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Jozsef Szilagyi
University of Nebraska-Lincoln, jszilagyi1@unl.edu

Nicholas Pinter
Southern Illinois University

Rob Venczel
Southern Illinois University

Application of a Routing Model for Detecting Channel Flow Changes with Minimal Data

Jozsef Szilagyi[1]; Nicholas Pinter[2]; and Rob Venczel[3]

Abstract: The discrete linear cascade model (DLCM) was applied for historical flow routing along the Nebraska City–Rulo section of the Missouri River in southeastern Nebraska. With the help of optimized model parameters it has been possible to identify the triggering mechanism responsible for historical changes in the stage-discharge relationship at Rulo over the 1952–2006 period, following construction of Gavins Point dam on the Missouri above Sioux City, Iowa. It was found that in the second part of the past century flood celerity in the study reach slowed by about 25%, most likely caused by an increase in the Manning roughness coefficient as a result of a large increase (almost a doubling) in the number of wing-dykes constructed over the reach within the same period. The ease of application and minimal data requirement (only discharge values at regular intervals) makes the DLCM a practical tool for stream-flow analysis. It also can serve as a preliminary investigative tool for more advanced and detailed hydraulic approaches that typically require a data-rich environment and significantly greater development time.

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Introduction

Recently, Pinter et al. (2001, 2006a,b) and Pinter and Heine (2005) revisited an updated version of the specific-gauge technique of Blench (1969) for detecting changes in channel flow dynamics in the Mississippi–Missouri River system and other rivers worldwide. With historical information on discharge, stage, channel width, and cross-sectional area, they were able to quantify changes in the stage–discharge relationship over time and determine the apparent causes of these shifts. For example, Pinter and Heine (2005) found that on the Lower Missouri River downstream of Nebraska changes in the stage–discharge relationship over the past 65 years were driven by decreased flow velocities (at three stations) and constriction in the channel cross-sectional area (at two stations). These changes resulted in progressively and significantly higher stage values over the study period for fixed flood discharge rates at all five stations investigated. These rating-curve shifts partially or completely counteracted the flood control benefits introduced by the large reservoirs [i.e., behind Fort Peck in Montana, Garrison in North Dakota, Oahe, Big Band, Fort Randall, and Gavins Point dams in South Dakota (Fig. 1)] on the Upper Missouri (Pinter et al. 2002).

The motivation of the present study is to demonstrate that, with the help of a physically based, simple flow routing technique, the discrete linear cascade model (DLCM; Szollosi-Nagy 1982), similar questions can be addressed even when information on channel cross-sectional area and/or channel width are absent.

The focus of the present study is a 104 km stretch of the Missouri River in southeastern Nebraska, between Nebraska City and Rulo (Fig. 1). This reach is immediately upstream of the portion of the river studied by Pinter and Heine (2005). Additional reasons for selecting this part of the river were: (1) staff gauges have been operating at the upstream and downstream ends of the reach without any datum shifts for the past 65 years; (2) this reach was the site of major civil engineering works after WWII to facilitate river navigation between the Mississippi River and Omaha; (3) the flow travel time within the reach is about a day, which is optimal when working with daily data; and (4) the reach has only minor tributaries.

Methods

The specific-gauge technique (fixed-discharge analysis) of Blench (1969) calculates changes in the stage–discharge relationship (i.e., the rating curve) at a specified cross section of a river as a quasi-time series. Within the study reach here, both Nebraska City and Rulo have had rating-curve data updated by the U.S. Geological Survey approximately biweekly for the past 65 years. By specifying a set of discharge values and corresponding tolerance intervals, a stage value—calculated as the mean of the stage values that belong to observed discharge rates within the tolerance inter-

[1]Associate Professor, Dept. of Hydraulic and Water Resources Engineering, Budapest Univ. of Technology and Economics, H-1111, Budapest, Hungary; and School of Natural Resources, Univ. of Nebraska–Lincoln, Lincoln, NE 68583-0997.

[2]Associate Professor, Dept. of Geology, Southern Illinois Univ., Carbondale, IL 62901-4324.

[3]Graduate Research Assistant, Dept. of Geology, Southern Illinois Univ., Carbondale, IL 62901-4324.

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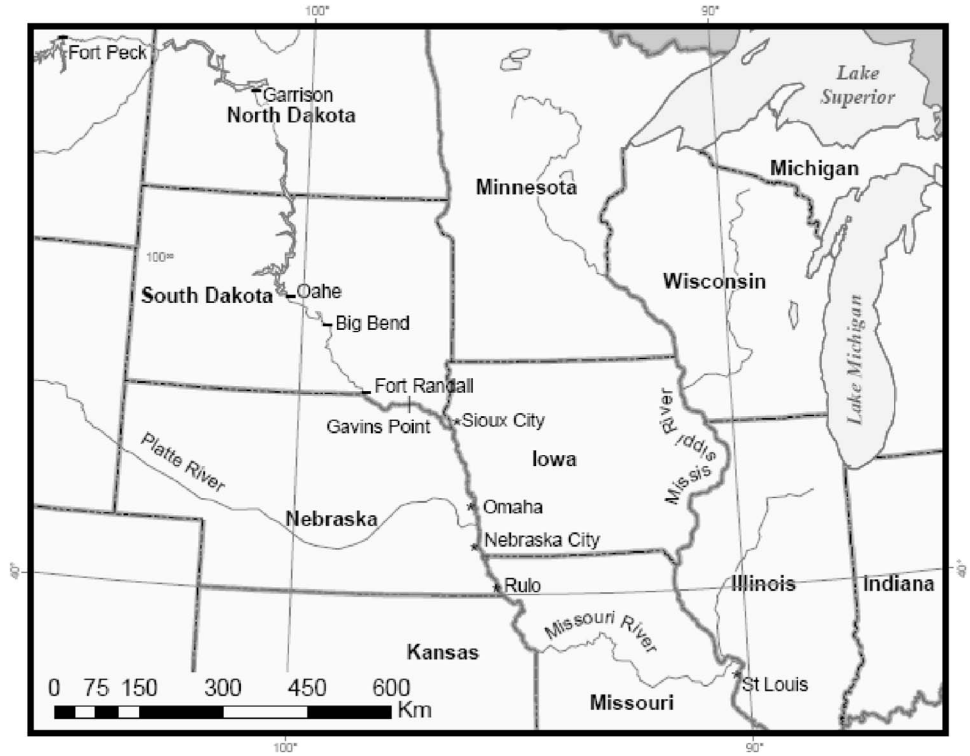

Fig. 1. Location of the study reach including locations of the mainstem dams on the Upper Missouri River

vals (if such observed flow-rate values exist at all in the given year)—can be assigned to each discharge value and plotted on an annual basis. Fig. 2 displays the resulting values for Nebraska City and Rulo, applying a tolerance interval of $\pm 10\%$ of the predefined flow rates. Note that with increasing fixed-discharge values the number of points in the graphs decreases as there are fewer large flood values.

As evident from Fig. 2, the same flow passes the two gauging stations with a historically increasing tendency in the corresponding stage values (with the exception of the smallest discharge rate at Rulo). For example, a flow of 3,000 $m^3\,s^{-1}$ in the 1950s did not typically reach the flood stage, but by the year 2000, it surpasses it by a meter or so, especially at Rulo. These observed tendencies are very similar to what was reported by Pinter and Heine (2005) for the gauging stations downstream in Missouri.

From the Manning formula, written for a wide, shallow (i.e., the width of the open surface, B, is much larger than the mean cross-sectional water depth, d) rectangular channel (i.e., the cross-sectional area, A, can be accurately estimated as Bd) under steady flow conditions (i.e., the friction slope is equal to the channel bottom slope, S_0 [–]), one obtains

$$\frac{Q}{d^{2/3}} = \frac{\sqrt{S_0}A}{n} \tag{1}$$

where n=Manning roughness coefficient $[L^{-1/3}\,T]$ and Q=discharge $[L^3\,T^{-1}]$. Assuming that the mean cross-sectional water depth, d, remains proportional to the observed stage, h, over time (which is not at all certain in general), the ratio of Q and h can only decrease (from Fig. 2) historically if the numerator decreases and/or the denominator increases in the right-hand-side of Eq. (1). The equation has three unknowns (S_0, A, and n), therefore additional information is needed for specifying the triggering mechanism of the observed historical change in the stage–discharge relationship (i.e., in Qh^{-1}). Although Pinter and Heine (2005) had such information for A (as well as B and d) in the form of historical measurements, such data may not always be available. Many times only the stage and discharge records exist for analysis, requiring extra equations for identifying the source of the detected change in flow dynamics.

Such extra information can be obtained by the application of a physically based flow-routing method in combination with a simplified version of the St-Venant equations of open-channel flow. The simplification is achieved by neglecting the inertial terms in the St-Venant equations, yielding the diffusion-wave equation (Cunge et al. 1980)

$$\frac{\partial Q}{\partial t} + c(Q)\frac{\partial Q}{\partial x} = D(Q)\frac{\partial^2 Q}{\partial x^2} \tag{2}$$

where

$$c(Q) = 5Q/3A \approx 5d^{2/3}\sqrt{S_0}/3n \tag{3}$$

[the right-hand side is based on Eq. (1)], and $D(Q) = A^2 d^{4/3}(2n^2 QB)^{-1}$ $[L^2\,T^{-1}]$, when written for a wide and shallow rectangular channel. Note that the flood-wave celerity term, $c(Q)$ $[L\,T^{-1}]$, includes the same two channel physical properties (S_0 and n) found in Eq. (1), their historical change potentially influencing the stage–discharge relationship. In fact, Eq. (3) is the same as Eq. (1) up to a constant multiplier.

With the help of a flow-routing model, the celerity value, or at least a representative mean value of it, can typically be specified. Historical change in the celerity value thus obtained then can help with identifying the causal mechanisms driving the observed change in the stage-discharge relationship of the study reach (Fig. 2) because the number of equations is increased to two containing the same three unknowns.

There are two properties of the Missouri River having a navigable channel that can be exploited in this analysis and may pertain to many other rivers of the world. First, because of poten-

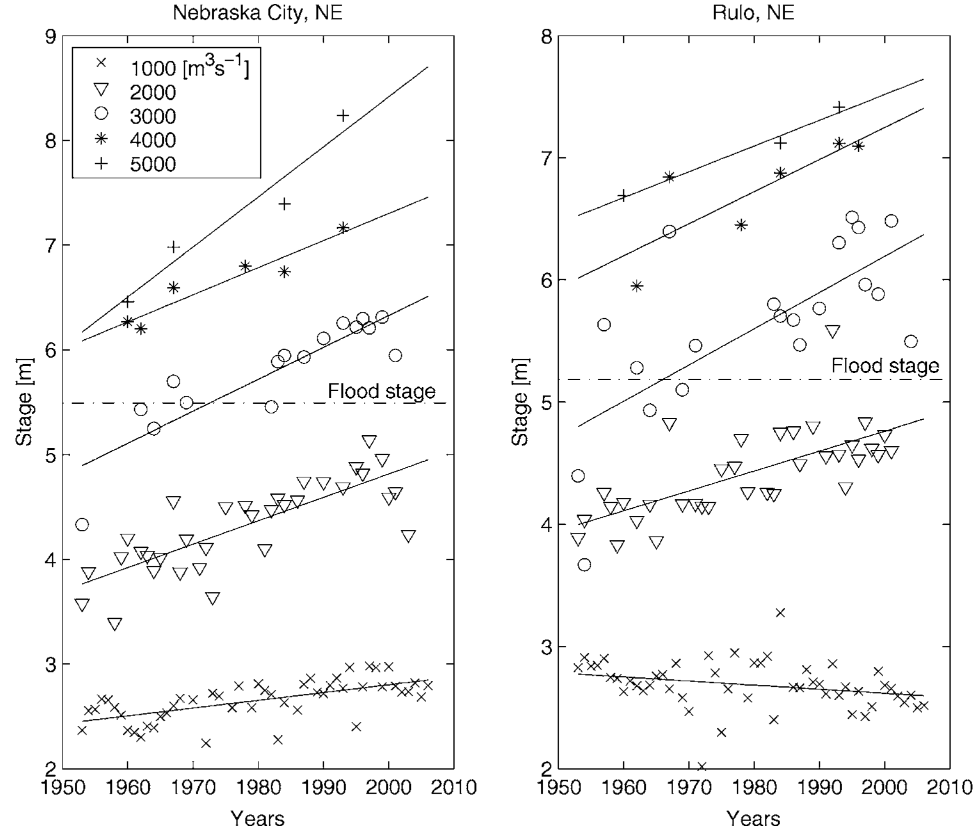

Fig. 2. Specific-stage diagrams of the Missouri River at Nebraska City and Rulo, Neb. with linear trend functions fitted. Note that data for each year are for hydrologic years that start October 1 the previous year and end September 30 the year shown.

tial historical channel straightening, dredging (to prevent sediment accretion) and construction of flow-training structures (e.g., wing-dykes) to maintain minimum navigational depth throughout the year, S_0 can be expected not to decrease over time. Second, for a channel that is constrained to a well-defined location by bank stabilization works and levees, if not a true proportionality between stage (h) and mean cross-sectional water depth (d), but at least that d should not decrease with increasing h over time for a given discharge, can be expected from a nondecreasing channel slope, S_0. The latter is illustrated in Pinter and Heine (2005) for the Missouri River gauging stations downstream of Nebraska.

A representative mean value of the nonlinear celerity term, $c(Q)$, can now be obtained by the application of the DLCM. The DLCM derives from the linear kinematic-wave equation, which is obtained from Eq. (2) with $D(Q)=0$ and $c(Q)=c$, a constant, substitution (Lighthill and Whitham 1955), via a backward spatial discretization scheme, written in a state-space framework. The outflow from the reach, $y_{t+\Delta t}$, at time $t+\Delta t$, where Δt is the time-increment of observation (here one day), is expressed as

$$y_{t+\Delta t} = \mathbf{H}(\mathbf{\Phi S}_t + \mathbf{\Gamma}_1 u_{t+\Delta t} + \mathbf{\Gamma}_2 u_t) \tag{4}$$

where u=inflow to the reach; $\mathbf{S}=m\times 1$ state vector made up of the stored water volumes of m uniform subreaches, each having a mean residence time of k^{-1}, within the model reach; $\mathbf{\Phi}=m\times m$ state-transition matrix; $\mathbf{\Gamma}_1$ and $\mathbf{\Gamma}_2=m\times 1$ input-transition vectors, and $\mathbf{H}=1\times m$ output vector (Szilagyi 2003). The elements of $\mathbf{H}$ are $[0,0,\ldots,k]$, the term at the ith row and jth column of the lower-triangle Toeplitz-band $\mathbf{\Phi}$ matrix can be written as $(k\Delta t)^{i-j}e^{-k\Delta t}[(i-j)!]^{-1}$, whereas the ith element of the $\mathbf{\Gamma}_1$ and $\mathbf{\Gamma}_2$ vectors become $\Gamma(i,k\Delta t)[k\Gamma(i)]^{-1}\{1+e^{-k\Delta t}[\Gamma(i,k\Delta t)]^{-1} - i(k\Delta t)^{-1}\}$ and $\Gamma(i,k\Delta t)[k\Gamma(i)]^{-1}\{i(k\Delta t)^{-1} - e^{-k\Delta t}[\Gamma(i,k\Delta t)]^{-1}\}$, respectively. Here Γ denotes the incomplete (with two arguments) and complete (with one argument) gamma functions.

The model has two parameters to be optimized, the number of linear storage elements (i.e., subreaches), m, and the storage coefficient, k. The ratio of the two parameters, mk^{-1}, yields an estimate of the time an upstream disturbance (in our case at Nebraska City) reaches the downstream station (at Rulo). Similarly, the Lkm^{-1} term represents the mean celerity of the reach of length L. Note that although the DLCM is a special discretization of the linear kinematic-wave equation, which has a zero diffusion coefficient (D), it provides an approximation of the linear diffusion-wave equation (i.e., $D>0$) via the finite spatial differences employed in its derivation (Szilagyi et al. 2005). This is not by chance, as otherwise it would hardly be of much use for flow routing.

The Nebraska City–Rulo reach of the Missouri River has a present-day channel length of $L=104$ km. Over this reach the river receives three larger tributaries, the Tarkio (long-term mean discharge, Q_m, of 6 $m^3\,s^{-1}$), the Nishnabotna ($Q_m=35$ $m^3\,s^{-1}$), and the Little Nemaha ($Q_m=9$ $m^3\,s^{-1}$) Rivers. For the Tarkio River, which is near Fairfax, Mo. and is the only and last station before the confluence, no discharge values are available after 1990, thus it had to be left out of the modeling. This omission does not introduce much uncertainty into the simulated daily values as the flow of the Tarkio River is a very small fraction of that of the Missouri River ($Q_m=1{,}200$ $m^3\,s^{-1}$ at Rulo). With the DLCM being a linear model, flow routing starting at the tributaries (i.e., at Hamburg, Iowa for the Nishnabotna, and at Auburn, Neb. for the Little Nemaha River) can be performed separately for the target station, Rulo. The parallel-routed discharges of

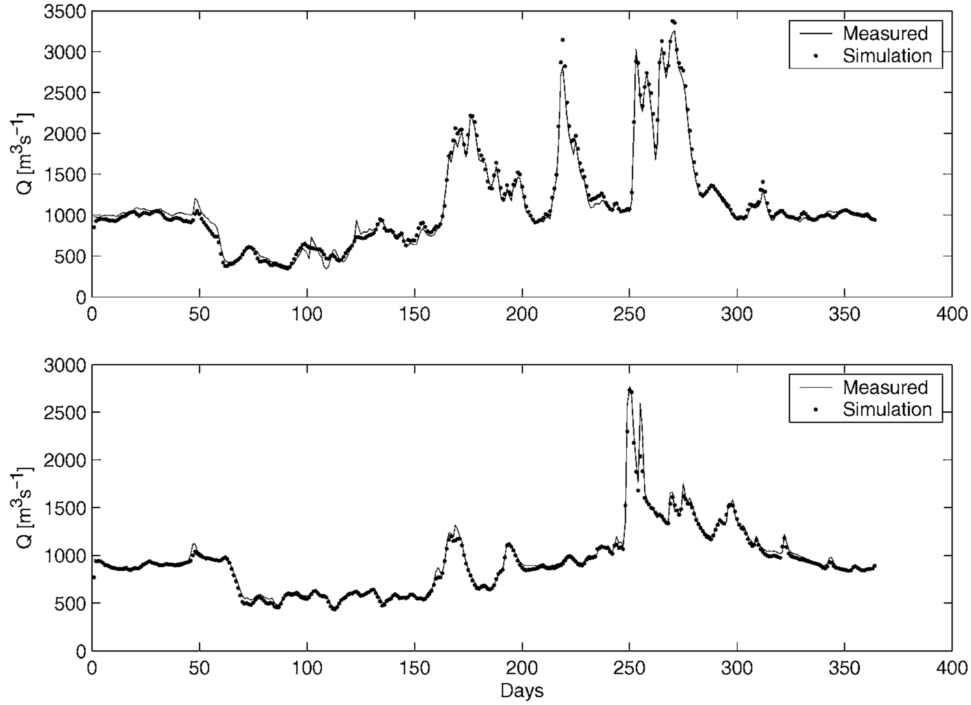

Fig. 3. Measured and simulated daily mean discharges of the Missouri River at Rulo, Neb.: (a) hydrologic year 1953; (b) hydrologic year 1995

the tributaries then were added to the main channel routing results to obtain a simulated flow value at Rulo. Such an approach requires the optimization of three pairs of m and k values. During optimization it turned out that the gauging station at Hamburg is so close to Rulo that no flow routing is necessary. As a result, the simulated daily discharge value at Rulo is the sum of the routed discharges originating from Nebraska City and Auburn, plus the actual discharge at Hamburg. This is possible because the daily discharge values available for flow routing for each station are *daily averages*, instead of the normally required instantaneous values measured at the same time of the day. This way only two pairs of m and k values had to be optimized in the end.

Model Results and Discussion

Two full years (Fig. 3), one from the 1950s (hydrologic year of 1953) and one from the 1990s (hydrologic year of 1995) were chosen for model parameter optimization and, therefore, for mean celerity value estimations. In selecting these two years we attempted to choose periods with similar water regimes as the celerity is typically a function of the discharge, as illustrated in Eq. (3). The selected years have similar water regimes with a mean daily discharge value of 1,128 $m^3\,s^{-1}$ for 1953 and 948 $m^3\,s^{-1}$ for 1995. By selecting two similar years we avoided the possibility that any detectable historical change in the celerity value would be caused by a difference in the mean discharge over the two periods rather than actual changes in channel properties. There were not too many other alternatives in choosing the two periods since one had to represent "base conditions," against which any changes are compared, consequently one period had to be from the beginning of the available discharge record and the other from the end. Note that the effects of the Upper Missouri reservoirs are clearly detectable in both years in the form of a sudden drop in flow around the start of December (i.e., around day 60 of each hydrologic year).

Model optimization for the main channel resulted in $m=3$ and $k=5.7\ \text{day}^{-1}$ for hydrologic year 1953 with a mean error, ε (i.e., the mean difference between simulated and measured values) of 20 $m^3\,s^{-1}$ and a standard deviation (σ_ε) of 73 $m^3\,s^{-1}$. For hydrologic year 1995 it yielded $m=3$, $k=4.3\ \text{day}^{-1}$ with $\varepsilon=-24\ m^3\,s^{-1}$ and $\sigma_\varepsilon=48\ m^3\,s^{-1}$. The corresponding Nash–Sutcliffe-type model efficiency coefficient (NSC), defined as

$$\text{NSC} = 100\left(1 - \frac{\Sigma_i(Q_i^s - Q_i)^2}{\Sigma_i(Q_i - Q_{i-1})^2}\right)\ (\%) \tag{5}$$

where Q^s=simulated and Q=measured mean daily discharge at Rulo, was 72% for 1953 and 76% for 1995. Note that a perfect simulation results in a NSC value of 100%, whereas a simulation that is worse than the 'naïve forecast' (i.e., $Q_i^s=Q_{i-1}$) leads to a negative NSC value.

In the 1950s it took about 0.53 day($=mk^{-1}$) for a floodwave to travel the Nebraska City—Rulo distance (assuming that the length, L, of the stream reach stayed a constant 104 km, the present day value), whereas the same travel time was about 0.7 day in the 1990s. This translates into a mean celerity value of 8.23 km h^{-1} in hydrologic year 1953 and 6.21 km h^{-1} in 1995, a significant ($\sim$25%) slowing.

As it was argued earlier that neither the mean channel depth (d) nor the channel bottom slope (S_0) decreased over time along the study reach, a decreased celerity means an increase in the Manning roughness coefficient (n) value in Eq. (3). An increase in the n value is very likely the result of a large increase in the number of wing-dykes constructed within the study reach over time. Although in the 1950s there were only about 340 of them, by the 1990s their number grew to about 660 (counted from maps by the Omaha District of the United States Army Corps of Engineers). Note that A could not drive the observed rating-curve

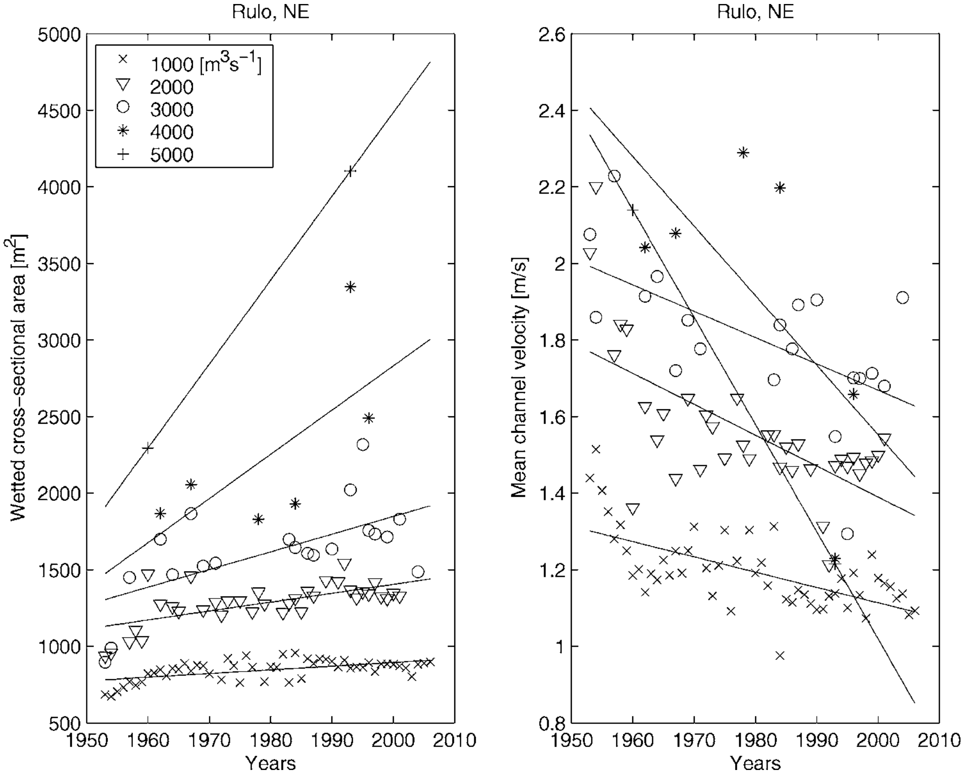

Fig. 4. Specific cross-sectional area and mean channel velocity diagrams for the Missouri River at Rulo, Neb., with linear trend functions fitted

change as doing so its value should have decreased over time in Eq. (1), whereas a simulated decrease in the celerity value of Eq. (3) requires that A (under a specified flow) must have increased over time. The contradiction is solved by accepting that A indeed increased for a given flow and realizing that the historical increase in the n value of Eq. (1) had to be even larger in magnitude.

During normal flow conditions the wing-dykes force flow into the center of the channel and, as a result, may increase channel conveyance. By increased conveyance we mean that the same flow can pass with decreased stage values over time, as can be seen in Fig. 2 at $Q=1{,}000\ \mathrm{m}^3\,\mathrm{s}^{-1}$ for Rulo. This is only possible if mean flow velocity increases and/or if the channel undergoes incision. The former is an intended purpose of wing-dyke construction for promoting river navigation, as a faster and spatially more concentrated flow typically hinders sediment accretion and the formation of channel sand bars.

During flood conditions, however, the flow normally contained within the wing-dykes, will not be restricted to the central section of the channel. Instead, the water will flow partially or fully over the wing-dykes, which, with their rough surfaces and their very presence, can increase the mean channel roughness significantly.

To check whether model optimization results and the ensuing conclusions are correct, i.e., that the observed historical increase in specific stages at Rulo is indeed caused by an increase in the Manning roughness coefficient value (n) of the channel and not by floodplain encroachment and/or a constriction of the channel cross-sectional area in Eq. (1)—historical wetted cross-sectional area (A) values available for Rulo were plotted (Fig. 4). A has increased over time for every fixed discharge value indicating the absence of any such encroachment or constriction over the duration of record. Consequently, floodplain encroachment and/or channel cross-sectional area constriction could not trigger the observed decrease (Fig. 2) in flow conveyance in Eq. (1). At the same time, the mean channel velocity, calculated as QA^{-1} decreased historically for every discharge value chosen (Fig. 4). This is in accordance with model results. Even the extent of the velocity change—from about $1.4\ \mathrm{ms}^{-1}$ in 1953 to $1.1\ \mathrm{ms}^{-1}$ in 1995 (a $\sim$22% decrease) at $1{,}000\ \mathrm{m}^3\,\mathrm{s}^{-1}$ and from about 2 to $1.5\ \mathrm{ms}^{-1}$ (a 25% decrease) at $2{,}000\ \mathrm{m}^3\,\mathrm{s}^{-1}$—agrees with the model-simulated 25% decrease in mean flood celerity.

Conclusions

In summary, the specific-gauge technique (Blench 1969; Pinter et al., 2001) is a convenient tool for detecting historical changes in channel flow dynamics. In addition, the discrete linear cascade model (Szollosi-Nagy 1982; Szilagyi 2003) combined with the diffusion-wave equation, can be successfully employed under only minimally restrictive conditions for identifying the underlying mechanisms of the observed change. These restrictions include: (1) mean channel bottom slope of the study reach cannot decrease over time, and; (2) at each target gauging station mean cross-sectional channel depth must remain more or less proportional to stage over time. On this basis, the DLCM can be used to identify possible triggering mechanisms of rating-curve changes even when the available data includes only stage and discharge measurements.

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