being deposited must equal the number being eroded.

14.3.3 Diffusion and the Diffusion Equation

In mathematical modeling of sediment processes, the sediment discharge potential is computed at each discrete cross section. These potentials reflect the current hydrodynamic forces in the flow field. However, the actual suspended sediment concentration profiles do not adjust immediately to changes in hydrodynamic forces. Both advection and diffusion are significant processes in the physics of adjustment.

The concepts of diffusion in turbulent flow are presented in Chapter 2 of this volume. For nonequilibrium sediment transport, the transport potential must be corrected for the advection-diffusion processes to account for conditions where the development length for equilibrium sediment transport is longer than the grid size δx. The correction for deposition is different from the correction for entrainment.

One approach to accommodating the diffusion process is to include the advection-diffusion equation in the entrainment and deposition calculations for material moving between the bed and the water column. Another approach is to approximate the diffusion process with entrainment and deposition coefficients. In either case the objective is to distinguish between the actual transport rate C_s' and the transport capacity C_s for the equilibrium condition.

Generally,

$$C_s' < C_s \quad \text{for} \quad U/x > 0 \tag{14-5}$$

and

$$C_s' > C_s \quad \text{for} \quad U/x < 0 \tag{14-6}$$

In the diffusion theory of sediment transport, the concentration of suspended load C is described by the convection-diffusion equation,

$$\frac{\partial C}{\partial t} + u\frac{\partial C}{\partial x} + \frac{\partial}{\partial x}\epsilon_x\frac{\partial C}{\partial x} = -\alpha\, w_s(C - C_*) \tag{14-7}$$

The first term in Eq. (14-7) accounts for a nonsteady concentration of sediment with respect to time. The second term accounts for the nonuniform distribution of concentration in the direction of flow. Each of those two terms is zero for equilibrium sediment transport with no local inflows. The third term accounts for the diffusion process.

The right-hand side of the equation accounts for the mass transfer between the bed and the water column. Mass transfer is based on the sediment deposition and the entrainment rates, where C_* is the equilibrium concentration of sediment or the potential carrying capacity of a specific flow, and α is a dimensionless coefficient that characterizes the rate at which the new carrying capacity is attained. The term $-w_sC$ represents the actual flux; the second term, w_sC_*, is the transport capacity flux. In other words, the rate of deposition (or entrainment) by the flow is proportional to the difference between the actual suspended load and the sediment transport capacity of the flow.

The value of α must be determined separately for the cases of deposition and entrainment because of the different physical forces that dominate. The deposition case is the simpler of the two because it depends on the settling velocity of the sediment particles. Zhang et al. (1983) propose the expression

$$\alpha = 1 + \frac{\mathsf{P}}{2} \tag{14-8}$$

where P is the Peclet number, defined as $6w_s/\tau U_*$.

In the case of entrainment, they propose the following relationship for α':

$$\alpha' = \frac{2}{\mathsf{P}} + \frac{\mathsf{P}}{4} \tag{14-9}$$

In solving Eq. (14-7) appropriate boundary conditions and initial conditions are required. The upstream boundary condition is given by

$$C = C_o \text{ at } x = x_o \tag{14-10}$$

where the subscript o designates values at the upstream boundary.

Another approach to accommodating the diffusion process in sedimentation modeling is using entrainment and deposition coefficients. In the case of deposition the settling velocities of the individual sediment grain sizes can be used to calculate the deposition coefficients. However, the entrainment process is not associated as strongly with the settling velocity of the sediment particles as it is with the hydrodynamic forces in the flow field. A surrogate parameter for estimating the entrainment coefficient is flow distance. Flow distance refers to the distance that the water-sediment mixture has to travel before the velocity and sediment concentration profiles reach equilibrium. The concept comes from physical modeling in a flume. Some claim that, in a flume, the distance from the headgate that is required for the flow to attain the theoretical vertical velocity-distribution profile predicted by the log-velocity distribution law is 100 times the flow depth. By similitude, the travel distance in the flume can be used to approximate requirements in the river. For example, use flow depth as the scaling parameter. Therefore, the distance in the river that is needed for the sediment concentration to increase from a lower to a higher equilibrium value could be approximated as a coefficient times the flow depth.

14.3.4 Allocation of Scour and Fill

In one-dimensional modeling the solution of the sediment continuity equation provides a change in the cross-sectional area. That end area change must then be allocated to each coordinate point across the cross section. Different computational models approach the allocation calculation differently. In any case the computation of sedimentation processes is one-dimensional, which, at best, relegates the allocation calculation to an approximation. Consequently, the shape of the cross section is not a question to address with a one-dimensional sediment model. Perhaps some observations of different conditions will aid in understanding the different modeling approaches to this issue.

Emmett and Leopold (1963) investigated scour and fill of the bed profile and of the channel cross section in both ephemeral and perennial streams. They used scour chains, so conditions during the passage of the hydrograph were not measured. However, in a stable river channel on a perennial stream, sediment tends to deposit in the crossings and to erode from the bends during a flood event. After the flood passes, the deposition/erosion sequence will switch, so the crossings will tend to erode and sediment will deposit in the bends. The distribution of erosion and deposition across a cross section will be shaped by the same hydraulic forces that shaped the initial cross sections. Therefore, deposition will not be horizontal nor will it fill the deepest portion of the cross section first. The allocation can be made as a veneer over the surface of the original cross section. Similitude suggests that both deposition and erosion can be applied to the cross section using the veneer concept. Thomas utilized this concept in developing HEC-6 (HEC-6 1977; 1993; Thomas 2002).

In the ephemeral channels of the arid southwest, visual observation suggests that the surface of the channel cross section is usually horizontal at the beginning and at the end of a runoff event. However, during the flood runoff it is reasonable to suspect that the cross section will be reshaped by hydrodynamic forces and sedimentation processes appropriate for flow through a river bend. That is, the secondary flow cells will move the thalweg toward the outside of the bend and will form the classical point bar pattern on the inside of the bend. In ephemeral streams the veneer concept is probably a poor approximation to actual sedimentation processes in the cross section during the passage of an flood event. Chang utilized that observation in developing FLUVIAL12 (1985).

A horizontal deposit is more likely in reservoir deposition than it is in a river channel. In a reservoir the bed material load seems to deposit in the original channel section first. It fills the channel feature, and the water-sediment mixture spills out laterally. When the reservoir level falls the channel will cut through the delta deposit in, perhaps, some new location. However, unless there is a change in the runoff discharges, the width and depth of the new channel will be very similar to those of the original channel. Consequently, a one-dimensional model is able to predict the rate of delta growth and the resulting water surface elevations even though it does not mimic the channel avulsion process.

The physics of sedimentation processes are such that a natural levee tends to build along the top bank of the channel (James 1985). Those forces are also active in reservoir deposition. Sediment size and water velocity are the significant parameters in determining how far sediment particles move away from the channel. This is not a one-dimensional process, and one-dimensional models approximate the process differently.

Some sedimentation models are built around the concept that the width and depth of a river channel will be adjusted to effectively reduce the streamwise variation in stream power as the river seeks to establish a new equilibrium. In such models, the allocation of scour and fill across a section for a time step is assumed to be a power function of the effective tractive force $\tau_o - \tau_c$. Chang proposes the equations for allocating scour and fill

$$\Delta z = \frac{(\tau_o - \tau_c)^m}{\sum_B(\tau_o - \tau_c)^m}\frac{\Delta A_b}{\Delta Y} \tag{14-11}$$

where

Δz = the local correction in channel-bed elevation;
$\tau_o = \gamma DS$ = local tractive force;
τ_c = critical tractive force;
m = exponent;
y = horizontal coordinate; and
B = channel width.
The value of τ_c is zero in the case of fill.

The m value in Eq. (14-11) is generally between 0 and 1; it affects the pattern of scour-fill allocation. For the schematic cross section shown in Fig. 14-1, a small value of m, say 0.1, would mean a fairly uniform distribution of Δz across the section; a larger value, say 1, would give a less uniform distribution of Δz, and the local change will vary with the local tractive force or will vary roughly with the depth. The value of m is determined at each time step so the correction in channel bed profile will result in the most rapid movement toward uniformity in power expenditure, or linear water surface profile, along the channel.

Equation (14-11) can only be used in the absence of channel curvature. The change in bed area at a cross section in a curved reach is

$$\Delta A_b = \frac{1}{r_f}\int r\,dz dr \tag{14-12}$$

where r_f is the radius of curvature at the discharge centerline or thalweg. Because of the curvature, adjacent cross sections are not parallel and the spacing Δs between them varies across the width. Therefore, the distribution of Δz given in Eq. (14-11) needs to be weighted according to the r-coordinate

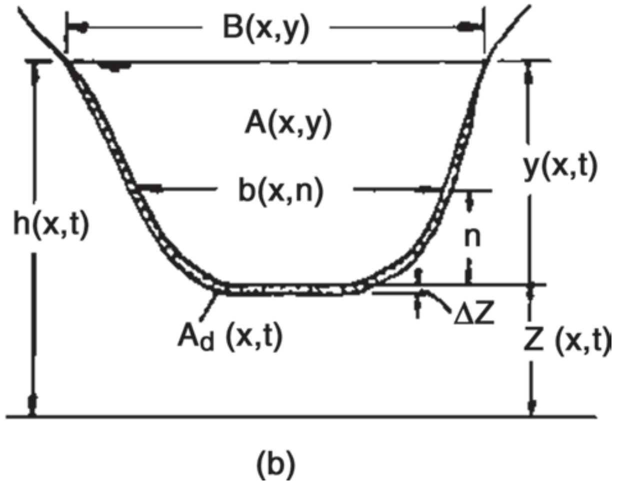

Fig. 14-1. Schematic cross-sectional change.

with respect to the thalweg radius r_f/r (Chang, 1985). The equation is

$$\Delta z = \frac{(\tau_o - \tau_c)^{m/r}}{\sum_B(\tau_o - \tau_c)^{m/r}} \cdot \frac{\Delta A_b}{\Delta r} \tag{14-13}$$

14.3.5 Channel Width

Channel width is one of the morphological variables listed previously. If the channel is too narrow for the runoff hydrology, the banks will erode, causing the channel width to increase. Likewise, in a channel experiencing bed erosion, banks will fail, resulting in channel widening. Modelers must accommodate changes to channel width, and the approach depends on the requirements of the model.

14.3.6 Planform and Bankline Migration

The natural alignment of an alluvial river channel is the result of hydrodynamic forces, sedimentation processes, and soil mechanics principles (see Chapter 8). The hydrodynamic forces are calculated from the conservation of energy, conservation of mass, and flow resistance. Sedimentation processes, as defined above, are the erosion, entrainment, transportation, and deposition of mixtures of sediment particles and the compaction of sediment deposits. Soil mechanics principles describe bank stability. However, channel-bed and bank materials are not homogeneous. The native materials range from inorganic and organic sediment particles to vegetation. The inorganic sediments range from cohesive clays to noncohesive boulders, and the organic sediments range from leaves to large woody debris. These different materials exhibit different strengths and weaknesses. They resist hydrodynamic forces via complex interactions that vary in time and space. For example, bank failure will remove trees and vegetation from the banks, resulting in a change of bank roughness. The effect of such a change in boundary roughness on energy dissipation is especially significant when the width/depth ratio is small.

For example, during the decade of the 1950s, creeks in northern Mississippi were converted into straight canals to improve drainage. The conversion changed the channel width and the slope. The first reaction of the creeks was erosion of the bed. Soon the bed had eroded so deeply that the banks became too high to remain stable, and bank failures occurred on a grand scale. As the channel became deeper, the bank-full water discharge increased, and that increased the amount of the total runoff energy that had to be dissipated on the channel bed and banks. The eroded banks not only were exposed to larger stresses from the larger channel discharges but also were exposed to erosive forces from raindrop impact.

At the same time that bank failure was increasing the channel capacity, it was reducing the hydraulic roughness. Two processes were involved. First, as the width/depth ratio increased, the effect of bank roughness on the composite hydraulic roughness of the channel cross section increased.

Second, the eroding banks removed the prevailing vegetation and prevented new growth. Consequently, the benefit of vegetation roughness on the banks was eliminated.

As the water and sediment mixture left the channelized reach at the downstream end, it entered the natural creek. However, the concentration of bed material load in the flow from the channelized reach was higher than could be transported in the natural creek. Transport capacity had returned to prechannelized conditions and was considerably less than in the channelized portion of the creek. Consequently, a deposition zone developed. The new deposits changed the current pattern, which initiated a new meander pattern. It is significant that where the channels were straightened during construction, they remained straight during the eroding phase of the channel evolution. However, when flow reached the deposition zone, channel meander intensified.

As time passed, all six channel parameters changed in the creeks of northern Mississippi. The amount of change showed significant variation from place to place. It is common to apply computational models to such problems and to use river morphology principles in developing the one-dimensional model. The channel evolution model proposed by Schumm et al. (1984) describes these processes.

The development of a river channel is often controlled by the microdistribution of its boundary materials along the stream corridor and not by the average of these distributions. Moreover, a single downed tree can realign an entire channel, change the channel pattern, and not change the channel width.

14.4 SIMILARITY BETWEEN COMPUTATIONAL MODEL STUDIES AND PHYSICAL MODEL STUDIES

A computational model study can be organized into ten tasks as follows:

1. Assemble available data from office files: maps, cross sections, suspended sediment measurements, bed load data, bed material measurements, soil types/sediment yield, hydrographs, water temperature, observed water surface profiles, reservoirs in the basin, construction activities.
2. Develop geometric data set and run a steady-state water discharge: run a 2-year peak discharge to identify trouble spots and data gaps.
3. Make a reconnaissance trip through the study area: identify locations of bank and bed instability; observe features that will aid in establishing n-values of the bed, banks, and overbanks; give particular attention to locations appearing to be trouble spots; prepare requests for additional/missing data.
4. Calibrate n-values: run the model in fixed-bed mode to compare calculated water surface elevations to observed values; add sediment and run the model in movable-bed mode; confirm that the calculated water surface approximates the observed value.
5. Develop the sedimentary data set: develop the bed gradation; develop the inflowing sediment concentration; select the transport function; develop the gradation of the inflowing sediment concentration.
6. Calibrate the model: estimate the channel-forming discharge in each segment; run a series of steady flows and confirm sediment delivery; run historical hydrographs and sediment concentrations and demonstrate that the model results will match specific gauge plots if data are available; confirm that model results match annual sediment yields.
7. Run base test: run the no-action condition using future conditions hydrology and sediment concentration.
8. Run plan test: define the conditions to be tested and organize into a series of model tests; install the conditions into the base test model, one at a time, and run.
9. Analyze results: compare the results of the plan test with those from the base test to evaluate how much impact sedimentation will have on the plan and how much impact the plan will have on stream system morphology.
10. Perform a sensitivity analysis: change the boundary condition values or the initial condition values by 25% and rerun; express model results as a comparison with those for the base test and the plans tested.

This list of tasks is not a recipe. It is suggested as tasks one can use to organize a model study. Exceptions to this organization are acceptable. However, it is desirable to document the reasons for exceptions.

The rational for these ten tasks comes from the similarity between computational model studies and physical model studies. That rationale is presented in more detail in the subsections that follow.

14.4.1 Model Limits

In physical model studies, the expression "model limits" refers to the limits of the prototype area that will be constructed in the model. The prototype refers to the actual project being studied. The space inside the model limits is the area that will be included in the model. Model construction is the process of molding the (x,y,z) dimensions of the prototype into the dimensions required for the scale model. Measurements of hydraulic parameters and the resulting sedimentation processes are made in the model area. The same concepts are followed in computational modeling. The process of converting the area of the prototype that is within the limits of the computational model into a digital representation of the prototype is called model development.

The location of model limits is not arbitrary. The inflow end of the model must be in a location where the inflowing water discharge and sediment concentration by particle size are known. The tailwater elevations at the outflow end must be known. Moreover, these known values must not be changed by any changes that happen within the model area during the simulation period. The data assembled in Task 1 will be valuable in establishing model limits.

14.4.2 Headgate and Tailgate

In physical model studies the main water supply at the upstream end enters the model area through a headgate that regulates the inflowing water discharge rate and the flow pattern. Flow leaves the model area at a tailgate that regulates the tailwater elevation. These facilities provide the necessary boundary conditions for the model study. In this case, boundary conditions do not refer to the geometry or surface conditions within the model area.

14.4.3 Boundary Conditions for the Computational Model

Mathematically, computational sedimentation modeling is an initial-boundary value problem. That is, there are more unknowns to be solved than there are equations. Therefore, the problem is conditioned by prescribing the missing unknowns at the inflow and outflow boundaries of the model. There are four boundary conditions: the inflowing water discharge, the inflowing sediment concentration by particle size, the tailwater elevation, and the water temperature.

The need for inflowing water and sediment loads form a requirement in the computational model that is analogous to the headgate of a physical model. The need for a tailwater elevation (i.e., base-level control) in the computational model is analogous to the requirement for a tailgate in a physical model study.

14.4.4 Survey Data for Initial Conditions

The initial geometry of the prototype in the model area is needed to establish the starting conditions for the model study. Surveyed data must have sufficient resolution to establish hydraulic and sediment controls throughout the model area. These data are used for model design and construction. Tasks 2 and 3, cited above, pertain to model design and construction.

14.4.5 Survey Data for Final Conditions

A final geometry of the prototype in the model area is needed at the end of a sufficiently long period of time to verify the computational model.

14.4.6 Model Calibration

Model calibration is a process used in both physical and computational modeling. It is the process of demonstrating that the model is behaving like the prototype. Although the parameters being observed in a physical model are often more detailed than those in the computational model, the concept of demonstrating agreement with the prototype is the same in both. Tasks 4, 5, and 6 pertain to calibration of the computational model.

14.4.7 Base Test and Plan Tests

To minimize model biases, the usual procedure in physical model studies is to run a base test in which existing conditions are extended into the future. The project being studied is then inserted into the model and the test is rerun. The impact of the plan is measured by comparing the model results of the plan test with those from the base test. This same procedure is suggested for computational modeling. Tasks 7 through 10 pertain to running the model tests and analyzing the results.

14.4.8 Selection of Physical Model versus Computational Model

One of the most difficult tasks is deciding whether to use physical modeling or computational modeling in a sedimentation study. Dimensionality and scale are important technical parameters in the decision. Time and cost are important economic parameters. Each project has specific needs that must be factored into decisions as that project is moved through the formulation process. In the early planning phase, preliminary estimates of sedimentation are adequate most of the time. A key consideration is whether the impact of sediment on the project, or the impact of the project on the stream system morphology, could reverse decisions about project feasibility. In the engineering and design phase, sedimentation questions must be resolved in detail. It may be necessary to switch from computational models to physical models to achieve the necessary detail. These general concepts are discussed more specifically in the following examples.

For example, if the decision involves how to align and position a navigation channel within the river cross section, the problem needs a physical model. This is a three-dimensional hydrodynamic problem having a movable boundary. Computational modeling of such processes is evolving, but it is still largely experimental. Physical modeling is appropriate for such studies, provided the modeling approach recognizes what the significant sedimentation processes are and includes those processes in model calibration. The accuracy of physical modeling is affected by the scale distortion. In rigid-boundary hydraulics, the scale distortion may not be a serious problem, but in the case of erodible-boundary hydraulics,the scale distortion may not be totally overcome. Consequently, the selection of the modeling materials is very important.

On the other hand, if the decision requires prediction of maintenance dredging for a navigational channel located in the deepest part of the cross section, a one- or two-dimensional model is adequate. The one-dimensional model will not predict where the deepest part of the cross section will be, but it will predict the size of cross section that is required to transport the inflowing sediment load.

Decisions involving flow in bends are three-dimensional. Also, when the decision requires predicting the concentration of sediment that would be diverted through an outflow structure, either a physical model or a three-dimensional computational model is required. Neither one- nor two-dimensional computational models account for the secondary flows that control the distribution of the bed material load.

Flow through an expansion or contraction can usually be treated as a two-dimensional process. Sedimentation processes can be analyzed with a two-dimensional model or, if conveyance limits that approximate the rate of expansion can be established, the calculation can be made with a one-dimensional model. Examples are a dike field or a sediment trap.

If the decision involves flood elevations, either in a reservoir or in an open river site, reliable predictions can be made without knowing exactly where the deepest part of the channel will form in the cross section. Such a problem can be evaluated using a one-dimensional computational model.

The performance of hydraulic structures is a three-dimensional problem. Decisions involving sedimentation processes should probably be analyzed with a physical model at this point in time. However, the utility of three-dimensional computational models is advancing at such a rate that one should consider that approach in the model selection phase of a study.

14.5 DATA TYPES AND RESOLUTION

14.5.1 Introduction

Generally, data requirements are grouped into two types. One type helps the engineer to understand the historical behavior of the prototype. The other data group is the data that are needed to develop and operate the computational model. The data used to understand the behavior of the prototype are summarized in the next paragraph.

14.5.1.1 History of Prototype The *project area and study area boundaries* should be marked on a project map to delineate the area needing data. Add the lateral limits of the study area and the tributaries to this study area map. *Bed profiles* from historical surveys in the project area are extremely valuable for determining the historical trends that which the model must reconstitute. Use *aerial photographs* and aerial mosaics of the project area to identify historical trends in channel width, meander wavelength, rate of bank line movement, and land use in the basin. Analyze *stream gauge records* to determine the annual water yield to the project area and the water yield from it. Obtain annual peak discharge frequency curves for the project. These are useful for assessing the historical stability of hydraulic parameters such as width, depth, velocity, slope, and channel pattern. Analyze the stage-discharge curves in and around the project reach for trends. It is important to work with measured data. Do not regard the extrapolated portion of a rating curve as measured data. An example of this is shown in Fig. 14-2 where the measured flows are less than 52.39 m^3/s (1,850 cfs) and the project formulation flows range up to 453.07 m^3/s (16,000 cfs). Hydraulic data such as measured water surface profiles, velocities, and flood limits in the project reach are extremely valuable. Local action agencies, newspapers, and residents along the stream are sources of information when field measurements are not available.

14.5.1.2 Model Development Developing the one-dimensional representation of a three-dimensional open channel flow problem is an art. It requires one to visualize the three-dimensional flow lines in the actual problem and translate that image into a one-dimensional model. This step will often require several iterations to arrive at an acceptable model. The concept is one of developing representative data. A successful approach is to creep up on a solution by first running an approximation of the problem using simplified geometry and hydrology and the best sediment data available. Next, a fixed bed model of the actual geometry should be developed and run using three steady-state water discharges: low-flow, median-flow, and high-flow. Sediment should be added to this model and run with the same three discharges. Finally, the actual hydrology should be run to verify model calculations, to run the base test, and to run the plan tests.

14.5.2 Geometric Data

Mobile-bed water surface profile models calculate the water surface elevation and the bed surface elevation as they change over time. It is necessary to prescribe the starting geometry. This is done using cross sections for one-dimensional models. After that, computations will either aggrade or degrade the cross sections in response to mobile-bed theory. The cross sections never change locations.

14.5.2.1 Cross-Sectional Layout and Spacing It is customary to view and lay out cross sections from left to right, facing downstream. As in fixed-bed calculations, it is important to locate the cross sections so that they model the channel contractions and expansions.

It is particularly important in mobile boundary modeling to recognize where conveyance limits are needed. That is, if it is not physically possible for flow to expand laterally to the full width of the prototype, then determine how much of the cross section will convey flow and set conveyance limits in the model. Conveyance limits can result from internal embankments or from the lateral rate of expansion of a flow jet.

There is no theory for spacing cross sections. Some studies have required distances as short as a fraction of the river width. Other studies have allowed cross sections to be

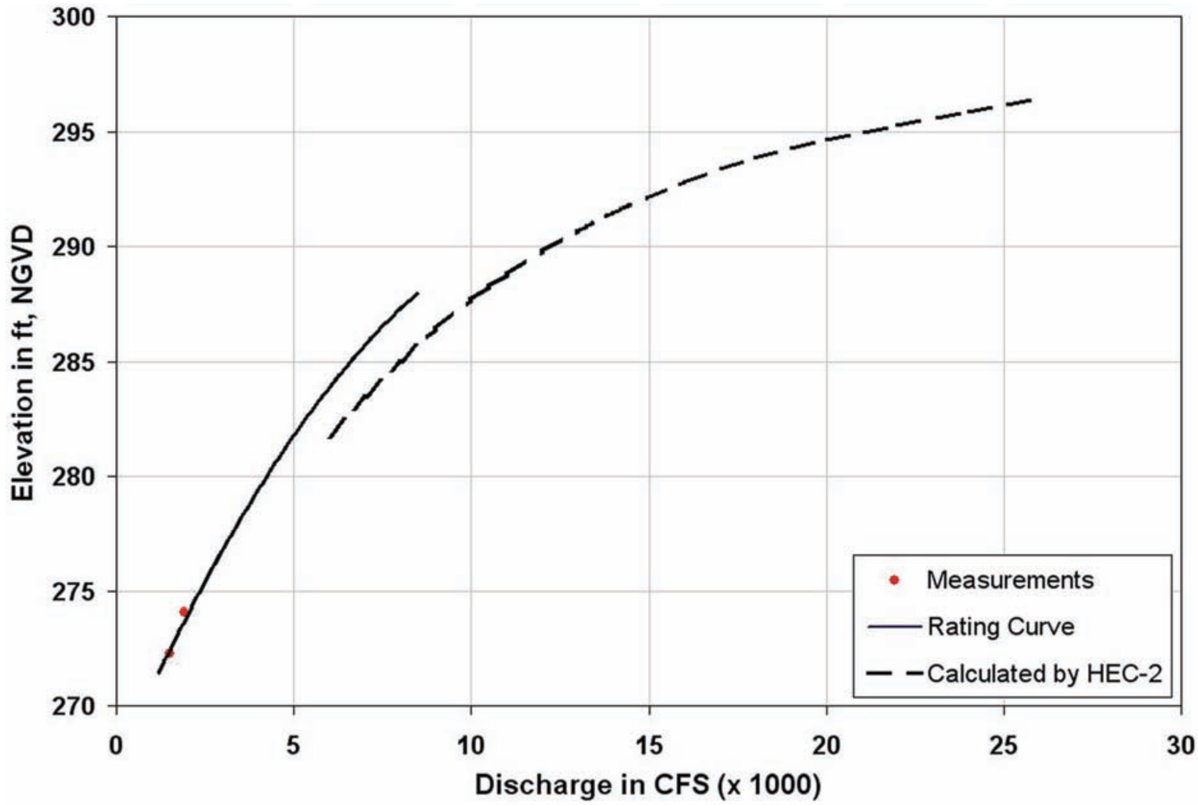

Fig. 14-2. Extrapolated discharge rating curve.

spaced from 16 to 32 km (10 to 20 mi) apart. The objective is to develop a model that will reconstitute the historical response of the streambed profile. The usual approach is to start with geometry that has already been developed for water surface profile calculations and transform it into geometry needed for sedimentation calculations.

There may be cases where cross sections must be eliminated from the data set to preserve model behavior. An example is a cross section in a bend or at a junction where the shape of the section is molded by three-dimensional hydraulic forces. It is not possible to reconstitute the shape of such sections with a one-dimensional hydraulic-sediment transport calculation. Those cases are the exception.

Document cross section locations for future reference using a layout map such as Fig. 14-3. River mile (or channel station) is suggested for the cross section identification number. It makes it much easier to use or modify an old data file if the cross sections are referenced by their position along the river rather than an arbitrary cross section number.

14.5.2.2 Hydraulic Roughness In a fixed-bed hydraulics study a range of *n*-values is typically chosen. The low end of that range provides velocities for riprap design, and the high end of the range provides the water-surface elevations for flood protection. In movable-bed studies such an approach is not satisfactory. The relationship between sediment transport and hydraulic roughness is too significant. Manning's *n*-values, which do not agree with that relationship, will either predict too much sand yield, too little sand yield, too much bed degradation, or too much bed aggradation. Analytical procedures that link *n*-values with hydraulic and sediment parameters are called bed roughness predictors. Models often provide bed roughness predictors. If so, modelers are encouraged to use these procedures in computational sedimentation. Brownlie (1983) developed a procedure for calculating the *n*-value in sand-bed streams. The procedure predicts the bed regime as well as the transition between upper and lower regimes.

Limerinos (1970) correlated field measurements to provide an equation for channel roughness in gravel-bed streams. Although not strictly a bed roughness equation, it was developed from data in which the channels were wide relative to their depth. Consequently, it can be used as a bed roughness predictor. The procedure does not predict bed regime.

Jarrett (1985) published a regression equation for composite channel roughness in Colorado streams. Although it may provide dependable results, it should be used as a composite channel roughness equation and not as a bed roughness equation.

Other methods for calculating *n*-values will surely become available as time passes, but the present bed roughness predictors are not substitutes for field measurements.

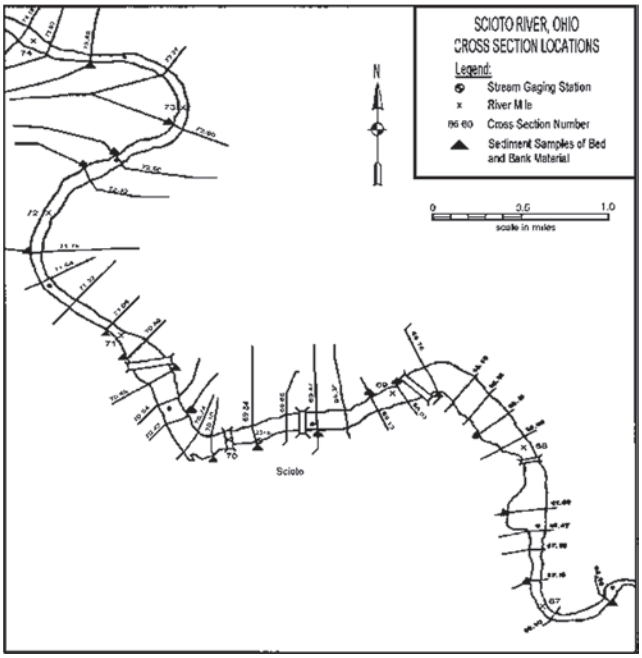

Fig. 14-3. Cross-sectional locations.

Measurements of water surface profiles and water discharges provide data that can be compared to model calculations. That is the most dependable technique for demonstrating hydraulic calibration. The second most dependable method is to reconstitute measured gauge records.

Regard data sets collected from a flood event as snapshots in time. When several of those snapshots are used along with the bed roughness predictors, the resulting calculation will account for variations in the hydraulic/sediment parameters during the entire runoff hydrograph. As a result, model performance will improve significantly.

In using bed roughness predictor equations, it is important to separate bed roughness from bank roughness. The equations do not include banks in the data set. Because bed roughness is completely tied to analytical equations, it cannot be used as the calibration parameter to match the calculated water surface elevation to historical flood profiles. That leaves bank roughness as the calibration parameter. There are no bank roughness equations, but the selection of n-values is not arbitrary. A systematic procedure for the selection of overbank n-values was developed by Arcement and Schneider (1989). They used Cowan's approach to associate n-value with surface grain, surface regularity, and surface vegetation materials. Their approach provides a systematic procedure for the selection of bank n-values, also. To apply this approach, document prototype conditions with photographs during the field reconnaissance.

The separation of bed from bank n-values requires that a composite channel n-value be calculated before the calculation of hydraulic parameters for the channel subsection. Compositing methods are described in Chow (1959).

Contraction and expansion losses, sometimes referred to as minor losses, are often included in sedimentation models. The information on contraction and expansion losses is more sparse than that for n-values. King and Brater (1963) give values of 0.5 and 1.0 for a sudden change in area accompanied by sharp corners and values of 0.05 and 0.10 for the best case. Design values of 0.10 and 0.20 are suggested. They cite Hinds (1928) as their reference. Values often cited by the U.S. Army Corps of Engineers are 0.1 and 0.3, contraction and expansion respectively, for gradual transitions. An acceptable alternative is to increase n-values to account for the effect of an irregular bank alignment.

14.5.3 Sediment Data

14.5.3.1 Size and Properties of Bed Sediment Reservoir The bed sediment reservoir is the space in the bed of the stream from which sediment can be eroded or onto which it can be deposited. This reservoir occupies the entire width of the channel, and in some cases the width of the overbank also. However, it might have zero depth, as in a concrete channel, at a rock outcrop, or over an erosion-resistant clay layer.

14.5.3.1.1 Gradation of the Bed Sediment Reservoir It is necessary to prescribe the gradation of sediment in the bed sediment reservoir. Section 14.5.3.1.2 gives insight into selecting sample locations for use in calculating an inflowing sand and gravel discharge rate. This section gives information to consider in selecting locations for sampling the bed. Studies need representative gradations for calculating sediment-transport capacity plus representative gradations for calculating streambed stability.

It is important to group bed samples according to geomorphological features and to select from the groups depending upon the purpose of the computation. For example, two samples were taken in the dry at 27 cross sections spaced over a 32 km (20-mi) reach of the creek in one study. One set of samples was near the water's edge and the other was from the point bar deposits about half the distance from the water's edge to the vegetated bank. These samples were considered as two populations, statistically, and sieved separately. The resulting gradations were plotted as bed gradation profiles, Fig. 14-4. The midbar samples were used to develop sediment transport rates for model calibration because they were taken from material deposited during high water. However, the results from the water-edge samples were used in the long-term simulations because the primary purpose of the study was to test for stream-bed erosion and these samples were coarser than the midbar population.

It is important to recognize that sampling the bed for a sedimentation study is an art. It is one of those activities that must result in providing representative data for a one-dimensional model. That means representative in the (x, y, z, t) coordinate system. It is common for one-dimensional models to develop a representative gradation for the bed surface at a cross section and to treat that bed as a homogeneous mixture

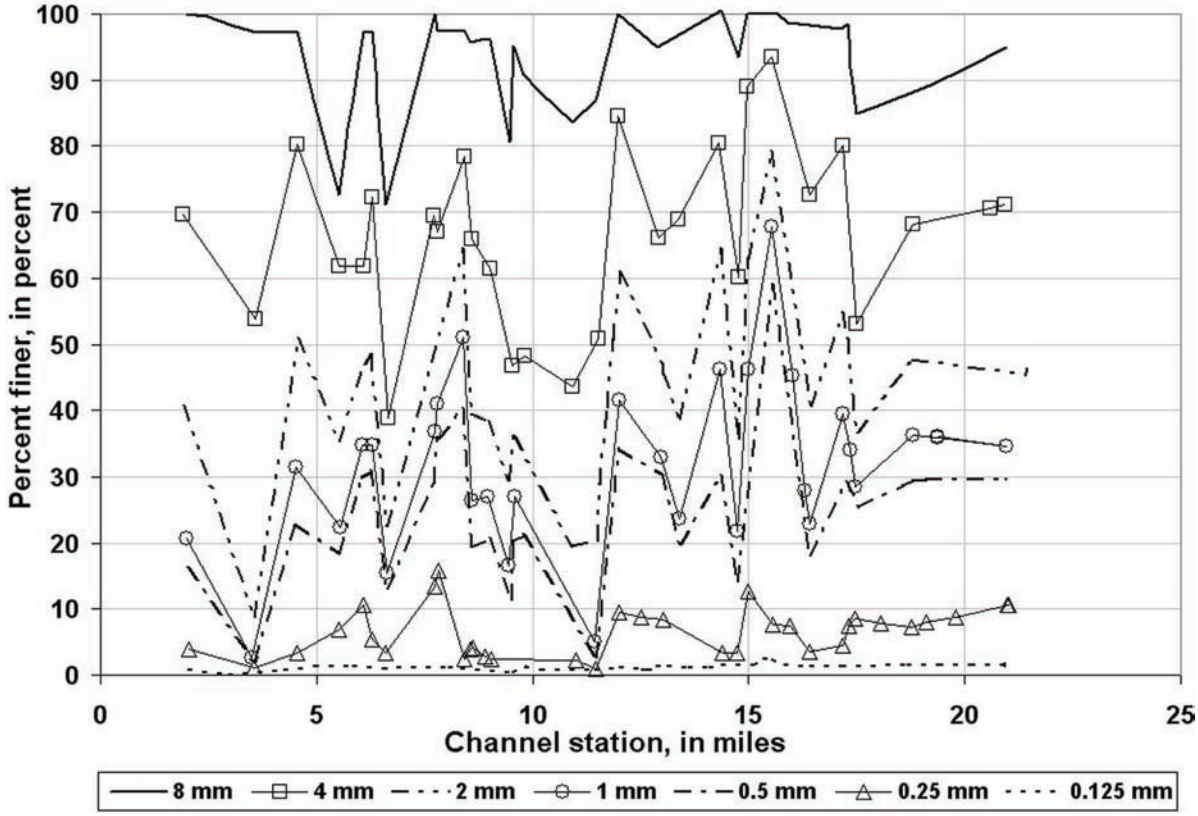

Fig. 14-4. Bed sediment profiles.

in the vertical. That is not adequate in cases where distinct layering is present in the bed-sediment reservoir. Bed layering is likely to be more of a problem on a coarse-bed stream than on a sand-bed stream. For example, bed layering is common at bridges because they are usually located at contractions. Pronounced bed layering can be created by a major flood runoff. In cases when layering is a problem, the model must be run in such a way as to approximate the effect of the change in bed gradation with respect to depth.

14.5.3.1.2 Sampling Concepts Sampling is largely a matter of experience. Sampling equipment and its operation have been standardized, but there are no standards for identifying the locations for collecting samples. The objective is to produce representative data. In this case "representative data" means a bed gradation curve that will produce the measured sediment concentrations in the flow field when used in concert with the representative channel hydraulics data.

In the absence of standards, the following general concepts are offered.

- The first choice of sample location is to sample in the dry. This allows the engineer to see the variability of the bed surface material and to collect samples that are representative of the active bed surface area.
- Use standard, calibrated sampling equipment and procedures. The Federal Interagency Sedimentation Project, located at the U.S. Army Engineer Waterways Experiment Station in Vicksburg, Miss., is responsible for standardizing sampling equipment.
- Organize sample sites into groups according to similar morphological features. Point or alternate bar samples probably provide the most representative gradations to use in calculating equilibrium sediment transport.
- In sand-bed streams, sample the depth of the active layer. If that is difficult to ascertain, favor about a 50 mm (2-in) depth, because that is the zone covered by the BM54 sampler.
- In gravel-bed streams, sample the surface layer and about 1 ft beneath the surface. Analyze the samples separately and composite the resulting gradations. Note the maximum size present on the bed surface, and include the larger sizes in bed gradation curves, because they will be necessary for bed stability calculations. Large sample volumes are recommended to avoid bias.
- Collect a sufficient number of bed and bank samples to provide a representative bed gradation for equilibrium sediment transport theory. The samples can be spatially weighted provided the distribution of sediment in the flow field is uniform over that same space. Otherwise, sample weighting should be adjusted in favor of the most active portion of the cross section for transporting bed material.

The sample locations cited for sediment transport calculations often miss the coarsest sizes in the stream bed. Therefore, also sample the stream bed in the geomorphological locations where coarser sediments are known to collect, such as deeper parts of the cross sections and the crossings. These samples will be important in bed profile stability calculations.

14.5.3.1.3 Variability of Samples There is often more variability from one side of the channel, or the point bar, to the other side than there is along the length of the sampled reach. Take a sufficient number of samples to be sure that this variability has been represented. A test of sufficiency is when the addition of one more sample does not change the composite bed gradation curve for the reach by a significant amount.

14.5.3.1.4 Test for Sufficiency The final test for sufficiency is to run the sampled gradations in the computational model using water discharges from the hydrograph prior to the time when samples were collected. The first event will entrain a high concentration from the new disturbed bed; subsequent iterations with that same water discharge should produce bed material load concentrations that match prototype measurements.

14.5.3.2 Size and Concentration of Inflowing Sediment Load

14.5.3.2.1 Inflowing Sediment Concentrations Occasionally suspended sediment concentration measurements, expressed as milligrams per liter, are available. These are usually plotted versus water discharge, Fig. 14-5. As in most cases, the concentrations in Fig. 14-5 show a great deal of scatter; however, such graphs are useful in developing or extrapolating the inflowing sediment data. It is desirable in most cases to develop the best estimate of the inflowing sediment concentration curve using the concentration graphs and then convert those values into sediment discharges in tons/day. That result is a sediment discharge rating curve, Fig. 14-6. The scatter is reduced from Fig. 14-5 but that is not because the correlation is better. It is because water discharge is being plotted on both axes. A scatter of about 1 log cycle is common in such graphs.

14.5.3.2.2 Grain Size Classes The total sediment discharge should then be partitioned into grain size classes. Table 14-1 shows the procedure that was developed for the Clearwater River at Lewiston, Id. Figure 14-7 is a graph of the sediment discharge by grain size class.

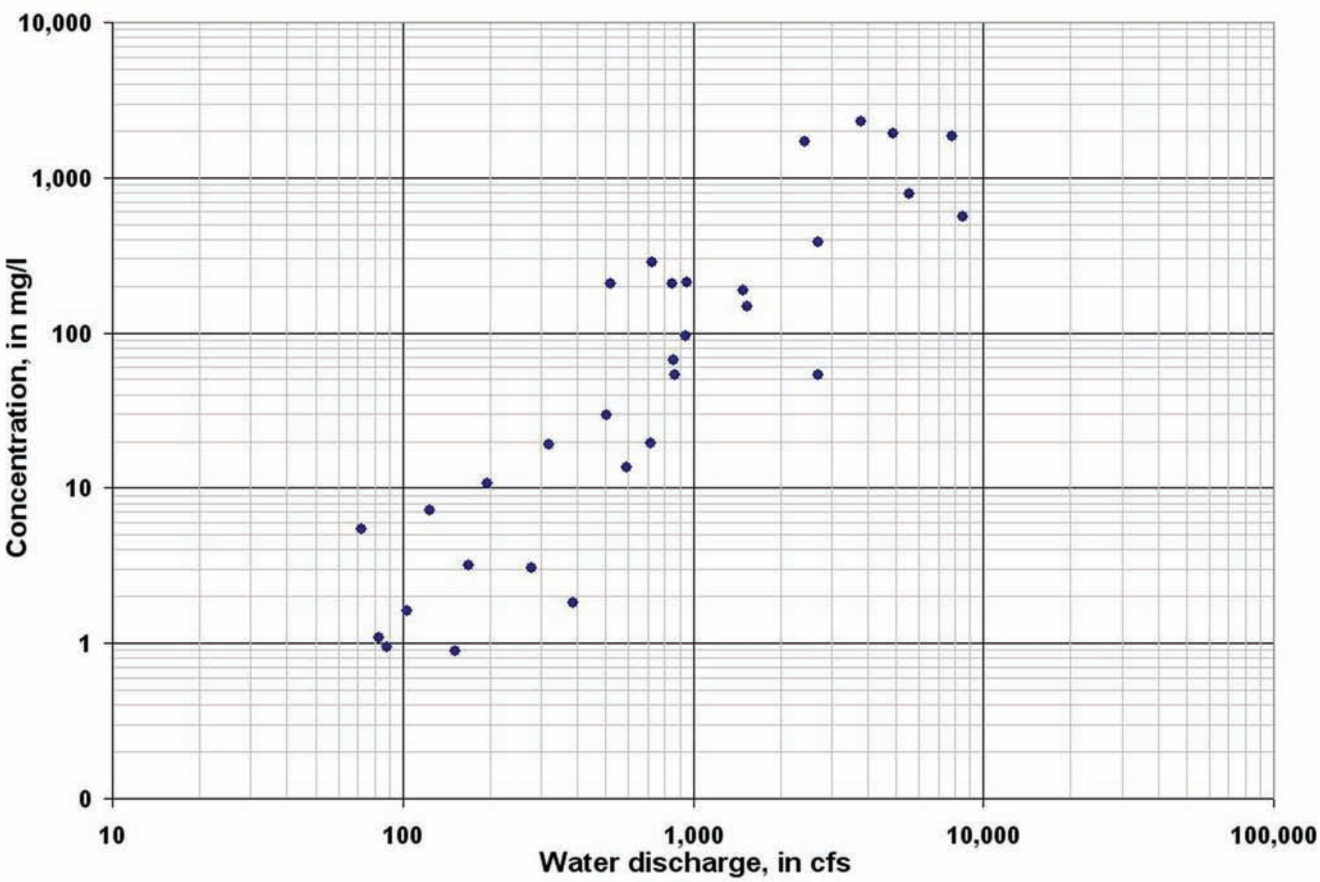

Fig. 14-5. Sediment concentration measurements.

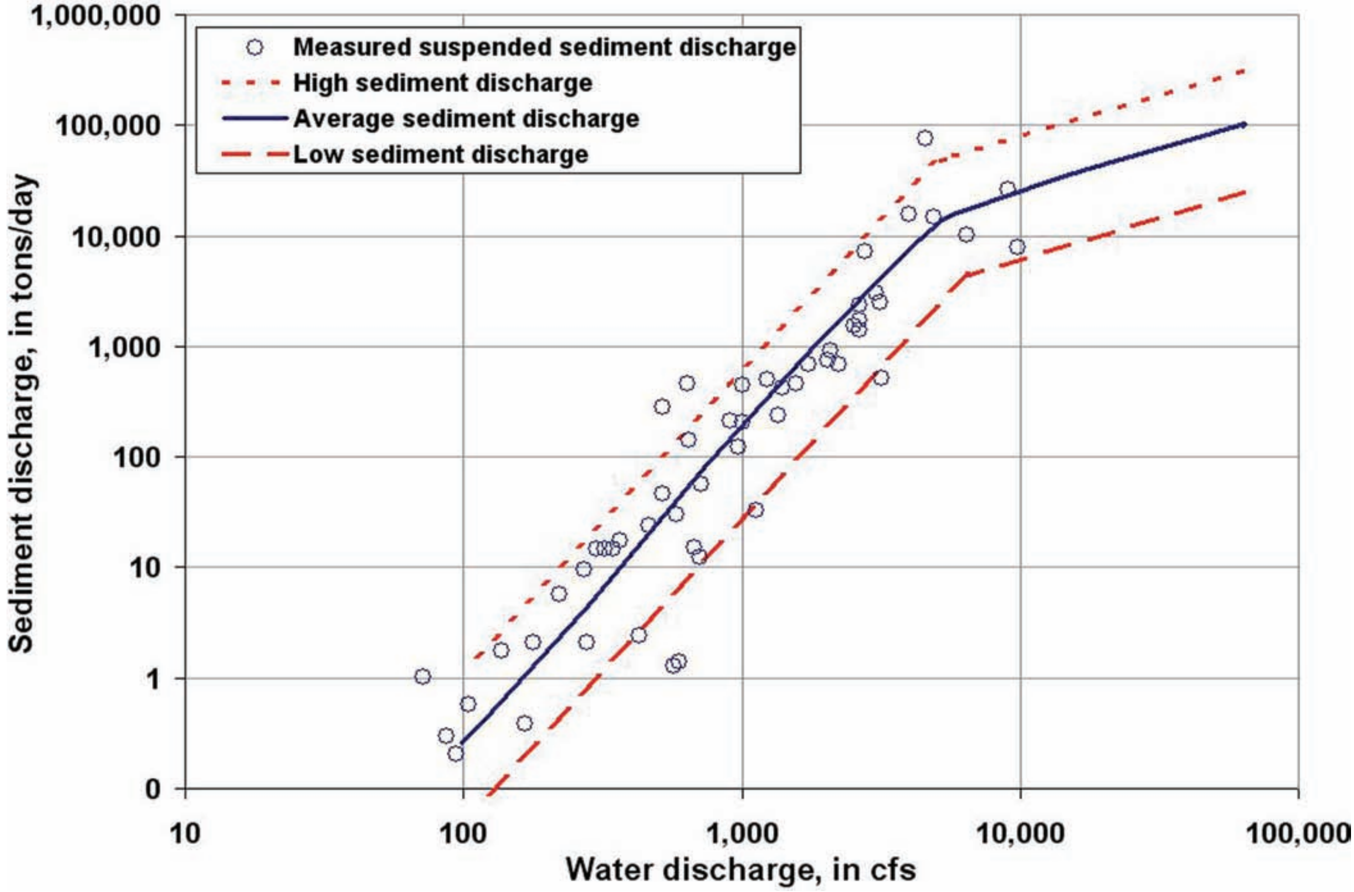

Fig. 14-6. Sediment discharge rating curve.

14.5.3.2.3 Calculating Sediment Inflow with Transport Theory When no suspended sediment measurements are available, the inflowing sediment boundary condition must be calculated with sediment transport theory. There is no theory for calculating the wash load concentration from sediment samples of the stream bed. This calculation can be made only for sand- and gravel-bed sediment using equilibrium sediment transport functions. The calculation should be made by particle size for the full range of water discharges in the study hydrograph.

Select the reach of channel very carefully for this calculation. The first choice is a reach approaching the project where the slope, velocity, width, and depth at one representative of the historical hydraulics. This reach should have a history of conveying the inflowing sediment load without aggradation or degradation. The selected reach should also have a bed surface that is in equilibrium with the sand and gravel discharge being transported by the flow. Finally, the selected reach should have locations where the bed gradation can be measured using standard procedures.

The second choice for calculating the inflowing sediment concentration is a reach within the project area. A location near the upstream end of the project is desirable. It is important that the selected location be a stable reach and have a history of conveying the inflowing sediment discharge without appreciable aggradation or degradation.

An example of an inappropriate location for calculating the inflowing sediment load is a reach within the project where dredging is performed.

Einstein made the following suggestions for choosing a river reach to apply his bed-load function. His suggestions are also appropriate for other equilibrium sediment-transport functions.

> In practical calculations of the bed-load function for a particular river reach, the length of the reach must be sufficient to permit adequate definition of the over-all slope of the channel. The channel itself should be sufficiently uniform in shape, sediment composition, slope and outside effects such as vegetation on the banks and

Table 14-1 Distribution of Sediment Load by Grain Size Class (Clearwater River at Lewiston, Idaho)

Grain size[a] diameter mm (1)	Classification (2)	Percent of total bed load[b] (3)	Bed load ton/day (4)	Percent of total suspended load[c] (5)	Suspended-load ton/day (6)	Total load Cols. (4) + (6) ton/day (7)
< 0.0625	silt & clay	0.04	1	54	216,000	216,001
0.0625–0.125	VFS	0.10	2	10	40,000	40,002
0.125–0.250	FS	2.75	52	13	52,000	52,052
0.250–0.500	MS	16.15	307	19	76,000	76,307
0.500–1.000	CS	13.28	252	4	16,000	16,252
1.000–2.000	VCS	1.19	23			23
2–4	VFG	1.00	19			19
4–8	FG	1.41	27			27
8–16	MG	2.34	44			44
16–32	CG	6.33	120			120
32–64	VCG	23.38	444			444
> 64	cobbles & larger	32.03	609			609
Total		100.00	1,900	100.00	400,000	401,900

[a]Values were read from the sediment load curve, 1972.74 measurements. (Total bed load, tons/day 1900.)

[b]These values were calculated by analyzing measured hydraulic parameters and measured bed loads using the computer program "Total River Sand Discharge and Detailed Distribution" by F. B. Toffaleti. (Total suspended load, tons/day 400,000.)

[c]These are representative values determined graphically by plotting the results of sieve analyses and developing a single percentage finer curve from all samples analyzed. (Total sediment load 401,900.)

Water discharge, cfs 200,000.

> overbanks, that it can be treated as a uniform channel characterized by an over-all slope and by an average representative cross section. (Einstein 1950, p. 45)

Having located a suitable reach, measure a sufficient number of cross sections to establish the width, depth, and slope of the channel in that reach. Composite the measured cross sections into a single cross section that is representative of the river just upstream from the project. Again, quoting from Einstein on the description of a river reach:

> One problem is that of determining how a number of cross sections can best be averaged. As the river reach is to be treated as a uniform channel with constant cross section and slope, in which only uniform flows are studied, a representative or average slope must be found, together with the average section. If a sufficiently long and regular profile exists for the river under consideration, the general slope of the reach should be taken from it. In the absence of such a profile, the slope must be derived from the cross sections themselves. Under all conditions, the cross sections must be tied together by a traverse which gives their relative elevations and the distance between them along the stream axis. Then the wetted perimeter and the wetted area are calculated for various water surface elevations. These are plotted in terms of the water surface elevation for each cross section.
>
> It is fairly common usage to construct the stream profile from the lowest points of the sections. This procedure is satisfactory for a long profile. If the reach is short, however, the use of a low-water surface is more satisfactory as the influence of insignificant local scour-holes is excluded. If such a low-water profile is not recorded when the sections are surveyed, a profile found from the area-curves may be substituted. A characteristic low-water discharge may be selected for the streams. The average velocity for such a flow can be estimated roughly. By division of the two one may find the corresponding low-water area of the cross sections. If the water-surface points which give this area at the different sections are connected, an approximate low-water surface is defined which represents a profile that is more regular and more representative than the profile of the low points of the bed.

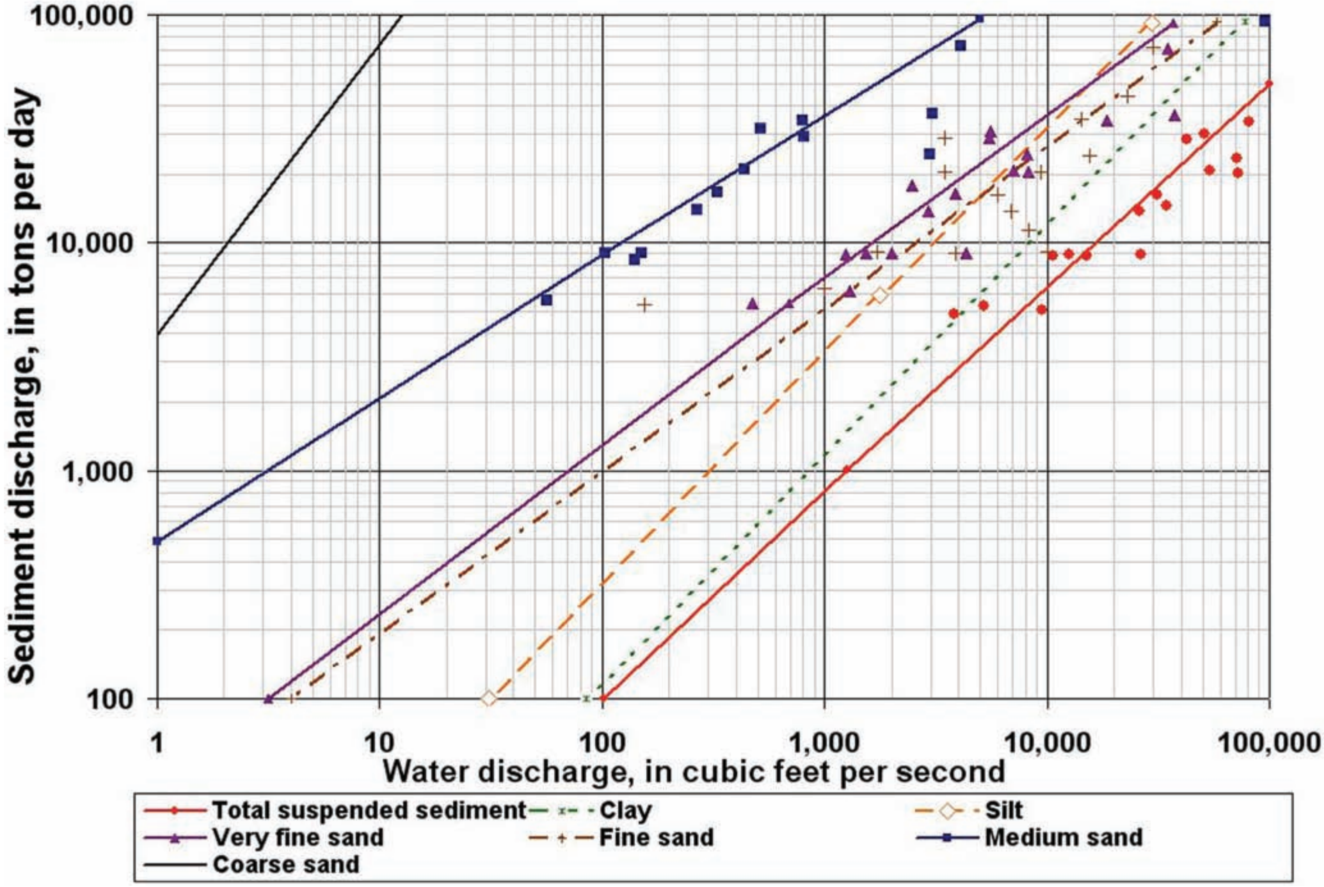

Fig. 14-7. Sediment discharge rating curve by particle size class.

> After the representative slope is selected, by fitting a straight line through the profile points, this slope may be used in averaging the cross sections. This can be done by sliding all the sections along this average slope line together into, for instance, the lowest section. (Einstein 1950, pp. 45–46)

Einstein applied this procedure to Big Sandy Creek near Greenwood, Miss. Eleven cross sections were selected. They were spaced roughly at three times the channel width for a total distance of 5181.6 m (17,000 ft) along the channel. Both bend and crossing sections were included. Using a low-flow water discharge and low-flow water velocity, the low-flow wetted area was calculated to be 4.65 m^2 (50 sq ft). When the elevation for 4.65m^2 (50 sq ft) was read from the individual cross section area-elevation curves and plotted versus channel station, the least-squares regression line through the points provided the channel slope. The slope was 0.00105 ft/ft in this example.

It is also important to develop a representative sample of the bed gradation for the equilibrium sediment transport analysis. In the example application of his bed load function to Big Sandy Creek Einstein wrote:

> The grain-size composition of the bed is determined by sampling. A bed which appears to be very uniform, such as that of Big Sand Creek, may be described by three to five samples. Each of the four samples listed . . . was a composite of three or four cores, taken in the same cross section at evenly spaced points over the total width of the channel. The individual samples were obtained by means of an auger or a pipe-sampler and were taken down to a depth of about 2 feet, the estimated depth of scour or active bed movement (Einstein 1950, pp. 45–46).

Treat the calculation of the inflowing sediment load as if it were a calculation to verify a sediment transport function. That is, locate the sample reach so that the point where the concentration is needed is at the downstream end. To the maximum extent possible, collect samples of the bed sediment over the same reach covered by the cross sections.

However, avoid using average geometry and bed gradation over extended lengths of a river when calculating the equilibrium sediment transport entering a study area. Instead, focus attention on the river channel approaching the point of interest. Use the following rules of thumb:

- Sample the bed surface gradation at and upstream from the gauge.
- Sample for a distance of 50 to 100 times the depth of flow. The idea is to provide a sufficient distance to allow the vertical distribution of velocity and sediment concentration in the water column to approach equilibrium conditions as depicted by hydraulic forces that are reasonably close the average over the reach.

14.5.3.2.4 Typical Bed Gradation on a Point Bar Figure 14-8 illustrates a typical bed surface gradation pattern on a point bar. Use such information to determine where to sample for a sediment transport calculation. Note that the typical grain sizes found on the bar surface form a pattern from coarse to fine, but there is no one location that always captures the precise distribution that will represent the entire range of processes in the prototype. The bed gradation controls the sediment discharge calculation. For example, true, multiple grain size transport functions like Laursen and Toffaleti show that the rate of transport increases exponentially as the grain size decreases, Fig. 14-9. There is no simple rule for locating samples. The general rule is "always seek representative samples." That is, select sampling locations very carefully and avoid anomalies that would bias either the calculated sediment discharge or the calculated bed stability against erosion.

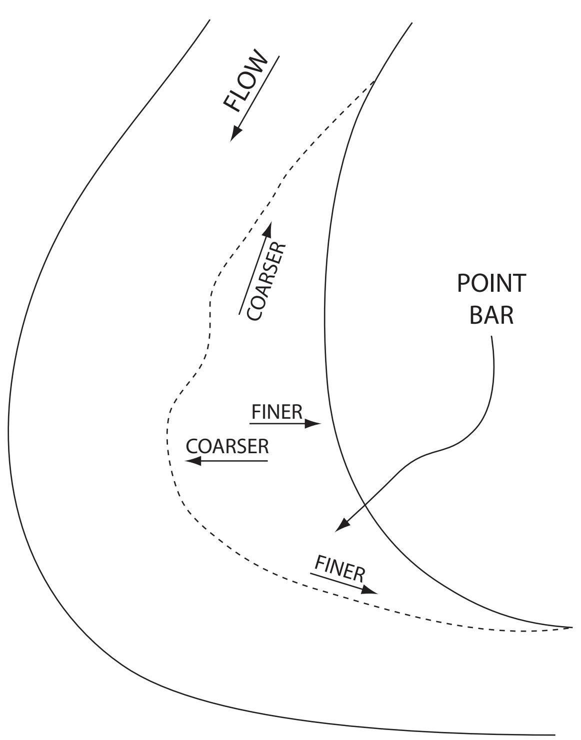

Fig. 14-8. Gradation pattern on a bar.

14.5.3.2.5 Sediment Inflow from Tributaries The sediment inflow from tributaries is usually more difficult to establish than it is for the main stem because there are usually fever data on the tributary. The recourse is to use the site reconnaissance to assess each tributary. For example, look for a delta at the mouths of the tributaries. Look for channel bed scour or deposition along the lower end of the tributary. Look for drop structures or other controls that would aid in stabilizing a tributary. Look for significant deposits if the tributaries have concrete linings. These observations guide the investigation of tributary sediment discharges.

14.5.4 Hydrologic Data

14.5.4.1 Main Stem Water Inflows Although a design water discharge is of interest, a single value is not adequate for a movable-bed computational model study. Simulating the change in channel behavior as the result of a flood requires the analysis of complete hydrographs. Consequently, the water discharge hydrograph must be developed. This step can involve manipulations of measured flows, or it can require a calculation of the runoff hydrograph.

Historical flows are needed for model calibration/verification because the model must reconstitute the historical behavior of the river, but future flows are needed to forecast the future stream-bed profile.

The length of the hydrograph period is important. Trends of a tenth of a foot per year becomes significant during a 50- or 100-year project life. On the other hand, a long-period hydrograph becomes a computation burden. Usually the bed profile changes are sufficiently slow to allow some aggregation of the forces involved. Therefore, aggregating the energy of a varying hydrograph into extended numbers of days is acceptable in most cases. For example, Fig. 14-10 shows the histogram for a year of mean daily flows.

In cases where measured flows are not available, the computational model still provides the framework for analyzing sedimentation in the project. Calculate hypothetical runoff hydrographs.

14.5.4.2 Tributaries Tributaries are a lateral boundary condition. They should be located, identified, and grouped as required to define the increase in water discharge as the drainage area increases along the project reach. The tributaries should be shown on the cross section location figure. Keep in mind that a 10% increase in water discharge will normally produce more than a 10% increase in bed material transport capacity. The transport relationship is nonlinear.

Often the tributaries are not gauged, and it is necessary to develop tributary inflows by analytical means. Table 14-2 illustrates such an approach in which six tributaries were grouped into three inflow points and their water discharges were calculated from the main stem discharge gage record.

Table 14-2 shows the 2-year flood peak at four locations along the project channel. These flood peaks were calculated by a rainfall-runoff-routing program from subbasins. The peaks were then subtracted to determine the three lumped tributary inflow values, column 3. The ratio of Q-main at River Mile 7.903, 13,648/17,692, was calculated to determine f-main, column 4. The tributary discharge, 4,044, was divided by Q-main, 17,692, to determine the f-trib coefficient, 0.2296, in column 5. These tributary discharge coefficients were then applied to all flows in the long-term hydrograph of the study to calculate tributary inflows.

14.5.4.3 Tailwater Elevation The tailwater elevation specifies the water surface elevation at the downstream end of the project. It is referred to as a tailwater elevation (or base level) because it establishes the energy gradeline at the downstream end of the model. It can be a stage-discharge rating curve, such as Fig. 14-2, or it can be a stage hydrograph.

When a backwater condition exists, such as at the mouth of a tributary or in a reservoir, use a stage hydrograph as the boundary condition. Be sure it covers the same period of time as the inflow hydrographs.

14.5.4.4 Water Temperature The final boundary condition is the temperature of the inflowing water sediment mixture. Develop representative values by month or season if measurements are locking.

14.5.4.5 Boundary Condition Changes over Time The historical water inflows, sediment concentrations, particle sizes, tailwater elevations, and water temperatures may change in the future. That possibility should be evaluated for each project and the appropriate modifications made to the water sediment boundary condition values.

14.5.5 Operating Rules

The usual procedure for controlling the water surface elevation in a large reservoir is to follow a prescribed rule curve. Similarly, low-head dams may be operated to control the water surface at a gauge several miles upstream from the dam. Diversion structures are designed to pass a prescribed water discharge. In these cases the depth of flow is being controlled by manipulation of hydraulic structures and not by friction losses. This creates a modeling requirement called “operating rules.” An operating rule is an internal boundary condition that will control the model.

14.5.6 Data Sources

14.5.6.1 General The data that will be needed to develop the model may come from office files, from other federal agencies, from state or local agencies, and from the team making the field reconnaissance of the project site.

14.5.6.2 U.S. Geological Survey (USGS) USGS topographic maps and mean daily discharges are used routinely in hydraulics and hydrology studies and are common

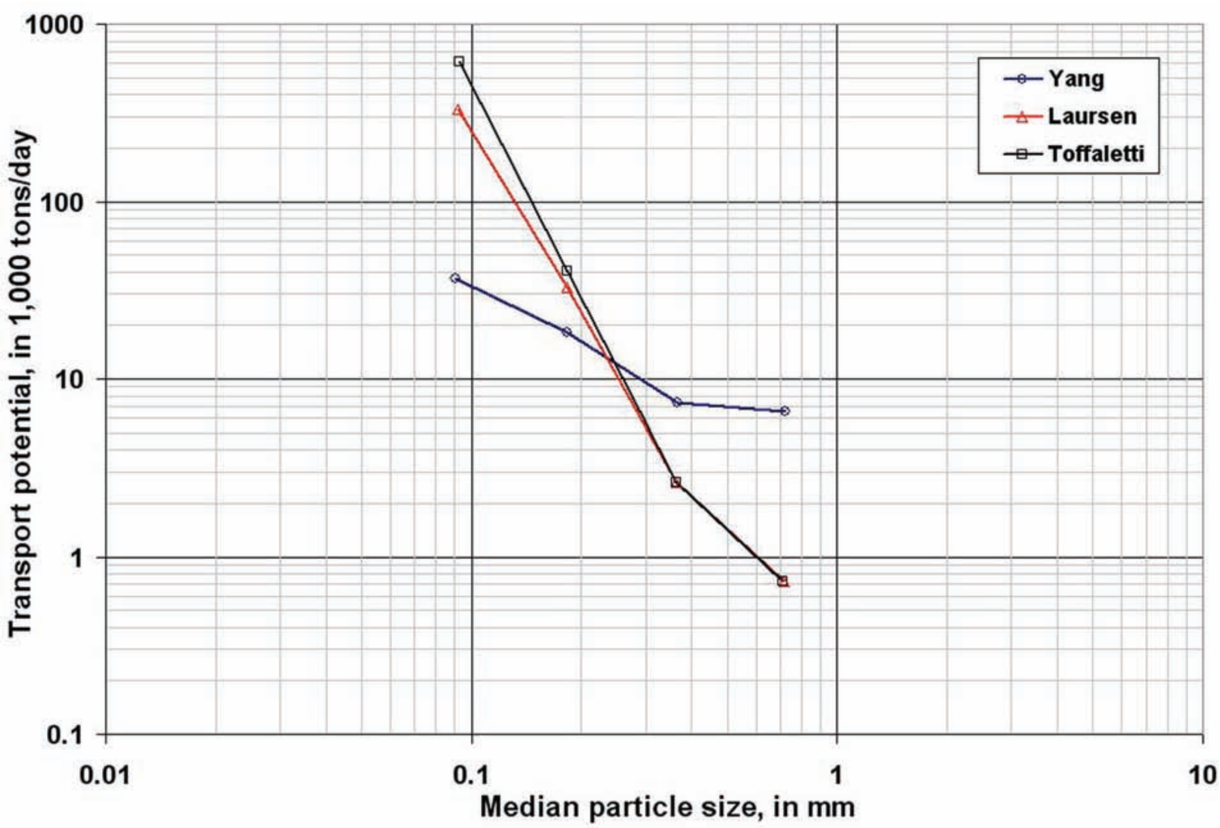

Fig. 14-9. Sensitivity of sediment transport potential to grain size.

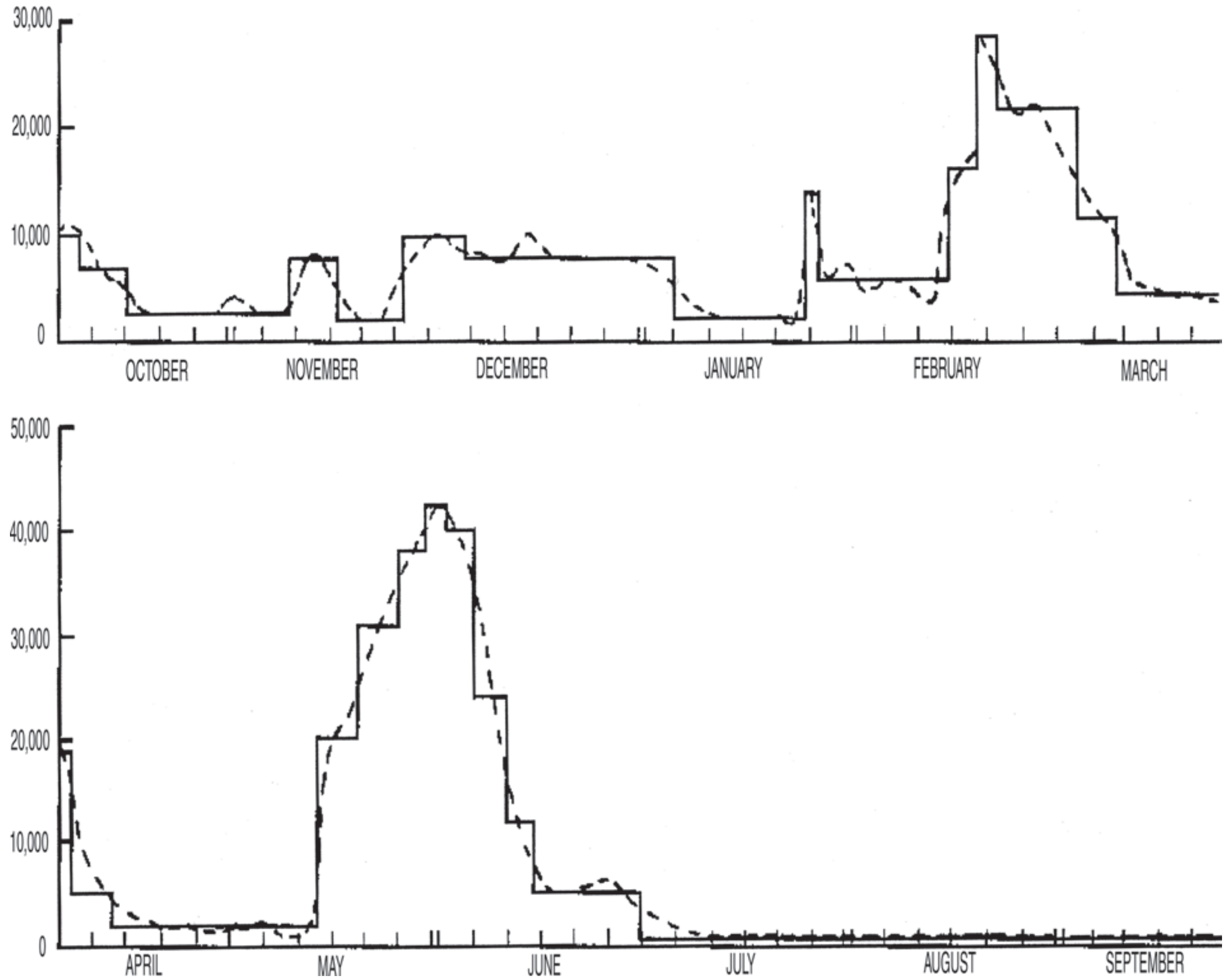

Fig. 14-10. Histogram for a year of mean daily

Table 14-2 Distribution of Runoff by Tributary, 2-Year Flood Peak

			Discharge coefficients	
River mile	Q-main cfs	Q-trib cfs	f-main	f-trib
0–7.903	17,692			
		4,044	0.7714	0.2286
7.903–11.942	13,648			
		4,777	0.6500	0.2699
11.942–17.346	8,871			
		1,696	0.8088	0.0959
17.346–21.005	7,175			

data sources for sediment studies, also. However, mean daily flows are often not adequate for sediment studies, and data for intervals less than 1 day or stage-hydrographs for specific events can be obtained, through strip-chart stage recordings, by special request. It may be preferable to use USGS discharge-duration tables rather developing such in house, and these are available from the state office of the USGS. Water quality data include suspended sediment concentrations and grain size distributions. Published daily maximum and minimum sediment discharges for the year and for the period of record are available, as are periodic measurements of particle size gradations for bed sediments.

14.5.6.3 National Weather Service (NWS) There are cases where mean daily runoff can be calculated directly from rainfall records and expressed as a flow-duration curve without detailed hydrologic routing. In those cases use the rainfall data published monthly by the National Weather Service for each state. Hourly and 1-day interval rainfall data, depending on the station, are readily accessible. Shorter interval or period-of-record rainfall data would require contact with the NWS National Climatic Center at Asheville, N.C.

14.5.6.4 Soil Conservation Service (SCS) The local SCS office is a good point of contact for historic and future estimates of land use, land surface erosion, and sediment yield. They have soil maps, ground cover maps, and aerial photos that can be used as an aid in estimating sediment yield. Input data for the universal soil loss equation one available for much of the United States. The SCS also updates reservoir sedimentation reports for hundreds of reservoirs throughout the country every 5 years, providing a valuable source of measured sediment data.

14.5.6.5 Agricultural Stabilization and Conservation Service (ASCS) This agency of the Department of Agriculture accumulates aerial photographs of crop lands for allotment purposes. However, those photographs will include the streams crossing those lands and are extremely valuable for establishing historical channel behavior, because overflights are made periodically.

14.5.6.6 U.S. Army Corps of Engineers Because the Corps gathers discharge data for operating projects and for those being studied for possible construction, considerable data from the study area may already exist. The Corps has acquired considerable survey data, aerial and ground photography, and channel cross sections in connection with flood plain information studies. Corps laboratories have expertise and methods to assist in development of digital models.

14.5.6.7 State Agencies A number of states have climatologic, hydrologic, and sediment data collection programs. Topographic data, drainage areas, stream lengths, slopes, ground covers, travel times, etc. are often available.

14.5.6.8 Local Agencies, Businesses and Residents Land use planning data are normally obtained through local planning agencies. Cross section and topographic mapping data are often available. Local agencies and local residents have some of the most valuable information to the engineer in their verbal and photographic descriptions of changes in the area over time, of channel changes from large flood events, of caving banks, of significant land use changes and when these changes occurred, of channel clearing/dredging operations, and other information. Newspapers and those who use the rivers and streams for their livelihood are valuable sources of data.

14.6 MODEL CALIBRATION

Computational studies fall into two general categories: (1) computational model studies and (2) computational analysis studies. Computational model studies are applications for which the model has been calibrated according to the formal procedures described in this chapter. Often the available field data are not sufficient to permit a formal calibration, but computational modeling is still the best method for analyzing the problem. In these cases model tests are devised so that engineering judgement can be used to assess the credibility of the calculated results. The resulting studies are called computational analysis studies.

Historically, there has not been a formal procedure for the calibration of computational models. Consequently, there has not been a formal definition of the word "calibration" It has been used to describe the initial work of adjusting a computational model until the calculated results matched whatever field data were available in the project area. This chapter proposes a formal calibration procedure. The word "calibration" will be reserved for those computational studies that have adequate field data to permit the implementation of the calibration procedure. Many studies will not have sufficient field data to calibrate models under this definition. Such studies will be beneficial because they will include the full computational capability of the model. Consequently, they will be called computational analysis studies rather than computational model studies.

Some believe that the word calibration should be reserved for instrumentation and have introduced the term "circumstantiation" to describe the process of demonstrating agreement between model and prototype. Their argument is based on the fact that model parameters are often adjusted using circumstantial evidence, whereas calibration is the result of scientific measurements. This chapter will continue the use of the term calibration.

14.6.1 Definitions

14.6.1.1 Calibration Calibration is the process of arriving at roughness coefficients, a sediment transport function, model parameters, and representative data on the study area that will allow the model to calculate values that agree with values measured in the prototype. For a one-dimensional model, the representative data are developed by transforming the three-dimensional (x, y, z) space of the prototype into a one-dimensional digital representation. Variables of interest are water surface elevations, velocities, depths, widths, hydraulic roughness values, the concentrations of sediment in the water column, the gradation of the sediment in the water column, the gradation of sediment in the bed, the delivery of sediment along the study area, and aggradation/degradation of the channel.

A model cannot be calibrated using a data set that was also used by that model to calculate some of its boundary conditions.

The field data used for model calibration should contain measured values of parameters similar to those for which the model is being calibrated. For example, if the purpose of the model study is to calculate aggradation or degradation of the channel profile, the calibration data should contain field data that include some historical information on the channel profile. The model is expected to reconstitute those measurements when it is provided with the boundary conditions that existed when the measurements were made. On the other hand, if the purpose of the model study is to evaluate the change in a water surface elevation as sedimentation processes change the channel over time, the calibration criteria should contain some field measurements of historical water surface elevations in the study area.

The examples in this section are given to illustrate the concept of model calibration. They are only a starting point for deciding what is required for a model to be calibrated. They are necessary, but they may not be sufficient to provide complete calibration for the general case. In principle, the requirements for calibration are based on the questions to be answered by the model results, and it is the responsibility of the engineer to develop and justify the steps that are required.

14.6.1.2 Verification A calibrated model is not necessarily a verified model. Verification is sometimes called a split record test. It demonstrates that the calibrated model will match the prototype during a period of time that is not used in calibration. The calibration parameters cannot be adjusted during model verification. Only the boundary conditions can be changed to those for the verification period.

14.6.1.3 Computational Modeling Computational modeling is the formal process of assembling data that provide the geometry of a study reach at two points in time and that provide a continuous record of the inflowing water discharge, the inflowing sediment load, and the downstream stages between those two points in time. Geometric data are provided by hydrographic/topographic surveys. The initial model geometry is developed from the first survey. The model is then run using the recorded hydrological and sedimentary boundary conditions, and the calculated results at the end of the simulation are compared with prototype values in the second survey.

14.6.1.4 Computational Analysis An alternative to computational modeling is computational analysis. Computational analysis is the application of a computational model to a problem in which model calibration is not possible. Many sedimentation studies are made where there are not adequate prototype data to calibrate the model. Perhaps there is only one survey of the prototype. Perhaps there are two surveys, but boundary condition data are not available during the time period between the two. Perhaps the river is so highly disturbed that computational modeling is not possible. Whatever the case, computational analysis allows the engineer to use the latest technology in mobile boundary computations in decision making. Such studies are very useful because they recognize that we live in a movable-boundary world. How the study area responds to the systematic application of hydrodynamic and sedimentation theories illustrates how sensitive the area is to sedimentation processes. Often one can gain sufficient understanding to predict how reliable a plan will be by comparing the calculated results from the plan test with those from the base test.

In some cases, a computational analysis will demonstrate that the questions being asked are sufficiently sensitive to sedimentation processes so that prototype data must be collected before proceeding with a design.

14.6.2 Fixed-Bed, Steady-State Hydraulic Calibration

Model calibration is approached in phases. The first phase is to reconstitute the water surface profile that was measured at the time the hydrographic survey was made. When the channel was dry during that survey, choose a low flow from the testing hydrograph. The purpose is to check the model geometry for consistency in width, depth, and slope and to check n-values. The mean error between calculated and measured water surface elevations should normally be within ± 153 mm (0.5 ft), or 10% of the flow depth, whichever is smaller. Because most surveys are made during low-flow-periods, this first test will be the low-flow test.

The next fixed-bed test should use a water discharge that is approximately the channel-forming discharge. In any case, the flow should not be out of banks for this test. The purpose is to check the geometry for consistency with regime concepts in channel widths and depths and to confirm channel n-values while all flow is still confined to the channel cross section. This n-value may be different from that developed for the low-flow condition. If so, the n-value should vary in the vertical later when hydrographs are included in the calibration process.

The final fixed-bed test should use the maximum water discharge in the testing hydrographs. At this point the calibration of the movable-bed model becomes more exacting than is usually performed for a fixed-bed calculation. That is, not only must the water surface elevation match known elevations but also the flow distribution between the channel and the overbanks must match the true prototype values. The only parameters available to achieve such a match are geometry and n-values. Geometry is usually more reliable than n-values, but it has been reduced to a one-dimensional approximation of the prototype. Therefore, ascertain that the cross sections and reach lengths are the best representation of the flow conditions in the prototype. Additional adjustments in both water surface elevation and the percentage of flow that is conveyed in the channel are made with the *n*-values. The process is neither random nor arbitrary. The resulting values must pass the test of "reasonableness." Keep the process of selecting *n*-values systematic. Base the estimates on physical conditions by using a procedure such as that of Arcement and Schneider (1989).

14.6.3 Fixed-Bed, Unsteady-State Hydraulic Calibration

When the hydraulic calculations include the unsteady-flow terms, as in the Saint-Venant equations, the calibration of the routing model involves storage in the geometric model. This is an important adjustment because the hydraulic results drive the sedimentation calculations. Calibration of an unsteady-flow model is such a formidable process that the reader is referred to unsteady-flow modeling procedures. The process will not be presented here. The same cautions apply as presented earlier for steady-state, fixed-bed calibration.

14.6.4 Movable-Bed, Steady-State Hydraulic/Sediment Calibration

Start with a steady-state discharge that approximates the channel-forming discharge. In the more arid regions where streams are ephemeral the 10-year flood peak is usually a reasonable value for these calculations. Elsewhere, a discharge about equal to the 2-year flood peak is usually a reasonable value. In a regime channel this calculation can be made with the channel full discharge. Ascertain that the model is producing acceptable hydraulic results by not only reconstituting the water surface profile but also plotting the water velocity, depth, width, and slope profiles. This test will often reveal width increases between cross sections that are greater than the expansion rate of the fluid and therefore require conveyance limits. Extremely deep bend sections will occasionally indicate velocities that are not representative of sediment transport around the bend, and the recourse is to eliminate them from the model. The results from running this discharge will also give some insight into how close the existing channel is to a regime condition. That is, if there is overbank flow, justify that it also occurs in the prototype and is not just a numerical condition.

It is useful to determine the model performance for the channel-forming discharge because, if the channel is near regime, this should cause very little aggradation or degradation. Before focusing on sediment transport, however, demonstrate that the Manning *n*-value for the channel is appropriate for the movable boundary. Make whatever adjustments are necessary to ensure that the *n*-value for the stream-bed portion of the cross section is in reasonable agreement with that from bed roughness predictors. Also, the sediment transport rate will usually be higher on the first computation event than it is on subsequent events because there is usually an abundance of fines in the bed samples that will be flushed out of the system as the bed layers are formed. The physical analogy is starting water flow down a newly constructed ditch. It is important to balance the sizes in the inflowing bed-material sediment load with transport potential and bed gradation. The scatter in measured data is usually sufficiently great to require smoothing, but the adopted curves should remain within that scatter.

It is useful to repeat this steady-state test for the maximum water discharge in the testing hydrograph. The key parameters to observe are water surface elevations, flow distribution between channel and overbanks, and velocities. However, each study is unique, and one should regard this paragraph as suggestions to illustrate thinking and not a list that is both necessary and sufficient.

14.6.4.1 n-Values The first approximation of *n*-values was coded into the original model. At this point refine those values using field observations of stage or velocity.

14.6.4.2 Water-Surface Profiles With movable-bed calculations active, it is important to recheck the model-to-prototype comparison for water surface elevations. Because prototype measurements are like snapshots in time, it is important to run prototype boundary conditions for a sufficiently long period for the bed surface profile to resemble that in the prototype when the data were surveyed.

14.6.4.3 Flow Distribution Reevaluate the calculated flow distribution, similar to that in the fixed-bed test, and adjust *n*-values or geometry if needed.

14.6.4.4 Coordination of Inflowing Gradation with Bed Sediment Gradation The bed gradation is prescribed as an initial condition for the bed sediment reservoir. The inflowing sediment discharge is prescribed as a boundary condition. These data sets are related. That is, if the prototype is in equilibrium, the inflowing sediment load will be transported by grain size class without excessive deposition or erosion. That requires confirmation because the calculation is sensitive to the transport function selected for the study. Observe model transport by grain size and adjust either the inflow or the bed gradations, depending on which is regarded as the weaker data set.

14.6.4.5 Sediment Yield One should confirm that the calculation sediment yields in the computational model match the annual yields for the watershed. Figure 14-11 is an example of such a comparison. It is important to develop consistent units before making such a comparison. That is, if the published sediment yields were calculated from suspended sediment measurements, then either convert the computational model results into the suspended sediment component of the total sediment load or add the load moving in the unmeasured zone in the prototype to the suspended measurements. The Toffaleti transport function will facilitate such an estimate by displaying the vertical distribution of sediment in the water column. Most other transport functions do not offer that feature.

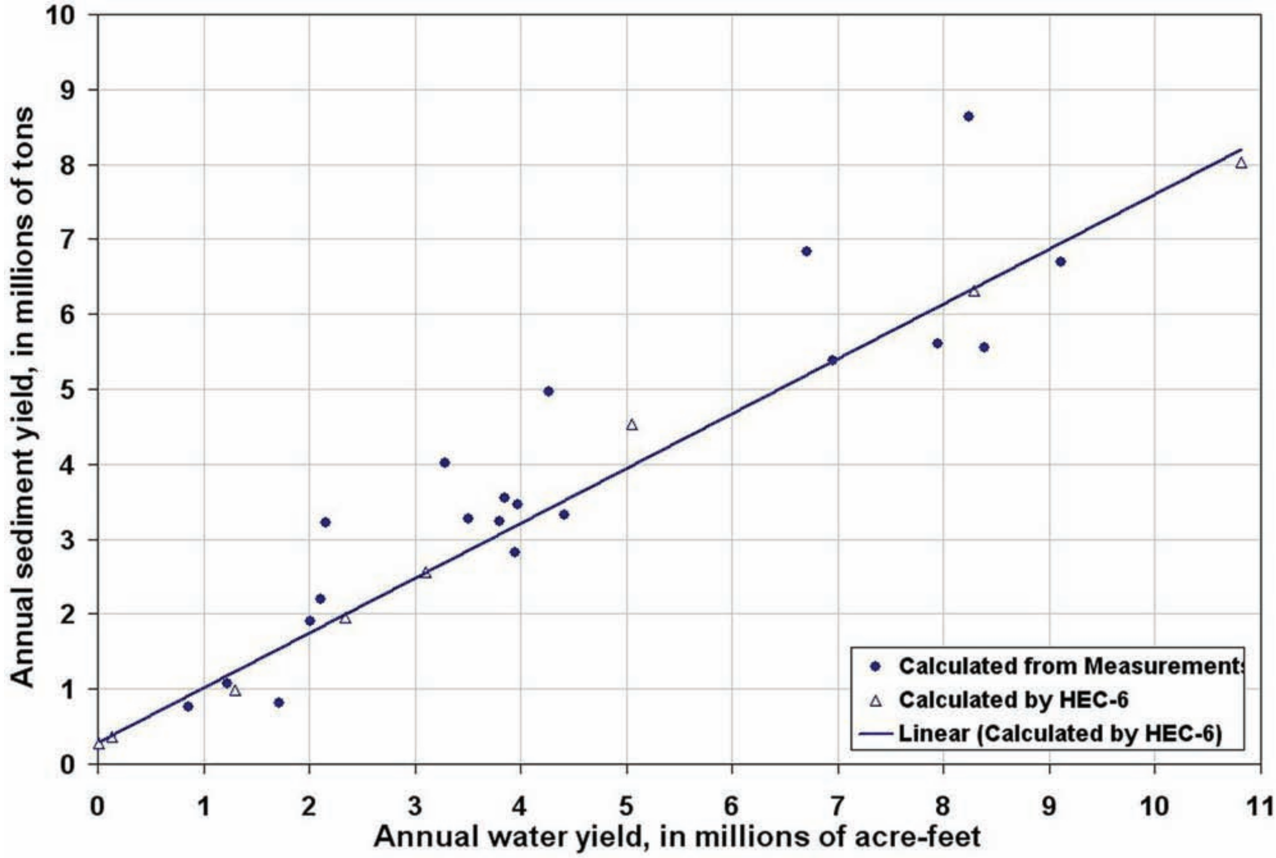

Fig. 14-11. Measured and calculated annual sediment yield.

14.6.4.6 Sediment Transport Profiles Another useful graphic is a plot of calculated sediment discharge versus river mile. It will show sources and sinks, which can then be one compared to the geometry data and to visual observations to test for reality. This will aid in improving the one-dimensional representation of the prototype to eliminate "numerical shadows" from the scour and deposition calculations.

14.6.4.7 Selection of Calibration Parameters Correlate model to prototype conditions in broad terms recognizing one dimensional approximations. For example, profiles of the bed elevation may exhibit little or no correlation with the prototype, but cross-sectional area changes should correlate with prototype behavior. Reconcile zones and amounts of aggradation and degradation by expressing accumulated volumes rather than depths. Reconcile accumulated weights passing each cross section.

14.6.5 Movable-Boundary, Unsteady-State Calibration

The final phase in model calibration is the movable-bed unsteady-state test. This is required for both unsteady-state and steady-state hydraulic calculations because the sedimentation equation is an unsteady-state equation. This phase utilizes both single-event and period-of-record hydrology. Of particular interest are the zones of deposition and erosion depicted during the passing of a hydrograph. When the stream is in a relatively equilibrium condition, these zones should cycle. When the graph of bed change versus time shows a trend to continue either deposition or erosion, investigate the cause. Resolve these issues starting with the most upstream location and continuing toward the downstream end of the model. Begin with the cross section showing the greatest change in bed elevation. Even if only one cross section is not responding properly, the model results downstream and sometimes upstream from it are not reliable.

14.7 BASE TEST

The most appropriate use of a movable-bed simulation is to compare an alternative plan of action with a base condition. In most cases the base condition is the predicted future behavior of the river in a "no-action future." In a reservoir study, for example, the base test would calculate the behavior of the reservoir reach of the river without the dam in place. In many cases, the base test simulation will show little or no net scour or deposition. These are the river reaches that are near equilibrium (i.e., where scour approximately equals deposition) under existing conditions.

Two sets of boundary conditions are needed: one set is for model calibration and the other set is for analyzing the plan test. That is, model calibration is a task designed to demonstrate that the model calculations match historical prototype behavior. The calibration task begins with a survey of the model area, called initial conditions, and runs to a second survey of the model area. It is necessary to know the boundary conditions that existed between those two surveys. That requirement dictates the selection of boundary conditions for model calibration.

However, boundary conditions for the base test must represent the future conditions in the project. They will be the same as those for the plan tests unless the plan tests are designed to investigate a change in the boundary conditions. For example, if the purpose of the model study is to investigate a channel modification, the base test would be conducted with the same boundary conditions as the plan test. The length of the simulation hydrograph for the base test is usually selected by considering the economic life of the project.

It is likely that the hydrology and sediment boundary condition values for the calibration period will not be representative of the long-term future hydrology and sediment boundary condition values. However, if the purpose of the model study is to investigate what will happen in a river as the result of a change in the boundary conditions, the base test values will be different from the plan test values, but they should contain the same period of time.

14.8 PLAN TEST

The project alternatives can be simulated by modifying the base data set appropriately. In case of a reservoir, a dam can be simulated by inserting operating rule data into the base test model. For a channel improvement project, cross-sectional geometry and roughness can be changed. If a major change is required, make the evaluation in steps. That is, change one parameter at a time so that the model results will be easier to interpret.

For example, it is best to analyze a channel modification project in three steps. First, change the hydraulic roughness values and run future flows in the existing geometry. Second, insert the modified cross sections and complete the analysis by running the alternative to be tested. Finally, add the contraction and expansion coefficients for the modified channel design and run the plan test.

Use model results from each of the above steps as an aid in predicting future conditions. Rely heavily on engineering judgment when analyzing model results. Look for surprises in the calculated results. These surprises can be used by the experienced river engineer to locate data inadequacies and to better understand the behavior of the prototype system. Any unexpected response of the model should be analyzed and should be justified before the results are accepted.

14.9 INTERPRETATION OF RESULTS

14.9.1 Form of Study Results

Results from the plan tests should be expressed in terms of change from the base case. This will provide an assessment of the impacts of proposed projects on the stream behavior. The impact of sedimentation on the performance of the project should be presented in units appropriate to the decisions which need to be made. For example, a flood channel will require maintenance to remove sediment deposits. These units are usually cubic yards. However, the parameter to measure in arriving at a maintenance schedule will most likely be the water surface. The results of the sedimentation study should include the locations of deposits and their resulting impact on the water surface profile.

14.9.2 Sensitivity Tests

It is desirable during the course of a study to perform sensitivity tests. Quite often part of the input data (such as inflowing sediment load) will be missing or will contain measurement error. The impact of these uncertainties on model results can be studied by modifying the suspected input data by $\pm\, x\%$ and rerunning the simulation. If little change in the simulation results, the uncertainty in the data is of no consequence. If large changes occur, the input data need to be refined. Refinement should then proceed by using good judgment and by modifying only one parameter at a time. Sensitivity studies performed in this manner will increase the modeler's understanding of model behavior and that understanding will aid in predicting the behavior of the prototype.

14.10 EXAMPLES TO ILLUSTRATE MODEL APPLICABILITY

One-dimensional computational models of sedimentation have been used in a variety of studies over the past three decades. Examples are

- to confirm land acquisition for a run-of-river reservoir that required simulating sedimentation processes in that reservoir for a 50-year life;
- to calculate the stability of a hydraulic fill prior to placing it across the Mississippi River to arrest the upstream movement of salt water from the Gulf of Mexico during low water;
- to predict the water surface profile in setting a levee grade in a backwater area of a reservoir;

- to reconstitute the degradation trend downstream from dams;
- to design erodible bed channels with bank protection and grade control structures;
- to predict general scour at a bridge crossing for bridge design or evaluation;
- to evaluate the impacts of instream sand and gravel mining;
- to predict maintenance dredging for existing and proposed navigation projects;
- to study degradation and aggradation in the development of the Atchafalaya Basin and Delta;
- to predict the stream-bed response of a river if water and sediment are diverted out;
- to predict stream stability and maintenance dredging for a flood protection project;
- to predict sedimentation processes following the removal of dams;
- to predict aggradation and degradation in channel modification projects:
- to design sediment traps; and
- to predict the bed roughness and resulting water surface profile due to the transport of sand and gavel through concrete channels.

14.11 AN EXAMPLE APPLICATION

An example was provided by Chang (1984) to illustrate the general points in this chapter. The actual study encompassed the lower 3 miles of the San Dieguito River in California, Fig. 14-12. Because this is only an illustration, the entire study area is not reproduced in Fig. 14-12.

14.11.1 Model Data

Data for the fluvial processes during the January to March 1993 flood were used for calibration. Channel geometry is defined by 43 cross sections selected along the reach. A total of 17 cross sections were surveyed before the flood and resurveyed soon after the flood. Cross sections not included in the survey were developed from the 1992 topographic map of the river channel. The map has a contour interval of 304.8 mm (1 ft).

Sediment particle-size distributions for the stream bed material are based on samples taken along the study reach. The stream-bed sediment is sand.

The runoff hydrograph for the January to March 1993 flood is shown in Fig. 14-13. This runoff was measured by the county of San Diego at the Hodges Dam. The flood that occurred on January 14, January 16, and January 18, 1993

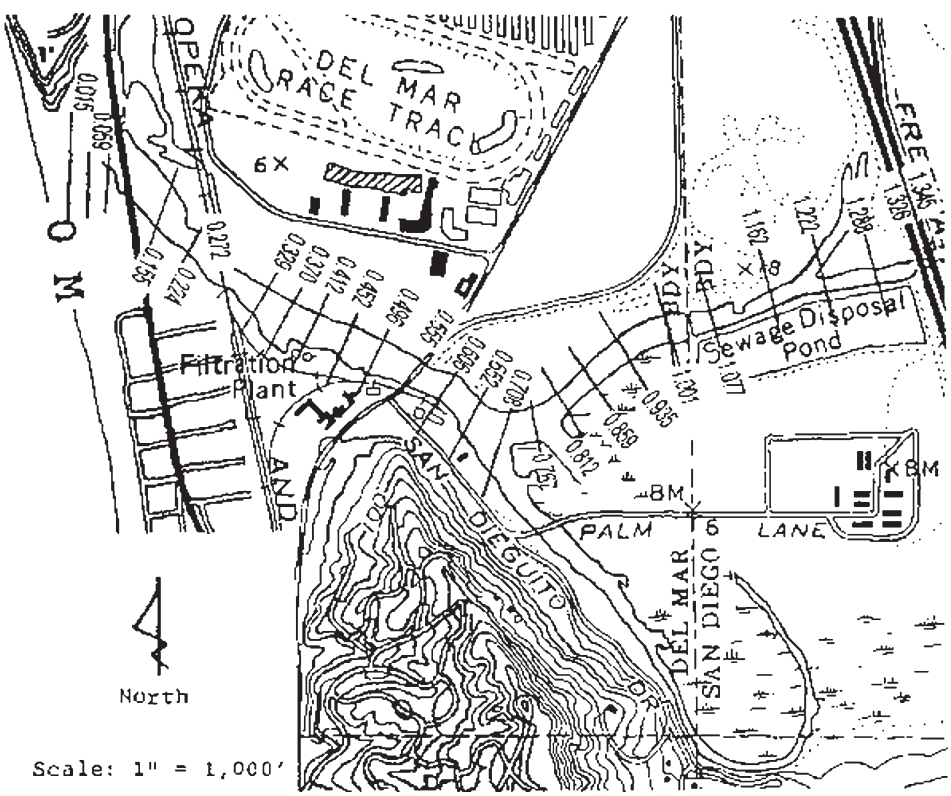

Fig. 14-12. Location of surveyed cross sections (multiply ft by 0.3048 to get m).

had three peaks. The discharges were 120.43 m^3/s (4,253 cfs), 188.39 m^3/s (6,653 cfs), and 120.43 m^3/s (4,253 cfs), respectively. The peak discharge of 188.39 m^3/s (6,653 cfs) has a return period of 14.7 years.

14.11.2 Selection of the Sediment Transport Formula

Numerous formulae have been developed for calculating sediment movement in sand-bed channels. Each one will predict a different sediment transport rate. The selection of the formula to use is best confirmed by comparing calculated results to measurements at the site. However, in an ephemeral stream such as the San Dieguito River, sediment discharge measurements can be made only during floods.

Consequently, data are scarce. Even when suspended sediment measurements are made, there is always the presence of the unmeasured zone near the bed. The substantial movement of the bed material in the unmeasured zone adds uncertainty to the measured data set.

Therefore, the approach to selecting the sediment transport function includes more than just a search for measured sediment concentrations. It also includes consideration of the physical conditions of the fluid, the hydraulic parameters of the flow, and the sediment characteristics of the stream bed. The engineer can start by comparing the hydraulic and sediment properties at the study site with those used in development of the transport function. The functions passing this test are then submitted to additional testing using measurements other than sediment concentrations.

Measured data include measured changes in channel dimensions. That is, the calculated rate and amount of erosion or deposition in a channel depend on the sediment transport formula used. If a formula overpredicts the sediment transport rate, it will calculate more deposition than the measured values. On the other hand, a formula that underpredicts the transport rate will show less deposition than the measured amount. At a cross section undergoing scour, the amount of scour is overpredicted by a sediment transport formula giving transport rates that are too high and vice versa. The use of calculated changes in bed elevation provides valuable information when the sediment transport function is being selected for a study. Stream channel changes in the lower San Dieguito River during the 1993 flood are characterized by channel-bed scour. The changes were significant.

In this study, the Ackers-White formula, the Engelund-Hanson formula, and the Yang formula (Vanoni 2006), were identified as possible choices. These functions were selected based on the extensive evaluation made by Brownlie (1983).

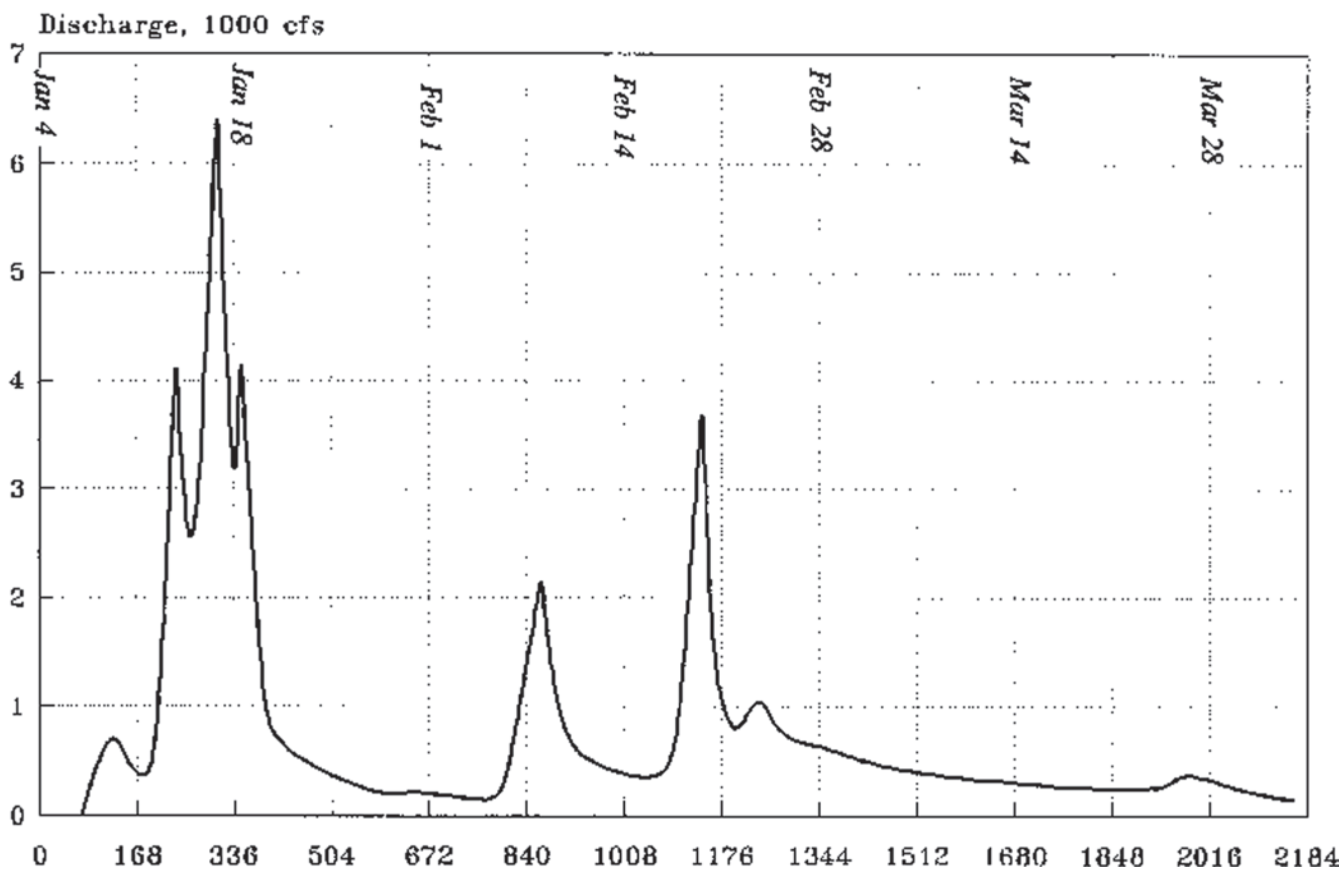

Fig. 14-13. Hydrograph for January to March 1993 floods (multiply cfs by 0.0283 to get m^3/s).

Each formula was tested to determine whether or not the computational model would simulate measured channel changes as described in the previous paragraph.

14.11.2.1 Calculated Sediment Delivery Sediment delivery is defined as the accumulated sediment load that passes a specified channel cross section. The equation is

$$Y_s = \int_T Q_s dt \tag{14-14}$$

where

Y_s = sediment delivery;
Q_s = sediment discharge;
t = time; and
T = the specified period of time.

Sediment delivery is widely employed by hydrologists working in watershed management. The quantity commonly used in their work is annual sediment yield. In computational modeling the accumulated sediment load is available, by particle size, at every cross section and for every computational time step. However, when the specified period of time for the sediment delivery calculation is a year, the sediment delivery becomes the annual sediment yield. Referring to the output from a computational model as sediment delivery preserves the historic definition of sediment yield.

The calculated sediment deliveries of bed material load based on the Ackers-White, Engelund-Hanson, and Yang formulas are shown in Fig. 14-14. These results were compared with the measured data in the final selection of the transport formula for this study.

In the general case the sediment discharge Q_s can be any part of the sediment load or it can be the total sediment load. In this case it pertains only to bed-material load. It was not necessary to include fine sediment, i.e., silt and clay, in this model because those particle sizes were not present in the samples of bed material. As a result, the conversion factor between volume and dry weight of sediment is 1633.9 kg/m^3 (102 lb/cu).

The shape of the sediment delivery graph identifies zones of erosion and deposition along a channel. The plot should be read in the direction of the water flow. A decreasing delivery in the downstream direction, i.e., a negative gradient for the delivery-distance curve, signifies that sediment is depositing into the channel bed and banks. On the other hand, an increase in the sediment delivery in the downstream direction indicates that sediment is being removed from the channel bed and banks. A horizontal sediment delivery plot indicates

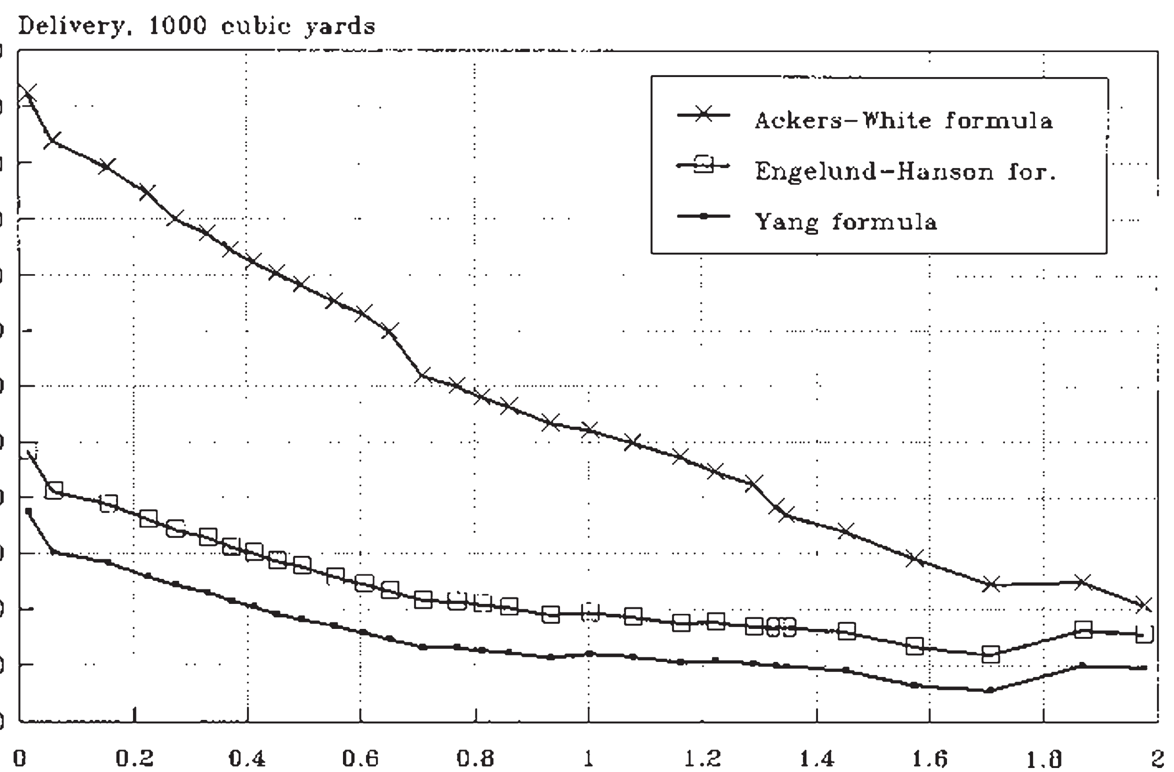

Fig. 14-14. Sediment delivery through the study reach (multiply cy by 0.7646 to get m^3).

zero deposition or erosion. Of course, the assumption in this illustration is that local sediment inflow or outflow is zero.

As depicted in Fig. 14-14, sediment delivery through the lower San Dieguito River during the 1993 flood was characterized by general erosion along most of the river reach. Although all three figures show the general trend of erosion, their quantities are nevertheless different. The total calculated erosion for the inlet channel and the west channel is shown in Table 14-3. The inlet channel is from the river mouth to river mile 0.713 and the west channel is from river mile 0.713 to Interstate 5 at river mile 1.345.

The delivery curves shown in Fig. 14-14 have different slopes for the inlet channel and the west channel, and the steeper slopes are in the inlet channel. The average slope for the delivery curve of a channel is the difference in delivery from one end of the reach to the other divided by the reach length. The average change in end area can also be calculated from measured cross sections and compared to the slope of the delivery curve as shown in Table 14-4. The row identified as "Measured" shows volumes calculated from measured cross-sectional changes integrated over the channel length.

Table 14-4 provides a direct comparison of the calculated channel changes with measurement. The amount of erosion is considerably overpredicted by the Ackers-White formula. It is slightly overpredicted by the Engelund-Hanson formula. The calculated results based on the Yang formula are similar to the measured values. For this reason, the Yang formula was selected for application on the lower San Dieguito River.

Both simulation and measurement show that the inlet channel underwent greater erosion than did the west channel. The modeler can use such information to understand the prototype. For example, a possible cause for this difference is that the inlet channel is replenished by beach sand after each episode of storm flow.

14.11.2.2 Reconstitution of the Measured Bed Profile Profiles of the calculated bed surface and water surface are shown in Fig. 14-15. The results based on the Yang formula are closer to prototype measurements than the results using the Ackers-White or the Engelund-Hanson formula. The differences are within 5%. For this reason, the Yang formula is selected for application on the lower San Dieguito River.

It should be noted that the version of Ackers-White formula used in this study is the earlier version. It was included in the Brownlie evaluation. Ackers and White have since modified this formula.

14.11.3 Calculated Changes in Channel Geometry

Calculated changes in river channel geometry are presented as changes in longitudinal channel-bed profiles and in channel cross sections. These changes reflect the spatial variations in sediment delivery described above.

14.11.3.1 Longitudinal Profiles Channel-bed profiles computed with the Engelund-Hanson and Yang formulae are generally similar (Fig. 14-15). The bed profiles at the peak flow are highly uneven, with the low points at channel bends and channel contractions. The channel-bed profiles become quite smooth toward the end of the flood. These results indicate that contraction scour is more pronounced during high flow and it becomes much less during low flow. The low point in bed profiles at a channel bend is related to deeper scour near the concave bank. This phenomenon will also be demonstrated by cross-sectional changes described in a later section. It can be seen from the relatively smooth channel-bed profiles at the end of the flood that channel-bed scour is at a maximum near the river mouth and it decreases gradually in the upstream direction.

For the simulated changes based on the Ackers-White formula, the extent of degradation is considerably greater than that based on the other two formulae. The deeper scour depths are related to the greater sediment delivery predicted by the Ackers-White formula.

14.11.3.2 Channel Cross Sections Calculated cross-sectional changes along the river reach are exemplified by those presented in Figs. 14-16 and 14-17. Each figure

Table 14-3 Simulated Sediment Deliveries (multiply cuyd by 0.7646 to get m^3)

	Simulated sediment delivery, cuyd		
Formula used	River mouth	Entrance of inlet channel	West channel
Ackers-White	520,000	309,000	184,000
Engelund-Hanson	205,000	107,000	83,100
Yang	151,000	65,800	48,800

Table 14-4 Calculated and Measured River Channel Erosion

	Total erosion, cuyd		Average change in cross section, sq ft	
Formula used or measured	Inlet channel	West channel	Inlet channel	West channel
Ackers-White	211,000	125,000	1,513	1,010
Engelund-Hanson	98,000	23,900	703	193
Yang	85,200	17,000	611	137
Measured	87,500	17,770	628	144

(multiply cuyd by 0.7646 to get m^3; multiply sq ft by 0.09290 to get m^2)

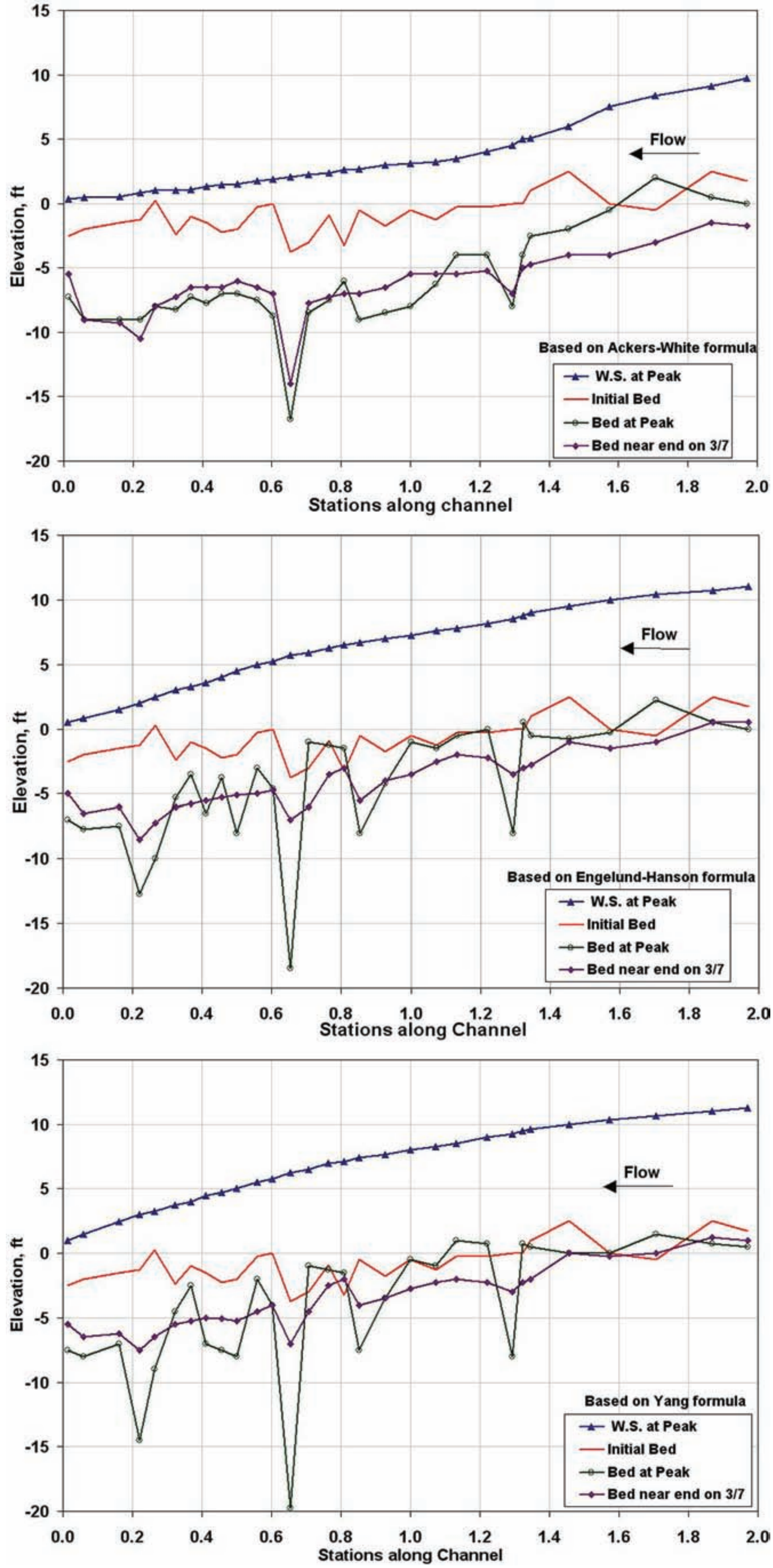

Fig. 14-15. Calculated bed and water surface profiles (multiply ft by 0.3048 to get m).

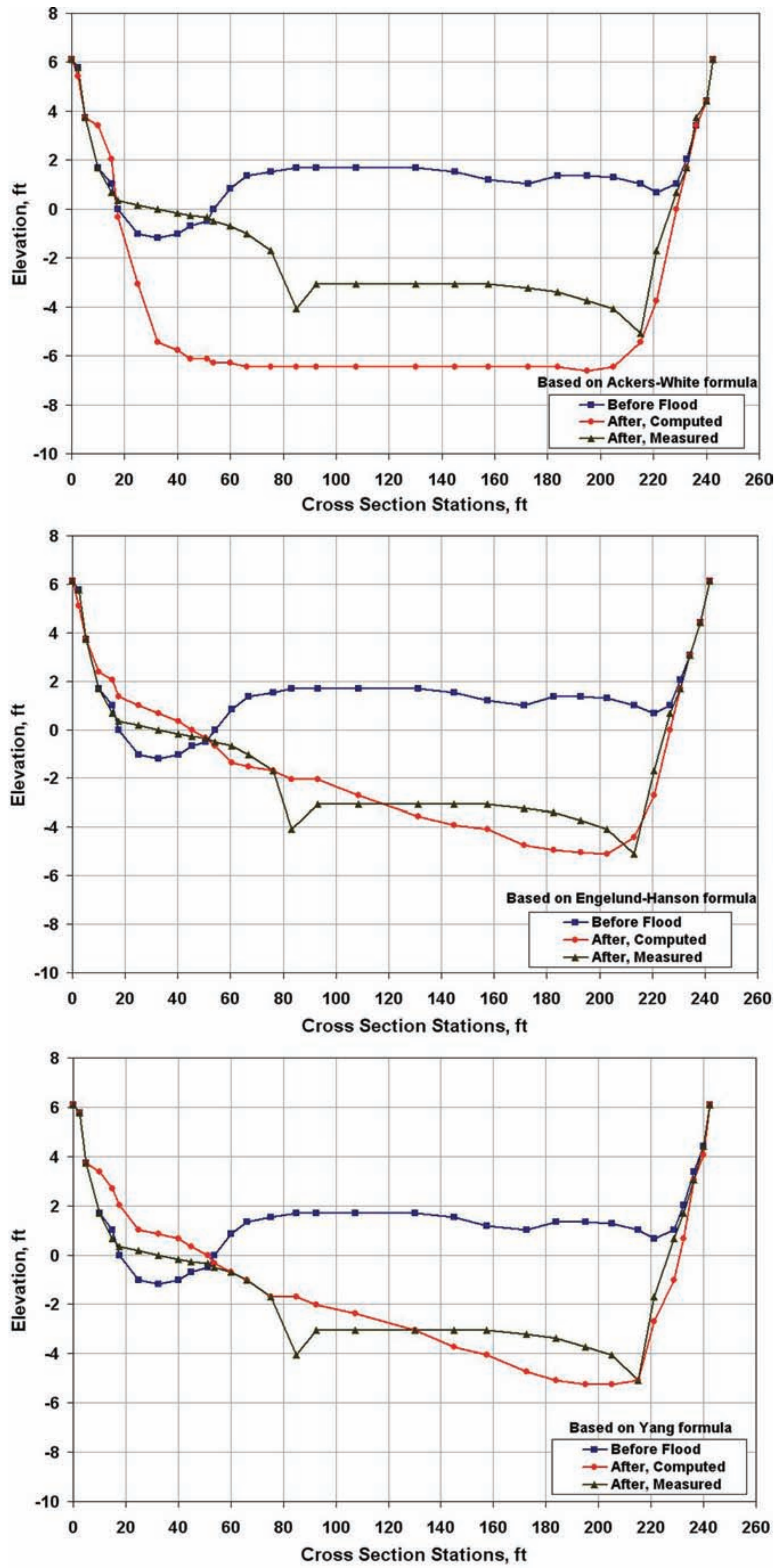

Fig. 14-16. Calculated and measured cross-sectional changes at Sect. 0.412 (multiply ft by 0.3048 to get m).

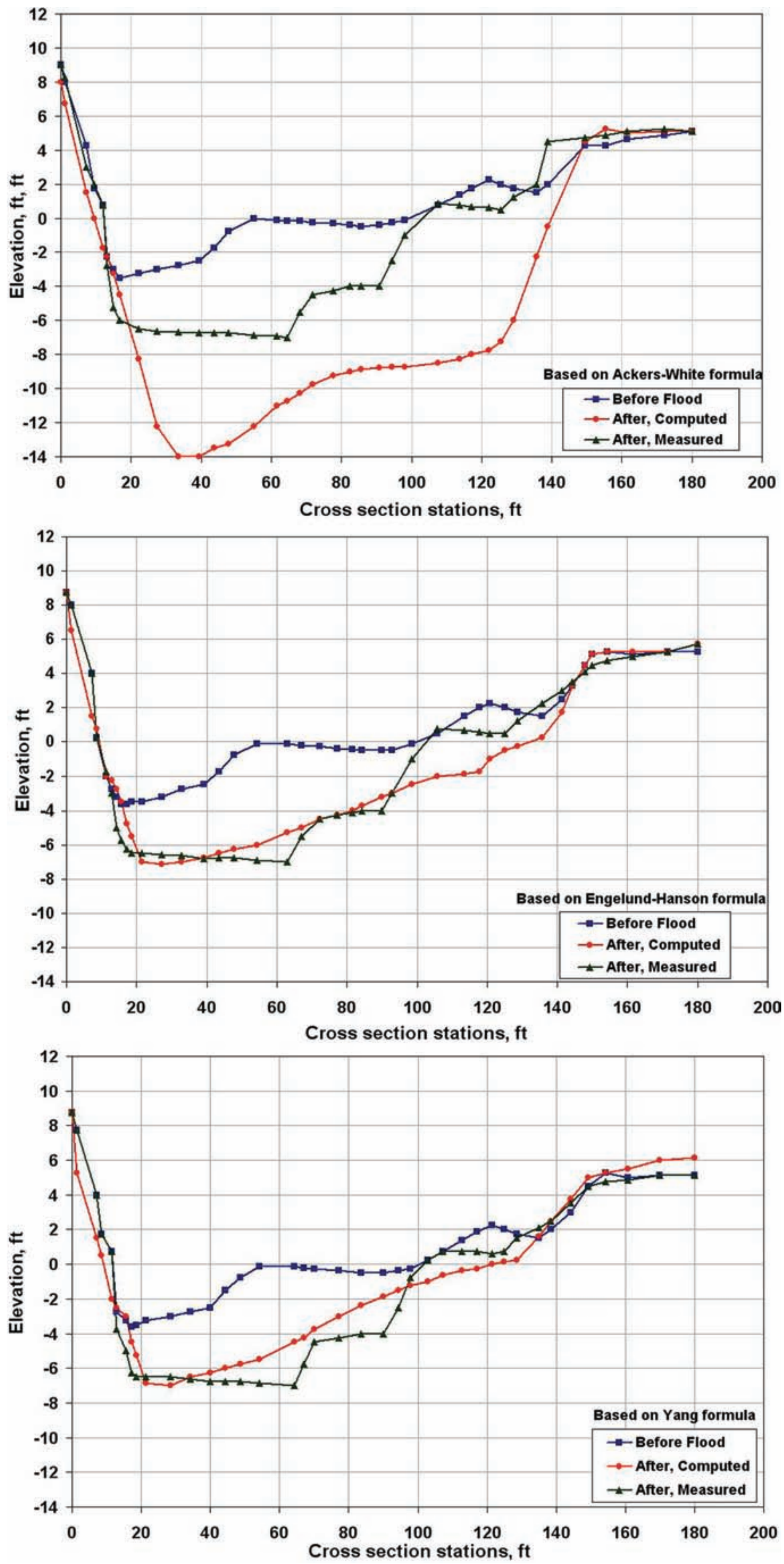

Fig. 14-17. Calculated and measured cross-sectional changes at Sect. 0.652 (multiply ft by 0.3048 to get m).

includes the following three channel-bed profiles: (1) The initial bed profile based on the preflood survey; (2) the calculated cross-sectional profile on the date of the postflood survey; and (3) the surveyed postflood cross-sectional geometry.

These figures provide comparisons of simulated and measured cross-sectional geometries. Cross-sectional changes simulated based on the Ackers-White formula far exceed the measured changes. It is therefore concluded that the results are unacceptable and that the Ackers-White formula cannot be used for this study.

Simulated cross-sectional changes based on the Engelund-Hanson formula are generally supported by the measurement. It can be seen that the net change in cross-sectional area as simulated tends to exceed the measured change. In other words, the scour is slightly overpredicted by the Engelund-Hanson formula. For the inlet channel, the overprediction is 12% averaged over the channel reach; the overprediction is 30% for the west channel, as summarized in Table 14-5.

Calculated cross-sectional changes based on the Yang formula are generally supported by the measurement. For the inlet channel, the scour is underpredicted by 2.7% averaged over the channel reach. For the west channel, the scour is overpredicted by 5%. It may therefore be concluded that the Yang formula is the most applicable to the lower San Dieguito River.

The simulated patterns of scour and fill are also used to demonstrate the complex channel geometry adjustments during floods. For the erosional changes, the scour pattern at a cross section is affected by the geometries of adjacent cross sections and channel curvature. Section 0.652 is located in a channel bend, and the shape of the cross section is influenced by the channel curvature. To approximate such morphological adjustments, the mathematical model must be able to calculate nonuniform patterns of deposition and erosion.

A general comparison of the calculated and surveyed cross-sectional profiles may be assessed as follows. The erosional changes as simulated by the Yang formula are clearly consistent with the survey. Any discrepancy between simulated and measured results may be attributed to the following factors.

1. A nonhomogenous horizontal distribution of the bed sediment (i.e., sediment particle sizes on and in the stream bed are not uniformly distributed at a cross section). The presence of coarse materials usually affects the pattern of channel changes, and a one-dimensional mathematical model does not account for such sediment distributions.
2. A horizontal distribution of the suspended sediment concentration in the inflowing water that is not in equilibrium with hydraulic forces at the current cross section. A one-dimensional mathematical model does not account for such sediment distributions.
3. Imprecision in measurements such as the size of the flood discharge, the river cross section, and the bed material composition.
4. Imprecision in computations related to the roughness coefficient, the sediment transport formula, etc.

Table 14-5 Comparison of Calculated and Measured Scour

	Simulated scour/measured scour	
Formula used	Inlet channel	West channel
Ackers-White	241%	678%
Engelund-Hanson	112%	130%
Yang	97.3%	105%

Despite the differences between the calculated and measured cross-sectional changes, the calculated change in cross-sectional area and the pattern of erosion and deposition along the longitudinal profile are consistent with the survey. This validates the model for predicting longitudinal profiles and general cross-sectional end area changes along the lower San Dieguito River.

14.12 AVAILABLE COMPUTATIONAL MODELS

The Subcommittee on Sedimentation, Interagency Advisory Committee on Water Data investigated available computational sedimentation models (Fan 1988). In 1986 they initiated a project on the “selection and proper use of computer models to estimate sediment transport.” At the end of phase two of that three-phase effort they had selected 12 models for comparison and evaluation. These are presented in Table 14-6.

The field of computational modeling is continually changing. New models are being released and old ones are being improved. Even when originally printed, Table 14-6 was not an exhaustive list of sediment models. Fan writes,

> Beginning in the summer of 1987, the Work Group made a survey of the computer sedimentation models developed and implemented in the United States. Public responses to the survey were prompt and overwhelming. Within 2 months, the Work Group received approximately 48 sedimentation models which are available both in federal agencies and in the private sector in the United States. (Fan 1988, p. 3)

It was from this submission of models that the Work Group selected those listed in Table 14-6 for further investigation.

The purpose of this section is to illustrate what is meant by “computational model.” It makes no endorsement of a specific model nor does it imply that all are equal in their performance and reliability. The list of available computational models will very likely be obsolete even before it is

Table 14-6 Currently Available Computer Models[a]

Model name	Background	Comments
HEC-6	Developed by William A. Thomas during the period 1968 through 1974 and released by the U.S. Army Corps of Engineers, Hydrologic Engineer Center, Davis, Calif. in 1976.	The model is designed to simulate one-dimensional, steady, gradually varied water and sediment flow problems.
TABS2	Developed by a team of researchers at the U.S. Army Waterways Experiment Station, Vicksburg, Miss. during the period 1977 through 1984 and released in 1984.	This is a fully two-dimensional, finite-element solution of the flow and sediment equations.
IALLUVIAL	Developed by F. W. Karim at the University of Iowa under contract with the U.S. Army Corps of Engineers, Omaha, Neb.	It is a one-dimensional, quasi-steady routing model.
STARS	This model is an outgrowth of a model originally developed by Albert Molinas at Colorado State University and was submitted under contract to the Bureau of Reclamation in 1983.	STARS has a unique feature of using a stream-tube concept to vary the hydraulic and sediment transport characteristics across a stream cross section.
GSTARS	Developed by Albert Molinas and Chih Ted Yang for the Bureau of Reclamation and released in 1986.	This is a generalized stream-tube model for alluvial river simulation.
ONED3X	Developed in 1987 by Vincent Lai, U.S. Geological Survey.	This is a coupled multimode method of characteristics.
CHARIMA and SEDICOUP	Developed from 1985 through 1987 by Forrest Holly, Jr., University of Iowa. Between 1986 and 1988, Dr. Holly developed SEDICOUP, a totally coupled program.	This represents the latest generation in a series of codes whose progenitor was IALLUVIAL The model is a partially coupled program for mobile bed simulation and can duplicate an IALLUVIAL computation. It is still under active development and modification.
FLUVIAL12	Developed in 1976 by Howard Chang of San Diego State University, Calif.	The model is intended for water and sediment routing in natural and man-made channels. The combined effects of flow hydraulics, sediment transport, and river channel changes are simulated for a given flow period.
HEC2SR	Developed in 1980 by Ruh-Ming Li of Simons, Li and Associates, Inc. (SLA).	This model is designed to simulate watershed sediment yield, aggradation, and degradation in a river basin. It incorporates a sediment-routine program into the HEC2 program developed by Bill S. Eichert, Hydrologic Engineering Center, U.S. Army Corps of Engineers
TWODSR	Developed in 1988 by Yung-Hai Chen.	This is a two-dimensional model based on an uncoupled, unsteady approach.
RESSED	Developed byYung-Hai Chen for the Canadian International Project Management (CIPM)-Yangtze Joint Venture.	Developed to study the Three Gorges Project on the Yangtze River in China. This is a simplified quasi-nonequilibrium model for reservoir and river erosion sedimentation related problems.

[a]The first six models are federally owned. The last five are privately owned (1986).

printed. However, the principles presented in this chapter will continue to be useful in evaluating and selecting a computational model.

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