CHAPTER 196

Calibration of Branched Estuary Models

by

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INTRODUCTION

Unsteady flow models of one-dimensional channels or of channel networks are commonly calibrated using a combination of stage (water surface elevation) and discharge observations. In tidally influenced areas these observations may extend in time over several tidal cycles. Where there is an important mean flow discharge such as in a river harbor, observations must be repeated for a number of representative mean flow conditions.

Stage observations are comomonly collected using recorders coupled to water surface floats mounted at various points on the channel banks. The initial costs of purchasing the recorders, installing them, and in the leveling necessary to determine datum are the primary expenses involved in collecting stage observations. For these observations the cost per observation decreases as the number increases.

In tidally influenced channels, accurate one-dimensional instantaneous discharge measurements are difficult if not impossible to obtain because of the rapidly changing flow velocity. In many locations, discharge measurements cannot be made throughout a tidal cycle because of safety restrictions. Finally, with respect to stage information, the cost of collecting discharge information can be extremely high. In addition, the cost per observation does not decrease with the number of observations.

With respect to the boundary conditons used to drive them, there are two main classes of unsteady flow models, the stage-stage models and the discharge-stage models. The former are driven by inputs of stage at all important external boundary points. The primary purpose of the stage-stage model is to determine the time history, or an average value of downstream discharge. Discharge-stage models are driven upstream by steady or time-variable observed or hypothetical discharges and downstream by observed or hypothetical stages. This type of model is used primarily in conjunction with solute transport models in predicting the location and concentration of disolved substnces, as in disolved oxygen modeling.

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In light of the difficulty, danger, and relative expense of collecting discharge measurements, as compared to stage observations the queston arises how best to locate in space and time the minimum number of discharge measurements required to calibrate an unsteady flow model. In fact, it should be asked if situations arise wherein unsteady flow models can be adequately calibrated without any discharge measurements. This paper is presented to discuss these questions. Illustrations are presented using field data collected to calibrate both stage-stage and discharge-stage flow models.

THEORY

Unsteady Flow Computation

The conservation of mass equation for a segment of one-dimensional open channel with no distributed inflow is

$$\frac{\partial A}{\partial t} + U\frac{\partial A}{\partial x} + A\frac{\partial U}{\partial x} = 0 \tag{1}$$

Wherein the independent variables are x downstream distance and t, time. The dependent variables are cross-sectional area, A, and downstream velocity U. The conservation of momentum equation is

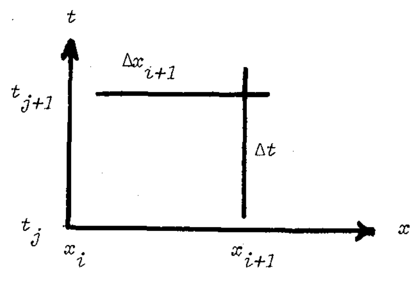
$$\frac{\partial U}{\partial t} + U\frac{\partial U}{\partial x} + g\left(\frac{\partial H}{\partial x} + S_f\right) = 0 \tag{2}$$

Wherein g is the gravitational constant, S_f is the friction slope and H is the stage. Cross-sectional area is expressed as a function of H and x. In (2) friction slope is expressed using Chezy's formulation

$$S_f = \frac{U^2}{C^2 D} \tag{3}$$

Wherein C is Chezy's discharge coefficient, the hydraulic depth, D, is A/T and T is the channel top width corresponding to a particular water surface elevation.

One of the most efficient schemes for solving the nonlinear hyperbolic partial differential equations 1 and 2 is Strelkoff's (1970) linear implicit technique presented here as modified by Bennett (1975). In this formulation, the following space-time convention is used,



and (1) becomes

$$\frac{1}{2\Delta t}\Big[T_{i+1}^{j}\;(H_{i+1}^{j+1}-H_{i+1}^{j})+T_{i}^{j}\;(H_{i}^{j+1}-H_{i}^{j})\Big]$$

$$+\frac{(U_{i+1}^{j}+U_{i}^{j})}{2\Delta x_{i+1}}\Big[(A_{i+1}^{j}-A_{i}^{j})+T_{i+1}^{j}\;(H_{i}^{j+1}-H_{i}^{j+1})-T_{i}^{j}\;(H_{i}^{j+1}-H_{i}^{j})\Big]$$

$$+\frac{(A_{i+1}^{j}+A_{i}^{j})}{2\Delta x_{i+1}}\Big[U_{i+1}^{j+1}-U_{i}^{j+1}\Big]=0 \qquad (4)$$

while (2) becomes

$$\frac{1}{2\Delta t}\;(U_{i+1}^{j+1}-U_{i+1}^{j}+U_{i}^{j+1}-U_{i}^{j})+\frac{(U_{i+1}^{j}+U_{i}^{j})}{2\Delta x_{i+1}}\;(U_{i+1}^{j+1}-U_{i}^{j+1})+$$

$$g\Bigg[\frac{(H_{i+1}^{j+1}-H_{i}^{j+1})}{x_{i+1}}+\frac{1}{2}\Bigg((S_f)_{i+1}^{j}+(S_f)_{i}^{j}+\left(\frac{\partial S_f}{\partial U}\right)_{i+1}^{j}(U_{i+1}^{j+1}-U_{i+1}^{j+1})$$

$$\left(\frac{\partial S_f}{\partial U}\right)_{i}^{j}(U_{i}^{j+1}-U_{i}^{j})+\left(\frac{\partial S_f}{\partial y}\right)_{i+1}^{j}(y_{i+1}^{j+1}-y_{i+1}^{j})+\left(\frac{\partial S_f}{\partial y}\right)_{i}^{j}(y_{i}^{j+1}-y_{i}^{j})\Bigg)\Bigg]=0 \qquad (5)$$

Bennett (1975) lists the advantages and disadvantages of the linear implicit formulation in solving the single reach unsteady flow problem. He also describes a general model which allows the efficient extension of the linear implicit technique to the computation of unsteady flows in networks of open channels. This model uses a coding system referenced to the end cross-sections of channel segments intersecting at interior junctions to inform the solution algorithm to apply interior boundary conditions and thus carry the implicit formulation from one channel segment to another. Specific techniques are introduced to reduce the core storage required and to expedite the inversion of the resulting sparse, wide-banded coefficient matrix. The calculations described in the remainder of this paper were made using the general model.

Calibration

Calibration of a mathematical model is the process of adjusting model parameters to obtain best-fit of model predictions to a set or sets of observations. The best-fit criterion is the objective function, a single unique function of differences between observations and the corresponding predictions. "Best-fit" is achieved when the objective function is minimized. The parameter values producing this minimum comprise the optimum parameter set.

A number of automatic optimization routines have been used to derive optimum parameters for unsteady flow models. Becker and Yeh (1972) and (1973) use the influence coefficient algorithm. A modification of their scheme was used by Bennett (1975) and is incorporated into the general model discussed above. Yih and Davidson (1975) found the Marquardt algorithm to be the most efficient in obtaining longitudinal dispersion coefficients. The main disadvantage in the use of these routines is the large number of complete simulation runs that must be made before the optimum is reached. In unsteady flow simulation this can become quite expensive.

The most commonly used objective function is the mean square error, the average of the squares of difference between observations and predictions. The square root of the mean square error, the RMS (Root Mean Square Error) is used in this paper. It has the advantage of being expressed in the same units as the original predictions and observations, and is, therefore, more easily comparable to the values of the individual variables themselves. When the observation consists of two or more noncommensurable quantities such as a stage and discharge it becomes difficult to define the objective function to give the correct weight to each type of observation. In this case some normalizing function should be used to commensurate the contributions of the two types of observations to the objective function. The definitions of the normalizing functions of course, influence the final result of the calibration; they, therefore, must be defined with great care. Becker and Yeh (1972) use velocity and depth errors directly in their objective function. For the shallow flows that they worked with this places about equal weight on both types of observations, however, for deeper flows it would emphasize the depth. Bennett (1975) used the average value of depth to normalize errors in stage and the average absolute value of discharge to normalize errors in discharge. This scheme probably places too much weight on discharge.

In this paper we want to investigate the possibility of calibrating the two basic types of unsteady flow models using stage data alone. We will therefore work primarily with the RMS of stage errors. However, we will continuously monitor the RMS of discharge deviations to determine the effectiveness of our techniques.

THE STAGE-STAGE PROBLEM

Three Mile Slough

The data chosen to illustrate the stage-stage problem was collected jointly by the U.S. Geological Survey and the California Department of Water Resources in July and August, 1959. As shown in Figure 1 Three Mile Slough is the first connecting channel upstream of the confluence of the Sacramento and the San Joaquin Rivers. It is a natural channel which has been stabilized and diked.

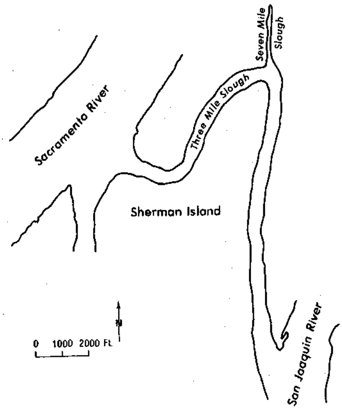


Figure 1. Three Mile Slough, a stage-stage problem.

At the two ends of the Slough the water surfacesin the rivers oscillate independently because of different travel times for tidal transients moving up the rivers from San Francisco Bay. Because of the independence of the water surface elevations at the two ends of the channel and because it is a relatively small cahnnel connecting the two larger bodies of water, Three Mile Slough provides a classical example of the stage-stage problem.

There are seventeen sets of cross section descriptions availale over the 3.2 mile length of the Slough, nine were used in the mathematical model. The side channel is a 2,300 foot segment of Seven Mile Slough presently closed off at the end away from Three Mile Slough. The only cross sectional properties available for Seven Mile Slough are average width and depth. These were used in the mathematical model. Little attention was paid to Seven Mile Slough during data collection because it was observed that this channel had little influence on the discharge picture in Three Mile Slough. Simulation results confirm this observation.

The tidal oscillations in the Sacramento and San Joaquin Rivers at the ends of Three Mile Slough were recorded digitally from direct reading floats at 15-minute intervals. The discharge observations used were obtained by summing the instantaneous contributions of individual discharge hydrographs for each of about twnety subsections at each end of the Slough. The individual hydrographs were prepared from standard discharge measurements made from boats. Each subsection was gaged about every 45 minutes.

Numerical Experiments

The investigation of the stage-stage problem, essentially a sensitivity analysis, was conducted as follows: (1) Fourier series were fit to the driving stages at the Sacramento (arbitrarily chosen as the upstream end of the Slough) and at the San Joaquin for a tidal day of twenty five hours. This was done, first, because it expedited interpolation between the observed stages, as stability conditions required a computaiton time interval on the order of six minutes rather then the fifteen minutes used in recording the data. Second, the analytical expressions for the driving stages allowed them to be shifted in time by any arbitrary increment, permitting the investigation of the influence of phase errors. The Fourier series stages were seldom as much as two-hundredths of a foot different than the original observaitons.

The mathematical model driven by the Fourier series stages was calibrated against upstream and downstream observed discharge by adjusting Chezy's C. The absence of intermediate discharge observations precluded the use of different C values for the three channel segments comprising the mathematical model. The optimum parameter value is C = 75 $ft^{1/2}$/sec. and the corresponding value of the normalized RMS discharge error is .123 (12 percent).

For use as observations in the sensitivity analysis the calibrated model was used to compute stages at river miles .66, 1.26 (the intersecitons of Three Mile and Seven Mile Sloughs) and 2.44 measured upstream to downstream. This error-free numerically generated set of stage observations was used to perform the sensitivity analysis of a typical stage-stage unsteady flow model subjected to perturbations of its parameters and boundary conditions. Note that because of rounding the hypothetical observations to the nearest 0.01 ft, the normalized stage RMS errors for the error free data set is .00015 (0.015 percent). Again, because actual observations of discharge, were used, this corresponds to a normalized error of 0.123 in this variable.

The Discharge Coefficient

The effect of varying the Chezy discharge coefficient C on normalized RMS stage error and normalized RMS discharge error is shown in figure 2. For the stage error, the largest quantity given on figure 2 represents an RMS error of only 0.01 ft (to the nearest hundreth) at each of the three observation stations. The stage errors are systematic, that is the water surface elevation for C values not equal to 75 $ft^{1/2}$/sec. are always slightly lower than for this value. Figure 3 shows for the extreme cases of C = 60 $ft^{1/2}$/sec. and C = 90 $ft^{1/2}$/sec, the influence of the variations on the discharge at the Sacramento River end of the slough. When C is greater than 75 $ft^{1/2}$/sec. the amplitude of the discharge fluctuation is noticeably larger than for this value, while for a lesser, the amplitude is less. A similar pattern is observed at the San Joaquin end.

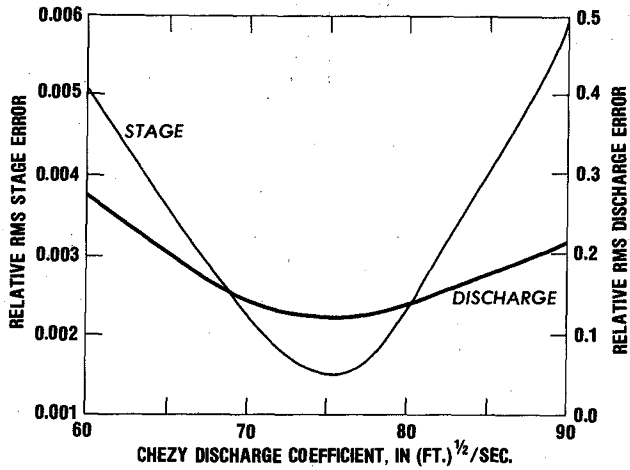


Figure 2. Effect of variation of Chezy C on normalized stage and discharge errors.

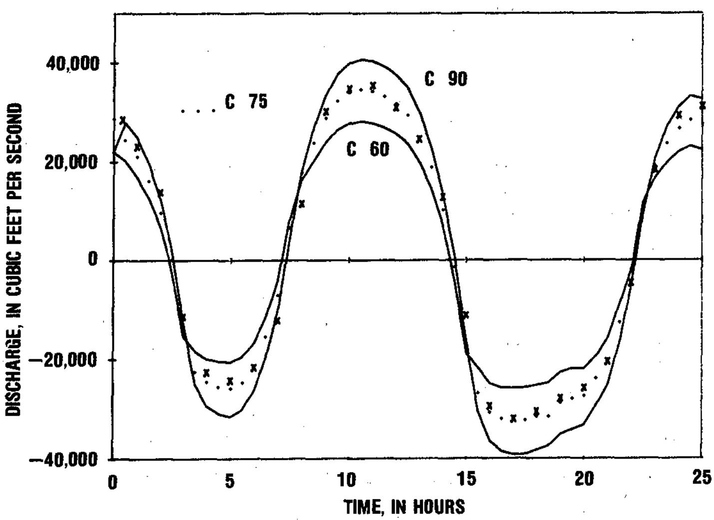


Figure 3. Observed and predicted discharge hydrographs for extreme values of Chezy C.

Using the error free stage observaiton set in performing the present sensitivity analysis has provided the answer to one of the questons posed earlier; namely, is it possible to calibrate a stagestage model using stage observations alone. The answer is no, because while for the nonoptimum C values the deviation of stage from that observed for C = 75 ft^{2}/sec is systematic, it is too small to be observed in a real stage record collected in the field.

Figure 3 makes it possible to answer another of the questions posed in the introduction namely, where best in space and time to place discharge measurements to make the best use of them. In calibrating a stage-stage model the best positions in time are at the two extremes of positive and negative discharge to define the amplitude of the discharge fluctuaiton. At least two positive and two negative peaks should be observed for each mean flow condition of interest. At least in a reach as short as Three Mile Slough, the spacial location of the discharge measurements is unimportant and measurements at one cross section should be sufficient.

At this point the main questions posed concerning the calibration of a stage-stage model using a bias-free data set have been answered. The remaining topics in this section concern the influences of errors in the calibration data on the final product and describe how to detect these errors using auxillary stage observations.

Cross-Sectional Area

The effect of variation of cross-sectional area on normalized RMS discharge error is shown on figure 4. The area ratio of this figure is a multiplicative factor applied in simulation to the true values of area and top-width of a cross section at any stage. At least for the magnitudes shown in figure 4, cross-sectional area has no disernable effect on the RMS stage error. Therefore, errors in cross-sectioanl area cannot be detected from stage records in the stage-stage problem.

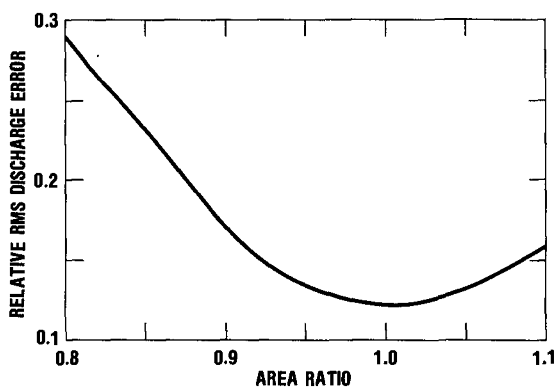


Figure 4. Effect of errors in cross-sectional area on discharge.

Decreasing the cross sectional area has the same quantitative effect on predicted discharge as decreasing the discharge coefficient, it increases the frictional resistance. Therefore, the amplitude of discharge fluctuation decreases as cross sectional area decreases. As can be seen from figure 4 for the same amount of area variation, the effect is more severe as the area decreases than as it increases. For relatively small errors in cross sectional area, say up to 20 percent a stage-stage model can probably be calibrated sucessfully. The C values will simply be erroneous and compensate for the errors in the cross sectional area. The section on discharge-stage models contains a discussion of the advantageous use of such compensating "errors" in calibrating a discharge-stage model.

Gage Datum

Assuming the intermediate gages are to correct datum, an error in the datum in one of the driving gages is distributed essentially linerally along the segment of channel between the driving gages. The error is easy to see because it appears as a vertical displacement between the observed and predicted water surface elevations at the intermediate gages. An approximate value for the error can easily be recovered by a simple linear extrapolation. Under normal operating conditions three to five hundredths of a foot should be detectable.

The effect of a positive datum shift at the upstream gage on the calculated tidal-cycle average downstream discharges for the simulation period are shown in table 1.

Table 1. Effects of datum shifts on downstream discharge.

Description	Average downstream discharge (cfs)	Steady uniform discharge (cfs)
Datum Shift		
" " = .05 ft	1572	6888
" " = .1 ft	3052	9741
" " = .2 ft	6116	13775

Also given in the table is a predicted approximate value for the steady-uniform flow in Three Mile Slough for C = 75 $ft^{\frac{1}{2}}$/sec where the amount of fall is equal to the datum shift. The unsteady nature of the flow causes the predicted average value to be less than that for steady uniform flow. The predicted average downstream discharge increases lineraly with datum shift, rather than with the square root of it as does the steady-unfiorm flow discharge. This is probably an isolated occurance because, of course, as shift increases average fall increases and tends to become dominant in determining the average value of discharge. As this happens the square root relationship has to come fully into effect.

The average value of 810 cfs for an observed downstream discharge seems large in comparison to the predicted value of 94 at zero shift. However, it should be kept in mind that 810 is somewhat less then two and one-half percent of the average of the absolute values of the observed discharges. This is well within measurement error for a single discharge measurement and surprisingly small when one considers the combined hydrograph technique that had to be used to convert the time-distributed measurements into the instantaneous values shown.

Driving Stage Timing

Errors in timing of driving stage gages can be caused by different speeds of the clocks driving the recorders, a missed punch, or erroneous starting or finishing times. They can be damaging because small errors are difficult to detect and, although it is apparently not so in the case illustrated here, small timing errors in driving stage can sometimes result in large errors in predicted discharge.

Figure 5 shows the effects of various time shifts of the stage record of the San Joaquin-gage on the normalized RMS stage error and normalized RMS discharge error. Figure 6 shows the effect of twelve miniute shift on the records produced at the three intermediate stations.

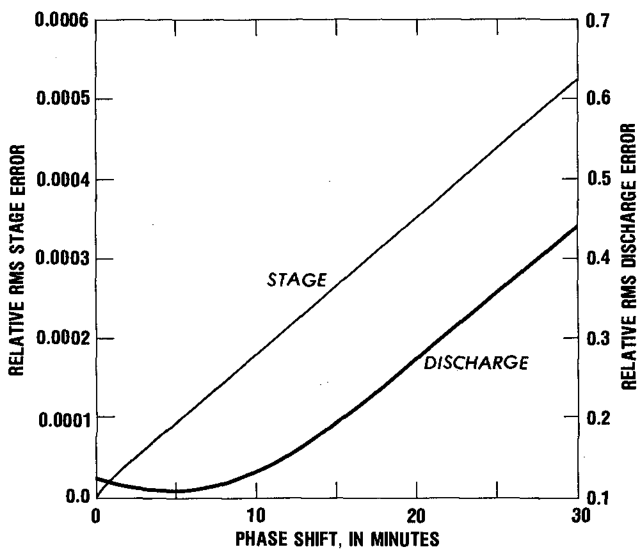


Figure 5. Effect of time phase shift of downstream gage on predicted stage and discharge.

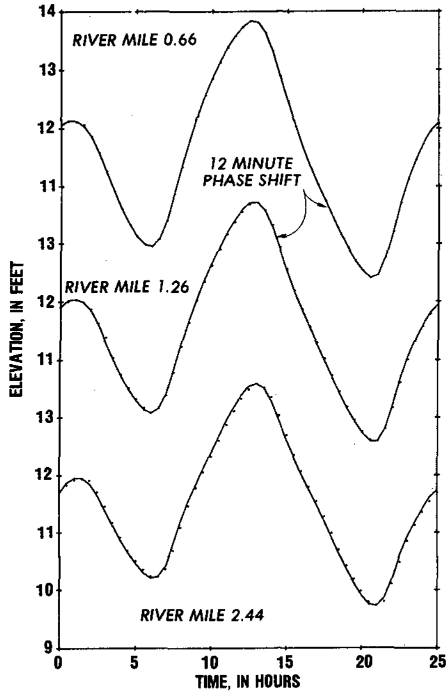


Figure 6. Effect of 12 minute time phase shift on predicted water surface elevations at the intermediate observation stations.

As can be seen from figure 6 the time shift is approximately lineraly distributed amongst the auxiliary stations. If we can make the big assumption that the clocks on the intermediate reocrders are accurate, the approximate value of the shift should be recoverable by linear extrapolation. For a real field situation where we can't be certain about the intermediate clocks either, but with conditons similar to the ones illusstrated here it should at least be possible to discern a shift as small as six minutes.

Figure 5 shows a minimum in the noramlized RMS discharge error at a value of time shift somewhere around five minutes. This indicates, first, that there was probably an undetected timing error of about this magnitude in the original stage records. It indicates, second, that this technique; that is, the construction of hypothetical shift versus normalized discharge error graphs, is a viable alternative to inspection of the intermediate stage records for finding possible timing errors.

In this example the increase in the nromalized RMS discharge error is not significant until the time shift becomes something between 12 and 18 minutes that is between two and three times the time computation step and around fifteen times the Courant time step. That this magnitude of shift is permissible should not, however, be taken as a general conclusion because a number of factors such as the rapidity with which the driving stages change, the phase difference between them, and the length of the reach being modeled determine the allowable value before serious errors result.

THE DISCHARGE - STAGE PROBLEM
Portland Harbor

The data selected to illustrate the discharge-stage problem comes from Portland Harbor, Oregon. As shown in figure 7, Portland Harbor consists of the lower 26 miles of the Willamette River from Willamette Falls to the Columbia River, the 21 miles of Multnomah Channel, and 56 miles of the Columbia River from Bonnieville Dam to Columbia City Oregon. Forty-nine cross sections in seven different channel reaches provided an adequate representation of harbor geometry for use in the simulation model. Input boundary conditions for running the model consist of the mean flow discharges for the Columbia River at Bonnieville and for the Willamette River at Willamette Falls and the Columbia River stage at Columbia City.

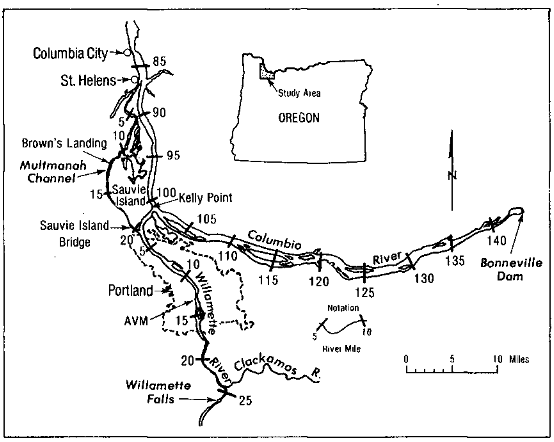


Figure 7. Portland Harbor, Oregon.

Cross section properties were obtained from recent soundings corrected to Columbia River datum and transcribed on a photomosaic of the Harbor by the Portland Distirict of the U.S. Army Crops of Engineers. Calibration data were collected during four short term intensive data collection periods characterizing a range of mean flow conditons on the two rivers. Referring to figure 7, measured discharges were available at the Acoustic Velocity Meter (AVM), Willamette River mile (RM) 1, and Sauvie Island Bridge. Observed water surface elevations were available at the AVM, Kelly Point, St. Helens, and the Vancouver Interstate bridge (Vancouver). The discharge at the AVM was obtained at hourly intervals from the calibrated accousitc flow meter, at RM 1 by boat measurement, and at Sauvie Island Bridge by bridge measurement, all by standard U.S. Geological Survey techniques. The water surface elevations were obtained from floats and recorded digitally at 15 minute intervals. Bennett (1975) describes this data set in greater detail.

Calibration Procedure

Bennett (1975) gives calibration curves for Chezy C versus characteristic discharge drawn for the seven segments of the Portland Harbor model. The calibraiton curves were derived from the data set described here and another set which covered a wider range of discharges but consisted only of obser-vaitions of stage and discharge at the AVM. In his study Bennett (1975) used the influence coefficient algorithm, an automatic optimization routine, to minimize an objective function which consisted of discharge prediction error normalized by local absolute mean discharge and water surface elevation prediciton error normalized by local mean depth. In contrast the object of the present study is to calibrate the model manually as though only stage information were available. The results of using the two calibration procedures will be compared in some detail, however, first it is necessary to describe the manual calibraiton procedure used.

The first step in the manual calibration procedure is the adjustment of the mean value of stage and the RMS stage error at Kelly Point by varying the Chezy coefficient C in the two channel segments of the Columbia River between Columbia City and Kelly Point. This can be done essenially independent of Multnomah Channel because the stage at Kelly Point is primarily dependent on the much larger discharge of the Columbia River. For a predominently unidirectional flow, aid in determining the amount of adjustment of the discharge coefficient required can be obtained by multiplying S_f by the length of reach and expanding the friction loss term as the first four terms in a power series in Chezy discharge coefficient. The resulting cubic equation in ΔC the required increment in the discharge coefficient, can be easily solved by syntheic division or Newton iteration.

For constant discharge, decreasing C has the effect of increasing the average predicted stage. A similar effect can be achieved by decreasing the cross-sectional area. Controlling the predicted stage by varying cross-sectional area has the advantage that C is left free, to a certain extent, to control the amplitude of the water surface elevation fluctuations. For the simulations reported here, the cross sectional-area of the Columbia River from Kelly Point to Columbia City was reduced by ten percent to allow

C to be large enough to force the amplitude of the stage fluctuations to that observed. This may be considered a questionable procedure because cross-sectional area is obtained from physical measurements made in the field. However, it should be realized that the cross sections used are at best random samples of those in the channel segments and as such may not be truly representative of the average conditions. Additionally, the field meaurements may be inaccurate or incomplete, therefore, it is reasonable to permit small changes in cross-sectional properties to expedite calibration.

The second step in the manual calibration procedure is the adjustment of the predicted stage at the AVM and Vancouver by varying the C's in the Willamette and in the Columbia above Kelly Point. Calibration for these two stages can proceed simultaneously yet independently because the water surface elevations at these two locations are functions of the independent discharge in the two rivers and of the stage at Kelly Point which has been established in the previous calibration step. Because of the large natural cross sectional areas in the Willamette below Willamette Falls, the stage at the AVM is insensitive to discharge coefficient for the lower discharges. For these discharges it is determined almost entirely by the water surface elevation at Kelly Point.

The final step in the manual calibration procedure was an attempt to minimize the RMS stage error at St. Helens by varying the discharge coefficient in Multonomah Channel. The results of this, however, were not necessarily good in terms of discharge prediction because calibration of Multonomah Channel is essentially a stage-stage problem. The water surface elevation at St. Helens is determined by those at Columbia City and Kelly Point independent of the discharge coefficient value in the Channel. The entire calibration process described here was completed for each data set in less then fifteen simulation runs, a considerable savings compared to the fifty to seventy-five required for the automatic optimization procedure described by Bennett (1975).

Results

For the four data sets, table 2 gives the component prediction RMS errors and the corresponding normalized RMS errors for simulations using the optimum parameters resulting from the calibration procedure described above. In the table, these errors are compared to the corresponding ones resulting from simulations using the optimum parameters derived in the study by Bennett (1975). As might be expected the normalized stage RMS errors for all four data sets are smaller for the parameters derived using the above procedure while the normalized RMS discharge errors are all larger.

Whether or not the increase in accuracy of discharge prediction evidenced by the earlier technique as compared to the one described here is sufficient to merit the additional cost of collection of the necessary discharge data is a question that has to be answered based on the purposes for which the model will be used. To add perspective, typical model behavior for the

Table 2. RMS component errors and normalized RMS errors for stage-only and combined objective function calibration procedures.

		Prediction error in feet LOCATION				Normalized Stage Error	Prediction error in cfs LOCATION			Normalized Discharge Errors
Date	Type	AVM	Kelley Point	St. Helens	Vancouver		AVM	River Mile 1.0	Sauvie Island Bridge	
5/13/76	Stage-only	.15	.11	.08	.14	.00511	4197	9823	1102	.448
	joint	.12	.18	.09	.20	.00683	3473	7877	2533	.344
8/1/73	Stage-only	.21	.18	.08	.15	.00605	5360	7636	1176	.444
	Joint	.20	.40	.08	.34	.01184	3460	5746	2509	.285
12/5/73	Stage-only	.21	.20	.05	.23	.00708	3602	1993	2161	.054
	Joint	.22	.22	.29	.26	.00911	3626	3065	510	.039
6/12/24	Stage-only	.06	.03	.16	.09	.00280	1523	3660	900	.088
	Joint	.22	.58	.08	.14	.00723	1502	1446	2779	.060

parameters derived from the two calibration procedures is shown by the stage hydrographs of figure 8 and the discharge hydrographs of figure 9. The hydrographs were computed using boundary conditions and optimum parameters for the data set of 5-31-73. Casual inspection detects little difference between the agreement of either set of predictions with the observed data.

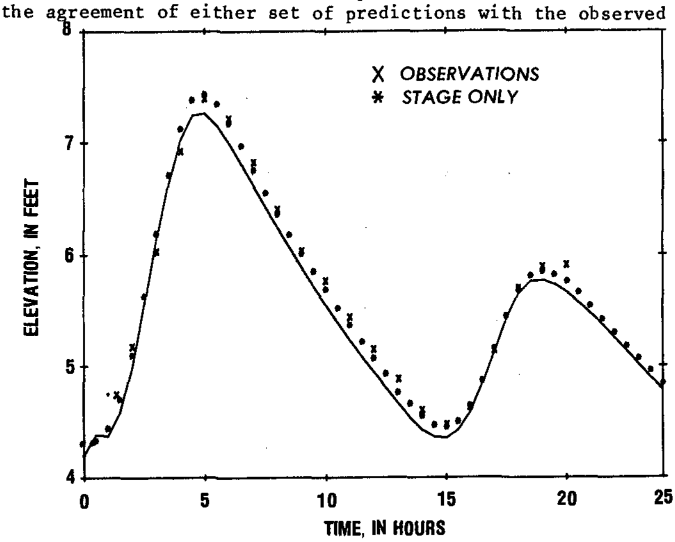


Figure 8. Predicted and observed stage hydrographs at Kelly Point for 5/31/73. Solid line curve reproduced from Bennett (1975).

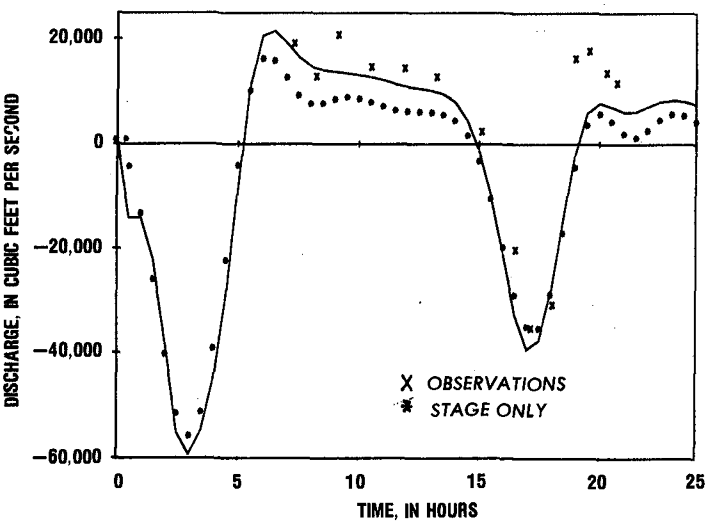


Figure 9. Predicted and observed discharge hydrographs at River Mile 1 for 5/31/73. Solid line curve reproduced from Bennett (1975).

Besides the accuracy of stage and discharge estimation another point which must be considered is the consistency of the parameter estimates obtained from the various data sets. Table 3 shows the C values for each of the seven channel segments of the harbor model. For the first three

Table 3. Channel segment discharge coefficients in $ft^{\frac{1}{2}}/sec$.

Channel segments Description	Date			
	5/31/75	8/1/73	12/5/73	6/12/74
Willamette: Falls to Multonomah Channel	90	90	90	90
Willamette: Multonomah Channel to Columbia	90	90	90	90
Multnomah Channel	85	85	85	85
Columbia: Bonneville Dam to Vancouver	60	60	60	60
Columbia: Vancouver to Willamette	195	200	96	120
Columbia: Willamette to Multnomah Channel	102	102	102	95
Columbia: Multnomah Channel to Columbia City	102	102	102	95

segments the values are the same for all data sets. The C vlaue for the fourth segment was not adjusted in calibration because of lack of stage data above Vancouver. The optimum C value for the fifth segment behaves eratically in a fashion which can not be explained. Finally, the C values for the 6th and 7th segments are essentially constant. This is much more consistent behavior than demonstrated by the optimum C values obtained by Bennett (1975).

The reasons for the consistent behavior bear some discussion. First the AVM stages are not sensitive to C at the lower discharges, therefore, the C values for the first two segments are essentially set by the 12/5/76 data set. Second, Multnomah Channel is really a stage-stage problem, and should be fine-tuned using discharge information. This is beyond the scope of the technique being investigated. Third, the C values in the lower Columbia apparently decrease slightly, but only slightly with increasing discharge. Finally, as mentioned above, no explanation has been found for the behavior of C values in the short segment of the Columbia from Vancouver to the Willamette.

As explained earlier the model's intended use dictates acceptable prediction accuracy. However, Table 2 and figures 8 and 9 indicate that, with the possible exception of Multnomah Channel, the Portland Harbor model could have been calibrated acceptably for most purposes in the complete absence of discharge measurements. Whether or not detailed information concerning Multnomah Channel behavior is important in the final study will dictate how much attention should be paid to its calibration. If its behavior must be predicted accurately Multnomah Channel should be calibrated as in the stage-stage problem; that is, with discharge information collected at the extremes of ebb and flood flow.

Conclusions

Calibration of a stage-stage model requires the use of observed discharge. The best time for making the required discharge measurements is near the peaks of maximum and minimum downstream flow. Like variations in the Chezy discharge coefficient, errors in cross-sectional area can not be detected using auxiliary stage observation stations. On the other hand, gage datum discrepancies on the order of 0.05 ft and timing errors in one of the driving stage records on the order of 6 to 12 min. can be detected by comparing the predicted and observed stage records from such stations.

In most water quality modeling situations, discharge-stage models can be satisfactorily calibrated using only stage observations. The data used should cover as nearly as possible the range of mean flow discharges to be encountered in prediction.

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