IC-55 SIMULATION OF THE DEFORMATION OF A RIVER CROSS SECTION BY A 1D-3D MODEL

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Changes in the form of an alluvial channel due to erosion or deposit of sediments is often modelled through the computation of the boundary shear stress. For engineering purposes, sediment parameters are not accurately known. Thus, shear stress should be computed by a simple and rapid method rather than an accurate one. First, we thus developed a geometrical method to compute the shear stress in an irregular cross section. This method called Merged Perpendicular Method was derived from the normal area method but gave more precise results. The objectives of this paper are, on the one hand, developing a method that might be applied to any cross section and, on the other hand, integrating it in a more general model of evolution of an alluvial channel. The results of a one dimensional hydrodynamic model is used as a basis, which decreases the computing time and also the number of initial data and of boundary conditions. For hydraulic computation in 1D we used RUBAR3. This code solves de Saint Venant equation by an explicit second order Godunov type numerical scheme 1D hydrodynamic equations are completed by the equation of conservation of the sediment mass (Exner equation) . The results obtained by the 1D model are distributed in the section transversely by the method developed in this paper. A 3D evolution of the bed is then possible. The 3 dimensions are the river axis, the transversal from one bank of the river to the other one and the vertical direction. The method is then applied to a real reach.

IC-82 SIMPLIFIED DESIGN METHOD FOR TRANSMISSION CANALS

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A transmission canal loses water through seepage and evaporation. For economy, it should be divided into sub-sections and the cross-section for each of the sub-sections must be designed separately. This adds cost of transition in between two sub-sections, but the transition cost is overcome by reduced cost of the cross-section. Optimal design parameters for transmission canal based on the Manning equation are not available yet. This paper presents design equations for the least cost transmission canal considering earthwork cost which may vary with depth of excavation, cost of lining, and cost of water lost as seepage and evaporation from irrigation canals of triangular, rectangular, and trapezoidal shapes. This optimization problem is some sort of a dynamic programming, which is complicated due to unknown number of subsections i.e. number of unknown constraints. The problem was expressed in dimensionless form and then solved numerically. The optimal design equations along with the tabulated section shape coefficients provide a convenient method for the optimal design of a transmission canal. These optimal design equations and coefficients have been obtained by analyzing a very large number of optimal sections resulted from application of optimization procedure in the wide application ranges of input variables. The analysis consists of conceiving an appropriate functional form and then minimizing errors between the optimal values and the computed values from the conceived function with coefficients. Using the proposed equations along with the tabulated section shape coefficients, the optimal number of subsections and corresponding cost of a transmission canal can be obtained in single step computations.

SIMULATION OF THE DEFORMATION OF A RIVER BY A 1D-3D MODEL

Saeed R. Khodashenas[1]

ABSTRACT

Deformation of various kinds of cross-sections was computed with the hypothesis that scour or deposit were directly related to shear stress computed by the Merged Perpendicular Method. Final stabilized cross section agrees with theoretical stable shape. The results of a 1D model which computes the volume of sediment eroded or deposited between 2 cross sections are used as a basis; these volumes are transversely distributed in every section in relation with shear stress. The method is then applied to a real reach.

KEY WORDS

shear stress, sediment transport, bed deformation, stable channel

INTRODUCTION

Most of the codes which were developed in the past decades, like HEC6 (Thomas and Prashum, 1977), IALLUVIAL (Karim and Kennedy, 1982), SEDICOUP (Holly and Rahuel,1990), BRALLUVIAL(Holly et al., 1985), CHARIMA(Holly et al. 1990), were built on a one dimensional approach. The one dimensional simulation of the evolution of riverbed appears to be not sufficiently complete. To solve the requirements of the engineers and the real problems of the rivers, research is thus carried out to develop the techniques of calculation of mobile bed by using other models: multiple strip, two-dimensional and, even, three-dimensional. Among the models of this type, one will quote: GSTARS (Yang et al., 1988), TABS2 (Thomas et al., 1985) and MOBED2 (Spasojevic et al., 1990) and USTARS (Lee et al., 1997). The multidimensional codes are often developed for the resolution of local or specific problems, their calculating time is considerable moreover they require a broad knowledge of the initial data and boundary conditions, which is not obvious in the majority of the cases, and, because of the lack of the data, they consider simplified assumptions, which will move away the final results from reality. Change in the form of an alluvial channel due to erosion or deposit of sediments is often modelled through the computation of boundary shear stress. For engineering purposes, sediment parameters are not accurately known. Thus, shear stress should be computed by a simple and rapid method rather than an accurate one. Khodashenas and Paquier (1999) developed a geometrical method to compute the shear stress in an irregular cross section. This method called Merged Perpendicular Method (MPM) was derived from the normal area method but gave more precise results. The objectives of this paper are, on the one

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hand, developing a method that might be applied to any cross section and, on the other hand, integrating it in a more general model of evolution of an alluvial channel.

CALCULATING OF SHEAR STRESS DISTRIBUTION BY MERGED PERPENDICULAR METHOD

After sharing the wetted perimeter into small segments, the mediator of every segment is drawn. Every mediator that intersects the previous mediator will be merged with it; the two mediators will have same continuation (Figure 1).

$$\hat{L}_{j,j-1} = \frac{1}{2}(\hat{L}_j + \hat{L}_{j-1})$$

$$\hat{L}_{j,j-1,j-2} = \frac{1}{3}\left(2\hat{L}_{j,j-1} + \hat{L}_{j-2}\right)$$

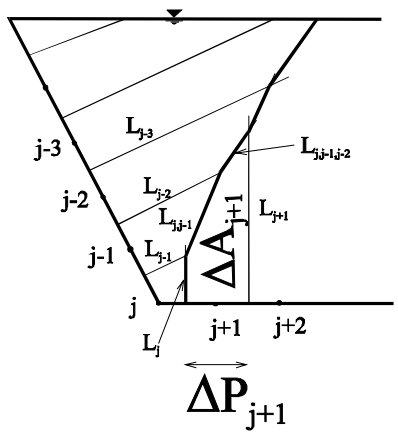

Figure 1- Merged Perpendicular Method (M.P.M.)

They join in a line of order 2. The direction of the new line is computed by weighted mean of the angles of the lines that intersect. Then, new lines can meet other normal and join into lines of higher order, the angle of which is weighted mean of the angles of the previous lines. This procedure continues till water surface. Area between the final lines is computed. Local boundary shear stress is computed by equation (1):

$$\tau = \gamma.R_h.S_f \qquad (1)$$

in which γ water specific weight, R_h hydraulic radius computed as the ratio of the area between 2 lines to the length of corresponding segment, S_f energy slope.

SIMULATION OF THE DEFORMATION OF A CROSS SECTION BY A 1D-3D MODEL

The following steps are proposed:

1- The results of a one dimensional hydrodynamic model is used as a basis, which decreases the computing time and also the number of initial data and of boundary conditions. For hydraulic computation in 1D, RUBAR3 was used. This code solves de Saint Venant equation by an explicit second order Godunov type numerical scheme (Paquier,1995).

2- 1D hydrodynamic equations are completed by the equation of conservation of the sediment mass (Exner equation). One size of sediment and constant Manning coefficient are supposed.

One dimensional deformation (or mean bed deformation), ΔS_{1D}, is computed by the Exner equation which is discretized as:

$$\Delta S = \frac{\Delta t \Delta Q_s}{(1-\lambda)\Delta X} \tag{2}$$

in which ΔX length of reach, Δt time step, λ porosity, Q_s =L.q_s sediment discharge, L width.

q_s sediment discharge rate that is computed by Meyer-Peter and Muller's relation (equation 3) (Graf and Altinakar, 1996) :

$$q_s = 8\sqrt{g d^3 (s_s - 1)}.\left(\zeta \tau^* - \tau_c^*\right)^{3/2} \tag{3}$$

in which d mean sediment size, $s_s=\rho_s/\rho$ relative density of sediment, g acceleration of gravity, $\zeta = \left(K_s / K_s'\right)^{3/2}$ is a roughness parameter in which K_s is total Manning-Strickler coefficient, $K_s' = 21/d^{1/6}$ grain Manning-Strickler coefficient, τ^* dimensionless shear stress and τ^*_c dimensionless critical shear stress.

3- The results obtained by the 1D model are distributed in the section transversely by the method developed in this paper. A 3D evolution of the bed is then possible; the 3 dimensions are the river axis, the transversal from one bank of the river to the other one and the vertical direction.

Change in bottom elevation in point j, $\Delta Z_{jinitial}$, is supposed to be proportional to the sediment discharge rate, q_s, computed in point j which means that in Meyer-Peter and Muller's relation τ^* and τ^*_c are computed in point j.

$$\Delta Z_{j\,initial} = \frac{8\Delta t}{(1-\lambda)\Delta X}\sqrt{g d^3 (s_s - 1)}.\left(\zeta \tau_j^* - \tau_{c_j}^*\right)^{3/2} \tag{4}$$

in which: τ^*_j dimensionless boundary shear stress in point j calculated by M.P.M., τ^*_c dimensionless mean critical shear stress and τ^*_{jc} dimensionless critical shear stress in point j that is computed by relation from (Ikeda ,1982):

$$\tau^*_{c\theta} = K\,\tau^*_{c0} \tag{5}$$

$$K = \frac{-\alpha\ {}^2\phi\ \theta + (\ {}^2\phi\ {}^2\theta + \alpha^2\ {}^2\phi\ {}^2\theta - {}^2\theta)^{.5}}{(1-\alpha\ \phi)\ \phi} \tag{6}$$

in which: τ^*_{c0} is dimensionless critical shear stress for horizontal bottom, $\tau^*_{c\theta}$ dimensionless critical shear stress for bank with slope θ, $\alpha = F_L / F_D$, F_L and F_D are respectively dimensionless lift and drag forces, ϕ angle of internal friction of sediment, θ side slope of cross section. For the tests here below, the following values are selected ϕ=35°, α=0.85

The total mass of the sediments that have moved, should not change between 1D computation and 2D computation, thus equality should be kept between the area of deformation directly obtained in 1D and the area of deformation obtained by adding the area related with all these segments (mentioned as 2D): $\Delta S_{2D}=\Sigma\Delta S_j=\Delta S_{1D}$. The areas from erosion

and deposition are separated, and different coefficients are applied to each area. Therefore, at first ΔS_{1D} and ΔS_{2D} are calculated, then, if the surface of deformation in 2D is larger than the surface of the deformation in 1D ($\Delta S_{2D}>\Delta S_{1D}$.), we understand that there is too much deposition and deposited surface is decreased. In the contrary case, if the surface of total deformation in 2D is smaller than the surface of the deformation in 1D ($\Delta S_{2D}<\Delta S_{1D}$.), we understand that there is too much erosion and eroded surface is decreased (Figure 2).

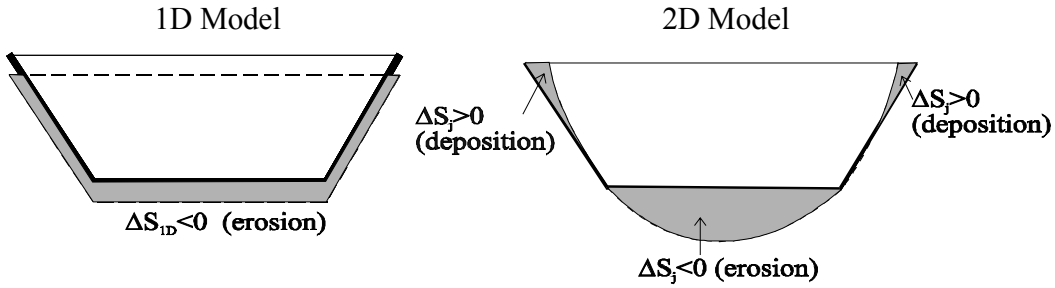

Figure 2- Difference between surface of deformation obtained in 1D and 2D

First case : when $\Delta S_{2D}>\Delta S_{1D}$

$$\sum \Delta \quad > \Delta \quad _{1} \Rightarrow \quad \sum_{\Delta S_{j\geq 0}} \Delta \quad + \sum_{\Delta S_{j<0}} \Delta \quad _{j} = \Delta S_{1D} \Rightarrow C_c = \frac{\Delta \quad _{1D} - \sum_{\Delta S_j<0} \Delta S_j}{\sum_{\Delta S\geq 0} \Delta S_j} \tag{7}$$

$$\begin{aligned} &if \quad \Delta S_j \geq 0 \Rightarrow \Delta z_{j\ final} = C_c \times \Delta z_{j\ initial} \\ &if \quad \Delta S_j < 0 \Rightarrow \Delta z_{j\ final} = \Delta z_{j\ initial} \end{aligned} \tag{7a}$$

Second case: when $\Delta S_{2D}<\Delta S_{1D}$

$$\sum \Delta \quad \leq \Delta \quad _{1} \Rightarrow \sum_{\Delta S_{j\geq 0}} \Delta \quad + \quad \sum_{\Delta S_{j<0}} \Delta \quad _{j} = \Delta S_{1D} \Rightarrow C_c = \frac{\Delta \quad _{1D} - \sum_{\Delta S_j\geq 0} \Delta S_j}{\sum_{\Delta S<0} \Delta S_j} \tag{8}$$

$$\begin{aligned} &Si \quad \Delta \quad _{j} \geq 0 \Rightarrow \Delta \quad _{j\ final} = \Delta z_{j\ initial} \\ &Si \quad \Delta \quad _{j} < 0 \Rightarrow \Delta \quad _{j\ final} = \quad _{c} \times \Delta z_{j\ initial} \end{aligned} \tag{8a}$$

VALIDATION OF MODEL BY COMPARISON WITH STABLE CHANNEL

In the steady and uniform conditions, in the absence of outside influences, a channel attains a stable shape. Diplas and Vigilar(1992) have studied the geometry of a stable channel. They proposed a fifth-polynomial bank profile to represent the shape of threshold bank.

Figure 3 shows that the developed method leads to a stabilised shape similar to the stable shape from Diplas and Vigilar(1992). This figure concerns an irregular cross section (Initial water depth h=9m, water discharge rate Q= 30 m^3/s, initial longitudinal slope S=0.0001).

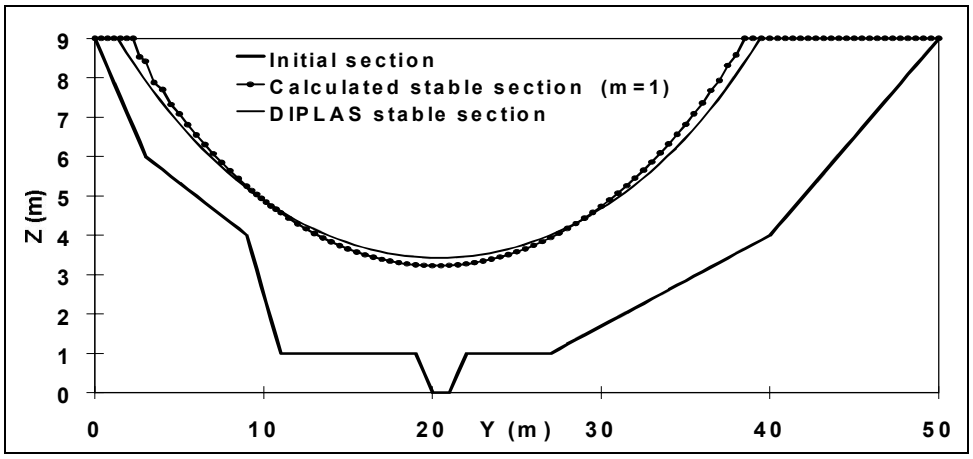

Figure 3- Comparison of calculated stable section with Diplas and Vigilar stable section

In a small flume, Stebbings (1963) sent a discharge into a flatbed of sediments in order to form a stable channel. The comparisons for three parameters (top width cross section, area and centerline channel depth of the stable channel) showed that the calculated values are in satisfactory agreement with the experimental data. Figure 4 shows the comparison for top width of stable section.

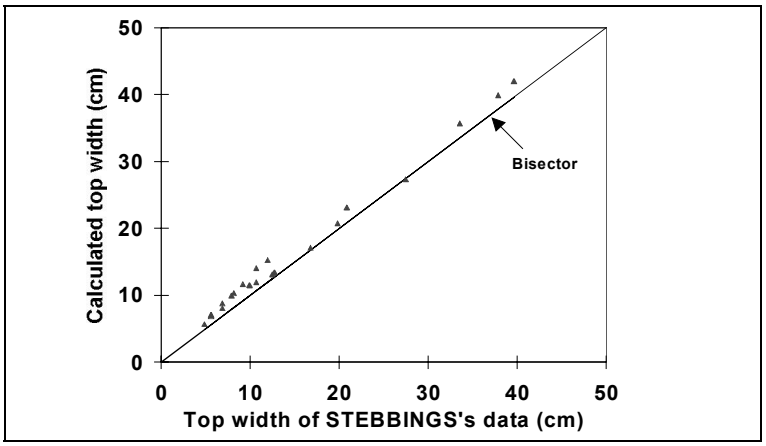

Figure 4- Comparison of top width of stable section

APPLICATION TO A REAL REACH

Figure 5 shows the map of the reach studied . This reach is situated between two dams, the Dam Seyssel and the Dam Motz on the Rhône river in the French Alps. The length of the reach is 2180 m and it includes 14 cross sections.

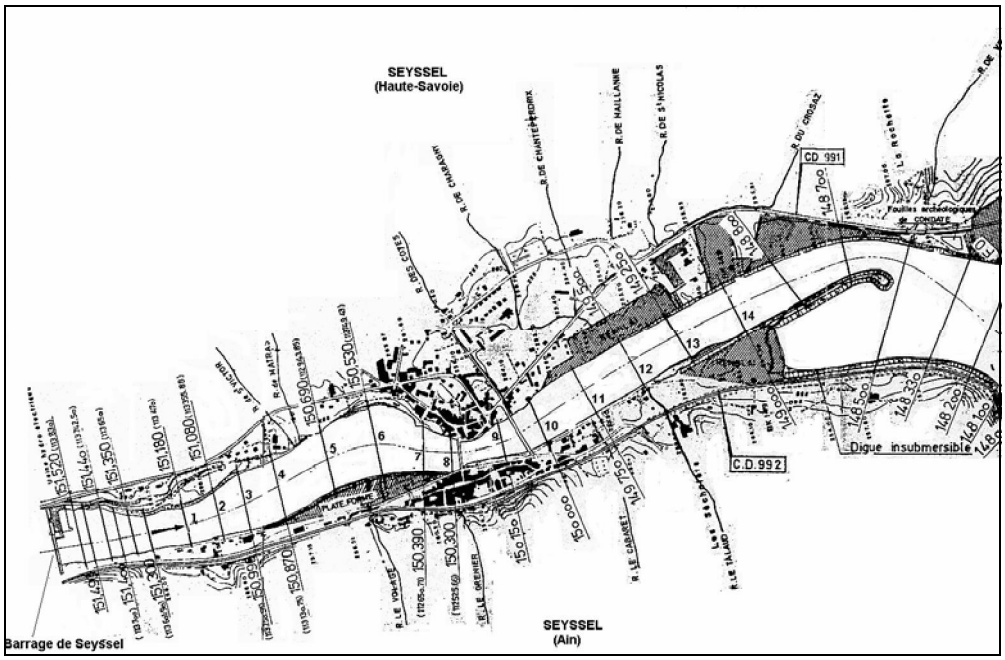

Figure 5- Situation of the studied reach of Rhône river

Initial and boundary conditions: The observations show that the diameter of the sediments in this part of Rhône is very large. A median diameter (d_{50}=3 cm) with a porosity (λ = %30) and a density of the sediments (ss=2,6) were proposed. Geometry of the bottom is defined by 14 cross sections for two dates June 1990 and April 1993. There is no information on the sediment discharge upstream of the reach. Maximum capacity of sediment transport in the first section upstream of the reach is then introduced in the reach.

RESULTS

Figure 6 shows the deformation calculated by the model over the period from June 1990 to April 1993 in the two sections 7, 8. Of course, one 3 years period is not sufficient to observe a significant deformation in the river, except if there are several high floods during this time. In the majority of the sections there is no significant deformation and the model confirms this point. In sections 7 and 8, the model is not far from measurements. In section 7, the calculated deformation has a shift to the left compared to measurements. This phenomenon can be due to the meander of section 7. In section 8, eroded sediment in the middle of the bed was a previous deposit of finer sediment.

Finally, the differences may be due: uncertainty of the sedimentological, and hydraulic data, errors to the topographical measurements used as references, existence of phenomena and obstacles like the meanders and the bridges which are not taken into account by the assumptions of the model, numerical errors of the computer code.

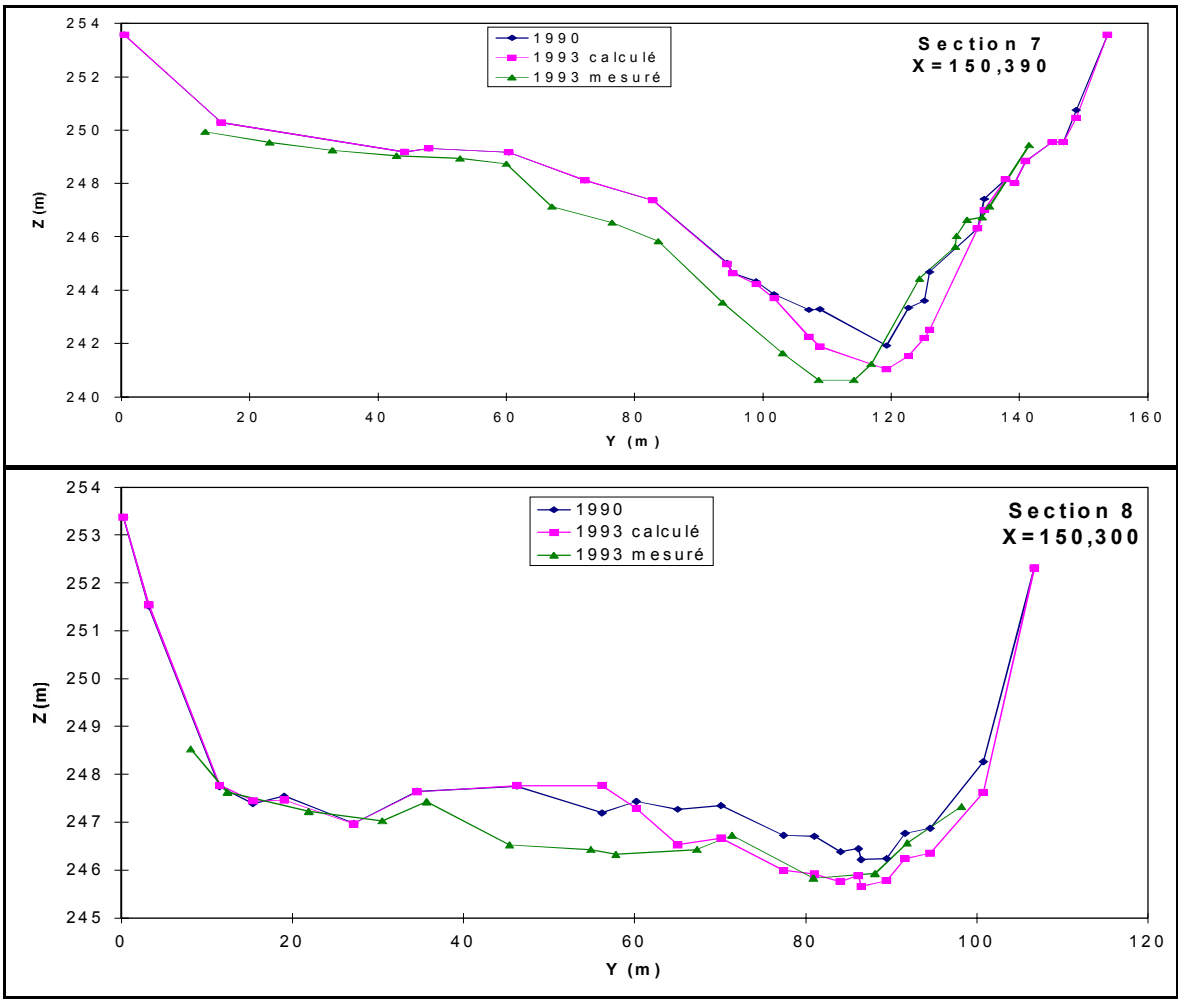

Figure 6- Deformation of the reach studied of Rhone during 1990-1993 in 2 sections 7,8

CONCLUSION

The introduction of a new geometrical method for computing boundary shear stress in classical 1D bed-load sediment model provides a model that changes the 3D topography of the river bed in a realistic way. Compared to theoretical examples or simple laboratory experiments, the accuracy of the model is sufficient.

The study of a real reach shows that the model gives rather satisfactory results. The difficulties in such real cases stand in the data requirements (precise topography, size of the sediments and the complexity of the real phenomena. Particularly two points could be included to the developed model: the influence of the meander could be represented by a correction to bottom shear stress of point j, τ_j; the variation of size of the sediment in places could be included in the critical shear stress of point j, τ_{cj}.

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