Appendix D
River Modeling - Lessons Learned

Section I
Defining River Geometry

D-1. Introduction

There is much similarity in the geometric data requirements of various river hydraulics models. This appendix describes common requirements, points out some differences between models, and presents methods that have been successfully used to model many different situations.

D-2. Geometric Data

a. River geometry. It is not feasible to replicate all topographic, land use, vegetative cover, soils types, etc. details in a digital representation of a river system at high resolution for hydraulic analyses. Therefore, key hydraulic features of the channel and floodplains must be identified by the engineer and included in any digital model. It is relatively easy to detect significant geometric variations of the floodplains because they are usually visible on maps (e.g. USGS quads.). The principal features of channel (i.e. in-bank) geometry are harder to detect because they usually cannot be seen on maps; their approximate locations can be found, however, with the understanding of geomorphology. Structures may constrict the flow, changing the hydraulics of the stream. The key to developing representative geometric data is the definition of the features that play significant roles in both the river's behavior and the numerical model's performance.

b. Cross section locations. Cross sections are located to serve two major purposes in river modeling: (1) to define the geometry of the river and floodplain, including the flow boundaries; and (2) to satisfy the computational accuracy requirements of the analytical method being used. With respect to the latter, for example, most river hydraulics numerical models provide interpolated computation points based on the properties of the input cross sections.

c. Unsteady flow vs. steady flow requirements. Steady flow models and unsteady flow models have different cross-sectional requirements. A steady flow analysis requires definition of only the active flow area (that is, the area which conveys flow), unless storage-outflow data is being developed for hydrologic routing. Unsteady flow simulation requires definition of both the active flow area and the inactive, or storage, areas. These storage areas are important because for most rivers, during flood flows, the speed of the flood wave is determined largely by storage rather than wave dynamics. Because steady flow cross sections may only define active flow areas, they may not be sufficient for unsteady flow analysis. Modifications to the cross sections may be needed to add storage.

(1) Another difference is the range of flows to be simulated. A steady flow model is often used to calculate water surface profiles for flood events, which are generally out of bank. In that case, it is seldom used for low flow, so the channel geometry may not need to be precisely defined. Often unsteady flow models, especially forecast models, are used to simulate a wide range of flows; therefore, the cross sections must include both low flow and overbank flow areas. An exception is a dam break model which, because of the magnitude and depth of flow, does not require detailed channel cross sections.

d. Pool-riffle sequence. A river generally forms a sequence of deep pools and shallow riffles. During low to moderate flow, the relatively high invert elevation of the riffle controls the water surface profile, backing water upstream. Pools and riffles are associated with meandering streams in which the flow is predominantly subcritical (although flow can be supercritical at the riffle). The pools occur on the outside of bends and the riffles occur in the straight sections connecting the bends. A pool-riffle sequence is shown in Figure D-1. The cross section through the pool is triangular shaped with maximum depth occurring toward the outside of the bend. The region on the inside of the bend, called the point bar, is typically exposed during low flow. The sections in the riffle tend to be rectangular in shape and much more shallow than those in pools. During low flow, the constricted cross sections at the riffles control the flow profile and the river becomes a sequence of small pools. As the flow increases, the impact of the riffles diminishes, becoming negligible at bank full flow.

(1) Because most data for river models is acquired to simulate larger flows, the pool-riffle sequence may not be included. Therefore, pool and riffle cross sections may be found at random throughout a cross section data file. The result is data which either simulates low flow at unrealistically low stages or yields unstable computations at low flow. The latter is caused by supercritical

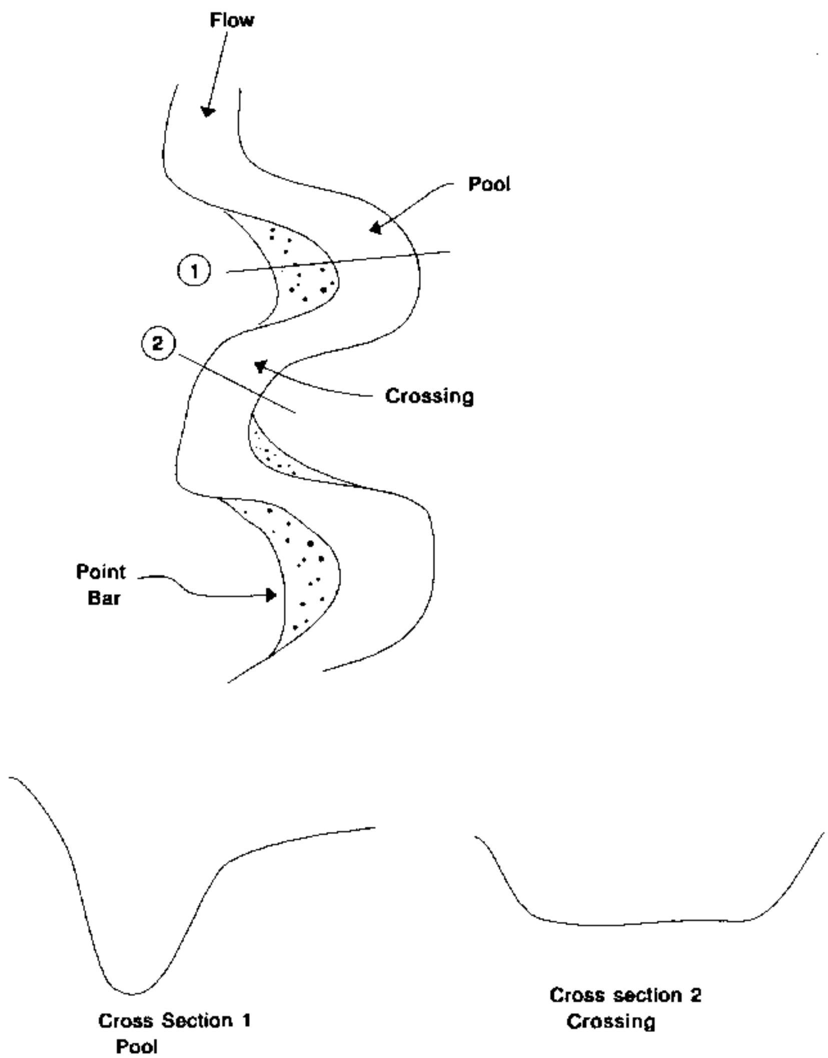

Figure D-1. Pool-riffle sequence in a river

flow occurring at the riffle. The supercritical flow could be real, or it could be caused by erroneous low tailwater resulting from a missing downstream riffle. The most common solution to both problems is to increase Manning's *n* for low flow, thereby raising the water surface. A better solution is to locate the riffles and obtain sections at them.

(2) Cross sections to be used for forecasts should include riffles, especially if detailed navigation soundings are available.

e. Active flow area. The entire width of the floodplain seldom actively conveys flow. The floodplain generally has irregular boundaries; it is constricted by landforms, roadway crossings, levees, etc. For most situations, the active flow area will not expand to the entire width of the floodplain before contraction into another constriction. The modeler must decide on the limits of flow and should draw these limits on maps. As a rule of thumb, flow contracts at a rate of 1 on 1 and expands at a rate of 1 (transverse) on 4 (streamwise). This rule can be modified depending upon the sinuosity of the stream and valley. Figure D-2 shows the limits of flow defined in a reach of the Salt River. For this reach, flow was limited on the left by a terrace and a levee shown as a solid line. On the right, flow was limited by a levee and the flow's ability to expand.

f. Storage areas. Storage areas are the regions of the floodplain outside of the active flow area. They may be ignored for a steady flow analysis but are crucial to unsteady flow analysis. Because of the irregularity of the floodplain boundaries, particularly near tributary junctions, the storage indicated by the cross sections is always less than the total actual storage of the floodplain. This underestimation of storage can cause a computed flood wave to arrive too early; consequently, the geometric data may need to be adjusted during calibration.

g. Method of specifying wetted perimeter. The wetted perimeter is defined as the length of the cross section along which there is friction between the fluid and the boundary. It is used to calculate the hydraulic radius which appears in the Manning and Chezy formulas. The hydraulic radius is

$$R = \frac{A}{W_p} \qquad \text{(D-1)}$$

where

A = active flow area
W_p = wetted perimeter

(1) Several models, most notably DAMBRK (Fread 1988) and DWOPER (Fread 1978), approximate the wetted perimeter as the topwidth. The topwidth is always less than the wetted perimeter, but if the width to depth ratio is greater than 10, this assumption is reasonable. Still, the conveyance of a section computed using this assumption will be greater than the conveyance using the true wetted perimeter. For narrow channels, with small width to depth ratios, the error from this assumption increases. Figure D-3 shows the relation between width-depth ratio and the increase in conveyance by assuming that the wetted perimeter is equal to the topwidth for a rectangular channel.

(2) The increased conveyance can be offset by increasing Manning's *n* values. However, the *n* values chosen for a steady flow model or an unsteady flow model that does not use the topwidth approximation will not be appropriate for models that do use the topwidth approximation. The engineer must be aware of how geometric and other data are used in any particular numerical model to properly prepare input data and interpret model results.

D-3. Developing Cross-Sectional Data to Define Flow Geometry

Cross-sectional data are used to determine the conveyance and storage of the river channel and overbank areas. It is customary to obtain the boundary geometry by measuring ground surface profiles (cross sections) perpendicular to the direction of flow at intervals along the stream and measuring the distances (reach lengths) between them. Use of digital terrain models is also appropriate.

a. Flow lines. For floodplain studies, flow lines should be sketched on a topographic map to estimate flow direction and determine cross section orientations.

b. Topographic maps. Cross sections of the overbank areas may be obtained directly from an accurate topographic map, if one is available. Otherwise, cross sections must be obtained by field or aerial surveys. It is

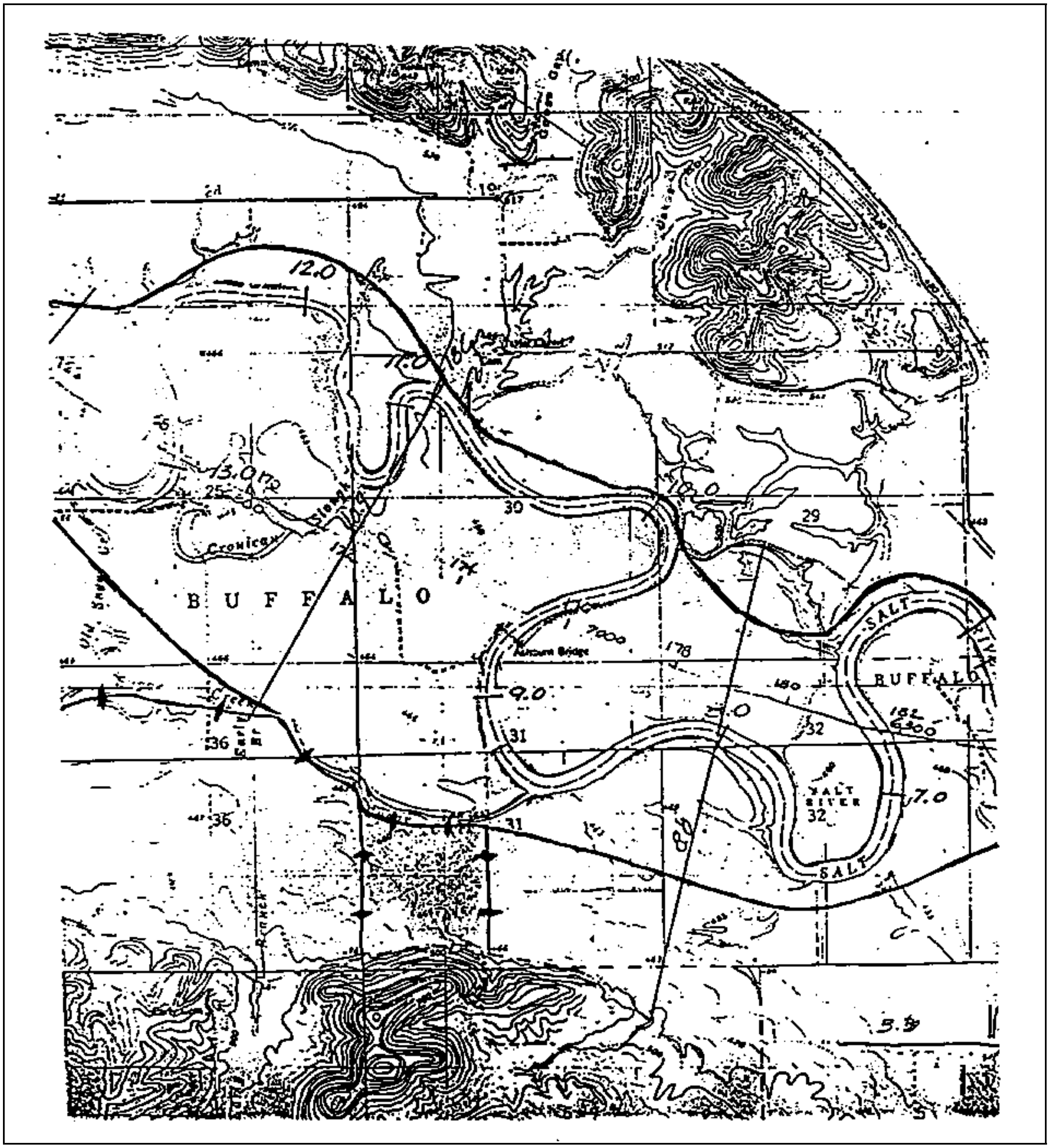

Figure D-2. Active flow limits along the Salt River

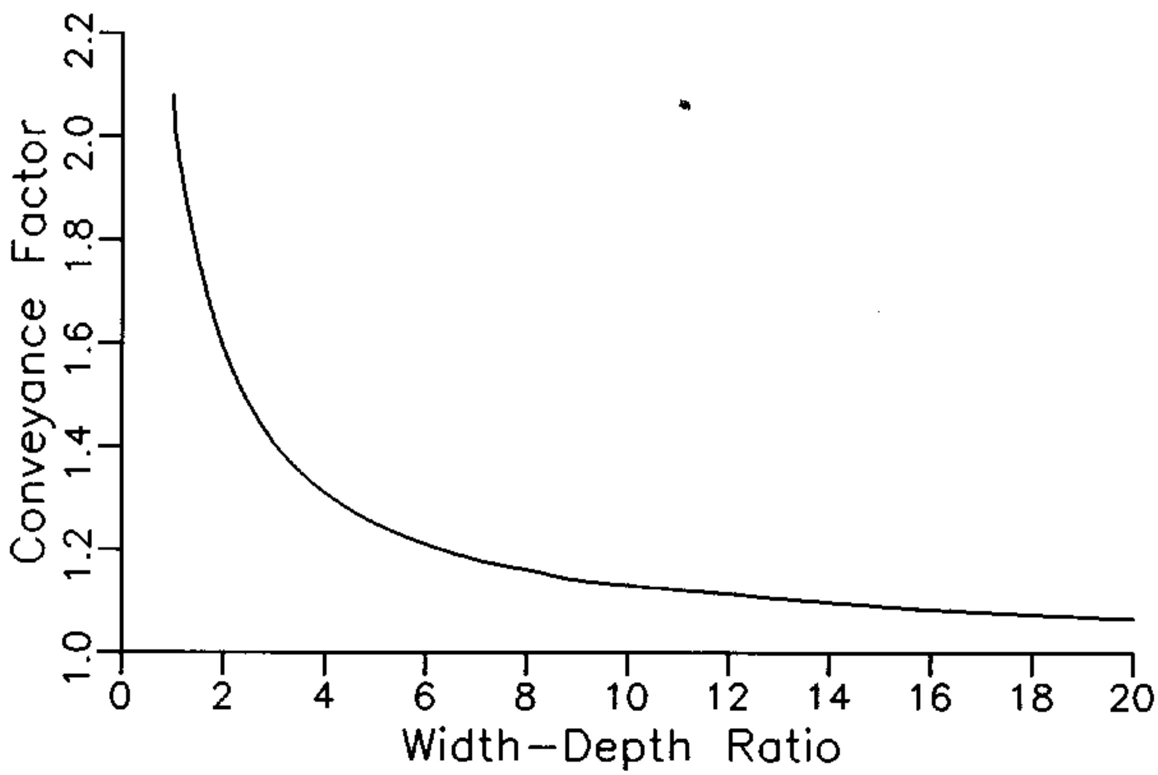

Figure D-3. The relationship between width-depth ratio and the increase in conveyance caused by assuming that the top width is the wetted perimeter for a rectangular channel

necessary to obtain the shape and slope of the channel from soundings of the river channel unless an accurate hydrographic survey is available. The thalweg of the stream should be located on a map so that cross sections may be identified by stationing or by river mileage measured along the thalweg. This also will facilitate measuring the reach lengths between cross sections.

c. Subdividing cross sections based on roughness variation. Cross sections obtained for water surface profile computations on rivers at flood stage should be divided into two or more segments that have different values of the friction coefficient *n*. These consist of the main channel areas, with relatively low value(s) of *n*, and one or more overbank areas which, because of vegetation and other obstructions to flow, generally have higher *n* values. Aerial photographs are valuable supplements to topographic maps and surveyed cross sections for determining the extent of vegetation and portions of cross sections having common values of *n*.

d. Subdividing cross sections based on depth variation. Parts of a cross section having the same roughness should be subdivided to reflect abrupt changes in depth. The effect of roughness variation tends to be reduced as the depth of flow increases.

e. Checklist for locating cross sections. If cross sections are located according to the criteria discussed in the preceding sections and the list of locations that follows, a reasonable initial definition of river and floodplain geometry should be acquired. Cross sections should be located:

(1) At all major breaks in bed profile.

(2) At minimum and maximum cross-sectional areas.

(3) At points where roughness changes abruptly.

(4) Closer together in expanding reaches and in bends.

(5) Closer together in reaches where the conveyance changes greatly as a result of changes in width, depth, or roughness.

(6) Between cross sections that are radically different in shape, even if the two areas and conveyances are nearly the same.

(7) Closer together where the lateral distribution of conveyance changes radically with distance.

(8) Closer together in streams of very low gradient which are significantly nonuniform, because the computations are very sensitive to the effects of local disturbances and/or irregularities.

(9) At the head and tail of levees.

(10) At or near control sections, and at shorter intervals immediately upstream from a control (subcritical flow).

(11) At tributaries that contribute significantly to the main stem flow. Cross sections should be located immediately upstream and downstream from the confluence on the main stream and immediately upstream on the tributary.

(12) At regular intervals along reaches of uniform cross section.

(13) Above, below, and within, bridges.

D-4. Developing Cross-Sectional Data to Satisfy Requirements of the Analytical Method

Some computational schemes treat each cross section as being located at the midpoint of a reach and use that single cross section to represent the entire reach for calculating energy losses. Other schemes (e.g., HEC-2) use cross sections to define hydraulic break points in the geometry, and properties of adjacent cross sections are averaged to calculate losses between them. Again, the engineer needs to be cognizant of the computational approach and assumptions of a particular model to properly prepare data.

a. Location of cross sections to represent adjacent reach conditions. Cross sections should be representative of the reaches adjacent to them, and located close enough together to ensure accurate computation of the energy losses. If the average conveyance between cross sections is used to estimate the average energy slope, then the variation of conveyance should be linear between any two adjacent cross sections.

b. Cross section location based on slope conditions. Cross sections should be located such that the energy gradient, water-surface slope, and bed slope are all as parallel to each other between cross sections as is pragmatic. If any channel feature causes one of these three profiles to curve, break, or not be parallel to the others, the reach should be further subdivided with more sections.

c. Spacing of cross sections on large rivers. On large rivers that have average slopes of 2 to 5 feet per mile or less, cross sections within fairly uniform reaches may be taken at intervals of a mile or more.

d. Spacing of cross sections in urban areas and on small streams with steep slopes. More closely spaced cross sections are usually needed to define energy losses in urban areas, where steep slopes are encountered, and on small streams. On small streams with steep slopes it is desirable to take cross sections at intervals of 1/4 mile or less.

e. Maximum reach lengths (distances between cross sections). One investigation (Barr Engineering Company 1972) recommends maximum reach lengths (measured down the valley) of: (1) 1/2 mile for wide floodplains and slopes less than 2 feet per mile, (2) 1,800 feet for slopes less than 3 feet per mile, and (3) 1,200 feet for slopes greater than 3 feet per mile.

f. Maximum reach lengths to achieve consistency between conveyance averaging methods. A profile accuracy study (U.S. Army Corps of Engineers 1986) used maximum reach lengths of 500 feet to compute consistent water surface profiles using different conveyance averaging methods.

g. USGS reach-length guidelines. A U.S. Geological Survey report (Davidian 1984) suggests that:

(1) No reach between cross sections should be longer than 75 - 100 times the mean depth for the largest discharge, or about twice the width of the reach.

(2) The fall of a reach should be equal to or greater than the larger of 0.5 foot or the velocity head, unless the bed slope is so flat that the above criterion holds.

(3) The reach length should be equal to, or less than, the downstream depth for the smallest discharge divided by the bed slope.

D-5. Reviewing Computed Results to Determine Adequacy of Cross-Sectional Data

The criteria presented in the preceding sections provide guidance for the location of measured cross sections and should help the engineer understand anomalies in computed profiles if not enough, or poorly located, cross sections are used. The focus is both on modeling the physical characteristics of the study reach and on meeting requirements of the method of analysis to obtain an accurate estimate of the energy losses. After the initial data are developed and the model executed, a review of the computed results is required to ensure that the spacing of cross sections is adequate. The following guidelines may be used to determine if additional cross sections are needed. Ideally, these would be surveyed in the field; however, interpolated sections or hydraulic parameters are frequently used.

a. Velocity change. Transitional cross sections should be added if the velocity change between cross sections exceeds ±20 percent. The accuracy of integration of the energy slope - distance relation is improved by use of relatively short reaches.

b. Energy slope change. Change in energy slope can also be used as a basis to evaluate cross section spacing. If the slope decreases by more than 50 percent, or increases more than 100 percent, the reach length may be too long for accurate loss calculations.

c. Flow distribution. The distribution of flow from cross section to cross section should be reviewed to ensure reasonable flow transitions. For example, HEC-2 allows flow in three flow elements; the channel and the two overbanks. A one-dimensional model, such as HEC-2, does not recognize the effects of geometry changes between cross sections on flow properties, so the program user must.

d. Conveyance ratio. It is suggested that the ratio of conveyances (K_1/K_2) between two adjacent cross sections satisfy the criterion: $0.7<(K_1/K_2)<1.4$ (Davidian, 1984). Shorter distances between cross sections may be needed, particularly in long reaches, if this criterion is not met. This criterion may be relaxed near structures such as bridges.

D-6. Other Considerations in Developing Cross-Sectional Data

Additional considerations for cross sections include:

a. End station elevations. The maximum elevation of each end of a cross section should be higher than the anticipated maximum water surface elevation.

b. Local irregularities in bed surface. Local irregularities in the ground surface such as depressions or rises that are not typical of the reach should not be included in the cross-sectional data.

c. Bent cross sections. A cross section should be laid out on a straight line if possible. However, a cross section should be bent if necessary to keep it perpendicular to the expected flow lines.

d. Avoid intersection of cross sections. Cross sections must not cross each other. Care must be taken at river bends and tributary junctions to avoid overlap of sections.

e. Inclusion of channel control structures. Channel control structures such as levees or wing dams should be shown on the cross section, and allowances in cross-sectional areas and wetted perimeters should be made for these structures.

D-7. Modeling Flow Geometry at Structures

a. Bridges and culverts. Roadway embankments restrict flow to narrow bridge and culvert openings when the upstream water surface is below the crown of the roadway. When the flow overtops the roadway, the embankment acts as a spillway. There are three elements to modeling a roadway crossing:

(1) Contract the active flow area into the bridge or culvert opening. Generally the flow contracts at an approximate rate of 1 to 1 although the boundaries of the

floodplain and stream meanders may interfere. The wetted area outside of the active flow area is storage. The top of the contraction should be set at the top of the roadway. Unsteady flow models solve the momentum equation; therefore, no expansion/contraction losses (commonly called eddy losses) need be applied through the contraction when using an unsteady flow model.

(2) Compute the head loss for flow through the bridge or culvert and over the roadway crown (commonly called weir flow). The structure itself can be modeled as an interior boundary condition. It is inconvenient to include the equations for bridges, culverts, and weirs directly in an unsteady flow program. One common approach (U.S. Army Corps of Engineers 1991b) is to develop a set of free and submerged rating curves for the structure. An example is shown in Figure D-4. The rating, which considers all possible flow conditions including pressure flow, inlet control, outlet control, open channel flow and weir flow, is usually computed in a preprocessor for an unsteady flow model. For perched bridges, or for bridges where the roadway is not overtopped, conveyance-based calculations, such as in HEC-2, may be preferable to the family of rating curves. For the normal bridge method, the bridge piers and deck are defined with the cross section data thereby reducing the conveyance.

(3) Expand the flow downstream from the bridge constriction. The cross section downstream from the bridge is usually a repeat of the upstream cross section. The flow will usually expand at an approximate rate of 1 (transverse) on 4 (streamwise). Some modelers, however, define the full flow cross section much closer to the bridge location.

b. Navigation dams. A navigation dam creates two flow conditions. During low flow the dam impounds a pool upstream, maintaining a minimum depth for navigation. During high flow, the gates are opened and the

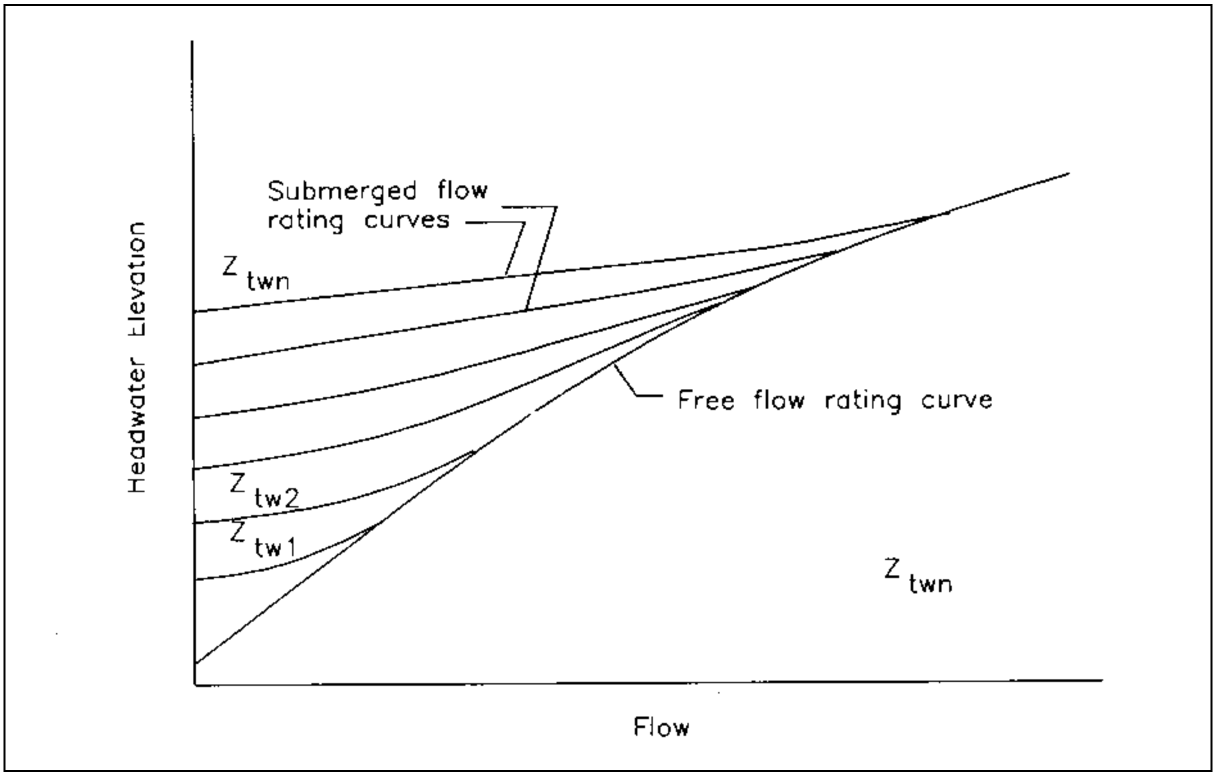

Figure D-4. A free flow rating curve and a set of submerged rating curves

river returns to a free flowing state. The dam generates a swell head upstream. Navigation dams can be found on nearly every major river in the United States and play a critical role in determination of the water surface profile and celerity of waves during low flow. Because of the pools and lack of bed friction during low flow, waves move quickly, approaching the speed of a gravity wave. River regulators have observed that the travel time from Lock and Dam 10 to St. Louis on the Mississippi River, a distance of 780 miles, is 2 days during low flows. In contrast, during a flood, the travel time is 10 days.

(1) Navigation pools are regulated to maintain a control point at or above a certain stage. The control points are located either at the dam or in the pool. The latter is called hinge pool operation. Figure D-5 shows Peoria Lock and Dam pool and tailwater stage hydrographs. Because the control point is located at the dam, the pool is maintained at a constant level until tailwater drowns the pool. Figure D-6 shows the pool, tailwater, and control point stage hydrographs for Lock and Dam No 26. The control point is maintained at a constant level by fluctuating the pool elevation until the tailwater submerges the pool.

c. Controls. Controls are natural or artificial structures which determine the upstream water surface profile. A control can be a dam, a falls, a rock outcrop, a drop structure, etc. The accuracy as well as the stability of a numerical model computation depends on the proper location and modeling of controls. The control may prevent supercritical flow upstream. For the Passaic River Basin in New Jersey, numerous low water dams, falls, and rock outcroppings control the water surface for low flow. Figure D-7 shows the maximum water surface elevation along the Passaic River. Note the jumps in the profile at the small dams.

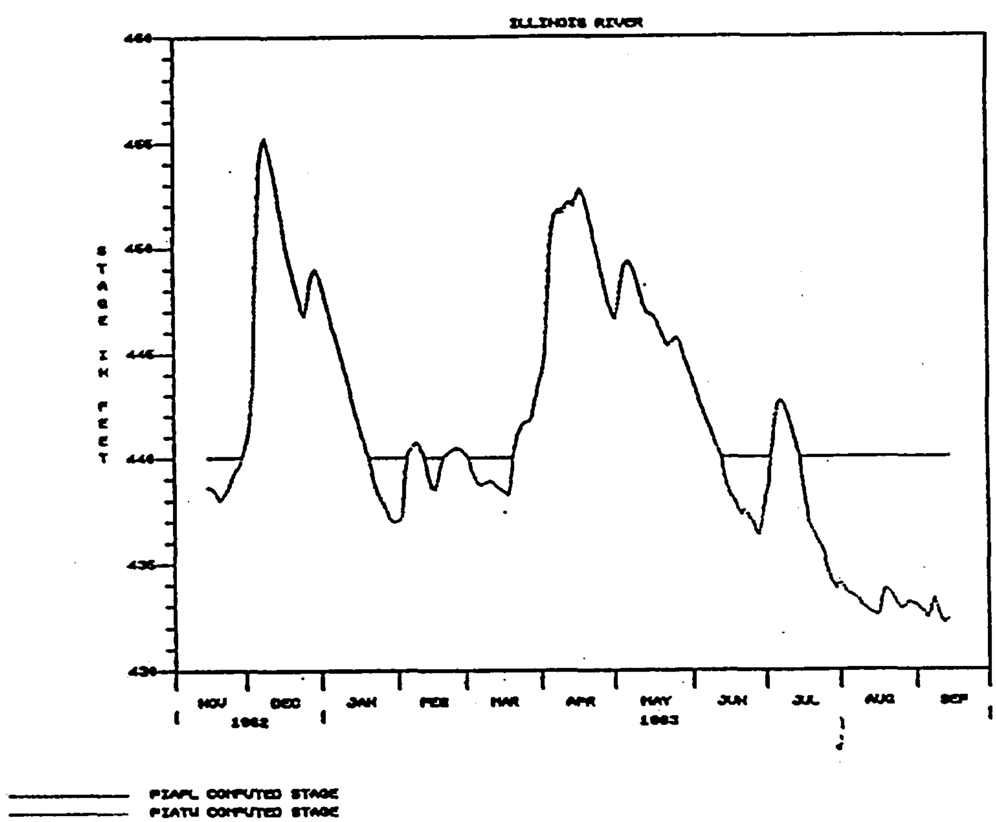

Figure D-5. Peoria Lock and Dam pool and tailwater hydrographs

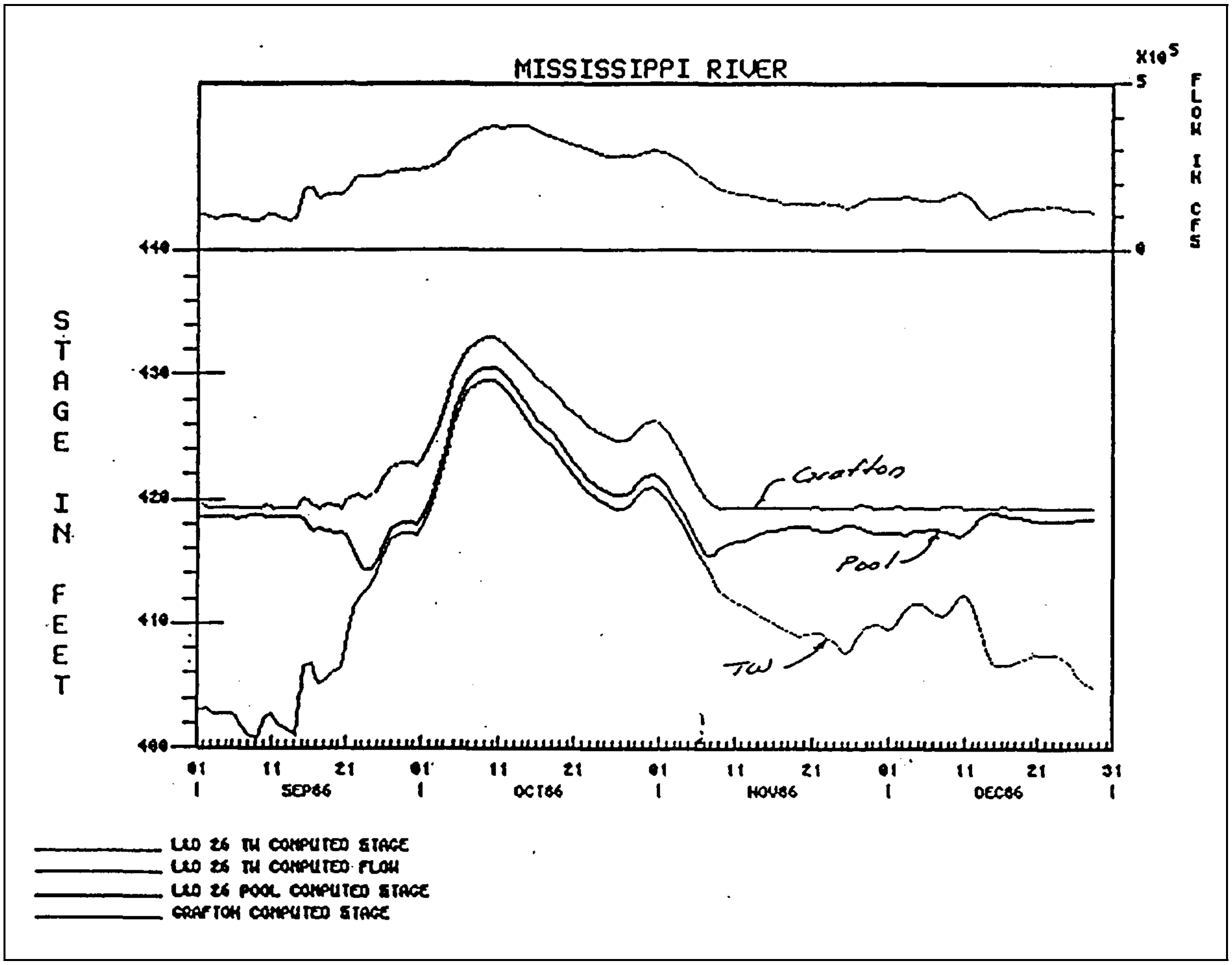

Figure D-6. Pool, tailwater, and control point hydrographs at Lock and Dam 26

d. Dikes. Dikes (also called wing dams or jetties) are narrow peninsulas of rock or timber built out from the river banks into the flow (Figure D-8). Dikes concentrate the flow in a section of the river, deepening the navigation channel for low flow and diverting flow away from the chutes around islands. The dikes are designed to create a more efficient and deeper navigation channel during low flow. There are at least three problems encountered when modeling dikes:

(1) Locating the dike field. Dikes are not marked on USGS quadrangle maps and sometimes are marked only on navigation charts. Cross sections will usually be located at a dike by chance. Check with the district potomologist for the location of dike fields.

(2) Modeling the effect of dikes. During low flow, the discharge is concentrated in the center of the channel inside the dike field. If the cross sections do not include the dike, the active flow area will be the full width of the channel (Figure D-8), which is not correct. The flow velocity will be too low resulting in slow wave celerities. The general trend in the water surface may be correct; but, the model will incorrectly simulate the timing and shape of small waves. For a forecast model, where simulation of the full range of flow is important, these small waves are critical because a poor simulation may detract from the credibility of the model or prevent the model from being used to regulate locks and dams. The only solution is to redefine the cross section invert for

active flow and augment storage, thereby modeling the effect of the dike.

(3) Nonstationarity of cross sections. Cross sections surveyed after the installation of a dike field will become increasingly inaccurate as time passes and the channel invert deepens. Thus, the low flow *n* values may need to be continually changed to adequately reproduce stages or the cross sections resurveyed.

e. Levees. Levees are earthen embankments which prevent floodwaters from inundating the floodplain. In steady flow modeling, levees are represented by constricted cross sections. For unsteady flow modeling, there is the constricted cross section, but also the added impact if the levees fail and the interior storage fills. When the flow is contained by the levees, their impact is usually a higher water surface. However, if the levees fail, the protected area becomes available for storage, cutting off a portion of the hydrograph. Figure D-9 shows the stage hydrographs for the Illinois River at Peoria from the 1 percent chance exceedance event with and without levee failures downstream. For this event, the failures decapitated the flood crest. Figure D-10 shows the maximum water surface profile for the 1 percent chance exceedance event. The failure of the levees reduced the flood profile by about 2 feet.

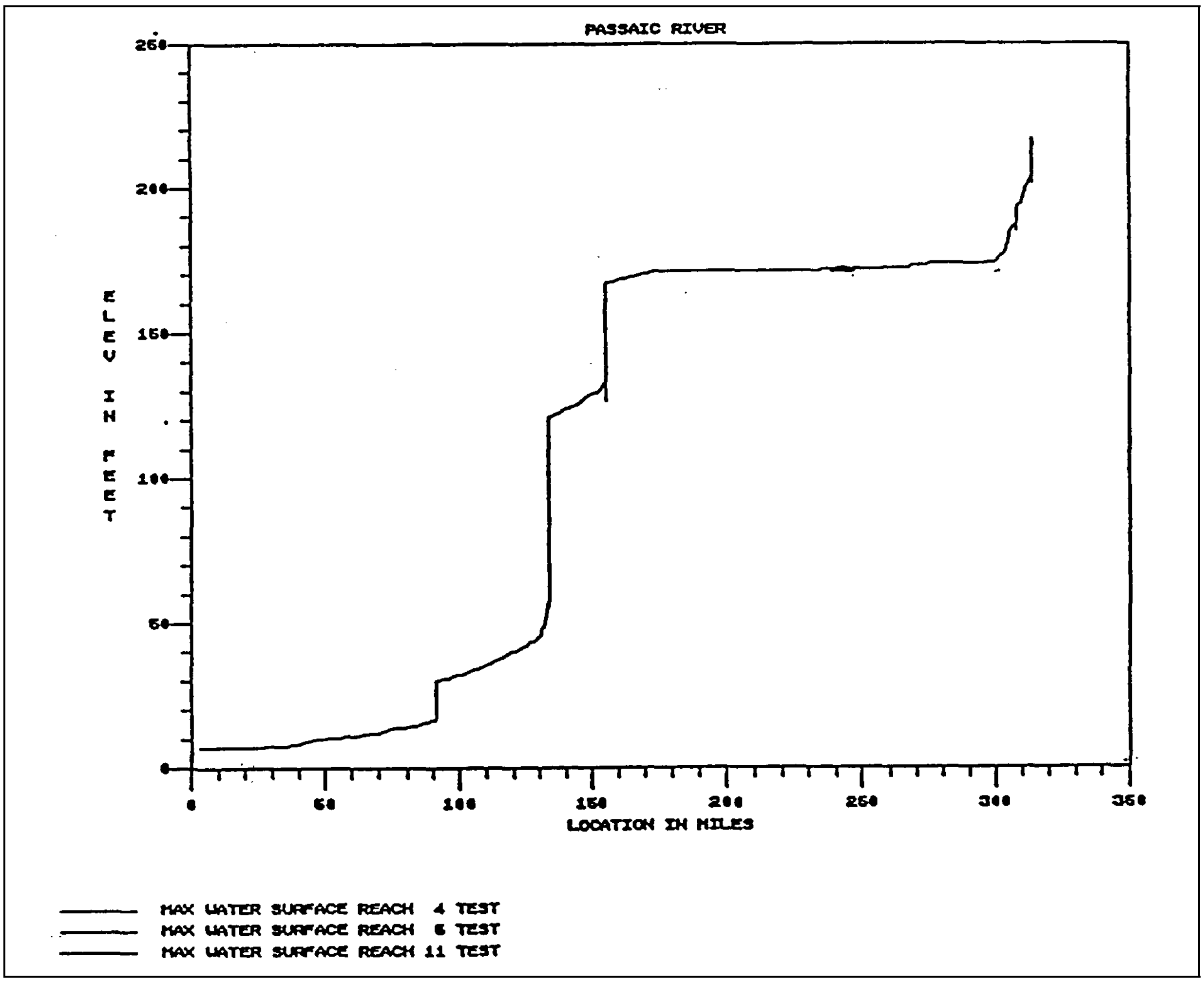

Figure D-7. The maximum water surface for a synthetic event on the Passaic River in New Jersey

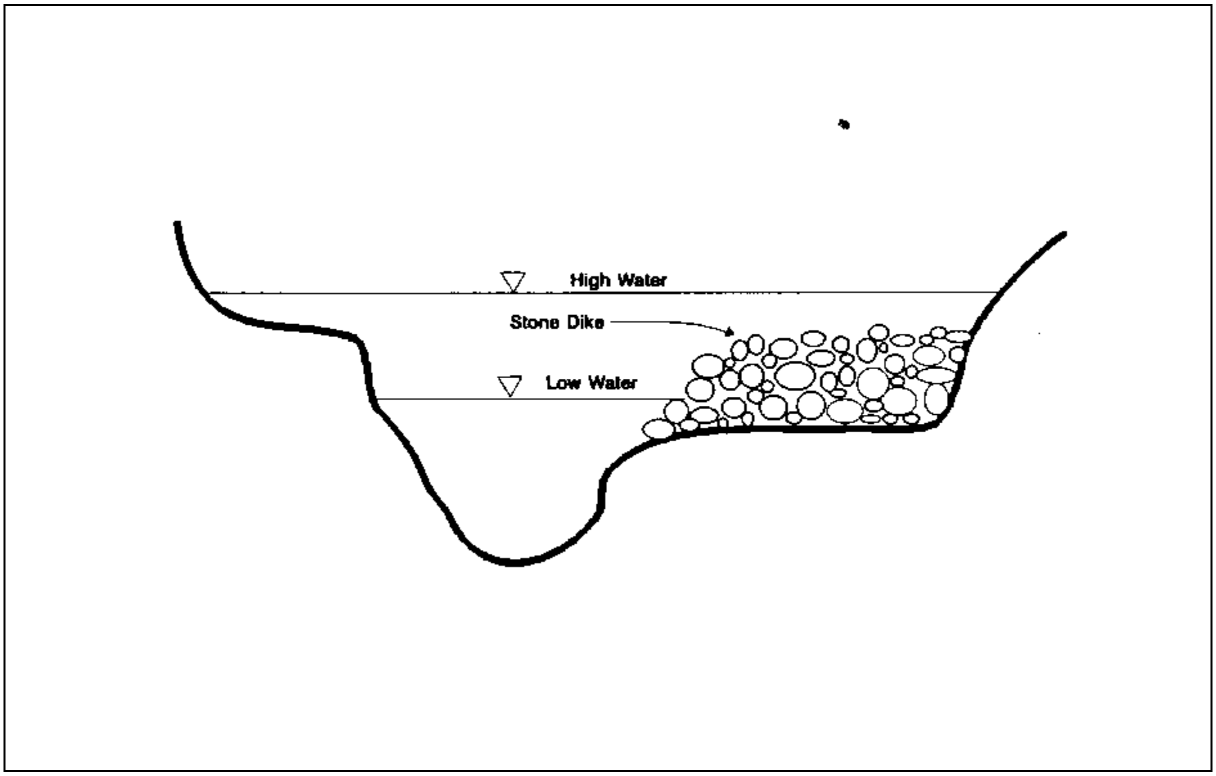

Figure D-8. River cross section including a navigation dike

(1) The failure of a levee is a dynamic event which can only be confidently simulated with unsteady flow. A levee breach typically forms at a low spot in the levee crown. When the levee fails, there is a draw down of the water surface at the breach (Figure D-11). The water from upstream is accelerated toward the breach. The flow downstream of the breach may reverse direction. Figure D-12 shows the flow hydrograph with and without levee failure at Peoria. The large flow spike resulted from the failure of a levee about 10 miles downstream. Figure D-13 shows flow hydrographs for the Illinois River at Kingston Mines, 13 miles downstream of Peoria. The flow hydrographs show the reversal in flow from the breach 3 miles upstream.

(2) The reproduction of levee failures may govern the success or failure of model calibration and the credibility of a forecast model. If a levee fails during an event being used for calibration (unknown to the analyst), it may be impossible to correctly reproduce the event and, thus, calibrate the model without compromising the integrity of the calibration. Moreover, for a forecast model, levee failures strongly influence the quality of the forecast. In October 1986, the failure of 22 levees on the Missouri River attenuated 80,000 cfs from the flood crest between Hermann and St. Charles on the Mississippi River. Without the correct simulation of these levee failures, the forecasted crest at Lock and Dam 26 tailwater and St. Louis would have been about 2.5 feet higher.

D-8. Developing Reach Length Data

Reach lengths are measured along the flow lines between cross sections. For HEC-2, three lengths are used to define the channel and the two overbank flow paths. A single discharge-weighted reach length computed from these is used by the program to determine the energy loss between cross sections.

a. Channel reach lengths. Channel reach lengths are usually measured along the stream thalweg, but they should be measured along a line through the estimated

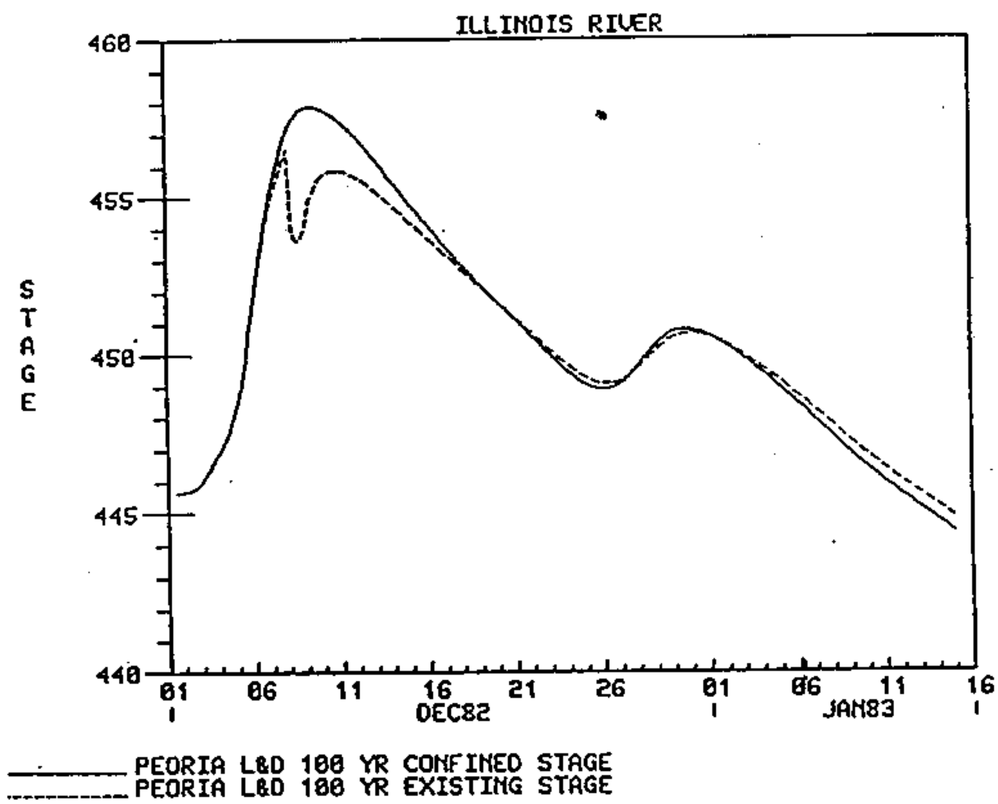

Figure D-9. Stage hydrographs for the Illinois River at Peoria with and without failures

center of mass of the flow if that line differs materially from the stream thalweg. In many cases, computed and estimated *n* values for overbank and channel flows are based on the same reach lengths for the overbank and channel areas. Defining the channel length based on the low flow channel course assumes that the flow will always follow the channel, even for flood flows.

b. Overbank reach lengths. If overbank flow follows a considerably shorter path than the main channel flow as in the case of a stream that meanders through the floodplain (in other cases it may be longer), and if computed or estimated *n* values used in the water surface profile computations do not include the effects of overbank reach lengths, then separate lengths should be measured for overbank and channel areas.

(1) Overbank reach lengths are measured along the center of mass of the flow element. Because this will vary for each discharge, the estimate should be based on the most important flood profile being analyzed.

(2) If the overbank cross section area is triangular in shape, with the deeper portion near the channel, the center of mass for the overbank area would be located one-third the distance away from the bank. Under these conditions, if the floodplain is sketched on a map, an overbank reach length can be scaled by measuring the length of the flow line located one-third of the distance from the channel bank to the end station.

(3) The expected flow path should be sketched on a map along with the locations of the cross sections. The computed results can then be evaluated in comparison with the expected flow path. Do the computed results conform with the expected? If not, the data may need to be adjusted based on the computed results.

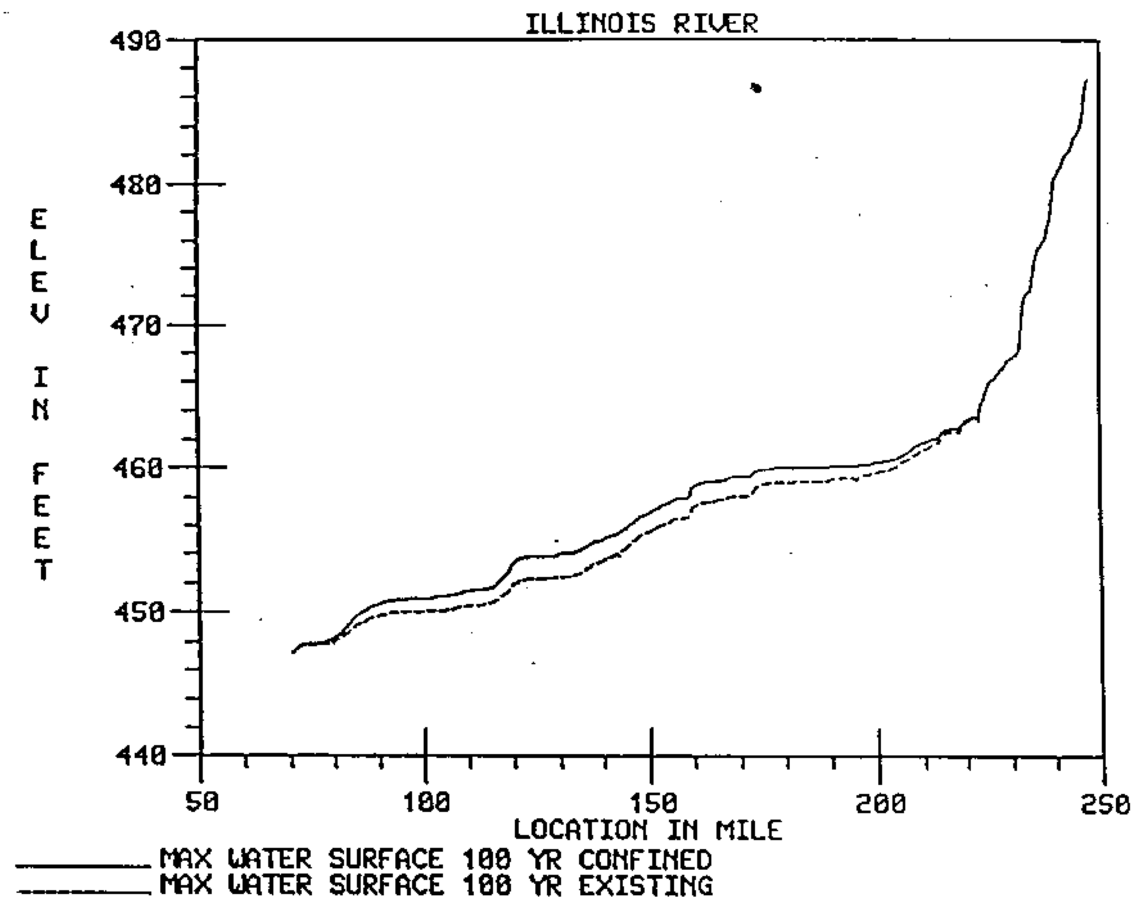

Figure D-10. The 100-yr maximum water surface along the Illinois River with and without levee failure

D-9. Survey Methods for Obtaining Cross Sections and Reach Lengths

The number of cross sections that are taken varies with study requirements and stream characteristics. Methods used to measure cross-sectional coordinates include field surveys performed with land surveying instruments, aerial spot elevations developed from aerial stereo models, topographic maps generated from aerial photography, and hydrographic surveys that are needed when the size and depth of streams preclude measurement by other means. Measurement errors for these methods are a function of industry adopted accuracy standards, equipment, terrain, and land surface cover (U.S. Army Corps of Engineers 1986).

a. Selecting a data collection method. Information has been developed for selecting an appropriate method of data collection for water surface profile computations (U.S. Army Corps of Engineers 1986). Commercially available field and aerial surveys and procedures intended to provide cross section data and topographic mapping are described therein. Key findings are as follows:

(1) Commercially available aerial and field surveys utilize up-to-date equipment and procedures to develop topographic and cross section data.

(2) The equipment used to perform aerial and field surveys continues to improve with emerging technology.

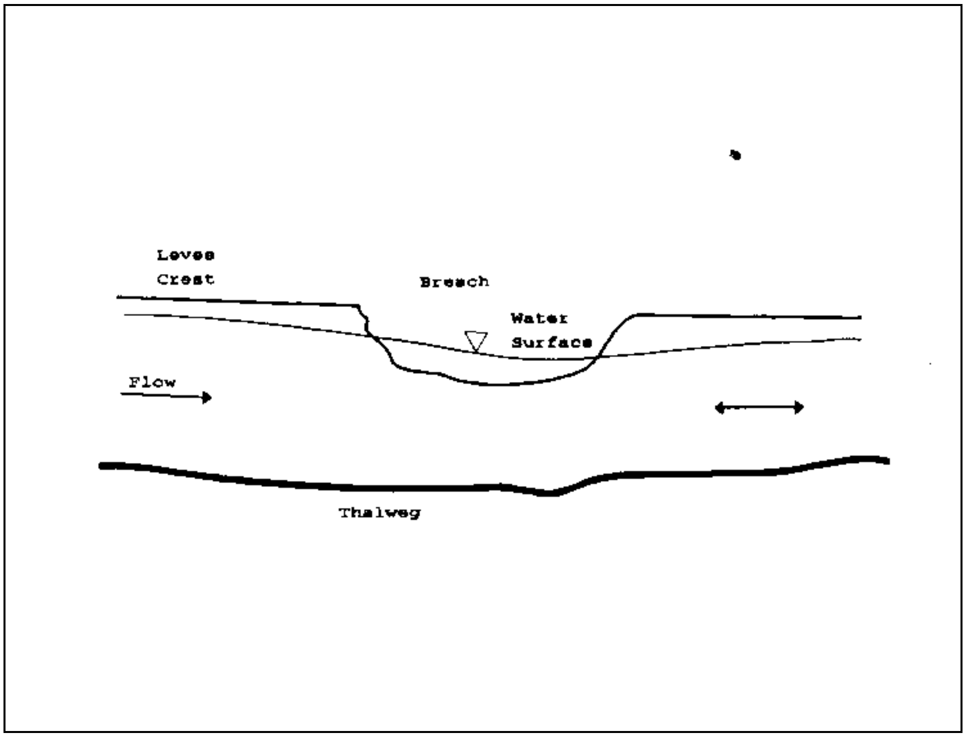

Figure D-11. The water surface of a river at a levee breach

(3) There are many potential sources of topographic and cross section data that should be investigated before setting up a field data collection program.

(4) Other project data needs may affect or even dictate the survey method for a specific project.

(5) When more than 10 to 15 cross sections are required, aerial surveys may be more economical than field surveys.

(6) The incremental costs to procure topographic mapping, in addition to cross section data, can be worth-while considering the value of the mapping obtained.

Section II
Energy Loss Coefficients

D-10. Variation of Manning's *n* with River Conditions

Manning's *n*, used widely in river hydraulic models to represent boundary roughness, varies with river conditions.

a. Variation of Manning's n with stage and flow. It is widely accepted that for the bed or channel portion of an alluvial stream Manning's *n* declines with rising stage and flow. The decline is caused by two factors: (1) a decline in the relative roughness and, (2) changes in bed forms. Relative roughness is the ratio of the height of the predominant projections in the bed geometry to the depth of flow. For an alluvial channel, the projections are the bedforms. As the depth increases the effect of these projections declines, hence, the decrease in *n* value. The effect of vegetation in the overbank is analogous.

(1) As the flow increases, the shear stress on the channel bed increases which can cause a dune bed to plane out and decrease in resistance (Simons and Richardson 1966). This phenomenon, which has been observed on the Lower Mississippi River, is shown in Figure D-14. For low flow the *n* value is about 0.06 and for high flow the *n* value is about 0.025. Simons also contends that roughness declines during the rising limb of the hydrograph and increases during the falling limb because of the looped rating curve effect.

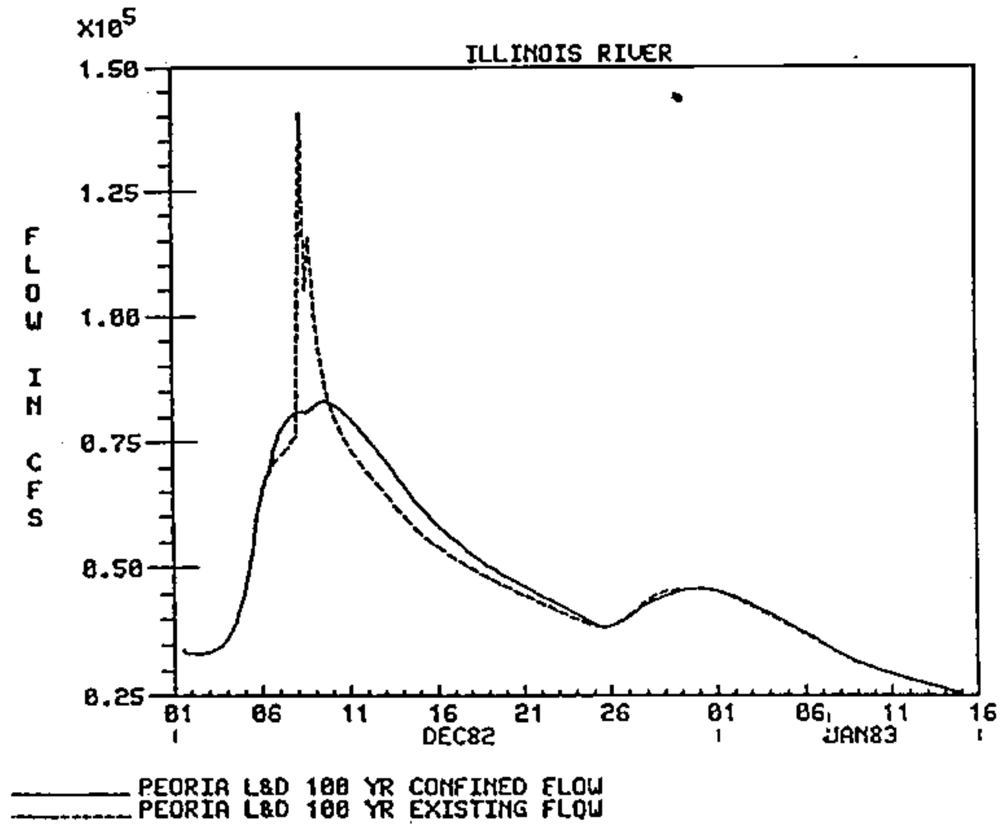

Figure D-12. Flow hydrographs for the Illinois River at Peoria with and without levee failure

(2) Manning's *n* is a function of both flow and stage. Use of the relation of *n* to flow is one of convenience. The flow relations can be defined for reaches of a stream. Stage relations apply to a specific cross section.

b. Variation of Manning's n with water temperature. The effect of water temperature was summarized by Vanoni (ASCE 1975). Lane et al. (1949) found that on the Lower Colorado River, sediment discharge increased with a decrease in temperature. Observations at Taylor's Ferry showed that in winter, when the water temperature dropped to 50°F, the sediment discharge was as much as 2-1/2 times larger than in summer when the temperature was 85°F. The increase was primarily in the suspended sediment load which agrees with theory because fall velocity decreases with decreasing water temperature.

(2) The U.S. Army Corps of Engineers (USACE 1969) studied a 7 mile reach of the Missouri River near Omaha, Nebraska, for some unsteady flow models. Figure D-17 shows the variation of temperature, discharge, velocity, and Manning's *n* with time for the Missouri River at Omaha, Nebraska, during 1966. The plot shows a reduction in Manning's *n* with temperature and a corresponding increase in velocity. The decrease in *n* was caused by a decrease in the height of the dunes and an increase in their length. Associated with the lengthening of the bedforms was a 50 percent increase in suspended sediment discharge.

(1) Colby and Scott (1965) in their study of the Middle Loop River in Nebraska discovered an increase in Manning's *n* with increasing water temperature (Figure D-15). The numbers beside the points are the discharges in cfs. The change in *n* was caused by the shift in bedforms as shown in Figure D-16. Profiles a, b, and c were taken on June 25, 1959, with discharge = 350 cfs and water temperature = 85°F; profiles d, e, and f were taken on December 5, 1959 with discharge

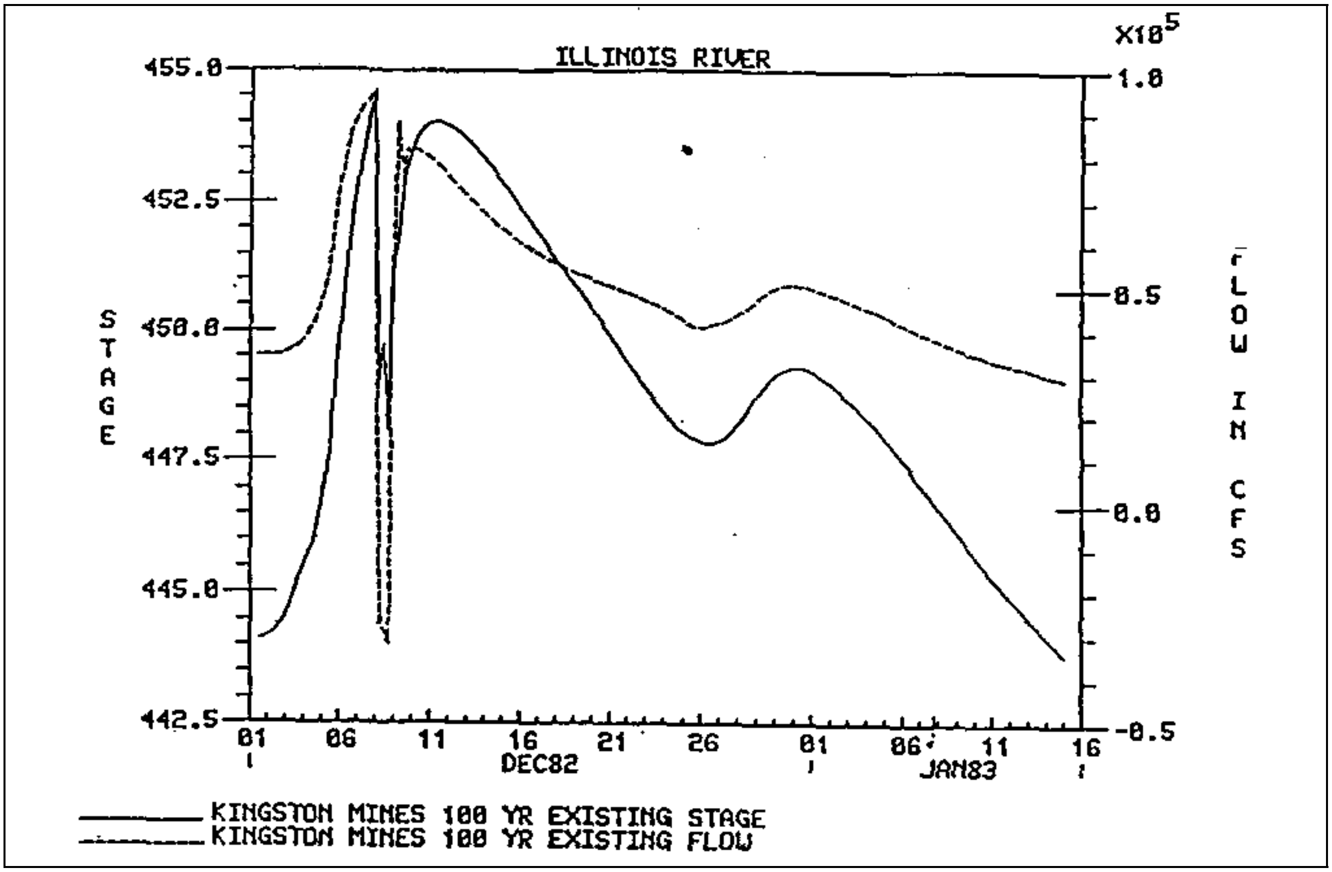

Figure D-13. Flow hydrographs for the Illinois River at Kingston Mines

= 350 cfs and water temperature = 39°F. During the winter, when the water temperature was low, the bedforms had a tendency to wash out (Figure D-16 d, e, and f) to a plane bed. During the summer, when the water temperature is warmer, the bedforms (dunes) were more pronounced (Figure D-16 a, b, and c).

(3) Carey (1963) studied a 200 mile reach of the Mississippi River above New Orleans. He observed that, as the water temperature lowered (80°F to 40°F), there was a tendency for the bed in the crossings to lower and for the height of the highest dunes to reduce. He also observed that, as the water temperature declined, the discharge for a given gage height increased.

(4) During water year 1983 (December 1982 through May 1983), three 10 percent chance exceedance peak discharges were observed at St. Louis. The discharge measurements taken during these events are plotted against the St. Louis rating curve in Figure D-18. The points are labeled with the date of the measurement and the observed water temperature. Note that the winter measurements are consistently below the curve. There is an increase in discharge for a given stage with decreasing water temperature. To further study this phenomena, the ratio of measured discharge to rated discharge versus time for stages over 20 feet was plotted at St. Louis for the period 1969 to 1983. The rated discharges were taken from the 1979 rating curve as compiled by the U.S. Geological Survey. The plot is shown in Figure D-19, and clearly shows the seasonal shift. Note that the transitions occur in April and November, but the exact timing of the transitions is not clearly defined.

D-11. Estimation of *n* Values

Conceptually, there are two major features in any reach: the channel and the floodplain. The friction force in the channel stems primarily from the bed sediment grains and bedforms, whereas the friction forces in the floodplain stem primarily from vegetation and, perhaps, structures. Decidedly different values of *n* can be expected for these regions and they should be differentiated.

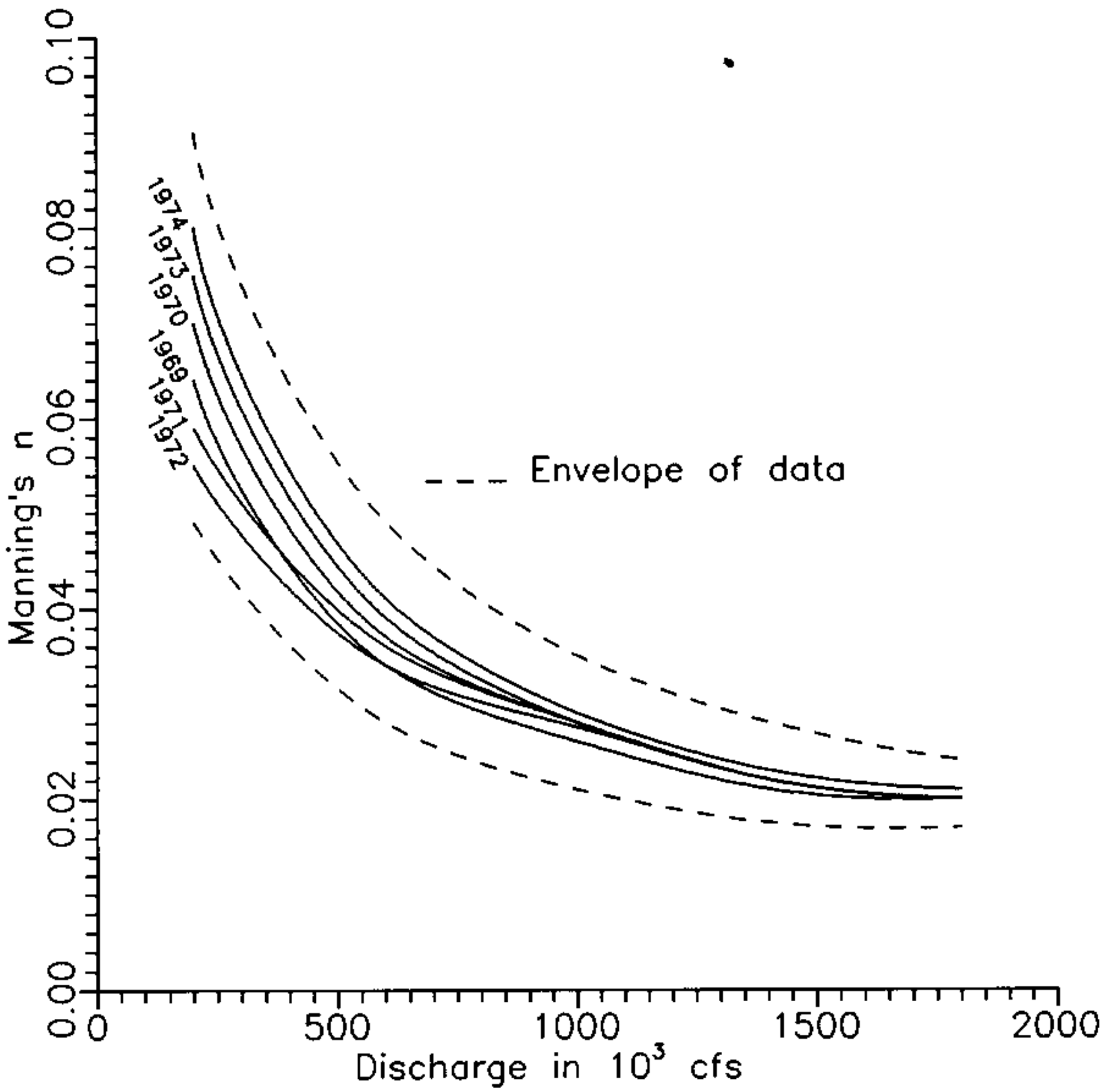

Figure D-14. Variation of Manning's *n* with discharge for the Mississippi River at Arkansas City (Source: St. Louis District, U.S. Army Corps of Engineers)

a. Overview of estimating methods. Selection of the proper value of the coefficient of friction, *n*, is very significant to the accuracy of the computed profiles. Manning's equation can be solved for *n* when discharges corresponding to observed water-surface profiles are known. If discharge measurements for the determination of *n* values are not available in Corps of Engineers' files they might be available from the U.S. Geological Survey or from other Federal or local agencies. If no records are available, values of *n* computed for similar stream conditions or values obtained from experimental data should be used as guides in selecting *n* values. Tables and photographs for selecting *n* values provided in hydraulics text books, such as Chow (1959), may be used. A contemporary summary of methods for predicting *n* values is given by USACE WES (1992).

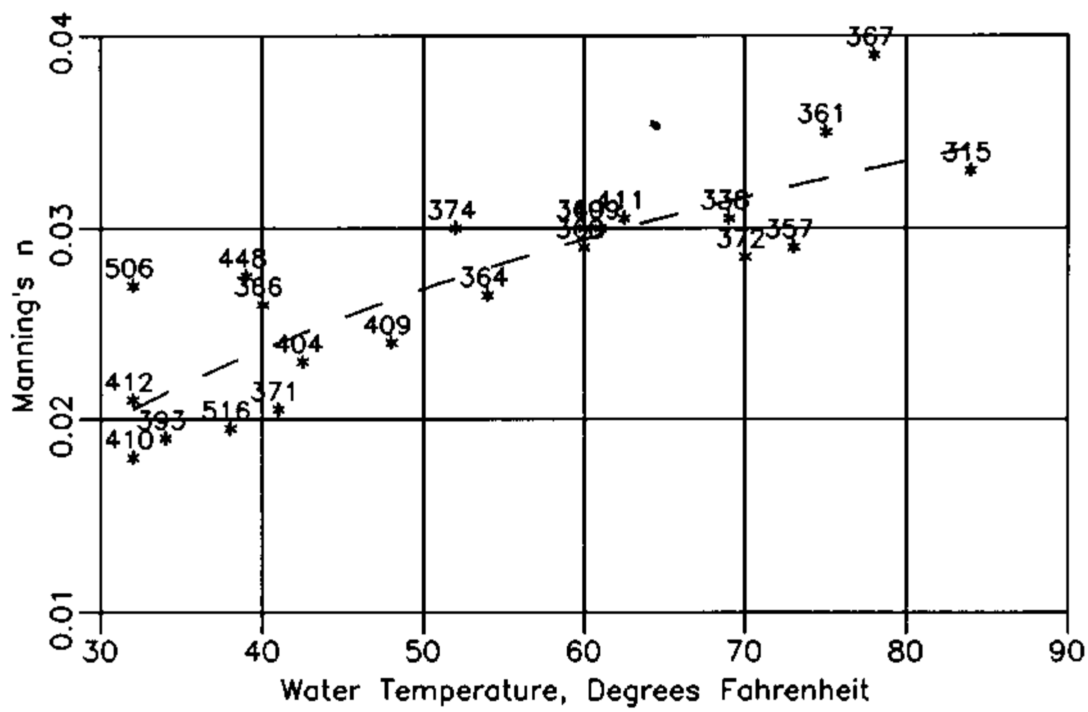

Figure D-15. Plot of Manning's friction factor *n* against water temperature for Middle Loup River at Dunning, Neb. (Colby and Scott 1965)

b. Experience, the best guide. The best guide for selecting *n* values is experience. What values have successfully been used previously in a region? Table D-1 presents a summary of *n* values for streams in the mid-west portion of the United States. For each category there is a fairly broad range of values. This range must be narrowed by field inspection and engineering judgment. Simons and Sentürk (1976, p. 225) state that dune bedforms are associated with *n* values from 0.018 to 0.035 and a plane bed is associated with *n* values from 0.012 to 0.016. Soundings show that free flowing streams have dune bedforms and that streams in backwater have nearly plane beds due to deposition of sediment. The values in Table D-1 are thus reasonably consistent with Simons' work. The higher upper limit for medium sized streams can be attributed to the greater impact of vegetation along the banks. High *n* values may be needed at low stages to mimic the effects of crossings.

c. Estimates based on observed data. Discharge measurements often include transverse variation in discharge in a cross section and give sounding depths so that cross sections can be plotted to compute area and hydraulic radius. Water surface slopes are obtained from profiles of high-water marks determined by field surveys or from records of stages at gaging stations if these are closely spaced. When discharge measurements are made to determine *n* values, it is desirable to also obtain

Table D-1
Values of Manning's *n* for Streams in the Mid-West

Stream Type	Value Range
Large rivers (over 500 ft wide)	0.020 - 0.035
Medium size rivers (less than 500 ft wide)	0.030 - 0.042
Strong backwater areas	0.015 - 0.025
Overbank:	
pasture	0.050 - 0.080
plowed field	0.040 - 0.070
cropland	0.050 - 0.080
woodland	0.070 - 0.150

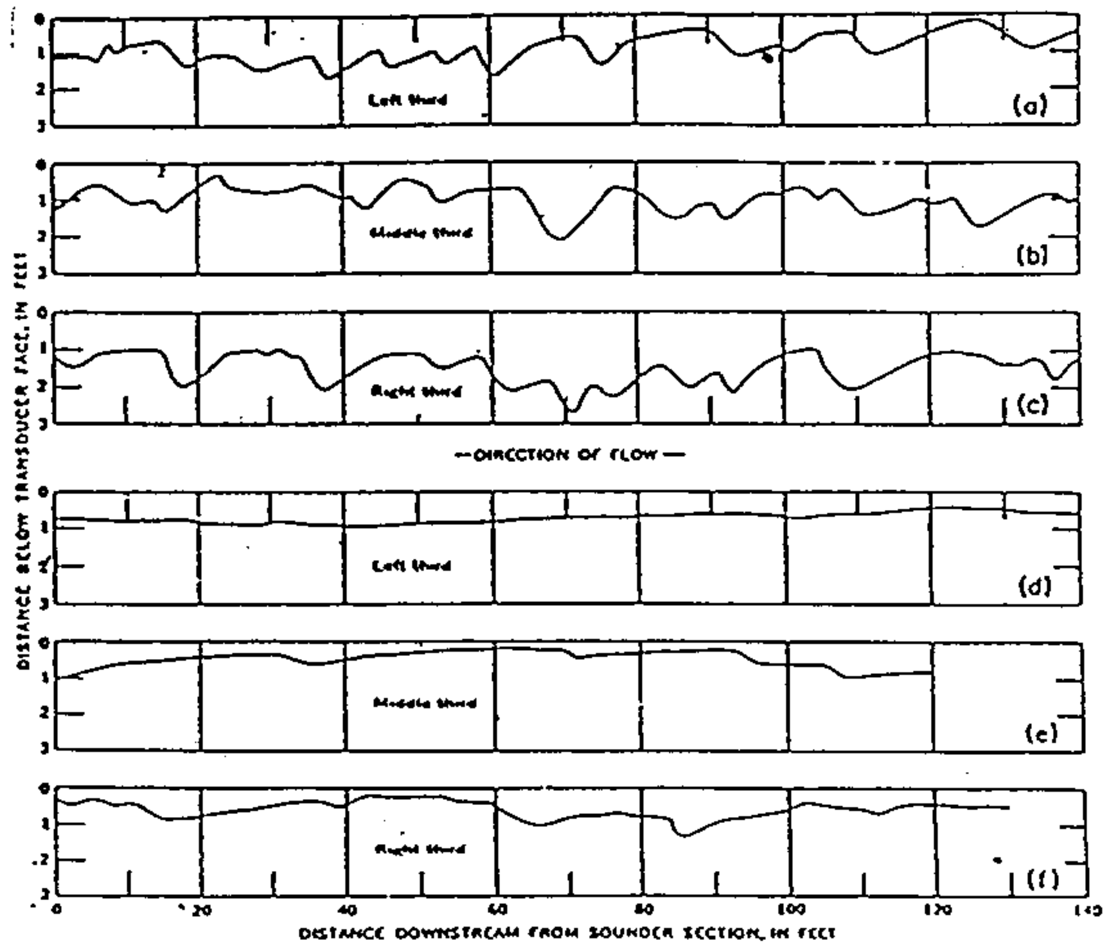

Figure D-16. Two sets of three longitudinal bed profiles each of Middle Loup River at Dunning, Neb., at high and low water temperature (Colby and Scott 1965)

watersurface slopes. Such data can be used to derive more reliable values of *n* than can be determined from high-water marks alone.

(1) From the water-surface slope, discharge, area, and hydraulic radius, the value of *n* can be computed from Manning's equation. In one method of calculation, uniform flow (for which the water-surface slope equals the friction slope) is assumed and approximate values of *n* are determined for overbank and channel areas of the cross section. As a check on nonuniform flow conditions, water surface profile computations should be made using the previously determined approximate values of *n* to obtain a comparison of the computed water-surface profiles with the observed profile. Unless reasonable agreement is obtained, the values of *n* should be adjusted by trial-and-error until the computed water-surface profile is in satisfactory agreement with observed values. The computations should be made for several discharges to obtain representative values of *n*.

(2) If the data show that *n* varies with stage, *n* should be determined from a curve of *n* versus stage or from the observed profile which most nearly approaches the stage of the desired profile. Generally, expansion

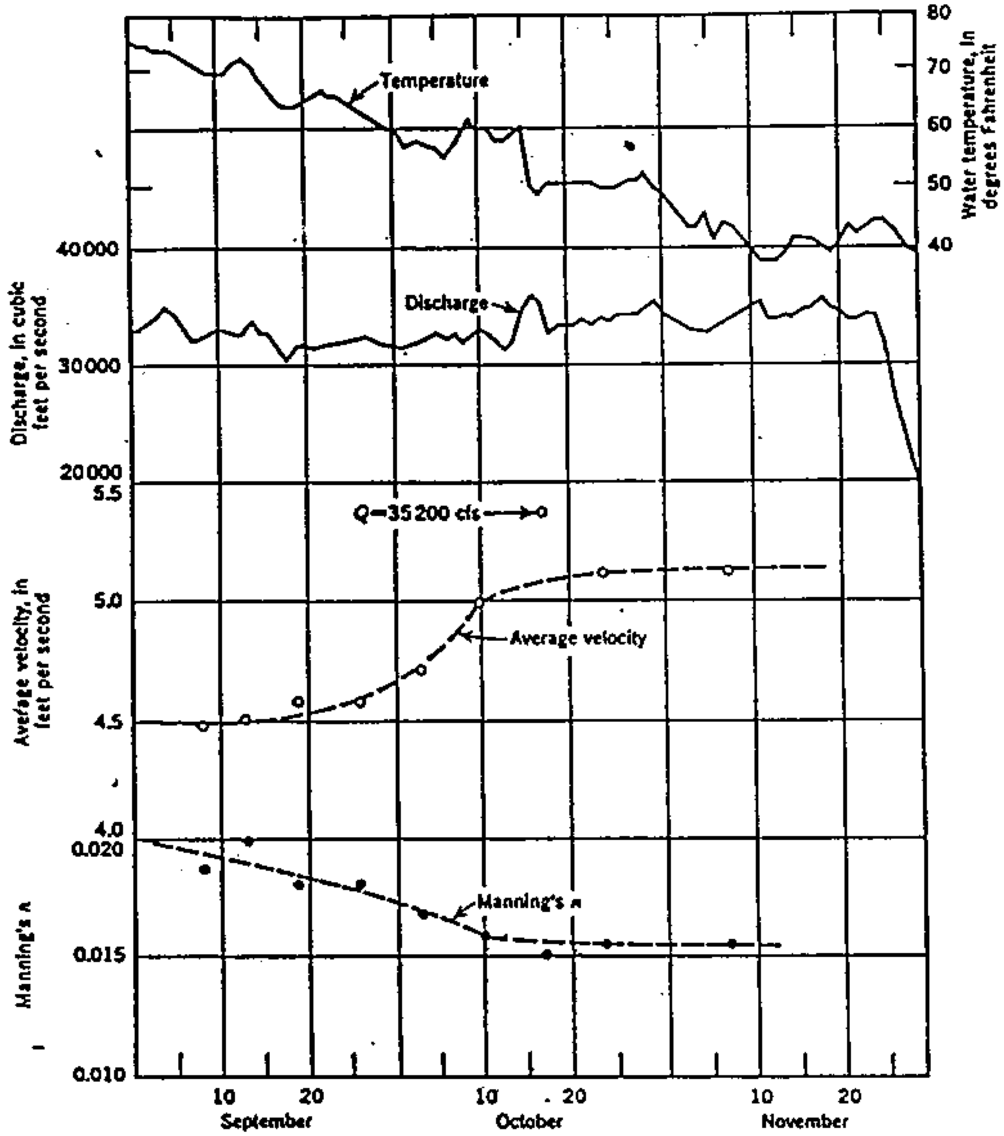

Figure D-17. Variation of water temperature, discharge, average velocity, and Manning's *n* for Missouri River at Omaha, Neb., during fall of 1966 (U.S. Army Corps of Engineers 1969)

and contraction losses should be considered separately in determining *n*; that is, not lumped into the *n* value.

(3) In determining *n* values from measured slopes and discharges, or in computing water surface profiles, superelevation in bends should be considered.

d. Estimates based on n values from similar reaches. When records of discharge measurements are not available, values of *n* determined for reaches of similar characteristics can serve as valuable guides in selecting proper values of *n*.

e. Estimates based on published guides. Tables of *n* values are provided to varying degree of detail in hydraulics texts and technical reports by Chow (1959), Barnes (1967), U.S. Army Corps of Engineers (1975), U.S. Department of Transporation (1984), Davidian (1984), USGS (1986), and WES (1992). Photographs to compare with field conditions are provided in Chow (1959) and Barnes (1967). Formulas have been derived to compute roughness coefficients by Beasley (1973), Chow (1959), Brownlie (1981), U.S. Department of Transportation (1984), and USGS (1986); they usually

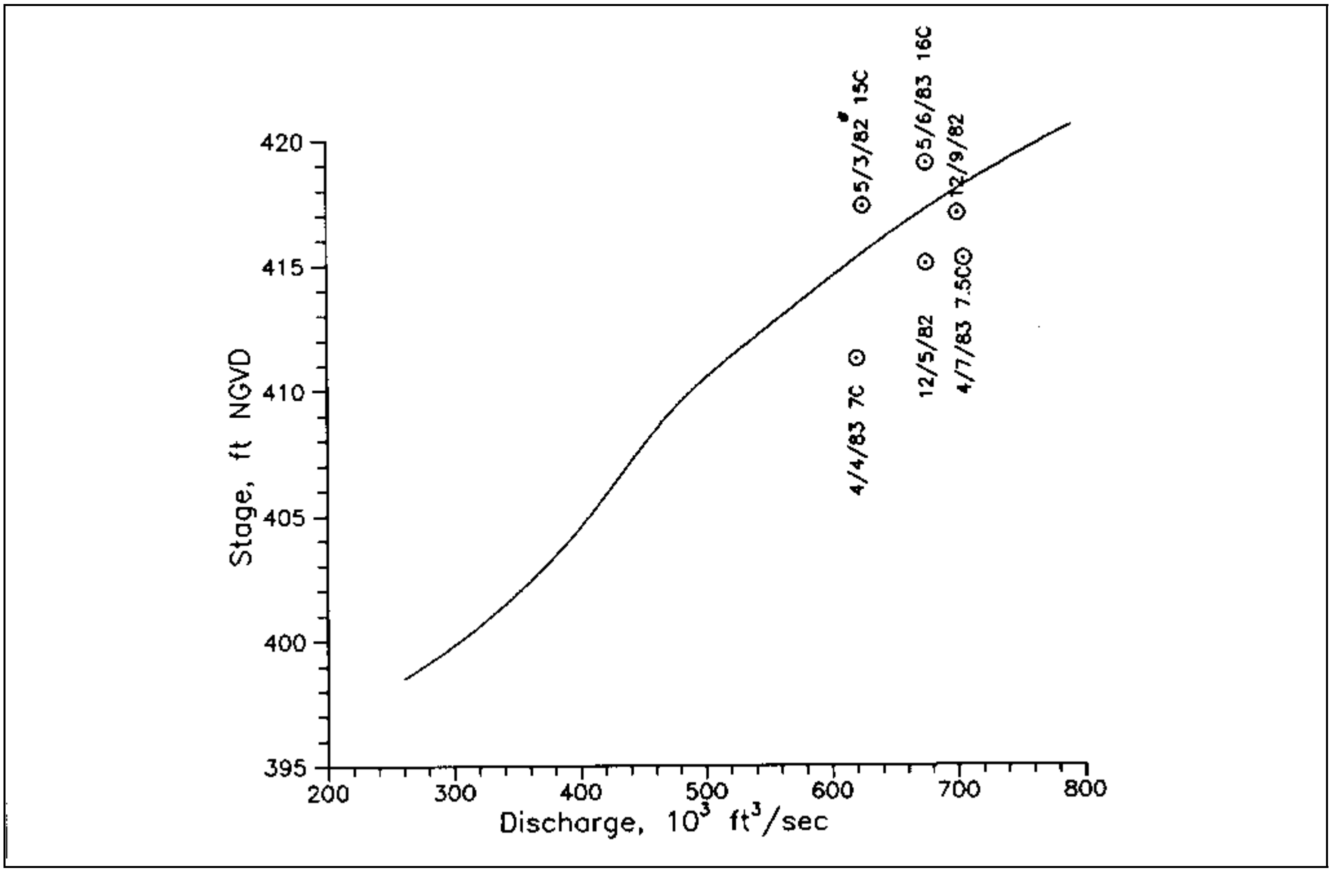

Figure D-18. Discharge measurements during the Dec. 1982, April and May 1983 floods at St. Louis, plotted with the St. Louis rating curve (Water temperature also shown in °C.)

require field samples of bed material and laboratory analysis of grain sizes.

(1) Chow (1959) compiled a table for flow over differing types of materials. He also presented photographs of differing stream conditions and the associated *n* values. Barnes (1967) computed *n* values for various streams in the United States. However, one should be cautious in using such computed *n* values. It is recommended that published values such as these be used only when the engineer is unfamiliar with the stream morphology.

D-12. Contraction and Expansion Losses

The coefficients of contraction and expansion are not as quantified as Manning's *n*. These coefficients are provided in backwater models such as HEC-2 and HEC-6 to account for losses associated with the contraction and expansion of flow due to changes in the size and shape of flow area. A range of values is given in the HEC-2 users manual; the lowest values apply to valley reaches in which the change in river cross section is relatively small, and the highest values apply to bridges and other locations where the change is more abrupt. Because these coefficients are applied to differences in velocity head between cross sections, the degree of change of velocity head governs their impact. In mild channels with small changes in velocity head, the impact is small; but in steep mountain streams where changes in velocity head are much greater, their impact may be critical to the solution (Hoggan 1989).

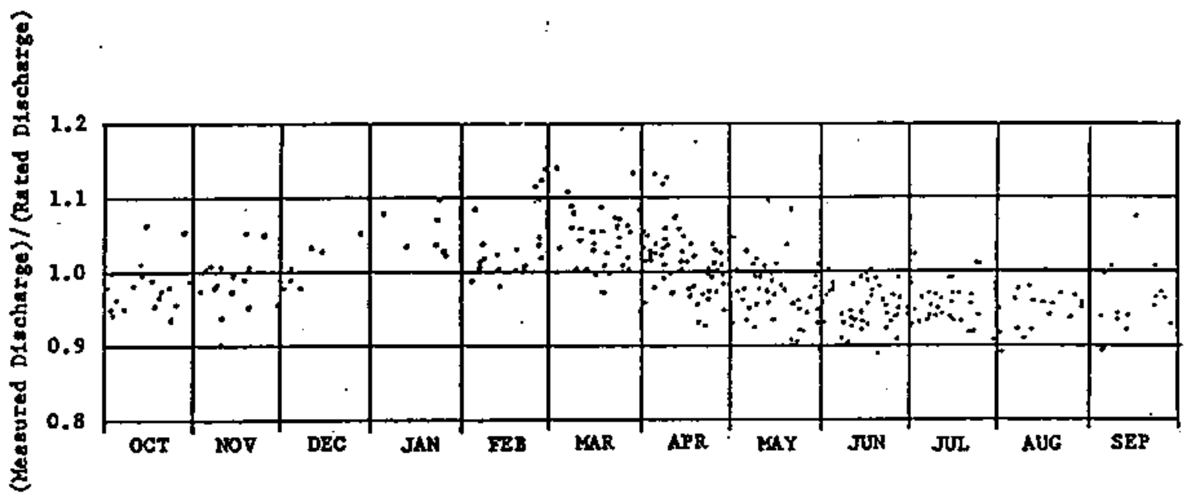

Figure D-19. Seasonal shift shown for the Mississippi River at St. Louis for stages over 20 feet from 1969 to 1983