

Author's response to Editor decision and comments from Reviewer

A methodological approach of estimating of resistance to flow under unsteady flow conditions

M.M. Mrokowska, P.M. Rowiński, M.B. Kalinowska

We would like to thank the Editor and Reviewer for quick positive response. In revised version we improved the passages pointed by the Reviewer: abstract, section 3.3.1 (kinematic wave concept) and conclusions. We also improved quality of 3 figures (Figs 9, 10, 11). We hope this revision is satisfactory for further processing of this paper.

We provide answers to comments from Reviewers below. For clarity each answer is structured as follows: (1) RC# comments from Referees, (2) AR# author's response. Below please find attached a marked-up version of the manuscript.

Answers to Comments of Anonymous Reviewer #1

RC#1. The resistance approach is a lot better this time. I have a few issues with section 3.3.1 where a kinematic wave is discussed with dh/dx different from zero. Actually, by definition, the kinematic wave approximation requires that dh/dx , dU/dx and dU/dt are all zero. What is presented here is quite confusing.

AC#1 We have rewritten some paragraphs of section 3.3.1. to make the kinematic wave concept more clear to the reader. We have stressed that it was introduced to evaluate dh/dx for non-kinematic wave, and for this reason it has many theoretical inconsistencies. The reader could find the derivation of the formula and the theoretical background in literature cited.

RC#2 The analysis that is presented is much improved but the conclusion and abstract should be reworked to highlight what is truly important from this study. For instance, to conclude that Manning is constant during floods is not a great advancement of our science. To conclude that the analysis of unsteady flows is so complex that it precludes analysis is deflating the purpose of this article. It is also not a real conclusion isn't it. To conclude that the "diffusive equation" applies well to "waves of diffusive character" is quite a genuine revelation. Some conclusive statements are contradictory. For instance, it is stated that "weak effect of flow unsteadiness on Manning n " on line 401 and then the authors pursue with "trend of Manning n with flow rate does not provide unique information on the variability of resistance." If unsteady flows do not affect Manning n , then what are you talking about in terms of "variability of resistance". Also, on line 395 it is stated that "it is strongly recommended to apply the methodology presented in this paper" and this sounds like a Viagra commercial. Why should the reader use your method? OK, one last one – why do you state: "we believe that when resistance relations are applied with an awareness of their constraints..." in your conclusions. Do readers need religious beliefs for this type of

analysis? My point is that the conclusions and abstract need to be thoroughly reworked and cleaned up.

AC#2 We have rewritten the abstract to follow the objectives, methods and conclusions. We applied remarks of the Reviewer to the Conclusions: rewritten unclear passages, removed unnecessary text and highlighted conclusions.

RC#3 I like the photo – thanks – and the role of vegetation is completely ignored. Would grasses bend over and change resistance to flow during the flood event?

AC#3 We cannot completely ignore the role of bank vegetation as it is a source of resistance when the water level rise. Other studies (cited in the paper – Szkutnicki, 1996) analysed the impact of vegetation. In the case presented in our paper, which is from the spring, the impact is not so significant as during the summer.

RC#4 I am not convinced that the second example is necessary. I would think that one good example would do the job.

AC#4 We have kept both cases. In our opinion the comparison between the cases is valuable for the reader. They are unique data from the same experiment showing the effect of wave subsidence along the channel. They are of different character in terms of type of wave, which makes it possible to draw conclusions on impact of formulae simplification on the reliability of results.

RC#5 There are quite a lot of figures and some could be eliminated. Some color graphs may not come well in the final paper. Many figures could be improved.

AC#5 In our opinion all figures are necessary in this version of the paper. The number of figures is reduced compared to first submitted version. We have improved Figs. 9, 10, 11 by changing colours to make data sets easier to distinguish.

A methodological approach of estimating of resistance to flow under unsteady flow conditions

Magdalena M. Mrokowska¹, Paweł M. Rowiński¹, and Monika B. Kalinowska¹

¹Institute of Geophysics Polish Academy of Sciences, Ks. Janusza 64, 01-452 Warsaw, Poland

Correspondence to: M.M. Mrokowska (m.mrokowska@igf.edu.pl)

Abstract.

This paper presents evaluation and analysis of resistance ~~to flow expressed as parameters~~: friction slope, friction velocity and Manning ~~n -coefficient~~ in unsteady flow. ~~Measurements of flow parameters~~ The methodology to enhance the evaluation of resistance by relations derived from flow equations is proposed. The main points of the methodology are: (1) to choose a resistance relation with regard to a shape of a channel and (2) type of wave, (3) to choose appropriate method to evaluate slope of water depth, (4) to assess the uncertainty of result. Beside critical analysis of existing methods, new approaches are presented: formulae for resistance parameters for trapezoidal channel, and translation method instead of Jones formula to evaluate the gradient of flow depth.

5 Measurements obtained from artificial dam-break flood waves in a small lowland watercourse have made it possible to apply ~~relations for resistance derived from flow equations. The first part of the paper provides suggestions on how to apply this method to minimize the uncertainty of the results. Proposed methodology enhances the reliability of resistance evaluation the method and to analyse to what extent resistance parameters vary~~ in unsteady flow, ~~and may be particularly useful in research~~

10 ~~investigating impact of flow unsteadiness on hydrodynamic processes. In the second part of the paper, the results of friction slope, friction velocity and Manning n are analysed.~~ The study demonstrates that ~~unsteadiness of flow has larger impact on~~ results of friction slope and friction velocity ~~than on Manning n~~ are more sensitive to applying simplified formulae than Manning coefficient.

15 Manning n , adequate as flood routing parameter, may appear to be misleading when information

20 on ~~unsteadiness of flow~~ trend of resistance with flow rate is crucial. Then friction slope or friction velocity seems to be better choice.

1 Introduction

Resistance is one of the most important factors affecting the flow in open channels. In simple terms it is the effect of water viscosity and the roughness of the channel boundary which result in friction forces that retard the flow. The largest input into the resistance is attributed to water-bed interactions.

Resistance to flow is expressed by friction slope S which is dimensionless variable or boundary shear stress τ which refers directly to the shearing force acting on the channel boundary, with the dimension of Pascal [Pa]. Alternatively, shear stress is expressed in velocity units [m s^{-1}] by friction (shear) velocity u_* , which is related to the shear stress and friction slope by the equation:

$$u_* = \sqrt{\frac{\tau}{\rho}} = \sqrt{gRS}, \quad (1)$$

where g – gravity acceleration [m s^{-2}], ρ – density of water [kg m^{-3}]. Shear stress and friction velocity are crucial in research on hydrodynamics problems such as bed load transport (Dey, 2014), rate of erosion (Garcia, 2007), contaminants transport (Kalinowska and Rowiński, 2012; Kalinowska et al., 2012), turbulence characteristics of flow (Dey et al., 2011).

On the other hand, in engineering practice the resistance is traditionally characterised by Manning n , Chezy or Darcy-Weisbach coefficients. The flow resistance equation (Eq. 12) relating flow parameters through Manning n was originally derived for steady uniform flow conditions:

$$n = \frac{R^{2/3}}{U} S^{1/2}, \quad (2)$$

where R – hydraulic radius [m], U – mean cross-sectional velocity [m s^{-1}]. Its application is accepted for gradually varied flows for which friction slope can be approximated by bed slope I . Manning n was supposed to be invariant with the water stage; however, research has shown that the resistance coefficient very often varies (Ferguson, 2010; Fread, 1985; Julien et al., 2002). Furthermore, the trend of n versus flow rate Q may be falling or rising depending on the geometry of wetted area. Fread (1985) reported, based on computations of n from extensive data of flood waves in American rivers, that the trend is falling when inundation area is relatively small compared to inbank flow area; in reverse case the trend is rising.

In unsteady flow additional factors affect flow resistance compared to steady flow. As Yen (2002) presents after Rouse (1965), besides water flow-channel boundary interactions represented by skin friction and form drag, resistance has two more components: wave resistance from free surface distortion and resistance due to local acceleration or flow unsteadiness. Consequently, in order to evaluate resistance in unsteady flow it might be not sufficient to approximate friction slope S by bed slope I .

Large variety of methods of bed shear stress and friction velocity evaluation have been devised in order to study the flow resistance experimentally. The majority of methods measure bed shear stress indirectly, e.g. using hot wire and hot film anemometry (Albayrak and Lemmin, 2011), a Preston tube (Mohajeri et al., 2012), methods that take advantage of theoretical relations between shear stress

and the horizontal velocity distribution (Khiadani et al., 2005), methods based on Reynolds shear stress (Czernuszenko and Rowiński, 2008; Nikora and Goring, 2000) or turbulent kinetic energy (Pope et al., 2006), or methods that incorporate double-averaged momentum equation (Pokrajac et al., 60 2006). These methods are impractical or even impossible to be applied during flood wave propagation. Instead, a number of authors recommend formulae derived from flow equations (Ghimire and Deng, 2011; Graf and Song, 1995; Guney et al., 2013; Rowiński et al., 2000); nonetheless, this method needs further development because scarce measurement data very often restrict the relationships on resistance to simplified forms which provide uncertain results. Among simplifications applied in literature there are simplifications of momentum balance equation terms and simplifications that refer to the evaluation of $\frac{\partial h}{\partial x}$ the gradient of flow depth. This method requires flow velocity and flow depth as input variables and for this reason its practical application is restricted. However, it is a good choice for research purposes.

In this study we apply formulae derived from flow equations to obtain values of friction slope, Manning n and friction velocity given data on flow parameters. The objectives of this paper are twofold: (1) to enhance the evaluation of resistance to flow by relations derived from flow equations and by providing relevant methodology, (2) to analyse to what extent friction slope, friction velocity and Manning n vary in unsteady flow. The first objective could be valuable for those who would like to apply relations derived from flow equations to evaluate resistance and its impact on hydrodynamic 75 processes, e.g. sediment transport, while the other could be of interest to those who use resistance coefficients in modelling practice.

The paper is structured as follows: Section 2 presents settings of dam-break field experiment and measurement data. Methodology of evaluation of friction slope, friction velocity and Manning n in unsteady flow with focus on detailed aspects of application of formulae derived from flow 80 equations is outlined in Sect 3. In Sect 4 results of computations of friction slope, friction velocity and Manning n are presented for field experiment. In Sect 5 conclusions are provided. The problem presented herein has been partially considered in the unpublished Ph.D. thesis of the first author of this paper (Mrokowska, 2013).

2 Experimental data

85 The data originate from an experiment carried out in the Olszanka which is a small lowland watercourse in central Poland (see upper panel of Fig. 1) convenient for experimental studies. The aim of the experiment was to conduct measurements of hydraulic properties during artificial flood wave propagation. To achieve this goal, a wooden dam was constructed across the channel, then the dam was removed in order to initiate a wave. Then, measurements were carried out at downstream cross- 90 sections. Two variables were monitored: the velocity and the water stage. Velocities were measured by propeller current meter in three verticals of a cross-section at two water depths. Water stage was

measured manually by staff gage readings. Geodetic measurements of cross-sections were performed prior to the experiment. An in-depth description of the experimental settings in the Olszanka water-course may be found in (Szkutnicki, 1996; Kadłubowski and Szkutnicki, 1992), and a description of similar experiments in the same catchment is presented in (Rowiński and Czernuszenko, 1998; Rowiński et al., 2000).

In the study, two cross-sections, denoted in Fig. 1 as CS1 and CS2, are considered. Cross-section CS1 was located about 200 m from the dam, and cross-section CS2 about 1600 m from it. The shape of the cross-sections is presented in the bottom panel of Fig. 1. Both were of trapezoidal shape with side slopes of $m_1 = 1.52$, $m_2 = 1.26$ and $m_1 = 1.54$, $m_2 = 1.36$ for CS1 and CS2, respectively (Fig. 2). The bed slope I was 0.0004 for CS1 and 0.0012 for CS2.

Two data sets are used in this study, denoted as follows: OI-1, OI-2. Other data sets provided qualitatively similar results and therefore, for simplicity, are not presented herein. The first set was collected in cross-section CS1 and the other in cross-section CS2 during the passage of the same wave on 26 April 1990 at the beginning of vegetation season when banks were slightly vegetated (Fig. 3). The bed was composed of sand and silt with no significant bed forms. Figure 4 illustrates the results of the measurements – the temporal variability of mean velocity (U) and flow depth (h). Mean velocity has been evaluated by the velocity-area method from propeller current meter readings and flow depth has been calculated from geodetic data and measurements of water stage. Please note the time lag between maximum values of U and h , which indicates the non-kinematic character of the waves. Consider that waves represent a one-dimensional subcritical flow, with a Froude number ($Fr = U/\sqrt{gh}$) smaller than 0.33. The loop-shaped relationship between flow rate (Q) and water stage (H) may be observed in Fig. 5. From the figure it can be seen that the rating curves are not closed, which is probably caused by too short series of measurement data.

115 3 Methods

The methodology of evaluating resistance to flow from flow equations is proposed. It comprises four questions that need to be answered to obtain reliable values of resistance:

1. What is the shape of the channel – is simplification of the channel geometry applicable?
2. Is it admissible to apply simplified formula with regard to the type of wave?
- 120 3. What methods of evaluating input variables, especially $\vartheta = \frac{\partial h}{\partial x}$ the gradient of flow depth, are feasible in the case under study?
4. What is the uncertainty of the input variables, and which of them are most significant?

In proceeding sections a thorough review of each questioned issue is given. Methods used in the literature are facilitated with critical analysis, and some new approaches are proposed by the authors.

125 3.1 Relations for resistance in unsteady non-uniform flow derived from flow equations

In this study, resistance to flow is evaluated by formulae derived from flow equations – the momentum conservation equation and the continuity equation. Here we propose to evaluate resistance to flow for dynamic wave from the relations derived from the St. Venant model for trapezoidal channel (Mrokowska et al., 2013):

$$130 \quad U(b+mh)\frac{\partial h}{\partial x} + \left(b + \frac{m}{2}h\right)h\frac{\partial U}{\partial x} + (b+mh)\frac{\partial h}{\partial t} = 0, \quad (3)$$

$$\frac{\partial h}{\partial x} + \frac{U}{g}\frac{\partial U}{\partial x} + \frac{1}{g}\frac{\partial U}{\partial t} + S - I = 0, \quad (4)$$

where t – time [s], x – longitudinal coordinate [m], b – width of river bed [m], h – here: the maximum flow depth in the channel section (trapezoidal height) [m], $m = m_1 + m_2$, m_1 and m_2 – side slopes
 135 [-] defined as $m_1 = l_1/h$ and $m_2 = l_2/h$. The cross sectional shape with symbols is depicted in Fig. 2. Equation (3) is the continuity equation and Eq. (4) is the momentum balance equation which the terms represent as follows: the gradient of flow depth (hydrostatic pressure term), advective acceleration, local acceleration, friction slope and bed slope. Further on, derivatives will be denoted by Greek letters to stress that they are treated as variables, namely $\zeta = \frac{\partial U}{\partial t}$ [m s⁻²], $\eta = \frac{\partial h}{\partial t}$ [m s⁻¹],
 140 $\vartheta = \frac{\partial h}{\partial x}$ [-].

The friction slope derived analytically from the set of equations is represented by the following formula:

$$S = I + \left(\frac{U^2}{g} \frac{b+mh}{bh + m\frac{h^2}{2}} - 1\right)\vartheta + \frac{U}{g} \frac{b+mh}{bh + m\frac{h^2}{2}}\eta - \frac{1}{g}\zeta \quad (5)$$

To evaluate friction velocity and Manning n Eq. (5) is incorporated into Eq. (1) and Eq. (2),
 145 respectively:

$$u_* = \left[gR \left(I + \left(\frac{U^2}{g} \frac{b+mh}{bh + m\frac{h^2}{2}} - 1 \right) \vartheta + \frac{U}{g} \frac{b+mh}{bh + m\frac{h^2}{2}} \eta - \frac{1}{g} \zeta \right) \right]^{\frac{1}{2}}. \quad (6)$$

$$n = \frac{R^{2/3}}{U} \left(I + \left(\frac{U^2}{g} \frac{b+mh}{bh + m\frac{h^2}{2}} - 1 \right) \vartheta + \frac{U}{g} \frac{b+mh}{bh + m\frac{h^2}{2}} \eta - \frac{1}{g} \zeta \right)^{\frac{1}{2}}. \quad (7)$$

Equations (5, 6 and 7) are considered in this study, as Olszanka watercourse has nearly trapezoidal cross-section.

150 Flow equations for rectangular channel or unit width are the most frequently used mathematical models to derive formulae on resistance. A number of formulae for friction velocity has been presented in the literature, e.g.:

– Graf and Song (1995) derived the formula from the 2D momentum balance equation:

$$u_* = [ghI + (-gh\vartheta(1 - (\text{Fr})^2)) + (\eta - h\zeta)]^{\frac{1}{2}}, \quad (8)$$

155 – Rowiński et al. (2000), and next Shen and Diplas (2010) applied the formula derived from the St. Venant set of equations:

$$u_* = \left[gh \left(I + \left(\frac{U^2}{gh} - 1 \right) \vartheta + \frac{U}{gh} \eta - \frac{1}{g} \zeta \right) \right]^{\frac{1}{2}}. \quad (9)$$

– Tu and Graf (1993) derived the equation from the St. Venant momentum balance equation:

$$u_* = \left[gh \left(I + \frac{1}{C} \eta - \frac{1}{g} \zeta \left(1 - \frac{U}{C} \right) \right) \right]^{\frac{1}{2}}, \quad (10)$$

160 where C – wave celerity [m s^{-1}].

3.2 Simplifications of relations with regard to type of flow

If the acceleration terms of momentum balance equation for dynamic wave (Eq. 4) are negligible, they may be eliminated, and the model for a diffusive wave is obtained. Further omission of the hydrostatic pressure term leads to the kinematic wave model, in which only the term responsible for gravitational force is kept. According to Gosh (2014); Dooge and Napiórkowski (1987); Julien (2002), in the case of upland rivers, i.e. for average bed slopes, it could be necessary to apply the full set of St. Venant equations. Aricó et al. (2009) have pointed that this may be the case for mild and small bed slopes. Moreover, artificial flood waves, such as dam-break-like waves (Mrokowska et al., 2013), and waves due to hydro-peaking (Shen and Diplas, 2010), are of a dynamic character. On the other hand, when the bed slope is large, then the gravity force dominates and the wave is kinematic (Aricó et al., 2009). Because of the vague recommendations in the literature, we suggest analysing whether simplifications are admissible separately in each studied case.

Below we provide simplified relations for diffusive wave which are applied in this study:

$$S = I - \vartheta, \quad (11)$$

175 which is equivalent to water surface slope,

$$u_* = [gR(I - \vartheta)]^{\frac{1}{2}}, \quad (12)$$

$$n = \frac{R^{2/3}}{U} (I - \vartheta)^{\frac{1}{2}}. \quad (13)$$

Relations for steady flow are as follows:

$$S = I, \quad (14)$$

$$180 \quad u_* = (gRI)^{1/2}. \quad (15)$$

$$n = \frac{R^{2/3}}{U} I^{1/2}, \quad (16)$$

3.3 Evaluation of the gradient of flow depth ϑ

The evaluation of ϑ is widely discussed in hydrological studies on flow modelling and rating curve assessment (Dottori et al., 2009; Perumal et al., 2004). The gradient of flow depth is evaluated based
 185 on flow depth measurements at one or a few gauging stations. Due to the practical problems with performing the measurements, usually only one or two cross-sections are used.

3.3.1 Kinematic wave concept

~~According to the kinematic wave concept, the gradient of flow depth is evaluated implicitly based on measurements in one cross-section:-~~

190
$$\vartheta_{\text{kin}} = \frac{\partial h}{\partial x} = -\frac{1}{C} \frac{\partial h}{\partial t}.$$

$$\vartheta_{\text{kin}} = \frac{\partial h}{\partial x} = -\frac{1}{BC^2} \frac{\partial Q}{\partial t}.$$

~~This approach is encountered~~ Paradoxically, kinematic wave approximation is widely applied in cases of non-kinematic waves where $\frac{\partial h}{\partial x} \approx 0$, e.g. in friction velocity assessment studies (Graf and Song, 1995; Ghimire and Deng, 2011). ~~However, this method has been challenged in rating curve studies~~
 195 ~~(Dottori et al., 2009; Perumal et al., 2004) due to its theoretical inconsistency.~~ As Perumal et al. (2004) presented, Jones introduced ~~the this~~ concept in 1915 in order to overcome the problem of $\frac{\partial h}{\partial x}$ evaluation in reference to ~~looped rating curves, i.e.~~ non-kinematic waves. ~~The looped shape of non-kinematic waves results from the acceleration of flow and~~ According to the concept, the gradient of flow depth ~~is evaluated implicitly based on measurements in one cross-section:-~~

200
$$\vartheta_{\text{kin}} = \frac{\partial h}{\partial x} = -\frac{1}{C} \frac{\partial h}{\partial t}, \quad (17)$$

$$\vartheta_{\text{kin}} = \frac{\partial h}{\partial x} = -\frac{1}{BC^2} \frac{\partial Q}{\partial t}. \quad (18)$$

~~The application of this method has been challenged in rating curve studies (Dottori et al., 2009; Perumal et al., 2004) due to its theoretical inconsistency, as it neglects attenuation and subsidence of a flood wave (Henderson, 1963). The kinematic wave, on the other hand,~~ has a one-to-one relationship between the water stage
 205 and flow rate, which is equivalent to a steady flow rating curve. ~~Both rating curves are illustrated in the, while non-kinematic wave is loop-shaped (upper panel of Fig. 6 after (Henderson, 1963)).~~

~~In this study, the impact of kinematic wave approximation on arrival time of $\frac{\partial h}{\partial x} = 0$ is analysed. Both approximations, Eqs (17) and (18), affect the time instant at which $\frac{\partial h}{\partial x} = 0$.~~ As shown in the ~~upper panel of Fig. 6 the figure,~~ in the case of a non-kinematic subsiding wave, the peak of the flow
 210 rate $\frac{\partial Q}{\partial t} = 0$ in a considered cross-section is followed by the temporal peak of the flow depth $\frac{\partial h}{\partial t} = 0$,

while the spatial peak of the flow depth $\frac{\partial h}{\partial x} = 0$ is the final one. For the purposes of this study the true arrival time of $\frac{\partial h}{\partial x} = 0$ is analysed. The bottom panel of Fig. 6 presents schematically the true arrival time of $\frac{\partial h}{\partial x} = 0$ for the non-kinematic wave, and the arrival time approximated by the kinematic wave assumption in the form of Eq. (17) and Eq. (18). Both formulae underestimate the time instant at
 215 which $\frac{\partial h}{\partial x} = 0$. As a matter of fact, from the practical point of view, the evaluation of the friction velocity is exceptionally important in this region, as intensified transport processes may occur just before the wave peak (Bombar et al., 2011; De Sutter et al., 2001).

In order to apply the kinematic wave approximation, the wave celerity must be evaluated. Celerity can be assessed by the formula derived from the Chezy equation (Eq. 19) (Henderson, 1963), and it
 220 is applied in this study.

$$C = \frac{3}{2}U. \quad (19)$$

Tu and Graf (1993) proposed another method for evaluating C :

$$C = U + h \frac{\partial U}{\partial t} / \frac{\partial h}{\partial t}. \quad (20)$$

However, we would like to highlight the fact that in Eq. (20) $\frac{\partial h}{\partial t}$ is in the denominator, which con-
 225 strains the application of the method. As a result, a discontinuity occurs for the time instant at which $\frac{\partial h}{\partial t} = 0$. When the results of Eq. (20) are applied in Eq. (17), the discontinuity of ϑ as a function of time occurs at the time instant at which $C = 0$, which is between $t(\frac{\partial U}{\partial t} = 0)$ and $t(\frac{\partial h}{\partial t} = 0)$. This effect is illustrated in the section on field data application (Sect 4.1).

We propose another approach for evaluation of ϑ , which is compatible with the kinematic wave
 230 concept, but does not require the evaluation of temporal derivatives, and for this reason may appear to be easier to be used in some cases. Let us assume a reference cross-section P0 and two cross-sections P1 and P2 located at a small distance Δs downstream and upstream of P0, respectively. Knowing the $h(t)$ relationship, let us shift this function to P1 and to P2 by $\Delta t = \frac{\Delta s}{C}$ in the following way: $h_1(t) = h_0(t - \Delta t)$, and $h_2(t) = h_0(t + \Delta t)$. The spatial derivative $\frac{\partial h}{\partial x}$ is next evaluated as follows:

$$235 \quad \vartheta_{wt} = \frac{\partial h}{\partial x} = \frac{h_2(t) - h_1(t)}{2\Delta s}. \quad (21)$$

The method is denominated as wave translation method, and is applied in this study.

3.3.2 Linear approximation based on two cross-sections

Because of the drawbacks of kinematic wave approximation, it is recommended to evaluate the gradient of the flow depth based on data from two cross-sections (Aricó et al., 2008; Dottori et al., 2009;
 240 Julien, 2002), which is, in fact, a two-point difference quotient (backward or forward). Nonetheless, a number of problematic aspects of this approach have been pointed out. Firstly, Koussis (2010) has stressed the fact that flow depth is highly affected by local geometry. Moreover, Aricó et al. (2008) have pointed that lateral inflow may affect the evaluation of the gradient of flow depth, and for

this reason the cross-sections should be located close enough to each other to allow the assumption
245 of negligible lateral inflow. On the other hand, the authors have claimed that the distance between
cross-sections should be large enough to perform a robust evaluation of the flow depth gradient.
The impact of distance between cross-sections on the gradient of flow depth has been studied in
(Mrokowska et al., 2015) with reference to dynamic waves generated in a laboratory flume. The
results have shown that with a too long distance, the gradient in the region of the wave peak is mis-
250 estimated due to the linear character of approximation. On the other hand, with a too short distance,
the results may be affected by fluctuations of the water surface which in such case are large relative
to the distance between cross-sections.

Another drawback of the method is the availability of data. Very often, data originate from mea-
surements which have been performed for some other purpose. Consequently, the location of gauging
255 stations and data frequency acquisition do not meet the requirements of the evaluation of the gradient
of flow depth (Aricó et al., 2009). The latter problem applies to the case studied in this paper.

Due to the linear character of a two-point (backward and forward) difference quotient, it is not
able to represent properly the peak region of a flood wave. In (Mrokowska et al., 2015) it has been
stated that for better representation of ϑ the central difference quotient should be applied. Due to not
260 enough measurement cross-sections for Olszanka watercourse, in this study only two-point differ-
ence quotient is applied.

3.4 Uncertainty of resistance evaluation

The results of resistance evaluation should be given alongside the level of uncertainty. In the case of
unrepeatable experiments Mrokowska et al. (2013) have suggested applying deterministic approach
265 – the law of propagation of uncertainty (Holman, 2001; Fornasini, 2008). Let us denote dependent
variable as Y (here: S , n or u_*), and independent variables as x_i . Then maximum deterministic
uncertainty of Y is assessed as:

$$\Delta Y_{\max} \simeq \sum_{i=1}^n \left| \frac{\partial Y}{\partial x_i} \right| \Delta x_i. \quad (22)$$

The method is valid under the assumption that the functional relationship describes correctly the
270 dependent variable. In this method the highest possible values of uncertainty of input variables are
assessed based on the knowledge of measurement techniques and experimental settings. Hence, it
provides maximum uncertainty of a result.

4 Results

4.1 Evaluation of the gradient of flow depth

275 As presented in Sect 2 a number of measurements were performed in Olszanka watercourse. Nonethe-
less, the location and the number of cross-sections constrain the evaluation of spatial derivative ϑ .

It is feasible to use the data from only two subsequent cross-sections: for data set OI-1, ϑ could be evaluated based on cross-sections CS1 and CS1a located 107 m downstream of CS1, and for OI-2 based on CS2 and CS2a located 315 m upstream of CS2 (upper panel of Fig. 1).

280 The following methods of evaluating ϑ are examined and compared:

- Linear approximation denoted as ϑ_{lin}
- Kinematic wave approximation in the form of the Jones formula (Eq. 17), denoted as ϑ_{kin} with C evaluated from Eq. (19)
- Wave translation (Eq. 21) denoted as ϑ_{wt} proposed in this paper with $\Delta s = 10$ m, and C evaluated from Eq. (19)
- 285 – Kinematic wave approximation (Eq. 17) with C evaluated from Eq. (20) which is denoted as $\vartheta_{\text{Tu\&Graf}}$.

As can be seen from Fig. 7, ϑ_{kin} and ϑ_{wt} provide compatible results. Nonetheless, huge discrepancies in the ϑ_{lin} values are evident compared to ϑ_{kin} and ϑ_{wt} . The reason for this is that the linear method is applied to data from two cross-sections, which are located at a considerable distance apart. Moreover, due to the linear character of this method, ϑ_{lin} is unsuitable to express the variability of the flood wave shape. As a result, it overestimates the time instant at which $\vartheta = 0$ when the downstream cross-section is taken into account (as in OI-1), and underestimates the time instant when the upstream cross-section is used (as in OI-2). Next, the lateral inflows might have an effect on the flow, and thus the estimation of ϑ by the linear method. When it comes to $\vartheta_{\text{Tu\&Graf}}$, the results are in line with ϑ_{kin} and ϑ_{wt} except for the region near the peak of the wave where discontinuity occurs. This occurs due to the form of Eq. (20), which cannot be applied if $\frac{\partial h}{\partial t} = 0$, as was theoretically analysed in Sect 3.3.1. Consequently, the method must not be applied in the region of a rising limb in the vicinity of the wave peak and in the peak of the wave itself.

300 4.2 Evaluation of resistance to flow

Friction slope S , friction velocity u_* and Manning n are evaluated by formulae for dynamic, diffusive waves and steady flow. Wave translation method is used to assess ϑ . Results evaluated by formulae for dynamic wave are presented with uncertainty bounds, which allow to assess if the results obtained by simplified methods lie within the acceptable bounds or not. Uncertainty bounds are evaluated by the law of propagation of uncertainty. The uncertainties of the input variables are assessed based on knowledge of measurement techniques and experimental settings as follows: $\Delta h = 0.01$ m, $\Delta U = 10\% U$ (measurement performed by a propeller current meter), $\Delta R = 0.01$ m, $\Delta \zeta = 0.0001$ [m s⁻²], $\Delta \eta = 0.0001$ [m s⁻¹], $\Delta \vartheta = 0.00001$ [-], $\Delta I = 0.0001$, $\Delta m = 0.001$, $\Delta b = 0.01$.

4.2.1 Evaluation of friction slope

310 In order to assess to which category of flood wave (dynamic, diffusive or kinematic) the case under study should be assigned, the terms of the momentum balance equation are compared. The results are shown in Fig. 8. All terms are evaluated analytically from measurement data. For data set OI-1, the bed slope and the maximum flow depth gradient are of magnitude 10^{-4} , and the acceleration terms reach the magnitude of 10^{-4} along the rising limb. For OI-2 bed slope is of magnitude 10^{-3} ,
315 the maximum flow depth gradient is of magnitude 10^{-4} , and other terms are negligible. However, the acceleration terms are of opposite signs, and the overall impact of flow acceleration on the results might not be so pronounced. The comparison between OI-1 and OI-2 shows that in cross-section CS1, which is closer to the dam, more terms of the momentum balance equation are significant. From the results for CS2 it may be concluded that the significance of the temporal variability of
320 flow parameters decreases along the channel. In the case of data set OI-1, along the rising limb local acceleration term is slightly bigger than the advective one, which may indicate dynamic character of the wave. On the other hand, it may be concluded that the wave for OI-2 is of a diffusive character.

Fig. 9 presents comparison between the results of friction slope evaluated by formulae for dynamic wave S_{dyn} (Eq. 5), diffusive wave S_{dif} (Eq. 11) and approximated by bed slope I (Eq. 14). Values
325 of S_{dyn} range in the following intervals: [0.00027, 0.00085] for OI-1 and [0.0013, 0.0015] for OI-2 with the maximum before the peak of wave. Difference between values of S_{dyn} for OI-1 and OI-2 is affected to large extent by difference of bed slope between cross-sections CS1 and CS2.

In the case of data set OI-1 S_{dif} slightly differs from S_{dyn} along the rising limb of the wave. There are regions in which the results for diffusive wave lie outside the uncertainty bounds of friction slope
330 evaluated by formula for dynamic wave. This is another argument for choosing the formula for dynamic wave along the rising limb of the wave in OI-1. For the falling limb formula for diffusive wave may be applied. Steady flow approximation is not recommended in this case as the values of bed slope fall outside the uncertainty bounds in both rising and falling limb. In the case of OI-2 results of friction slope for both approximations – diffusive wave and steady flow are within uncer-
335 tainty bounds. However, the formula for diffusive wave is recommended, as it reflects the temporal variability of friction slope. With steady flow formula the information about friction slope variability during the propagation of wave is not provided. Before the peak of wave $S_{\text{dyn}} > I$ and after the peak $S_{\text{dyn}} < I$.

4.2.2 Evaluation of friction velocity

340 Figure 10 presents the comparison of the results of friction velocity evaluated by dynamic $u_{*\text{dyn}}$ (Eq. 6), diffusive $u_{*\text{dif}}$ (Eq. 12) and steady flow $u_{*\text{st}}$ (Eq. 15) formulae. The results for friction velocity are in line with the results of friction slope. Values of $u_{*\text{dyn}}$ range in the following intervals: [0.031, 0.052] for OI-1 and [0.057, 0.061] for OI-2 with the maximum before the peak of wave.

As can be seen from Fig. 10, the results for friction velocity in OI-1 obtained by the formula
345 for dynamic wave and the formula for diffusive wave agree well with each other along the falling
limb. The slight difference along the rising limb of the wave between the results occurs, as u_{*dif} falls
outside uncertainty bounds. This is caused by the acceleration terms, which appear to be significant
in OI-1 along the leading edge (Fig. 8). Consequently, in this region, the application of formula
for dynamic wave may be considered, while for falling limb formula for diffusive wave may be
350 applied. In the case of OI-1 u_{*dyn} and u_{*st} differ from each other. The results for steady flow formula
fall outside the uncertainty bounds along the substantial part of the wave, which indicates that the
application of steady flow approximation is incorrect. In the case of OI-2 diffusive wave formula may
be applied, as u_{*dyn} and u_{*dif} agree well with each other. Moreover, discrepancy between results for
dynamic wave and steady flow is smaller, and steady flow approximation might be considered in
355 friction velocity evaluation. However, then the information on maximum value of resistance along
rising limb is missing.

4.2.3 Evaluation of the Manning coefficient

Figure 11 presents the comparison of the results of Manning n evaluated by dynamic n_{dyn} (Eq. 7),
diffusive n_{dif} (Eq. 13) and steady flow n_{st} (Eq. 16) formulae.

360 Values of n_{dyn} range in the following intervals: [0.015, 0.039] for OI-1 and [0.024, 0.032] for
OI-2. The values of Manning n correspond with the values assigned to natural minor streams in the
tables presented in (Chow, 1959). The minimum values of OI-2 correspond with "clean straight, full
stage, no rifts or deep pools", while the minimum value of OI-1 does not match n for natural streams
presented in the tables. The maximum values may be assigned to "same as above, but more stones
365 and weeds". The Manning n coefficients have been evaluated in a completely different way for the
measurement data from this field site by Szkutnicki (1996); Kađubowski and Szkutnicki (1992).
In that study, n was treated as a constant parameter in the St. Venant model, and its value was
assessed by optimising the model performance. The authors have reported that for spring conditions,
 $n \in [0.04, 0.09]$. In this analysis, the results are smaller.

370 Results for n_{dyn} , n_{dif} and n_{st} follow the same trend achieving minimum values for time instant
of U_{max} . The results for Manning n obtained by the formula for dynamic wave and the formula
for diffusive wave agree well with each other in both cases: OI-1 and OI-2. Results obtained by
formula for steady flow differ slightly from n_{dyn} along the rising limb of OI-1, and lie on the edge
of uncertainty bounds, while n_{st} agree well with n_{dyn} in the case of OI-2. Consequently, Manning n
375 may be approximated by formula for diffusive wave along the rising limb of OI-1, while along the
falling limb of OI-1 and for OI-2 steady flow approximation may be applied.

4.3 The variability of resistance to flow during flood wave propagation

The variability of resistance in unsteady flow is very often analysed in terms of flow rate Q , and Manning n is considered as a reference variable (Fread, 1985; Julien et al., 2002). It seems reasonable to compare Manning n and friction velocity vs. flow rate Q . The comparison is illustrated in Fig. 12. As can be seen from the figure, Manning n decreases with increasing flow rate. This trend is characteristic of the majority of streams with inbank flow (Chow, 1959), which has been observed by Fread (1985) when the inundation area was relatively small compared to inbank flow area. This is the case considered herein, as the experiment was performed under inbank flow conditions. The reverse trend has been observed by Julien et al. (2002) for flood waves in the River Rhine. The authors discussed extensively impact of bed forms on Manning n . However, we would like to emphasise another aspect – the shape of inundation area which determines the reverse trend. In (Julien, 2002) interpretation of rising n as rising resistance is qualitatively correct, while in the case of Olszanka watercourse false conclusions may be drawn from the analysis of Manning n , that the bulk resistance decreases with flow rate. As the results for friction velocity show, the maximum values of resistance are in the rising limb of the waves, before the maximum flow rate Q .

5 Concluding remarks

~~Relations for Proper determination of resistance parameters: friction slope, friction velocity and the Manning coefficient have been derived from flow equations. Analyses based on experimental data have shown that resistance is highly uncertain and difficult to determine Manning coefficient in unsteady flow is very often hampered by scarcity or high uncertainty of input data.~~ However, we believe that when resistance relations are applied with an awareness of their constraints, and proper effort is made to minimise the uncertainty of the input data, ~~the method of resistance evaluation it~~ is likely to provide obtain reliable results. ~~For this reason, in-depth description of how to determine resistance for unsteady flow has been provided~~ To facilitate the evaluation of resistance parameters, we have proposed the methodology which provides means to enhance reliability of results obtained by relations derived from flow equations. The methodology ~~proposed in the paper should be helpful in reaching a compromise between scarcity of data and the correctness of simplifying assumptions.~~ ~~Beside~~ comprises four questions which help to judge if simplifications with regard to shape of a channel and type of wave are admissible, to decide which method of $\frac{\partial h}{\partial x}$ evaluation is the best in the case under study, and to evaluate the uncertainty of results. Besides critical analysis of existing methods we have proposed some new approaches: the formulae for ~~friction slope, friction velocity and Manning n resistance parameters~~ for trapezoidal channel and wave translation method instead of Jones formula to evaluate $\frac{\partial h}{\partial x}$. ~~The analysis of $\frac{\partial h}{\partial x}$ evaluation has shown that it is constrained by the spatial data, and this is the weakest point of application of relations for resistance. Hence, this element needs particular attention when resistance parameters are evaluated.~~

The paper has demonstrated the application of ~~resistance relations~~ proposed methodology to experimental data; hence, the detailed conclusions drawn in the study apply to similar cases. ~~Analysis~~ The methodology has been applied to assess if the simplified formulae are admissible. The analysis
415 of terms of momentum balance equation has revealed that in the provided identification of the type
of waves. In the first case – Ol-1, which is closer to the dam, the wave has dynamic character along
rising limb and diffusive character along falling limb. In the second case - Ol-2, the wave is of
diffusive character with relatively small difference between water slope and bed slope. Thanks to
the uncertainty analysis the reliability of the results of resistance parameters obtained by simplified
420 formulae has been assessed.

~~Comparison between the results of simplified formulae for~~ The analysis revealed that for S , u_*
and n has shown that evaluation of friction slope and friction velocity is more sensitive to flow
unsteadiness than Manning n . For this reason S and u_* should not be approximated by formulae
for steady flow, and the type of flow (dynamic or diffusive) should be carefully studied, and it is
425 strongly recommended to apply the methodology presented in this paper. The reliability of resistance
expressed by friction slope and friction velocity is critical for studies on the impact of variability of
flow resistance on hydrodynamic phenomena, e.g. sediment transport, which are amply represented
in literature.

~~When it comes to steady state formula is unacceptable, while for Manning n , in the case presented~~
430 ~~in this study the the steady flow approximation is admissible when the wave is of diffusive character,~~
~~and diffusive approximation is satisfactory for dynamic flow. This weak effect of flow unsteadiness~~
~~on~~. Hence, Manning n is less sensitive to simplifications of formulae than S and u_* . It is an as-
set when n is considered as a parameter in flood routing practice, because the reliability of the
reliability of results is less dependent on quality and quantity of data used. However, it constrains its
435 application in studies on variability of resistance in unsteady flow. Moreover, trend of The study has
demonstrated that S and u_* are better choice than Manning n with flow rate does not provide unique
information on the variability of resistance when information on variability and trend of resistance to
flow during flood wave propagation is required.

Flood wave phenomena are so complex that it is currently impossible to provide a comprehensive
440 analysis, and the problem of resistance to flow in unsteady non-uniform conditions still poses a chal-
lenge. For this reason, more research on resistance in unsteady non-uniform conditions is necessary.

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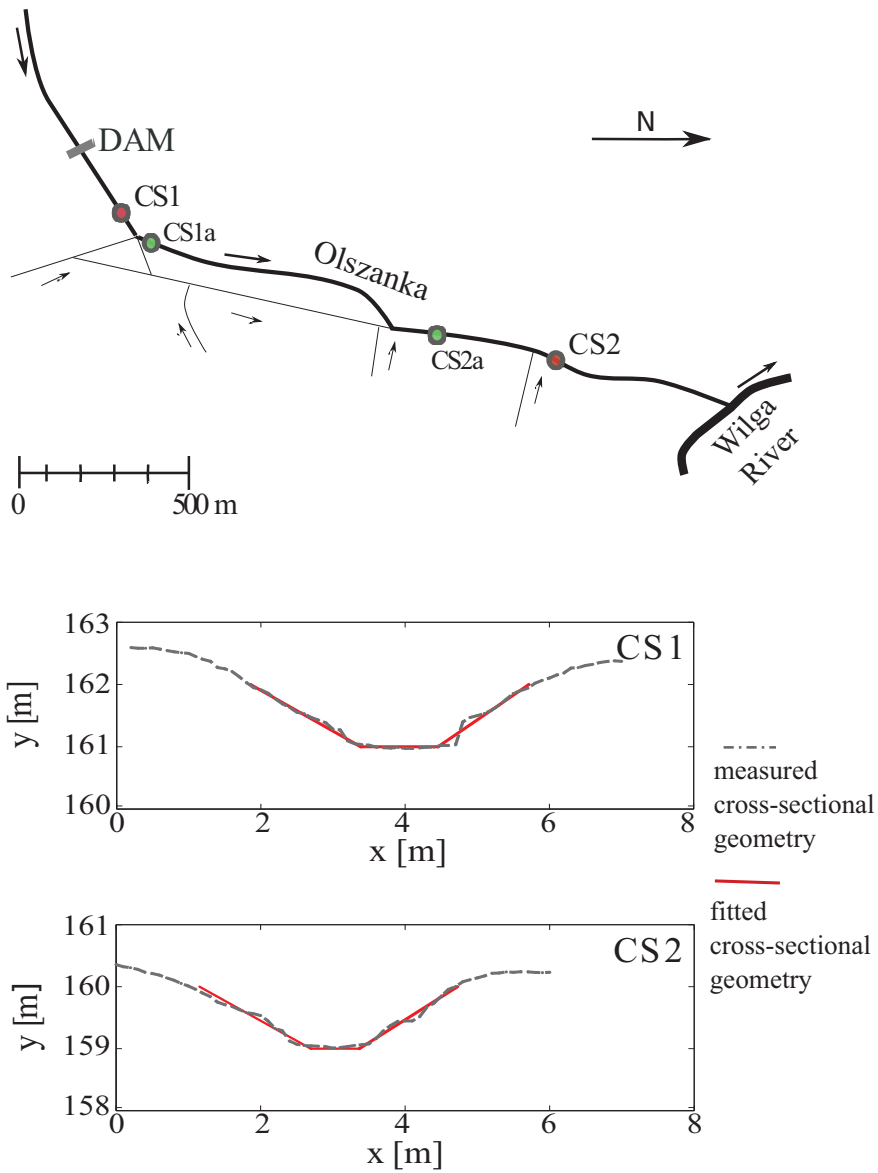


Figure 1. The site of the experiment in Olszanka watercourse (upper panel), and the shape of measurement cross-sections CS1 and CS2 (lower panel).

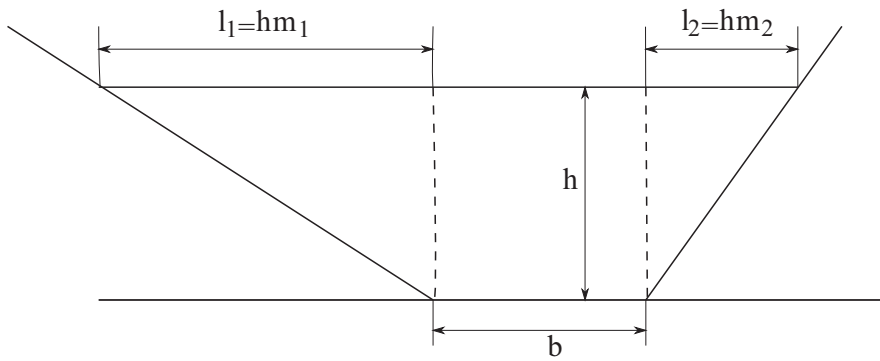


Figure 2. Trapezoidal cross-section of a channel with definitions of symbols used in the text.



Figure 3. Experimental reach of Olszanka watercourse. (courtesy of Jerzy Szkutnicki)

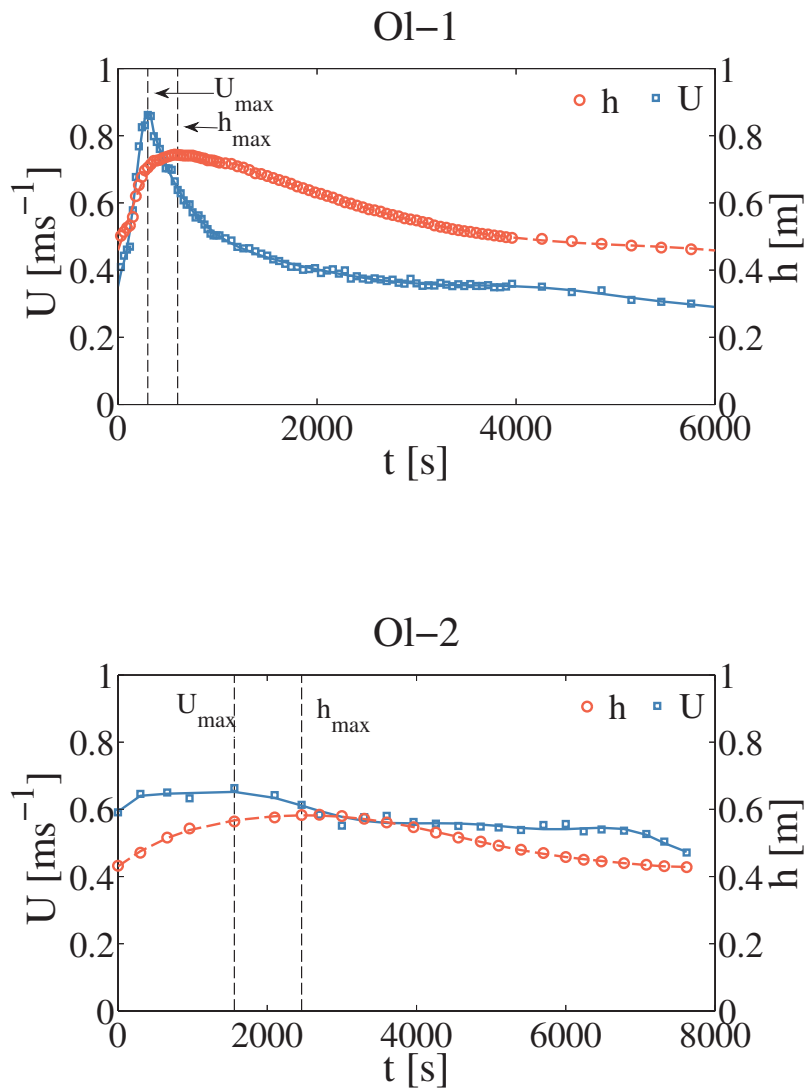


Figure 4. Temporal variability of flow depth h and mean velocity U for experimental flood waves in Olszanka watercourse.

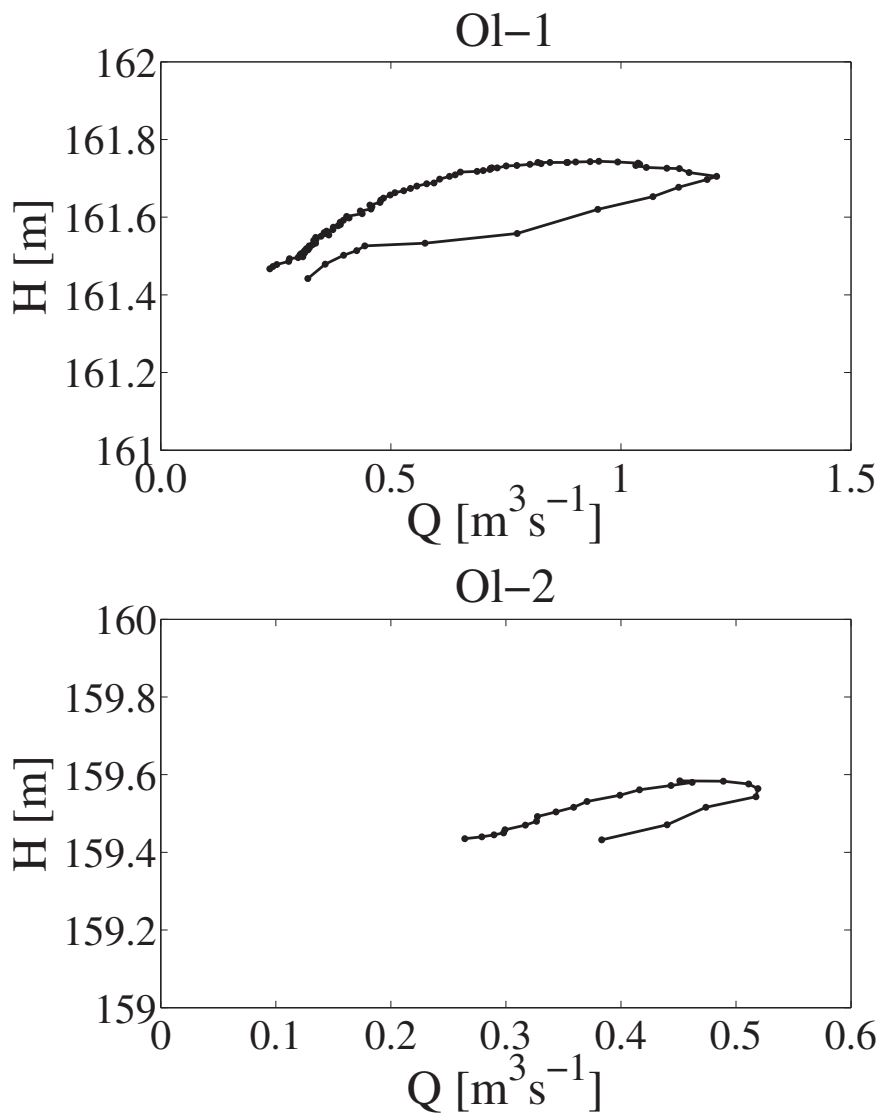


Figure 5. Rating curves of experimental flood waves in Olszanka watercourse.

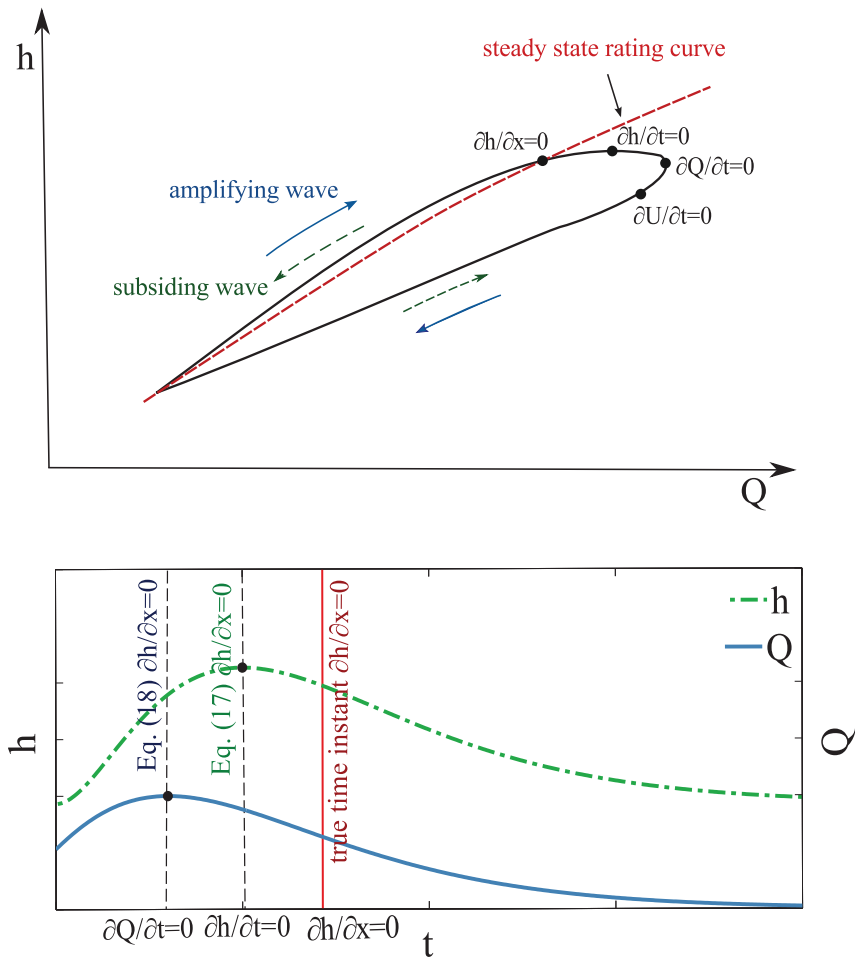


Figure 6. Comparison between rating curve for flood wave and steady flow with characteristic points, based on (Henderson, 1963) (upper panel), and impact of kinematic wave approximation (Eqs. (17) and (18)) on the assessment of time instant at which $\frac{\partial h}{\partial x} = 0$ (lower panel).

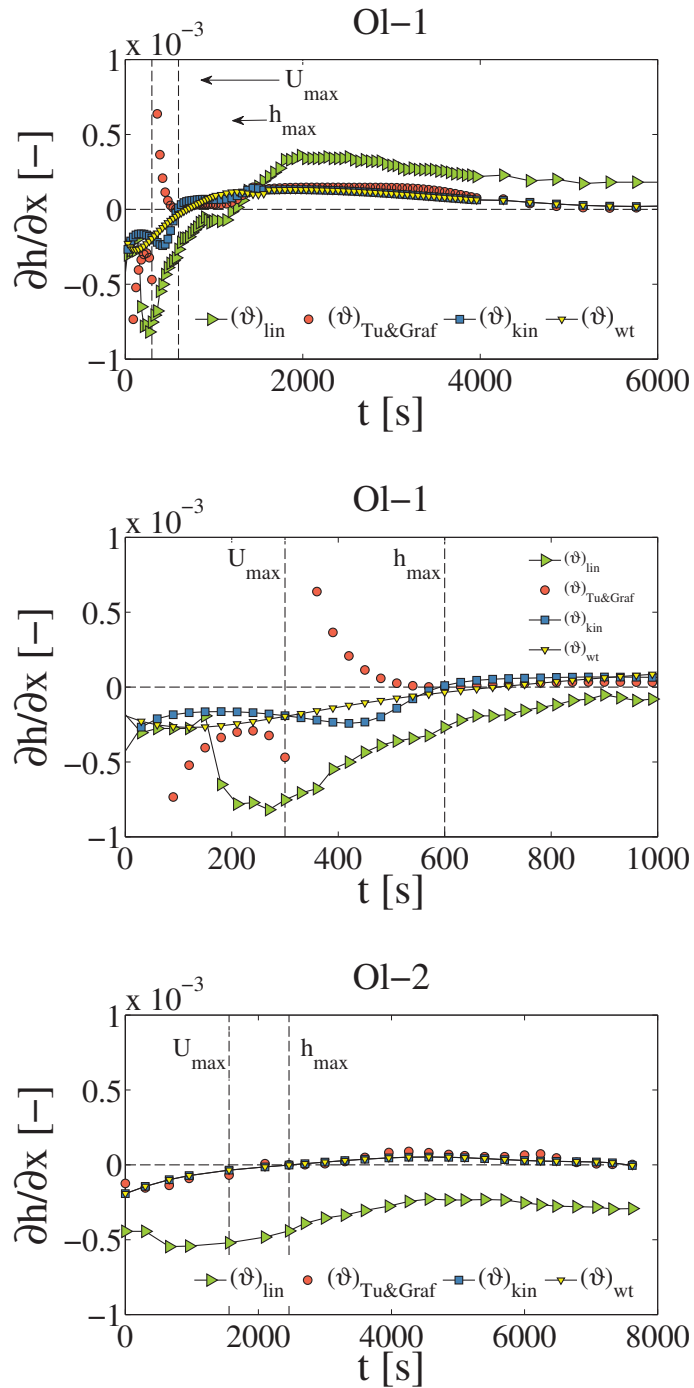


Figure 7. Temporal variability of gradient of flow depth $\vartheta = \frac{\partial h}{\partial x}$ for experimental flood waves in Olszanka watercourse. Middle panel shows a zoom of the rising limb of the wave for Ol-1.

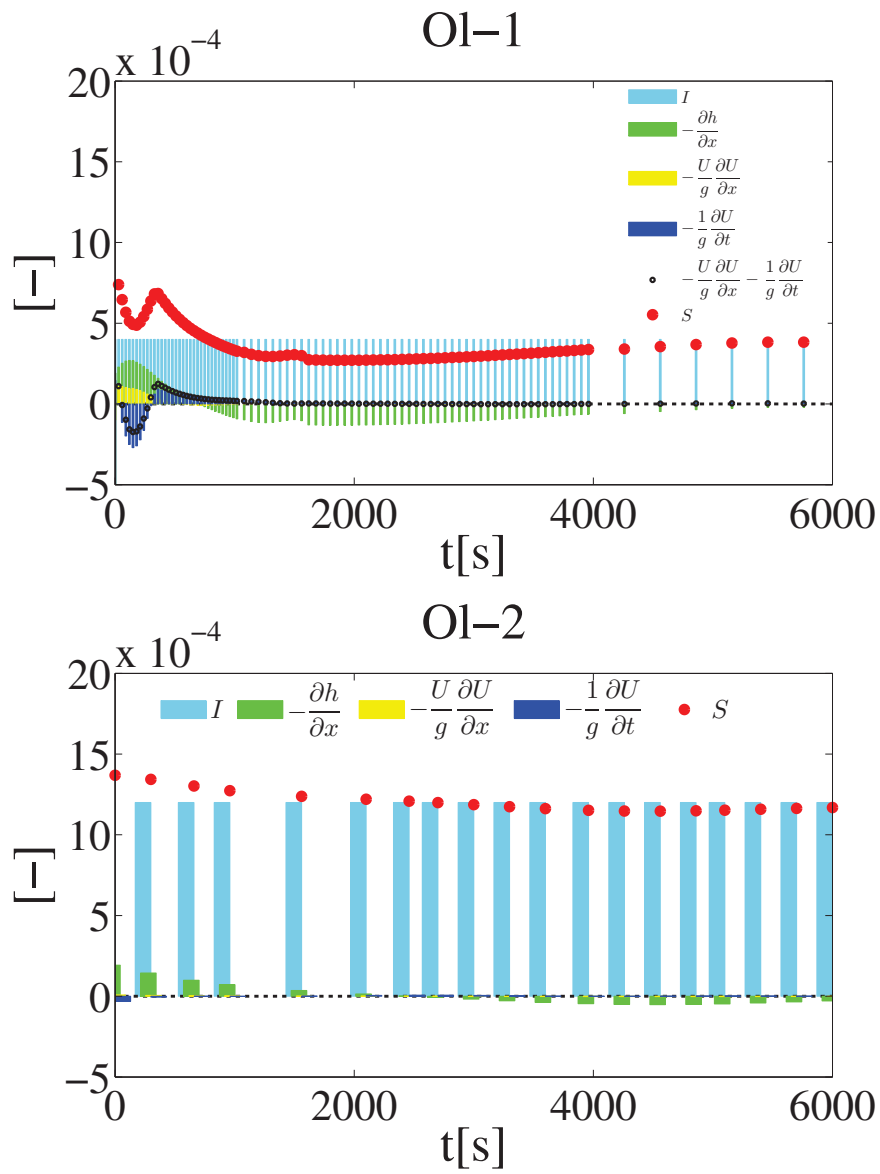


Figure 8. Comparison of terms of the momentum balance equation for experimental flood waves in Olszanka watercourse.

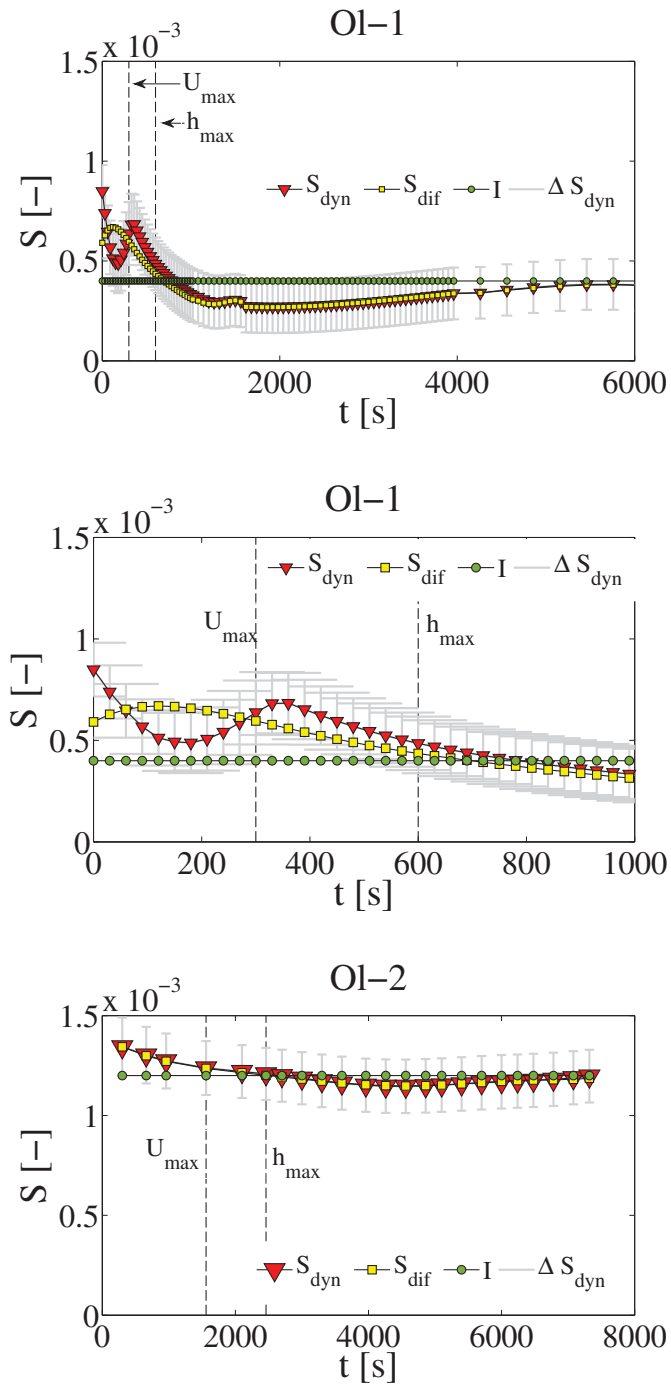


Figure 9. Comparison of friction slope evaluated by formulae for dynamic S_{dyn} , diffusive wave S_{dif} and steady flow I with uncertainty bounds ΔS_{dyn} for experimental flood waves in Olszanka watercourse. Middle panel shows a zoom of the rising limb of the wave for Ol-1.

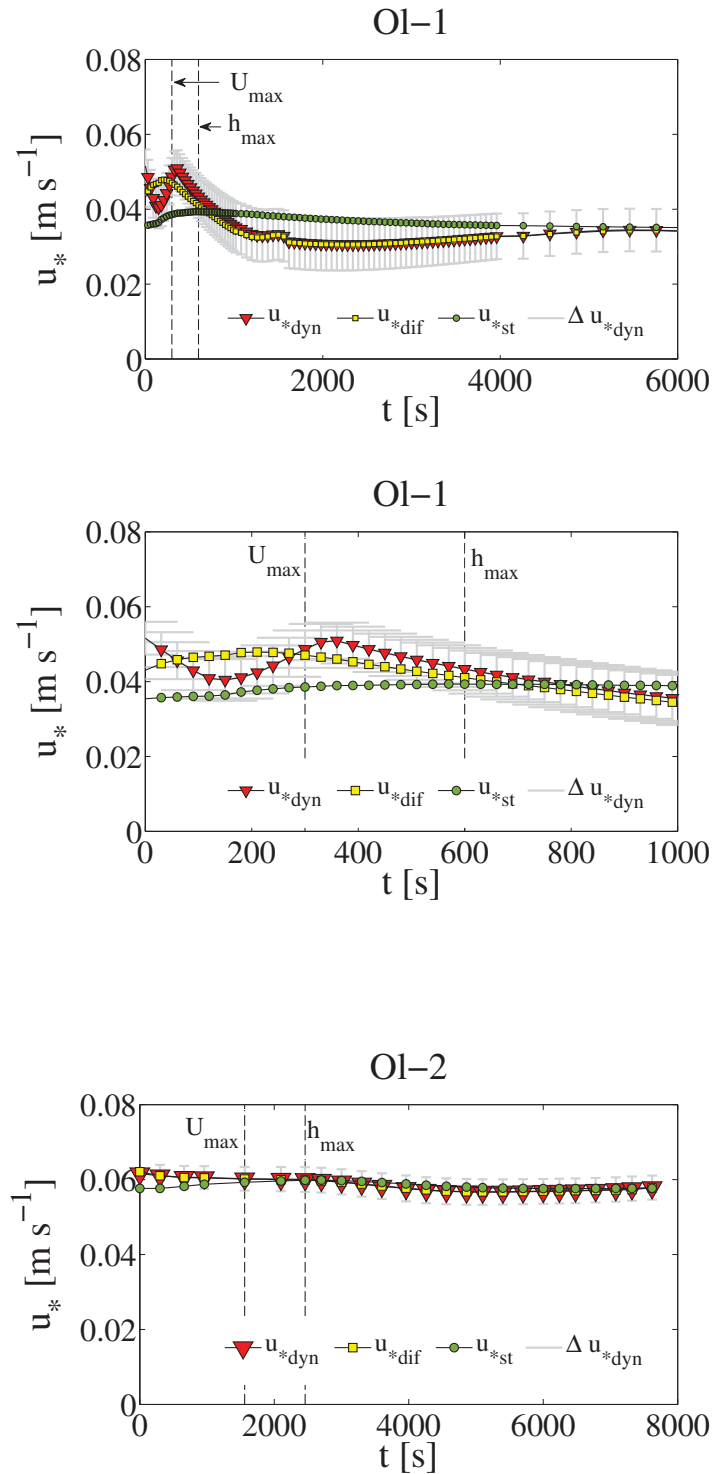


Figure 10. Comparison of friction velocity evaluated by formulae for dynamic u_{*dyn} , diffusive wave u_{*dif} and steady uniform flow u_{*st} with uncertainty bounds Δu_{*dyn} for experimental flood waves in Olszanka watercourse. Middle panel shows a zoom of the rising limb of the wave for OI-1.

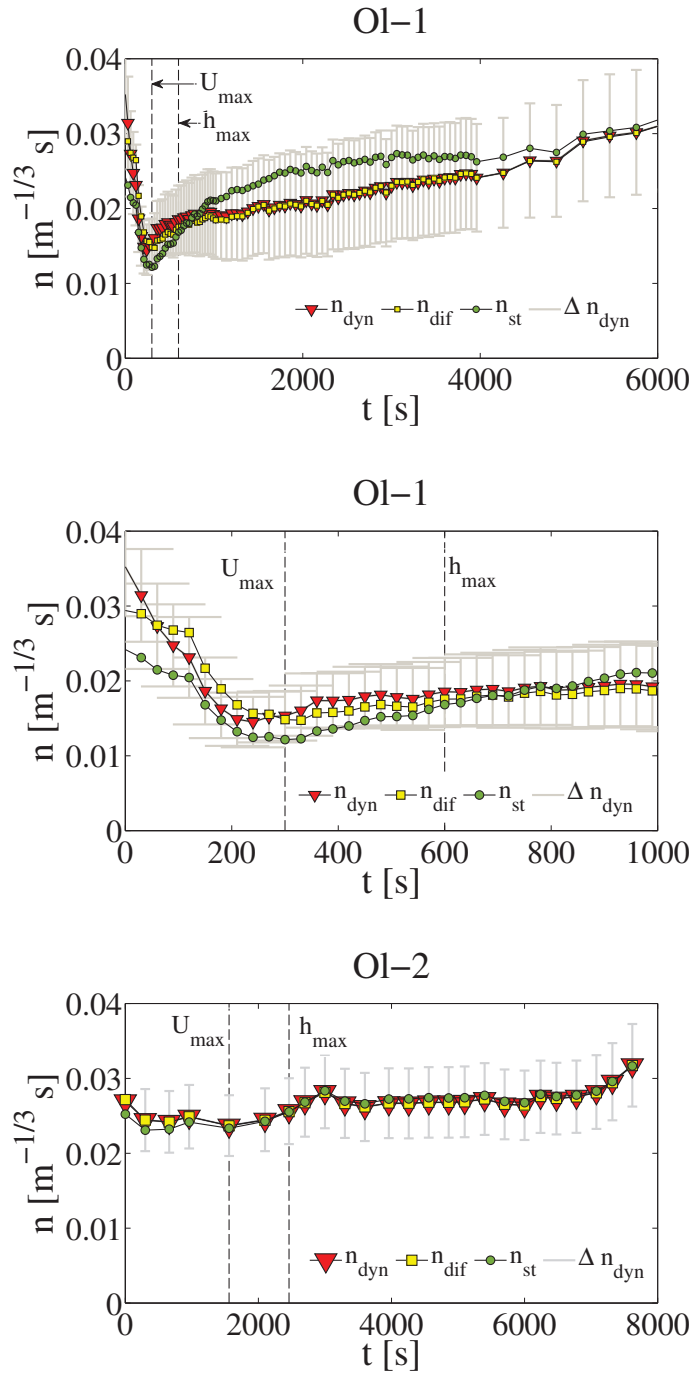


Figure 11. Comparison of Manning n evaluated by formulae for dynamic n_{dyn} , diffusive wave n_{dif} and steady uniform flow n_{st} with uncertainty bounds Δn_{dyn} for experimental flood waves in Olszanka watercourse. Middle panel shows a zoom of the rising limb of the wave for OI-1.

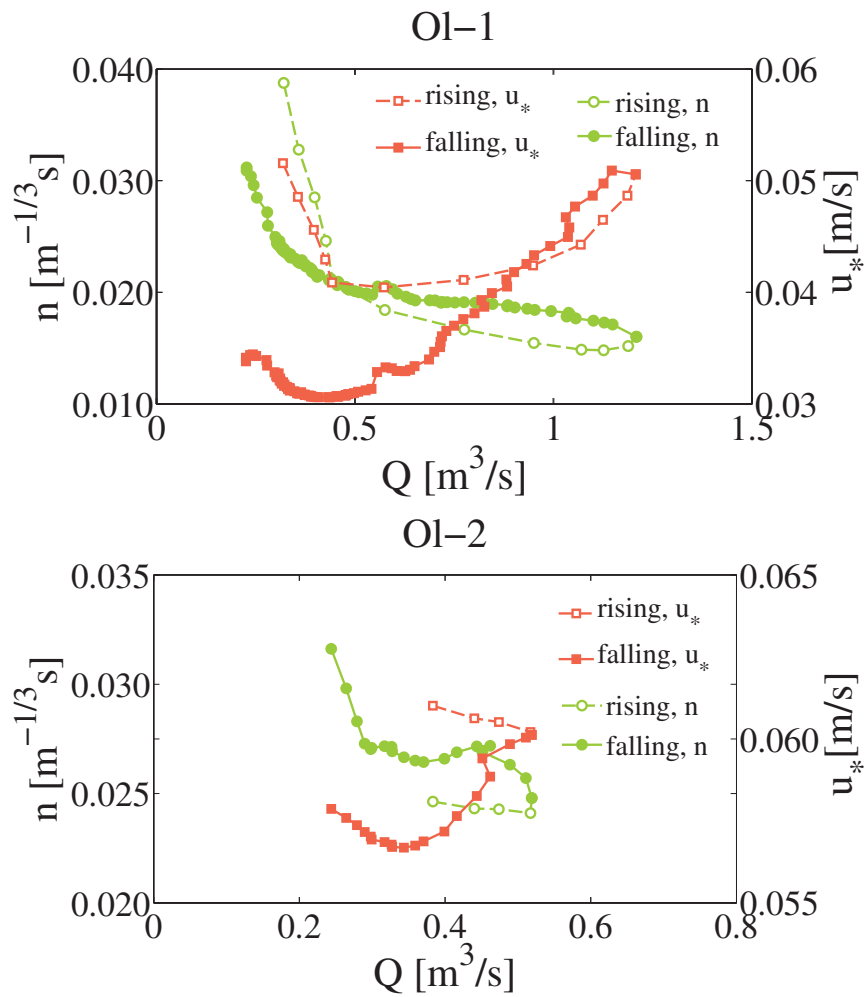


Figure 12. Comparison of the relation of Manning n vs flow rate Q and friction velocity u_* vs Q along rising and falling limbs of waves for experimental flood waves in the Olszanka watercourse.