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A simplified 2D model for meander migration with physically-based bank evolution

Davide Motta¹, Jorge D. Abad², Eddy J. Langendoen³, Marcelo H. Garcia⁴

Abstract

The migration rate calculated by numerical models of river meandering is commonly based on a method that relates migration rate to near-bank excess velocity multiplied by a dimensionless coefficient. Notwithstanding its simplicity, since the early 1980s this method has provided important insight into the long-term evolution of meander planforms through theoretical exercises. Its use in practice has not been as successful, since the complexity of the physical processes responsible for bank retreat, the heterogeneity in floodplain soils, and the presence of vegetation, make the calibration of the dimensionless coefficient rather challenging. This paper presents a new approach that calculates meander migration rates using physically-based streambank erosion formulations. The University of Illinois RVR Meander

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model, which simulates meandering-river flow and bed morphodynamics, is integrated with the streambank erosion algorithms of the US Department of Agriculture channel evolution computer model CONCEPTS. The performance of the proposed approach is compared to that of the more simple classic method through the application to several test cases for both idealized and natural planform geometry. The advantages and limitations of the approach are discussed, focusing on simulated planform pattern, the impact of soil spatial heterogeneity, the relative importance of the different processes controlling bank erosion (hydraulic erosion, cantilever, and planar failure), the requirements for obtaining stable migration patterns (centerline filtering and interpolation of bank physical properties), and the capability of predicting the planform evolution of natural rivers over engineering time scales (i.e., 50 to 100 years). The applications show that the improved physically-based method of bank retreat is required to capture the complex long-term migration patterns of natural channels, which cannot be merely predicted from hydrodynamics only.

Keywords: meander migration, migration coefficient, bank erosion, planform shape, computer model

1. INTRODUCTION

The modeling of meandering-river migration requires the simulation of the following processes: hydrodynamics, sediment transport, bed morphodynamics, and bank erosion. The hydrodynamic modeling resolves the mean and turbulent flow fields: e.g., primary and secondary flows, Reynolds stresses, and turbulent kinetic energy among other hydrodynamic parameters. In
bends, both curvature-driven (Prandtl's first kind) and turbulence-driven
(Prandtl's second kind) secondary flow can be present and alter the mor-
phology of the bed and banks, which then affects the anisotropy of the flow
(Akahori and Schmeeckle, 2002; Blanckaert and de Vriend, 2005). Because
the shear stresses exerted by the flow on the bed and banks control the
erosion and transport of the boundary materials, their modeling is critical.

Modeling of the sediment transport in meandering streams simulates the
transport of sediments as a combination of bed and suspended load, because
of the complex flow and the possible large difference between bed and bank
material particle sizes. Bed load is quantified using empirical formulations
(Garcia, 2008), and its direction is determined by the near-bed flow direction
corrected by the effect of bed slopes (Seminara and Tubino, 1989; Kovacs and
Parker, 1994; Talmon et al., 1995; Mosselman, 2005; Abad et al., 2008). The
suspended load is calculated using an advection-diffusion equation, where
the diffusion coefficient is related to the turbulence characteristics of the flow
(Lyn, 2008). Abad et al. (2008) show the application of this methodology
for the case of laboratory meandering channels.

Modeling of the bed morphodynamics provides the bed morphology at
different spatial scales, which allows for reproducing the feedback between
bed structures and flow field (Best, 2005), like the disruption of secondary
flows due to migrating bedforms (Abad et al., 2010) and the interactions be-
tween suspended sediment particles and bed morphology (Schmeeckle et al.,
1999).

Modeling bank erosion allows for simulating the migration of the me-
andering channel, which in turn affects hydrodynamics, sediment transport
and bed morphodynamics. In a bend, faster and deeper flow develops near
the outer bank, which causes bank erosion (Thomson, 1879). At the inner
bank, a point bar commonly forms, promoting bank accretion. Widening in
meandering channels may happen when outer bank retreat exceeds the rate
of accretion of the opposite bank (Nanson and Hickin, 1983).

Models of different degrees of sophistication have been used to simulate
freely meandering channels. For example, Ruether and Olsen (2007) per-
formed three-dimensional (3D) numerical modeling using Reynolds Averaged
Navier-Stokes (RANS) equations, a $k-\epsilon$ model for turbulence, an advection-
diffusion equation for suspended sediment, and van Rijn (1984)’s formulation
for bed load. Bolla et al. (2009) developed a 3D analytical model for low-
sinuosity meanders and steady bed morphology. However, these models need
coupling with bank erosion and meander evolution submodels to simulate
planform changes for engineering and geological time scales.

Two-dimensional (2D) analytical models for long-term river migration,
again valid for low-sinuosity meanders and steady bed morphology, were
developed, among others, by Ikeda et al. (1981), Blondeaux and Seminara
Sun et al. (1996), Sun et al. (2001), Zolezzi and Seminara (2001), and Lan-
caster and Bras (2002). These models calculate the migration rate based
on a method independently introduced by Hasegawa (1977) and Ikeda et al.
(1981). This method relates the migration rate to the near-bank excess ve-
locity multiplied by a dimensionless coefficient, and is referred to as the
classic or MC (Migration Coefficient) approach hereafter. The dimensionless
coefficient is obtained by means of calibration against field data and is typi-
cally a very small number ($10^{-7}$-$10^{-8}$). From a theoretical perspective, this method has provided fundamental insight into the planform evolution of meandering channels, but it has not been as successful in practical applications, where the calibration of the dimensionless coefficient can be challenging and unable to capture the observed migration patterns. Constantine et al. (2009) sought to establish a relation between the migration coefficient and measurable physical characteristics of the channel boundary materials using data from the Sacramento River, California, USA. This enables the estimation of the migration coefficient directly from field data for streams where historical data are unavailable or controlling conditions have changed.

Natural meander patterns show that meander migration is not continuous in time or space, giving rise to spiky or, in general, irregular and complex planform shapes. Part of this complexity derives from the hydrodynamic conditions. Frascati and Lanzoni (2009) were able to reproduce features observed in nature such as upstream- or downstream-skewed simple bends, compound bends, and multiple loops using a suitable hydrodynamic model that accounts for the full range of morphodynamic regimes in combination with the classic migration-coefficient approach for bank erosion. On the other hand, the complexity of the bank erosion processes due to heterogeneity of floodplain soils and vegetation can also produce complex meander planform patterns and bend shapes. The simple approach using a calibrated migration coefficient cannot adequately capture bank erosion complexity at the sub-bend scale, because it predicts a smooth centerline. There are many additional limitations to the MC approach. The linearity of the expression implies that the only bank retreat mechanism considered is particle-by-particle erosion (also
termed hydraulic or fluvial erosion). It does not explicitly account for local, episodic mass failure mechanisms like cantilever, planar, rotational, and seepage-induced failures, which can temporarily change local bank retreat rates thereby altering migration patterns. The formulation does not account for an erosion threshold. Further, it does not consider the effect of the bank geometry either, since it assumes vertical sidewalls. Finally, the classic approach does not consider the impact of the vertical heterogeneity of the bank materials and the associated differences in erodibility and shear-strength of the soils.

With the ongoing effort in both the United States and Europe to re-naturalize highly modified streams, it cannot be expected that assessment studies using the classic migration method will accurately simulate the response of meandering streams to in-stream and riparian management practices over engineering time scales (that is, a few years to decades or, in general, periods before cutoff occurrence). A new physically-based approach is therefore needed, which explicitly relates meander migration to the processes controlling streambank erosion.

This paper presents a new modeling approach that merges the functionalities of the RVR Meander toolbox (Abad and Garcia, 2006), which is a 2D long-term meander migration model based on Ikeda et al. (1981)’s model, with the physically-based streambank erosion algorithms of the CONCEPTS (CONservational Channel Evolution and Pollutant Transport System) channel evolution model (Langendoen and Alonso (2008); Langendoen and Simon (2008); Langendoen et al. (2009)). Darby et al. (2002) and Rinaldi et al. (2008) carried out similar efforts, however only for short reaches and simu-
lation periods. The paper describes the new physically-based methodology for computing river migration and presents model tests for idealized (sine-generated and Kinoshita curve) and observed planforms. The computed planform evolution is compared to that obtained with the classic method based on a migration coefficient.

2. MODEL DESCRIPTION

The modeling platform is composed of two main components. The first component simulates the hydrodynamics and bed morphodynamics. The second component simulates the channel migration. Since the main goal of this paper is to evaluate the performance of the new approach for bank retreat as compared to the classic method, it is coupled with a simple physically-based analytical model for hydrodynamics and bed morphodynamics of meandering streams, which is based on the model of Ikeda et al. (1981).

2.1. Hydrodynamics and bed morphodynamics model

Ikeda et al. (1981)’s model for hydrodynamics and bed morphodynamics provides an analytical solution of the 2D depth-averaged shallow water equations through linearization and adimensionalization techniques. However, the model does not explicitly solve for the morphodynamics of the bed, but prescribes it. The model herein adopted is a slightly modified version of that developed by Ikeda et al. (1981) and Johannesson and Parker (1985), and details on the derivation of the solution are presented by Garcia et al. (1994). Its main theoretical limitations are that coupling between hydrodynamics and sediment dynamics is absent and that the lateral redistribution
of streamwise momentum due to secondary currents is not taken into consider-
ation (Camporeale et al., 2007). The first issue can only be overcome
by using a more refined model which fully couples flow, sediment transport,
and bed morphodynamics (Johannesson and Parker, 1989b; Zolezzi and Sem-
inara, 2001). Without such coupling, only the sub-resonant behavior can be
described and resonance cannot emerge, since curvature is the only forcing
for the flow (Lanzoni et al., 2006; Frascati and Lanzoni, 2009). The second
issue can be indirectly addressed in Ikeda et al. (1981)’s model by increasing
the factor controlling the bed transverse slope, which itself depends on the
secondary flow (Johannesson and Parker, 1989a). This has justified the use of
this kind of models both in the study of long-term meandering river dynam-
ics (Howard and Knutson, 1984; Sun et al., 1996; Stolum, 1996; Edwards and
Smith, 2002) and in practical applications (Johannesson and Parker, 1985;
Garcia et al., 1994; Camporeale et al., 2007). Below we only present the
2D solutions of flow velocity, flow depth, bed elevation, and bed shear stress
used by our model.

Figure 1 defines the coordinate system and planform and cross section
configurations. The governing equations and their solution are expressed in
intrinsic coordinates: \( s^* \) is streamwise coordinate and \( n^* \) is transverse coor-
dinate. From hereon the superscript star indicates values with dimensions,
whereas the omission of the superscript star indicates a dimensionless quan-
tity.

Assuming that the ratio of channel width to radius of curvature is much
smaller than one, it is possible to derive a solution of the depth-averaged
flow velocity components in \( s^- \) and \( n^- \)-directions (\( U \) and \( V \), respectively) and
the flow depth \((D)\) that is composed of the solution for a straight channel (identified by the subscript \(ch\)) and a perturbation due to channel curvature (identified by the subscript \(1\))

\[
(U(s, n), V(s, n), D(s, n)) = (1, 0, 1) + (U_1(s, n), V_1(s, n), D_1(s, n))
\]  

The solution is normalized as \(U = U^*/U^*_ch\), \(V = V^*/U^*_ch\), and \(D = D^*/D^*_ch\), where \(U^*_ch\) and \(D^*_ch\) are the reach-averaged velocity and flow depth at a particular time. The perturbation variables \(U_1\), \(V_1\), and \(D_1\) are

\[
U_1(s, n) = a_1'(n)e^{-a_2's} + n \left( a_3'C(s) + a_4'e^{-a_2's} \int_0^s C(s)e^{a_2's} ds \right)
\]  

\[
V_1(s, n) = \frac{a_2'}{2}e^{-a_2's} \left( 2 \int_0^n U_1(0, n)dn - nU_1(0, n) + U_1(0, 1) \right) + \frac{a_4'}{2}(nU_1(s, n) - U_1(s, 1)) + \frac{a_5'}{2}(n^2 - 1)
\]

\[
D_1(s, n) = C(s)n \left( F^2 + \alpha \right)
\]
where $C = B^*C^* = -B^*d\theta/ds^* = B^*/R_0^*$ is curvature, $B^*$ is channel half-width, $\theta$ is the angle between the channel centerline axis and the horizontal axis, $R_0^*$ is local radius of curvature, $F_{ch}$ is reach-averaged Froude number, $\alpha$ is a coefficient relating transverse bed slope to curvature, and $a^\prime_1(n) = U_1(0, n) + nC(s = 0)$

$$a^\prime_2 = C_{f,ch}s_1\beta$$

$$a^\prime_3 = -1$$

$$a^\prime_4 = \beta C_{f,ch}\left(F_{ch}^2 + (\alpha - 1 + s_1) - \frac{D_{ch}^*}{U_{ch}^*} s_2(F_{ch}^2 + \alpha)\right)$$

$$a^\prime_5 = (1 - \alpha - F_{ch}^2)C_{f,ch} d_{ch} B^*/U_{ch}^* C$$

$$a^\prime_6 = \frac{U_{ch}^*}{D_{ch}^*} \frac{F_{ch}^2}{F_{ch}^2 + (\alpha - 1) \frac{U_{ch}^*}{D_{ch}^*} - s_2(F_{ch}^2 + \alpha)}$$

$$s_1 = 2$$

$$s_2 = \frac{U_{ch}^*}{C_{f,ch} \frac{\partial C_f}{\partial D^*}} = -\frac{5 U_{ch}^*}{D_{ch}^*} \sqrt{C_{f,ch}}$$

where $\beta = B^*/D_{ch}^*$ and the friction coefficient $C_f$ reads (following Engelund and Hansen (1967))

$$C_f = \left(6.0 + 2.5 \ln \left(\frac{D^*}{2.5 d_s^*}\right)\right)^{-2}$$

where $d_s^*$ is sediment particle size.

The bed shear stress components in $s$ and $n$ directions are calculated as

$$(\tau_s^*, \tau_n^*) = \rho C_f(U^*, V^*) \sqrt{U^{*2} + V^{*2}}$$

where $\rho$ is the density of water.
Note that the parameter $s_2$, defined by Eq. 12, derives from the assumption that the friction coefficient varies in space according to the variations in the water depth $D^*$. Assuming a spatially constant friction coefficient (equal to the reach-averaged value calculated with Eq. 13 using the reach-averaged depth), then $s_2 = 0$ and the hydrodynamic solution by Johannesson and Parker (1985) is recovered.

The above expressions allow for calculating the depth-averaged flow, bed shear stress, and bed elevation distributions at any given time step and for any given channel planform shape represented by the curvature of the channel centerline. Following Johannesson and Parker (1985)

$$C = -\left(\frac{dx}{ds}\frac{d^2y}{ds^2} - \frac{d^2x}{ds^2}\frac{dy}{ds}\right)$$  \hspace{1cm} (15)

where $x$ and $y$ are the dimensionless Cartesian coordinates.

In order to avoid the propagation of numerical errors related to the computation of the channel curvature, we use the three-point curvature smoothing method suggested by Crosato (1990) after every time step

$$C_i = C_{i-1} + 2C_i + C_{i+1} \frac{4}{4}$$  \hspace{1cm} (16)

where $i$ is cross section index. This filter removes spurious node-to-node oscillations in calculated curvature caused by the inaccuracies of the curvature calculation method (Eq. 15).

### 2.2. Channel migration

The normal bank retreat rate $\xi^*$ is defined as
\[ \xi^* = \frac{dn_b^*}{dt^*} \]  

where \( n_b^* \) is the transverse coordinate of the outer bank. Assuming that channel width is locally constant, the normal displacement of a meandering channel centerline equals that of the outer bank (Eq. 17). The coordinates of a point \( P \) located on the channel centerline then migrate as follows

\[
\frac{dx_P}{dt} = -\xi \sin \theta \\
\frac{dy_P}{dt} = +\xi \cos \theta
\]

where \( x_P = x_P^*/B^* \) and \( y_P = y_P^*/B^* \) are the dimensionless coordinates of the point \( P \), and time and displacement are normalized as \( t = t^*U_{ch}^*/B^* \) and \( \xi = \xi^*/U_{ch}^* \), respectively.

2.2.1. Classic approach for migration

Hasegawa (1977) and Ikeda et al. (1981) introduced the classic meander migration approach, which linearly relates \( \xi(s) \) to the dimensionless perturbation velocity at the outer bank \( U_1(s, n = 1) \)

\[ \xi(s) = E_0U_1(s, n = 1) \]

where \( E_0 \) is a calibrated erosion coefficient which depends on bank soil properties and riparian vegetation as well as the hydraulic properties of the flow. Eq. (20) assumes that the force of the flow exerted on the bank erodes the soil particles. Odgaard (1987), Hasegawa (1989), and Pizzuto and Meckelnburg (1989) have shown that this assumption agrees with observations of natural meanders.
2.2.2. Proposed approach for migration

Our new methodology relates migration rate directly to the physical processes controlling bank retreat, i.e. hydraulic erosion and mass failure, using the bank erosion methods of Langendoen and Simon (2008) and Langendoen et al. (2009). The new approach is also referred to as PB (Physically-Based) method hereafter. The bank erosion method accounts for natural bank profiles and the presence of horizontal soil layers. This is more realistic than the classic approach, which assumes vertical banks with vertically homogeneous soil properties represented by the erosion coefficient $E_0$. In the PB approach, the simulated retreat of the banks determines the migration of the centerline. Thus, the use of a migration coefficient calibrated against historic centerlines is therefore avoided. We further assume that eroded bank material is carried in suspension by the flow and transported out of the modeled channel.

The lateral hydraulic erosion rate $E^*$ for each bank-material layer is modeled using an excess shear stress relation, typically used for fine-grained materials

$$E^* = M^* \left( \frac{\tau^*}{\tau_c^*} - 1 \right)$$  \hspace{1cm} (21)

where $M^*$ is the erosion-rate coefficient (with dimensions of length over time) and $\tau_c^*$ is the critical shear stress. We assume that the bank shear stress $\tau^*$ equals the near-bank bed shear stress predicted by the hydrodynamic model at $n = \pm 1$. Note that at the banks the magnitude of the shear stress is equal to that of the shear stress in the streamwise direction ($\tau = \tau_s$) since $V = 0$.

Cantilever failures occur when overhanging blocks of bank material, generated by preferential retreat of more erodible layers at depth or simply by
the erosion of the bank below the water surface, fail (Figure 2a). For given unit weight and shear-strength properties, the extent of the overhang (or undercut) determines its stability. Because failed material is immediately transported out of the modeling reach, we can therefore assume that stability can be assessed using an arbitrary undercut threshold.

Figure 2: Bank failure mechanisms: (a) cantilever failure and (b) planar failure.

Planar failure (Figure 2b) is analyzed using a limit equilibrium method in combination with a search algorithm to determine the smallest factor of safety (stability factor), which is the ratio of available shear strength to mobilized shear strength. Shear strength is a combination of cohesive and frictional forces. The bank is unstable if the factor of safety is smaller than one, and a failure is then simulated. Potential failure blocks are subdivided in vertical slices, and a stability analysis is performed for each slice and for the entire failure block. Three different methods can be used for the computation of the factor of safety (Langendoen and Simon, 2008): (1) ordinary method, which does not consider interslice forces; (2) Janbu simple method, which
considers only interslice normal forces; and (3) Morgenstern-Price method, which considers both interslice normal and shear forces. The analysis also considers the possible formation of a tension crack on the floodplain behind the eroding bank face.

Note that, whereas hydraulic erosion is a continuous process in time (as long as the critical shear stress is exceeded), cantilever and planar failure processes are episodic. Details of the bank stability analysis are found in Langendoen and Simon (2008).

There are two alternatives to compute the centerline migration. The first option (Option 1, Figure 3a) consists of calculating the centerline displacement $\xi^* \Delta t^*$ at each cross section from the lateral displacement of the toe of the right bank $S_{i, right}^*$ and that of the toe of the left bank $S_{i, left}^*$

$$\xi^* \Delta t^* = \frac{\left[ S_{i, left}^* (t^* - \Delta t^*) - S_{i, left}^* (t^*) \right]^2}{2} - \frac{\left[ S_{i, right}^* (t^*) - S_{i, right}^* (t^* - \Delta t^*) \right]^2}{2}$$ (22)

Alternatively, the intersect of the bank and the water surface can be considered instead of the bank toe. After each time step $\Delta t^*$ the new width of each $i$-th cross section is then:

$$(B_i^*)_{new} = (S_{i, right}^*)_{new} - (S_{i, left}^*)_{new}$$ (23)

In order to migrate the dimensionless centerline its displacement is normalized by the half width of the channel $B^*$. The new dimensioned geometry can be recovered by multiplying by the half width. Because the channel width changes after each iteration the dimensionless centerline coordinates $x$ and $y$
are rescaled as:

\[(x_{\text{new}}, y_{\text{new}}) = (x_{\text{old}}, y_{\text{old}}) \frac{B^*_\text{old}}{B^*_\text{new}} \] \hspace{1cm} (24)

Use of this option will result in a new channel width that varies along the stream. However, the hydrodynamics model assumes a constant width, which we define as the minimum width among all cross sections:

\[B^*_{\text{new}} = \min(B^*_{i,\text{new}}) \] \hspace{1cm} (25)

The choice of the minimum width is governed by the fact that the model imposes a slip boundary condition at the sidewalls, which means that only the central region of the channel, where the effect of the sidewalls is not present, is actually modeled. Hence, for a series of cross sections characterized by different widths, the only constant-width channel which can represent the central region for all cross sections is that having a width equaling the minimum width among all the cross sections. Moreover, a sensitivity analysis of the hydrodynamic solution (2-4) showed that changes in bed shear stress \(\tau^*_s\) along the outer bank are relatively larger for increasing channel width than decreasing channel width. Therefore, the selection of minimum width minimizes possible errors in calculated long-term migration rates introduced by this method.

The second centerline-migration option (Option 2, Figure 3b) equates the dimensioned centerline displacement \(\xi^* \Delta t^*\) to the displacement of the outer bank. We define the outer bank of a cross section as the bank which experiences more erosion. If the outer bank is the left bank, the dimensioned centerline displacement is
Figure 3: Centerline migration options for the proposed physically-based approach: (a) Option 1 and (b) Option 2. Bank toe displacements (EL and ER at left and right bank respectively) determine centerline (CL) migration. Alternatively, the intersect points between banks and water surface can be used.

\[ \xi^* \Delta t^* = S^*_{i,\text{left}} (t^* - \Delta t^*) - S^*_{i,\text{left}} (t^*) \]  

(26)

otherwise it is

\[ \xi^* \Delta t^* = S^*_{i,\text{right}} (t^* - \Delta t^*) - S^*_{i,\text{right}} (t^*) \]  

(27)

Again, \( S^* \) can indicate either the bank toe or the intersect between bank and water surface. As in the classic approach we assume that the width of the channel is constant. Therefore, the inner bank displacement equals that of the outer bank.
The above migration options satisfy the constant-width requirement of Ikeda et al. (1981)’s model. However, note that a more robust description of channel migration requires a hydrodynamic model, which considers, besides the interactions between channel curvature and flow-bed topography, also the interactions associated with width variations and the mutual width-curvature interactions (Luchi et al., 2010).

2.2.3. Regridding and smoothing

Crosato (2007) demonstrated that simulating long-term channel migration eventually introduces spurious oscillations into the centerline geometry due to non-equidistant, increasing cross section spacing caused by centerline elongation. Following Sun et al. (1996) we use Parametric Cubic Splines (PCS) to extract a set of equally-spaced nodes after each migration step. When the amplitude of meander bends increases, regridding of the nodes using PCS introduces additional centerline nodes and reduces the spacing between consecutive nodes.

Redistribution and addition of centerline nodes require the interpolation of bank geometry and materials. Right and left bank geometry at a newly introduced node are obtained by interpolation of the bank geometry of the two existing centerline nodes located immediately upstream and downstream of the new node. Figure 4 illustrates the interpolation procedure. First, main chords are defined that connect the toe and floodplain points of the existing banks. Minor chords are then generated by connecting existing points on one left or right bank profile to interpolated points on the other left or right bank profile. The two points defining a minor chord have the same proportional distance to the main chords bounding the bank profiles. A tolerance
on minimum distance avoids closely-spaced minor chords. Linear interpolation along minor and main chords provides the bank geometry at the new centerline node. Soil properties for each bank-material layer are similarly obtained.

Figure 4: Sketch of the procedure for bank geometry interpolation.

The physically-based migration approach may locally produce large changes in centerline curvature, which requires the use of a filter to smooth the migrated centerline in order to avoid numerical instability. Our model adopts the Savitzky-Golay smoothing filter (Savitzky and Golay, 1964), which is a low-pass filter for smoothing noisy data and is applied to a series of equally-spaced data values, in this case the coordinates of the centerline nodes. The main advantage of this approach is that it tends to preserve features of the distribution such as relative maxima, minima and width, which are usually flattened by moving averaging techniques. The Savitzky-Golay filter is a generalization of the moving window averaging method. It uses a polynomial of degree $k$ to perform a local regression on a series of at least $k + 1$ nodes.
Following Fagherazzi et al. (2004) and Legleiter and Kyriakidis (2006), the model employs 2nd and 4th-degree polynomial filters with averaging windows of 5 to 13 nodes. In contrast to the curvature filter (16), the use of the Savitzky-Golay filter is optional because it is not meant to filter spurious node-to-node oscillations in channel centerline geometry but to smooth very large curvature gradients that may arise, for example, for highly skewed bends.

3. MODEL APPLICATIONS

We evaluate the performance of the new physically-based channel migration method for three different cases. Table 1 reports the run parameters. Since reach-averaged Froude number, friction coefficient, and half-width to depth ratio change over time as the reach-averaged depth varies with changing sinuosity, Table 1 reports the values corresponding to a straight channel inclined at the valley slope. The channel centerline alignment of the first two cases follows a sine-generated and a Kinoshita curve, respectively. The goal of these two cases is to illustrate the features and limitations of the new approach, the variety in calculated planform shapes, and the differences between the planform shapes simulated by the classic approach and our new approach. The third case tests the model against the observed migration of a reach on the Mackinaw River in Illinois, USA, and compares the results to those obtained using the classic approach. Since high curvatures may develop during the simulations, the friction coefficient computed using Eq. 13 can become undefined because of a negative local depth at inner banks. Therefore, we assumed $s_2 = 0$ and a spatially constant, reach-averaged friction
Table 1: Test case run parameters. \((Q\) is discharge; \(2B^*\) is channel width; \(S_0\) is valley slope; \(F^2_0\) is squared Froude number for straight channel characterized by valley slope; \(C_{f0}\) is friction coefficient for straight channel characterized by valley slope; \(\beta_0\) is half-width to depth ratio for straight channel characterized by valley slope; and \(\alpha\) is transverse bed slope parameter).

<table>
<thead>
<tr>
<th>Case</th>
<th>Shape</th>
<th>(Q) (m³/s)</th>
<th>(2B^*) (m)</th>
<th>(S_0)</th>
<th>(F^2_0)</th>
<th>(C_{f0})</th>
<th>(\beta_0)</th>
<th>(\alpha)</th>
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<td>0.146</td>
<td>0.0069</td>
<td>22.16</td>
<td>5</td>
</tr>
<tr>
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<td>Mackinaw River</td>
<td>46.2</td>
<td>38</td>
<td>0.0009</td>
<td>0.109</td>
<td>0.0082</td>
<td>17.06</td>
<td>5</td>
</tr>
</tbody>
</table>

A coefficient was used in the simulations.

### 3.1. Case 1: sine-generated channel

The centerline of a sine-generated meandering channel is expressed as

\[
\theta = \theta_0 \sin \left( \frac{2\pi s}{\Lambda} \right)
\]

where \(\theta_0\) is the angle \(\theta\) at the crossover point and \(\Lambda\) is the length of the channel centerline over one meander wavelength.

This case considers a 2,040-meter long (2,000-meter long sinuous section with 20-meter long straight entrance and exit sections) and 30-meter wide channel with \(\Lambda = 250\) m and \(\theta_0 = 55^\circ\), which corresponds to a relatively low sinuosity \(\Omega = 1.27\) (Figure 5). 511 equally-spaced nodes describe the initial centerline, yielding a node spacing of 4 m. In order to accurately represent the channel planform evolution we limit the maximum grid spacing by imposing \(\Delta s^* < 0.9B^*\). The initial channel cross section geometry is trapezoidal with a bottom width of 30 m, a top width of 34 m, and a bank height of 2 m (therefore the bank slope is 45°).
We simulated the centerline evolution for a 300-year period employing a time step of 0.2 years and centerline-migration option 2 using the displacement of the bank toes to compute the migration distance. We did not apply the Savitzky-Golay smoothing filter. Table 1 lists the various simulation parameters. We conducted two runs in which only fluvial erosion and cantilever failures of homogeneous banks were simulated for two different sets of critical shear stress ($\tau^*_c$) and erosion-rate coefficient ($M^*$) values. The erosion-rate coefficient was $5 \cdot 10^{-7}$ m/s for each run, while critical shear stress equalled 11.5 and 12 Pa, respectively. The maximum size of the cantilever overhang was arbitrarily set to 0.1 m.

Figure 5 shows the simulated shear stress distribution in streamwise direction ($\tau^*_s$) at the start of the simulation. The combination of width to depth ratio, sinuosity, Froude number, and friction coefficient produces a peak bank shear stress ($> 12$ Pa) just upstream of the crossovers, whereas it is below $\tau^*_c$ at the channel apices. This case therefore resembles that of a confined meandering river, e.g. Nicoll and Hickin (2010).

Figure 6 shows the simulated migration pattern. The use of a critical shear stress threshold for hydraulic erosion prevents some channel portions
from migrating. As a consequence, the simulated migration pattern after 300 years is strongly skewed and locations characterized by high curvature gradients arise. The lower critical shear stress results in a higher downstream migration rate. The simulated planform is very similar to that of confined meandering channels as is shown by the observed series of river bends along the Beaver River, Canada (Figure 7).

Figure 6: Simulated migration pattern of a sine-generated channel using the physically-based bank retreat method with two different critical shear stress values for hydraulic erosion. Flow is from left to right.

---

Figure 8 shows the comparison between the simulated migration pattern obtained using the classic method and the new physically-based method. The migration coefficient values used in the MC approach are $E_0 = 2 \cdot 10^{-9}$, $E_0 = 3 \cdot 10^{-9}$, and $E_0 = 4 \cdot 10^{-9}$. The PB and MC methods predicted different planform shapes. The MC approach cannot reproduce the pattern predicted by the PB approach. As the PB approach, the MC method also predicted a downstream migration of the meander bends with little growth of the meander amplitude, however it cannot produce the strong asymmetry simulated by the PB approach. From Figure 8 it is evident that the bank
retreat model strongly affects the migration pattern. The MC approach is only capable of simulating such strong planform asymmetry by assuming non-erodible valley boundaries. Howard (1992) and Howard (1996) did follow this approach for the confined Beaver River.

3.2. Case 2: Kinoshita channel

Kinoshita-generated meandering channels are described by (Kinoshita, 1961)

\[ \theta = \theta_0 \sin (\kappa s) + \theta_0^3 (J_s \cos (3\kappa s) - J_f \sin (3\kappa s)) \]  

where \( J_s \) and \( J_f \) are the skewness and flatness coefficients respectively, \( \kappa = \)
After 300 years, MC 1, $E_0 = 2 \cdot 10^{-9}$; MC 2, $E_0 = 3 \cdot 10^{-9}$; and MC 3, $E_0 = 4 \cdot 10^{-9}$. Flow is from left to right.

$2\pi / \Lambda$ is the wave number.

We consider a 4,200-meter long (4,000-meter long meandering section with 100-meter long straight entrance and exit sections) and 30-meter wide channel with $\Lambda = 500$ m, $J_s = \pm 1/32$ (positive for upstream-skewed configuration and negative for downstream-skewed configuration), $J_f = 1/192$ and $\theta_0 = 110^\circ$, which corresponds to a high sinuosity $\Omega = 3.28$. These parameters result in locally high-curvature reaches which violate the assumption of mild curvature of Ikeda et al. (1981)’s model. However, the below model tests were designed to minimize the impact of the high-curvature reaches on the model results. The initial cross section geometry is trapezoidal with a bottom width of 30 m, a top width of 34 m, and a bank height of 2 m. 421 equally-spaced nodes describe the initial channel centerline, which yields a node spacing of 10 m.

We assessed the performance of the new approach for four different scenarios: (1) a comparison with the MC method regarding the evolution of
upstream- and downstream-skewed meander bends; (2) spatially heterogeneous floodplain soils; (3) sensitivity analysis of the centerline migration method (cf. Section 2.2.2) and centerline smoothing method (cf. Section 2.2.3); and (4) influence of planar failures on centerline migration. Table 1 lists the main parameters used in the simulations.

3.2.1. Evolution of upstream- and downstream-skewed meander bends

We performed a 5-year simulation using: a time step of 0.2 years; centerline migration option 2 using the displacement of the bank toe to compute the migration distance; and the Savitzky-Golay smoothing filter with second order polynomial regression, an averaging window of 5 nodes, and applied every 10 iterations. We only considered hydraulic erosion and cantilever failure processes of homogeneous banks with $\tau_c = 5 \text{ Pa}$ and $M^* = 5 \cdot 10^{-7} \text{ m/s}$.

Figures 9 and 10 show the simulated centerline migration patterns for upstream- and downstream-skewed meander bends, respectively. The downstream-skewed meander bends migrate faster than those skewed upstream, as was already speculated by Abad and Garcia (2009). Also, while the upstream-oriented bends tend to preserve their orientation, the downstream-oriented bends are changing their orientation towards upstream. In fact, as pointed out by Lanzoni et al. (2006), the hydrodynamic model used here can only describe sub-resonant morphologic regimes and is therefore bound to produce, in homogeneous soil, upstream-skewed planform features. However, as later illustrated in the paper, floodplain soil heterogeneity can preserve downstream-oriented bends (Figure 15), which motivated the use of an initial downstream-skewed configuration for the simulations in Figures 10 and 15. Compared to the PB approach, the MC approach, with $E_0 = 2.5 \cdot 10^{-7}$,
simulates similar trends, see Figures 11 and 12. The results of the PB approach, however, show a more dramatic tendency towards the formation of necks for the downstream-skewed configuration, which is commonly observed for natural rivers. In Figure 9 and especially in Figure 10 some portions of bend are below the critical shear stress for hydraulic erosion, enhancing the spatial discontinuity of the migration process and generating zones characterized by very strong local curvature and curvature gradient. Although the above scenarios are idealized, they capture the preferential migration of only portions of a bend, which is often observed in nature such as in the case of the Pembina River in Alberta, Canada (Figure 13).

![Simulated migration pattern and shear stresses using the physically-based migration method for an upstream-skewed Kinoshita channel. Flow is from left to right.](image)

**Figure 9:** Simulated migration pattern and shear stresses using the physically-based migration method for an upstream-skewed Kinoshita channel. Flow is from left to right.

### 3.2.2. Spatially heterogenous floodplain soils

We conducted two simulation scenarios to study the effect of floodplain soil heterogeneity on meander migration. The first scenario considers an upstream-skewed Kinoshita channel with different soil critical shear stresses for the left-most 67% and right-most 33% of the river valley (Figure 14). The
erosion-rate coefficient is the same for each soil ($M^* = 5 \cdot 10^{-7}$ m/s), whereas the critical shear stresses are 4 and 10 Pa, respectively. In the second scenario the Kinoshita channel is downstream-skewed and the upper and lower halves of each meander bend are located in two different floodplain soils. The two soils alternate in downvalley direction (Figure 15). The erosion rate coefficient is the same for each soil ($M^* = 5 \cdot 10^{-7}$ m/s), whereas the critical shear stresses are 5 and 8 Pa, respectively. These idealized scenarios are able to show
Figure 12: Comparison of the simulated migration patterns obtained with the MC and PB approaches for a downstream-skewed Kinoshita channel. Flow is from left to right.

Figure 13: Preferential migration of bend portions at lobes A and B in the Pembina River in Alberta, Canada (Parker et al., 1982). Flow is from left to right. A reach characterized by very strong local curvature and curvature gradient is present in the downstream portion of lobe B.

that differences in bank soil properties can invert the migration pattern of upstream- and downstream-skewed Kinoshita channels in homogeneous soils (cf. Figures 9 and 10).
Figure 14 shows the results of the first scenario. The more erosion-resistant soil (i.e., soil A) locally inhibits channel migration. Figure 14a shows that upstream skewness increases compared to the homogeneous scenario (cf. Figure 9) if the majority of the channel is located within floodplain soil B (the more erodible soil). If the majority of the channel is located within soil A, the migration of the meander bends located within floodplain soil B causes these bends to become downstream-oriented (see Figure 14b).

Figure 15 shows the results of the second scenario. The cyclic variation in soil erosion resistance increases the migration rate of every other meander. As a consequence, the tendency of the downstream-oriented bends to change their orientation towards upstream is moderated (cf. Figure 10).

3.2.3. Sensitivity to migration and smoothing methods

We performed a sensitivity analysis to study the effects of the centerline migration (see Section 2.2.2) and smoothing (see Section 2.2.3) methods.
Figure 15: Simulated migration pattern of a downstream-skewed Kinoshita channel using the physically-based migration method. The floodplain comprises two different soils: in zone A, $\tau^*_c = 8$ Pa and $M^* = 5 \cdot 10^{-7}$ m/s; and in zone B, $\tau^*_c = 5$ Pa and $M^* = 5 \cdot 10^{-7}$ m/s. Flow is from left to right.

Table 2 lists the 16 cases evaluated. We also analyzed the impact of using either the displacement of the bank toe or that of the intersect between water surface and bank profile in calculating centerline migration. The Kinoshita channel was upstream-skewed and its configuration is the same as for the above test cases. The simulation period was 10 years.

Figure 16 presents the most relevant results of the analysis. There are only minor differences between the various cases in the zones characterized by high curvature (indicated with circles in Figure 16a), because shear stress only exceeds the critical shear stress downstream of the bend apex (cf. Figure 9). Figure 16b shows that near the crossover points, however, the centerline migration method has the largest effect on the planform evolution. The simulated centerline migration in cases 3 and 7 is significantly different from that in cases 11 and 15. The migration rates are lower when using Option 1 since the displacement of the eroding outer bank is partially counteracted.
Table 2: Sensitivity analysis test cases (IF = Iteration Frequency, i.e. iteration interval for the application of centerline filtering or bank interpolation).

<table>
<thead>
<tr>
<th>Case</th>
<th>Centerline migration method</th>
<th>Bank point used for migration</th>
<th>Filtering</th>
<th>Bank interpolation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Option 1</td>
<td>Toe</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>2</td>
<td>Option 1</td>
<td>Toe</td>
<td>Yes (IF = 10)</td>
<td>No</td>
</tr>
<tr>
<td>3</td>
<td>Option 1</td>
<td>Toe</td>
<td>No</td>
<td>Yes (IF = 10)</td>
</tr>
<tr>
<td>4</td>
<td>Option 1</td>
<td>Toe</td>
<td>Yes (IF = 10)</td>
<td>Yes (IF = 10)</td>
</tr>
<tr>
<td>5</td>
<td>Option 1</td>
<td>Water surface intersect</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>6</td>
<td>Option 1</td>
<td>Water surface intersect</td>
<td>Yes (IF = 10)</td>
<td>No</td>
</tr>
<tr>
<td>7</td>
<td>Option 1</td>
<td>Water surface intersect</td>
<td>No</td>
<td>Yes (IF = 10)</td>
</tr>
<tr>
<td>8</td>
<td>Option 1</td>
<td>Water surface intersect</td>
<td>Yes (IF = 10)</td>
<td>Yes (IF = 10)</td>
</tr>
<tr>
<td>9</td>
<td>Option 2</td>
<td>Toe</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>10</td>
<td>Option 2</td>
<td>Toe</td>
<td>Yes (IF = 10)</td>
<td>No</td>
</tr>
<tr>
<td>11</td>
<td>Option 2</td>
<td>Toe</td>
<td>No</td>
<td>Yes (IF = 10)</td>
</tr>
<tr>
<td>12</td>
<td>Option 2</td>
<td>Toe</td>
<td>Yes (IF = 10)</td>
<td>Yes (IF = 10)</td>
</tr>
<tr>
<td>13</td>
<td>Option 2</td>
<td>Water surface intersect</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>14</td>
<td>Option 2</td>
<td>Water surface intersect</td>
<td>Yes (IF = 10)</td>
<td>No</td>
</tr>
<tr>
<td>15</td>
<td>Option 2</td>
<td>Water surface intersect</td>
<td>No</td>
<td>Yes (IF = 10)</td>
</tr>
<tr>
<td>16</td>
<td>Option 2</td>
<td>Water surface intersect</td>
<td>Yes (IF = 10)</td>
<td>Yes (IF = 10)</td>
</tr>
</tbody>
</table>

by that of the eroding inner bank (Figure 3a). This then raises the question which method is more appropriate. Observations show that many meandering alluvial streams, in the long term, maintain a roughly constant width even while actively migrating (Ikeda et al., 1981). Alluvial streams accomplish this by balancing erosion at one bank with deposition at the opposite bank. We therefore suggest Option 2 for long-term simulations in which the assumption of constant width is reasonable from empirical observations. The model, however, does not provide a physically-based description of the deposition processes which lead to the reconstruction of the inner bank. Option 1 requires the selection of a representative channel width to calculate the hydrodynamics and the bed morphodynamics because Ikeda et al. (1981)’s analytical model assumes a constant channel width (see also Section 2.2.2). Obviously this introduces an error, which however can be considered neg-
ligible compared to other approximations introduced in the modeling if the simulation period is not too long (a few years). Then, the differences between the hydrodynamics and bed morphodynamics computed for a constant-width channel are not too different from those calculated for a varying-width channel. We therefore suggest to use Option 1 for relatively short-term simulations, which are associated with small longitudinal changes in channel width resulting from net erosional processes. A more accurate description of the hydrodynamics and morphodynamics of varying-width channels requires either a fully non-linear 2D depth-averaged numerical model or, as previously mentioned, an analytical solution which also considers width variations, such as that developed by Luchi et al. (2010). Note that the cross section geometry obtained with Option 1 is not that corresponding to morphologic equilibrium, since only the flow in the central region of the channel is actually computed. However, bank shape is in equilibrium after the bank erosion processes are solved.

Figure 16: Comparison of simulated migration patterns for the different test cases listed in Table 2. Flow is from left to right.
3.2.4. Effects of planar failure mechanics on channel centerline migration

We studied the impact of planar bank failures on migration patterns by varying the shear strength of the bank soil. The Kinoshita channel is upstream-skewed with a similar geometry as used earlier. However, the initial streambanks are vertical and comprise a single soil with $\tau^*_c = 5$ Pa, $M^* = \frac{5}{10^{-7}}$ m/s, and a saturated unit weight of $18$ kN/m$^3$. The cohesion of each soil was $5$ kPa, whereas the friction angle was varied as $26^\circ$ (soil 1) and $20^\circ$ (soil 2). We used the Morgenstern-Price bank stability analysis method for the computation of the factor of safety. The lateral displacement of the intersect of bank and water surface determined centerline migration.

At a monitoring cross section located in the downstream portion of one of the Kinoshita bends, a planar failure (factor of safety is 0.91) occurs at the beginning of the simulation in the case of soil 2 (Figure 17d). The bank then retreats due to the combined action of hydraulic erosion and cantilever failure, preserving the shape of the bank profile. No planar failures occur in the case of soil 1 during the simulation period at the cross section considered (Figure 17c); the soil shear-strength and limited bank height inhibit bank failure. In both cases the bank retreat rate decreases in time since the channel tends to straighten at the location considered with consequent decrease of the streamwise shear stress at the bank. At the end of the first year of simulation, bank retreat is greater in the case of soil 2, because of the occurrence of planar failure. After ten years, however, bank retreat is greater for the case of soil 1, due to the evolution of the local hydrodynamic conditions, which are determined by local and upstream curvatures.

The impact of planar failures is mainly limited to the shape of the cross
Figure 17: Impact of the planar failure mechanism on the simulated migration pattern for an upstream-skewed Kinoshita channel. Flow is from left to right. (a) Simulated migration pattern after 10 years with soils 1 and 2; (b) detail of the simulated migration pattern (centerlines every one year for soil 2 and final centerline only for soil 1) with location of the monitoring node; (c) evolution of the left bank geometry at the monitoring node with soil 1; (d) evolution of the left bank geometry at the monitoring node with soil 2.

section, while the migration rates and distances are only weakly affected (Figures 17a and b). However, mass failures will generally affect the local flow field and failed material may temporarily protect the bank toe from eroding. We do not consider these effects here because the focus is on long-term migration patterns at the reach scale. The results shown in Figure 17 seem to confirm the observations by Constantine et al. (2009), who found a correlation between migration coefficient ($E_0$) and erosion-rate coefficient ($M^*$),
implying that the effects of mass failure mechanisms on channel migration rate can be accounted for by adjusting $M^*$. 

3.3. Case 3: Mackinaw River

We tested the new meander migration model against the observed migration of a reach on the Mackinaw River in Illinois, USA, and compared the results to those obtained using the MC approach. The reach is located in Tazewell County about 15 kilometers upstream of the junction of the Mackinaw River with the Illinois River between the towns of South Pekin and Green Valley (Figure 18c). Figure 19 shows aerial imagery of the study reach in the years 1951 and 1988.

An equidistant grid of 300 nodes with a spacing of 13.33 m describes the initial channel centerline (year 1951). Channel width is 38 m and valley slope is 0.0009 m/m (Garcia et al., 1994). The mean sinuosity of the study reach is 1.34. From an analysis of the discharge record between 1922 and 1956 at the USGS station 05568000 near Green Valley, we derived a model discharge of 46.2 m$^3$/s, which is between the average value (20.9 m$^3$/s) and the maximum value (61.6 m$^3$/s) of the mean annual streamflow over the period. Table 1 lists the values of the other parameters which characterize the simulation. The upstream boundary for modeling was set in a straight reach. Since the velocity distribution is not known there, a uniform profile in the transverse direction was assumed for simplicity.

Simulated bank retreat in the PB method is a combination of hydraulic erosion and cantilever failures. Because no bank-material data are available, we assumed homogeneous bank material and calibrated $M^* = 1.2 \cdot 10^{-6}$ m/s and $\tau_c^* = 9$ Pa. This rather large value of critical shear stress accounts for
the effects of temporary bank toe protection by failed bank materials and the presence of riparian vegetation visible in Figure 19, and implicitly accounts for the absence of a transfer function from near-bed to near-bank shear stress and the omission of shear-stress partitioning between skin friction, responsible for hydraulic erosion, and bedform friction. Further, we used: (a) channel centerline migration Option 2 using bank toe displacement; (b) a 2nd-order Savitzky-Golay filter with an averaging window of 5 nodes applied every 10 iterations; and (c) interpolation of bank physical properties every
Figure 19: Historic aerial photographs of the Mackinaw River study reach in the years 1951 and 1988. Flow is from right to left.

10 iterations. For the MC approach, we calibrated two alternative values of migration coefficient $E_0$: $5.0 \cdot 10^{-7}$ and $6.5 \cdot 10^{-7}$.

Figure 20 compares the simulated centerlines using the MC and PB methods to that observed in 1988. The simulated channel centerline using the PB method agrees well with that observed away from the boundaries of the model reach. In terms of planform shapes, the PB approach can capture the growth of the four upstream lobes (L1, L2, L3, and L4), which preserve their symmetry while migrating. The MC method produces shapes for lobes L1,
L3, and L4, which are characterized by strong upstream skewness. Lobes L1 and L3 develop a compound-loop shape which cannot be reproduced by the MC approach. The PB method also performs remarkably well also in the downstream portion of the reach (lobes L5, L6, and L7). Tuning the value of the migration coefficient (MC 1 or MC2 in Figure 20) can match the observed pattern in one bend or the other, but in general the predicted migration is biased in terms of lateral migration (especially in the downstream portion of the reach) and downstream migration.

As a measurement to quantify model performance, we calculated the ratio of the area between simulated and observed centerlines to the length of the observed centerline, which is equivalent to an average distance between simulated and observed centerlines. This distance is 94.6 m (about 2.5 times the channel width) for the MC 1 simulation, 126.5 m (3.3 times the channel width) for the MC 2 simulation, and 67.1 m (1.8 times the channel width) for the PB simulation. Therefore the prediction error using our new method is respectively 29 and 47% smaller than that of the classic method.

4. CONCLUSIONS

To quantify the migration of meandering streams researchers have been using an empirical formulation that relates channel migration rate to excess near-bank velocity and a migration coefficient. This approach requires the calibration of the migration coefficient against historic channel centerlines, and therefore does not explicitly relate channel migration to the processes controlling streambank retreat. A new physically- and process-based method was developed that relates channel migration to the streambank erosion pro-
Figure 20: Comparison between historic and simulated 1988 channel centerlines of the Mackinaw River study reach. MC: migration coefficient method (MC 1, $E_0 = 5.0 \cdot 10^{-7}$; MC 2, $E_0 = 6.5 \cdot 10^{-7}$). PB: physically-based method ($M^* = 1.2 \cdot 10^{-6}$ m/s and $\tau^*_c = 9$ Pa). Flow is from right to left.

cesses of hydraulic erosion and mass failure. Hence, channel migration depends on measurable soil properties, natural bank geometry, distribution of riparian vegetation, and both vertical and horizontal heterogeneity of flood-
plain soils. This approach is suitable for long-term simulation of migration patterns of natural rivers.

The presented test cases show that the planform shapes obtained with the physically-based migration method differ from those produced using the classic approach. The new approach is able to simulate features such as high skewness and sharp necks, which are commonly observed in nature. In particular, it is capable of modeling downstream skewness of meander bends (Figure 14b), compound loops (Figure 20), “rectangular” shapes (Figure 8), and preferential migration of some portions of a bend (Figures 9 and 10). The test cases also show that spatial heterogeneity of floodplain soils is as important as the simulated hydrodynamics in determining the planform evolution. Mass failure mechanisms like planar failures are important but can be represented by modifying the resistance to erosion parameters $\tau_c^*$ and $M^*$ to quantify long-term migration rates. Application of the proposed approach to the Mackinaw River in Illinois, USA, showed significant improvements over the classic approach in predicting the migration of natural streams.

Notwithstanding the above improvements, the model still has limitations regarding the simulation of hydrodynamics and bed morphodynamics. Most importantly, the model assumes: (a) uniform bed material; (b) constant water discharge (therefore, the impact of unsteady flows on river migration is not taken into account); (c) constant channel width to compute the hydrodynamics and bed morphodynamics; (d) eroded bank soils are transported as suspended load out of the modeling reach; and (e) there is no net aggradation or degradation along the channel. In particular, the last two limitations highlight the need of coupling sediment transport and bank erosion: since gravi-
tational bank failure depends on bank height, bed aggradation/degradation due to sediment transport imbalance or deposition of failed bank material at the toe could affect bank erosion.

Future studies will evaluate: (1) the effects of vertical bank-soil heterogeneity and bank shear stress distribution on migration patterns; (2) the impact of horizontal soil heterogeneity on migration patterns; (3) the need of more advanced linear models of meandering river hydrodynamics (e.g., Zolezzi and Seminara (2001)), possibly accounting for width variations (Luchi et al., 2010); (4) the impact of centerline filtering on the migration patterns, especially for very long-term scenarios; and (5) model applications to natural rivers with measured bank-soil physical properties.

The current version is a stand-alone platform for Windows and Linux operating systems. It is currently being migrated to a GIS (Geographical Information Systems) environment. It can handle the incorporation of other linear and fully nonlinear 2D numerical models. The model presented can be applied for river restoration and remeandering projects, in particular to assess the degree of instability of meandering channel designs.

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6. REFERENCES


