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Investigation into the Hydraulic Characteristics of Channels with Flood Plains

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Abstract

Some experimental results of flow on a straight and smooth open channel with symmetrically complex cross-section ("compound" or "two-stage" channel) are presented from the Hydraulic Laboratory of Department of Hydraulic Structures at the Warsaw Agricultural University. These results are used to assess the performance of several discharge calculation methods. The analyses consist of calculation methods, and consist of comparisons between the measured and calculated values for discharge and friction factors at vertical plains of the interfaces between the deep and shallow part of the channel. This article presents the analyses of the open channel flow executed to test the applicability of some existing formulas, methods and other approaches to estimating the discharge capacities of compound channels.

1. Introduction

The paper presents results of an experimental study of flow on a straight and smooth open channel with symmetrically complex cross-section ("compound" or "two–stage" channel) which are compared to some flume data from others laboratories.

"Two stage" channels are models of rivers with flood plains inundated when the total depth of water H exceeds the depth of main channel h. Then the flood plain has to carry a part of total discharge and a "two-stage" flow is induced with interaction and turbulent mixing between the significantly faster flowing water in main channels and more slowly moving flow over flood plains. The large velocity gradient caused by this difference results in turbulence and in momentum transfer from the main channel to flood plains. In consequence of these phenomena a longitudinal zone of pairs of vortices with vertical axis (Fig. 1) may be observed near the interfaces between the deep and shallow parts of the channel. Vertical plains of the interfaces will then experience important shear stresses in flow direction. There are also additional vortices with horizontal axis. The structure of the

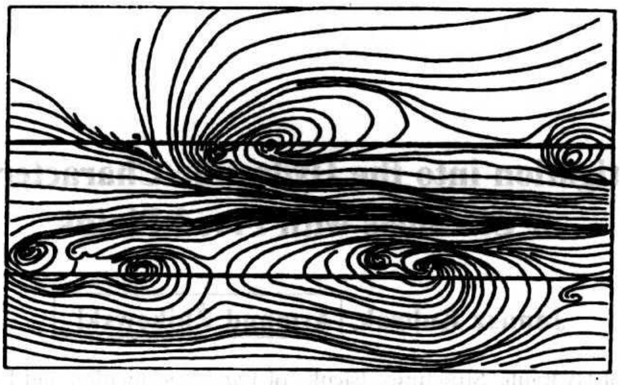

Fig. 1. Vortices with vertical axis

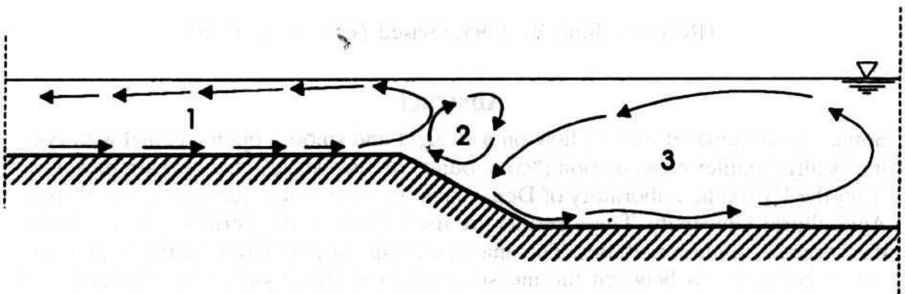

Fig. 2. Vortices with horizontal axis

secondary flow is shown in Fig. 2 (from Rhodes and Knight 1994 adapted from Shiono and Knight 1989). These highly complicated flow patterns are induced by 3-dimensional flow fields on channel. For computations of discharge capacity in such circumstances Knowledge of the following four shear stresses or forces is required: shear within the mixing layer between the flows on the two parts of the channel, shear about the sloping bank of the deeper section, forces between the bed of main channels or flood plains and water above them. In additionally, results of momentum interchange due to secondary flows must be known.

The complexity of "two-stage" flow and the importance of its proper assessment stimulated many investigators to perform numerous studies to gather more knowledge about interaction i.e. the transfer of the mass and momentum, and stresses in the channels. Such investigations have a long history, from Zheleznyakov (1950) to Rhodes (1994). In Western Europe the first paper by Zheleznyakov dating from the year 1950 was not published, and his founding and the problems of "two–stage" were unknown till 1964, when Sellin gave an account of his laboratory investigations on interaction between the waters flowing in the main channel and flood plains.

In the three decades since then and particulary in recent years numerous laboratory studies on „two-stage" flow have been conducted and described. All these investigations were limited to steady and uniform flow on the straight channels with smooth or not notably rough boundaries. Field experiments were made occasionally, only in exceptional cases. Several studies brought many noteworthy and useful findings both about the mechanism of interaction between the flows in subsections, and about its consequences.

Most of the studies aimed at seeking methods and formulas for quantifying the results of flow processes in compound channels. In such investigations the influence of hydraulic and geometric parameters on the interaction were given mostly in terms of:

- reduction or partial loss of discharge capacity of deeper subsection or of the total compound channel (Sellin 1964; Nicollet and Uan 1979; Christodoulou 1992; Ackers 1993),
- apparent shear stress at the interface between the deep and shallow subsection (Myers and Elsawy 1975; Myers and Brennan 1990; Baird and Ervine 1984; Wormleaton et al. 1982; Wormleaton and Merrett 1990).

According to Myers there are 2 forces resisting the water weight component acting in the direction of flow, which are as follows:

- shear force between the perimeter of channel and the water above and in the channel,
- shear force on the interface between the fast and slow flow of water.

Notwithstanding that many researchers gained from their investigations very remarkable findings, methods and formulas for quantifying the interaction in channel given in literature and in textbooks (Chow 1959), have only limited practical value and cannot be reliably generalized.

This may be due to the fact that experiments designed for obtaining methods or formulas were executed on too short university flumes, i.e. insufficient in size for generating the high Reynold's numbers required to minimise the scale effects.

There are some other obstacles in the development of physically sound and improved methods, formulas and other approaches for quantifications of the interaction between deep and shallow flow. These include the complexity of the phenomenon itself, too little understanding of flow resistance, insufficient data base and (cit) "tendencies to give to investigations an academical orientation". (Rajaratnam and Achmadi 1979; Mc Kee et al. 1985; Wormleaton and Merrett 1990; Knight and Siono 1990; Christodoulou 1992; Ackers 1993).

Perhaps the main reason for the above described situation is the one named in a statement by Schoemaker (Ackers and Schoemaker 1991) in his discussion of four papers dealing with the results of investigations by Myers and Brennan, Wormleaton and Merrett, Knight and Shiony and Elliot and Sellin (cit.): "The four papers on the results of the experiments on 'large scale' SERC Flood Channel Facility show clearly the absence of a generally accepted theoretical pattern in analysis of data" (Ackers and Schoemaker 1991).

Calculations of discharge capacity in engineering practice are based on dividing the channel cross section into separate, hydraulically homogeneous subsections, with the assumption that there are no interactions between them. The conveyances of these subsections are then determined as for an open channel of a regular shape (Manning, Strickler, Darcy-Weisbach, Prandtl-Colebrook) and the sums of partial capacities give the total discharge. The correct location of the interfaces is unknown, and in the calculations their positions must be assumed. Most frequently it is assumed that the interfaces are created by two vertical planes (1–1 Fig. 3b) passing through the junctions between the bed of flood-plains and the wetted perimeter of the main channel (scheme 1). Such a recommendation (scheme 1) can be found in textbooks e.g. Chow (1959); after Houk (1918): "Calculation of flow in open channels. Technical report for the Miami Conservancy District, Ohio".

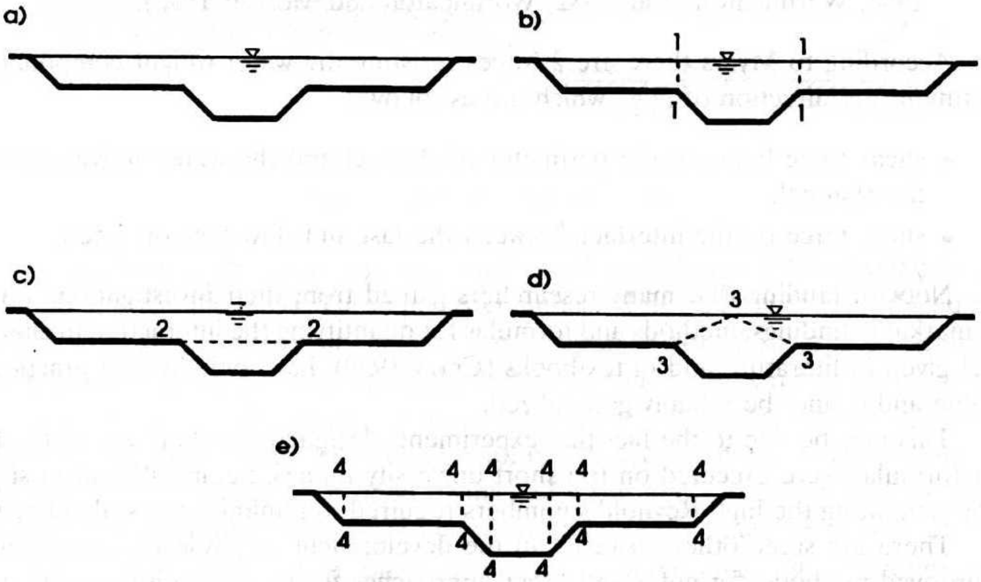

Fig. 3. Schemes of compound section dividing into separate subsections
a – scheme 0; b, c, d, e – schemes 1, 2, 3 and 4

The horizontal (2–2 Fig. 3c) and diagonal (3–3 Fig. 3d) planes passing through the same junctions as the vertical interfaces (Schema 2 and 3) are used rather rarely and scheme 4 (Fig. 3e) – least frequently.

Discharge capacities calculated with application schemes 1, 2, 3 and 4 are larger than the values measured in laboratories. The largest conveyance is provided by scheme 4 the smallest one – scheme 2. The overestimation, as happens frequently in practice, while using scheme 1 may amount to 20% and in some cases may reach the value of 30%.

The calculation of discharge of compound channel as a single, undivided unit (scheme 0) may give a spurious assessment of capacity caused by a sudden reduction in hydraulic radius when the water rises above the bed of flood plains. The capacity calculated by single channel method may be much smaller than the discharge measured in the laboratory. The opinions that this methods can be used in hydraulic calculation of two-stage flow are exceptional (Brandt and al. 1993; Brandt 1994) or limited to the cases with high ratios $(H-h)/h$.

Rating curves obtained for channels with flow above the bed of flood plains from laboratory investigations display the following features (Sellin 1964, Wormleaton and al. 1982; Wormleaton and Merrett 1990; Myers and Brennan 1990; Smart 1992):

- the relations discharge/stage have very good correlation coefficients and low errors estimate,
- when the H/h is only slightly greater than 1, the rate of discharge increases with H more slowly, and in some cases when the flood plain is rough and water on it shallow, a discontinuity of the stage-discharge relation in the region of bank-full depth may be observed. In such cases, the discharge of the total compound channel may be smaller than the discharge of the main channel by a stage lower than the bank-full depth. It would probably be more useful to calculate the conveyance with the assumption that the cross section is divided as in scheme 3,
- when H/h is greater than 1, the increase in the rate of discharge rises rapidly to a point where main channel and flood plains have roughly the same discharge capacity. The equalisation causes diminishing of momentum transfer from deep to shallow subsections.

The difference between the discharge of the compound channel calculated as the sum of capacities of separated hydraulically homogenous subsections may rise until the ratio $(H-h)/H$ reaches the value of approximately 0.2–0.3 and then decreases.

The investigations into the compound channel gained great popularity among many researchers and the number of studies of the subject has increased noticeably in the last decade (Ackers 1993). The scientists consider that complexity of the problem needs further, intensive and improved experimentation on more channels of different sizes, roughness and slopes.

Even though many investigators are sceptical about experimental results and the possibility of their generalization, they are still determined to continue and improve the investigation and to broaden their scope.

Some Authors assume that such determination is justified by the hope of obtaining reliable data from experiments on flow with high Reynolds numbers in large flumes. Reliable findings should afford numerous results from models of different sizes and so to assess the scaling effects. It is expected that (cit.): "this would then not only provide a set of data which could be more easily transferred to prototype rivers, but would also, by enabling investigation of scaling effects, increase the potency of the large amount data available from smaller laboratory flumes" (Wormleaton and Merrett 1990).

In recent years, an increasing number and greater scope of experiments, as well as use of larger flumes were more frequent and observable than some time ago. An outstanding example of this new development is the programme of investigations sponsored by the British Science and Engineering Research Council (SERC) which founded a long (56 m) compound channel arrangement in Wallingford.

Investigations in Wallingford were coordinated and included straight skewed and meandering channels and sedimentation problems.

2. Researches

2.1. Experimental Arrangement

The investigations described in this paper were performed in the Hydraulic Laboratory of the Department of Hydraulic Structures, Faculty of Land Reclamation and Environmental Engineering at the Warsaw Agricultural University – SGGW on a model $L = 16$ m long placed in a 20 m flume, 2.10 m in width. The water surface parallel as close as possible to the bed of the flume has a 0.0005 slope.

The model depth of the main channel was $h = 0.15$ m, the greatest water depth $H = 0.30$ m, (ratio $(H - h)/H \leq 0.50$). All other dimensions are shown in Figs. 4 and 5. The capacity of pump station was – 0.5 m^3/s.

The geometric parameters of the 3 subsections of the channel divided by vertical interfaces passing through junctions between the bed of flood plains and wetted perimeter of the main channel (scheme 1): wetted perimeters P_C, P_F, P_T wetted area A_C, A_F, A_T hydraulic radii R_C, R_F, R_T and friction factors f_C, f_F, f_T are shown given in Table 1 where subscripts C, F and T mean main channel, flood plain and total respectively – i.e. the undivided cross-section.

Calculation of P, A and R for subsections without interaction do not require additional explanations.

The friction factor after Prandtl-Colebrook for the smooth channel is:

$$\sqrt{\frac{1}{f}} = 2\log\left(\frac{Re\sqrt{f}}{2.51}\right) \tag{1}$$

with

$$Re = \frac{4VR}{\nu} \tag{2}$$

which gives

$$\sqrt{\frac{1}{f}} = 2\log\left(\frac{4.51\sqrt{gR^3S}}{\nu}\right) \tag{3}$$

for $g = 9.81$ m/s^2, $\nu = 1.15 \cdot 10^{-6}$ m^2/s and $S = 0.0005$ we have:

$$\sqrt{\frac{1}{f}} = 2\log 275380R^{1.5}. \tag{4}$$

Most flumes used in other laboratories were 6–15 m long and 0.6–1.2 m wide; the 3 largest flumes were approximately 50 m long (SERC in Wallingford, Nicollet and Uan i WES). The shortest of the known models had the following dimensions: $L = 4.57$ m, $b = 0.114$ m, $B = 0.457$ m, $H \leq 0.598$ m, $h = 0.045$ m (Sellin 1964). The flume at Warsaw Agricultural University is longer than most laboratory facilities, still not as long as that in Wallingford.

The water level and slope in the hydraulic laboratory in Warsaw were controlled by an adjustable weir located at the end of the flume and measured by manual pointer gauges.

Velocity of the flow was measured at 138 points with 10 mm vertical spacings at lines located in cross-section as shown in Fig. 4. The measurements were executed by 10 mm diameter miniature propeller current meters.

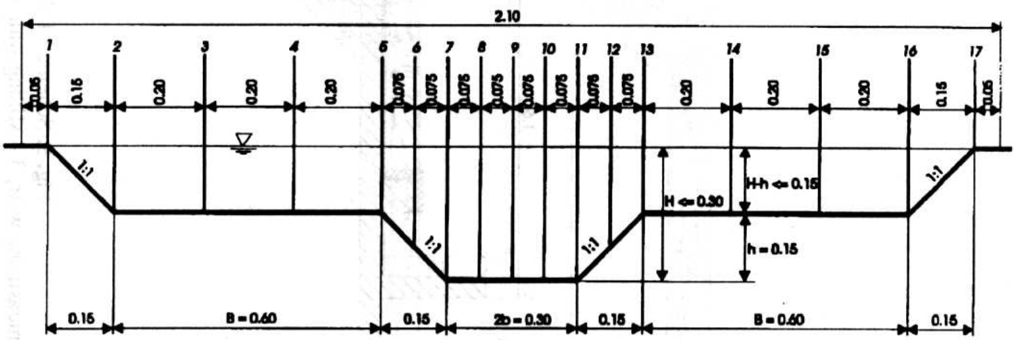

Fig. 4. Cross section of the model

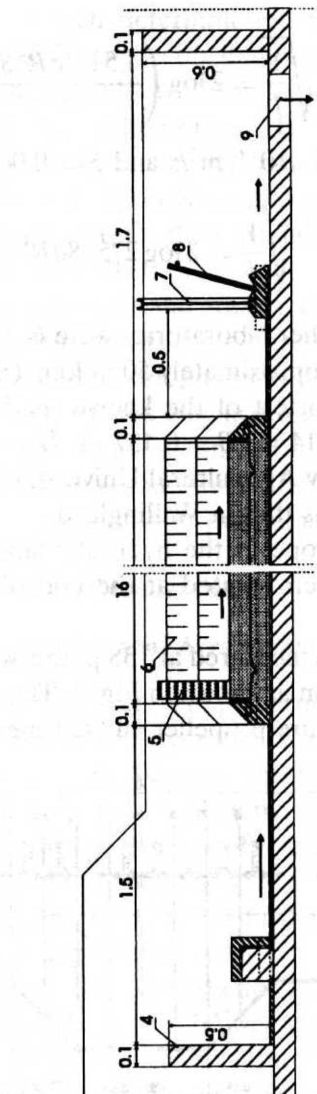

Fig. 5. Scheme of the experimental arrangement
1 – from supply line, 2 – measuring weir, 3 – stilling wall, 4 – spillway, 5 – second stilling wall, 6 – model, 7 – adjustable weir, 8 – flashboards, 9 – outfall

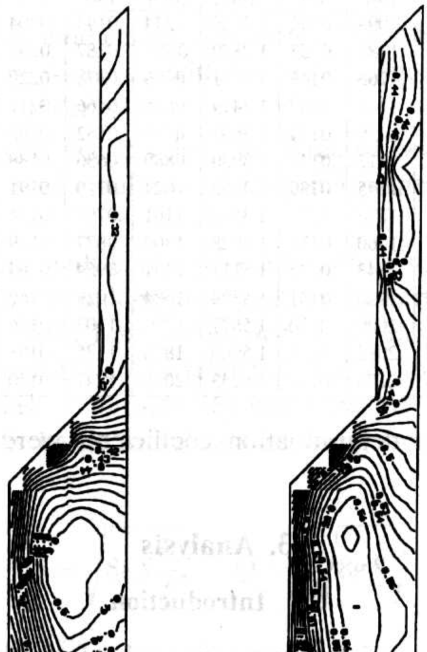

Fig. 6. Isovels and velocity profiles from measurements of laboratory models
a) $(H-h)/h = 0.3$, b) $(H-h)/h = 0.5$

Table 1. Geometric parameters of main channels, flood plain, and total cross–section

H [m]	$(H-h)/h$ [–]	P_C [m]	A_C [m^2]	R_C [m]	f_C [–]	P_F [m]	A_F [m^2]	R_F [m]	f_F [–]	P_T [m]	A_T [m^2]	R_T [m]	f_T [–]
.17	.118	.7243	.0795	.1098	.0156	1.2566	.0244	.0194	.0304	1.9808	.1039	.0525	.0202
.18	.167	.7243	.0855	.1181	.0153	1.2849	0369	.0287	.0256	2.0091	.1224	.0609	.0191
.19	.211	.7243	.0915	.1263	.0149	1.3131	.0496	.0378	.0229	2.0374	.1411	.0693	.0183
.20	.250	.7243	.0975	.1346	.0147	1.3414	.0625	.0466	.0211	2.0657	.1600	.0775	.0176
.21	.286	.7243	.1035	.1429	.0144	1.3697	.0756	.0552	.0198	2.0940	.1791	.0855	.0170
.22	.318	.7243	.1095	.1512	.0141	1.3980	.0889	.0636	.0188	2.1223	.1984	.0935	.0165
.23	.348	.7243	.1155	.1595	.0139	1.4263	.1024	.0718	.0191	2.1505	.2179	.1013	.0161
.24	.375	.7243	.1215	.1678	.0137	1.4546	.1161	.0798	.0174	2.1788	.2376	.1090	.0157
.25	.400	.7243	.1275	.1760	.0135	1.4828	.1300	.0877	.0169	2.2071	.2575	.1167	.0153
.26	.423	.7243	.1335	.1843	.0133	1.5111	.1441	.0954	.0164	2.2354	.2776	.1242	.0150
.27	.444	.7243	.1395	.1926	.0131	1.5394	.1584	.1029	.0160	2.2637	.2979	.1316	.0148
.28	.464	.7243	.1455	.2009	.0130	1.5677	.1729	.1103	.0156	2.2920	.3184	.1389	.0145
.29	.483	.7243	.1515	.2092	.0128	1.5960	.1876	.1175	.0153	2.3202	.3391	.1461	.0143
.30	.500	.7243	.1575	.2175	.0127	1.6243	.2025	.1247	.0150	2.3485	.3600	.1533	.0141

The correlation and determination coefficients were $R = 0.998$ and $R^2 = 0.997$.

3. Analysis

3.1. Introduction

The following analyses consist of comparisons between the laboratory measured and calculated values of conveyances and friction factors of compound channels. The comparisons were necessary in order to the the applicability of some of the numerous existing formulas, methods or other approaches for estimating the discharge capacities of compound channels.

The examinations of the formulas and other approaches were conducted in two different groups:

- the first consisting of the methods of discharge calculation based on the assumpation that there is no interaction between the parts of a channel. In such cases the channel is divided into separate, hydraulically independent subsections, and for each of them the conveyances and friction factors were calculated as for homogeneous wholes. The sum of these discharges gives the total conveyance of the compound channel,
- the second one consisting of 7 formulas chosen from among published approaches. The calculation of the channel conveyances and the friction factors were carried out on the basis of selected formulas and the results of the computations were compared to the measured values.

3.2. First Group

The comparisons were carried out for:

- the undivided compound channel (scheme 0) for which the discharges were computed as for the whole i.e. with hydraulic radius as the ratio of the total area and total perimeter,
- three compound channels divided into three of two subsections by sets of two vertical, one horizontal or two diagonal planes ((schemes 1,2 and 3) shown in pt. 1),
- the channel divided with 6 vertical planes into 7 subchannels (scheme 4). The friction factors of independent sections (schemes 1, 2, 3 and 4) were calculated with formula 1 and dimensions from Table 1.
  Thus obtained values h were in relations (3, 4, 5) to obtain the conveyance. The same formulas or formula (6) can serve to calculate friction factor on the basis of discharge or velocity resulting from the measurements in laboratory:

$$V = \sqrt{8gS\frac{R}{f}}, \quad Q = A\sqrt{8gS\frac{R}{f}} \tag{5}$$

for $g = 9.81\ \mathrm{m/s^2}$, $S = 0.0005$.

$$V = 0.198\sqrt{\frac{R}{f}}, \tag{6}$$

$$Q = 0.198A\sqrt{\frac{R}{f}}. \tag{7}$$

The friction factor calculated from the measured value is:

$$f = \frac{8gSR}{V^2} = 0.0392\frac{R}{V^2}. \tag{8}$$

V – average velocity in the main channel resulting from the measuring on the model.

The error EQ of calculated conveyance was:

$$EQ = \frac{Q_T - Q_M}{Q_M}\,100\% \tag{9}$$

where:

Q_T – total calculated conveyance (the sum of subsections capacities),

Q_M – total conveyance from the measurements.

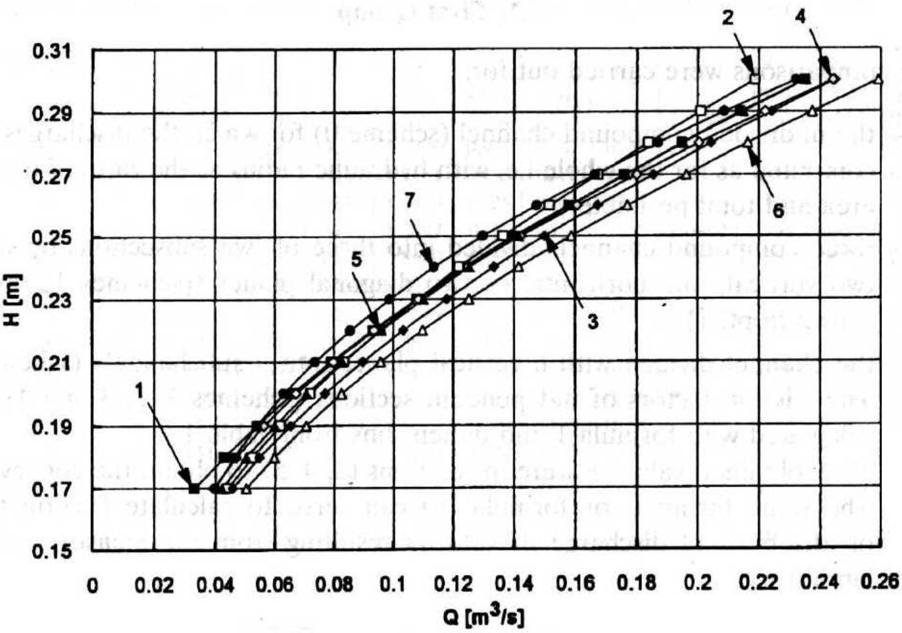

Fig. 7. Stage/discharge curves
1 – for scheme 0; 2 – for scheme 1 but with apparent shear stresses at vertical interfaces; 3, 4, 5, 6 – for schemes 3, 4, 5, 6; 7 – measured in laboratory

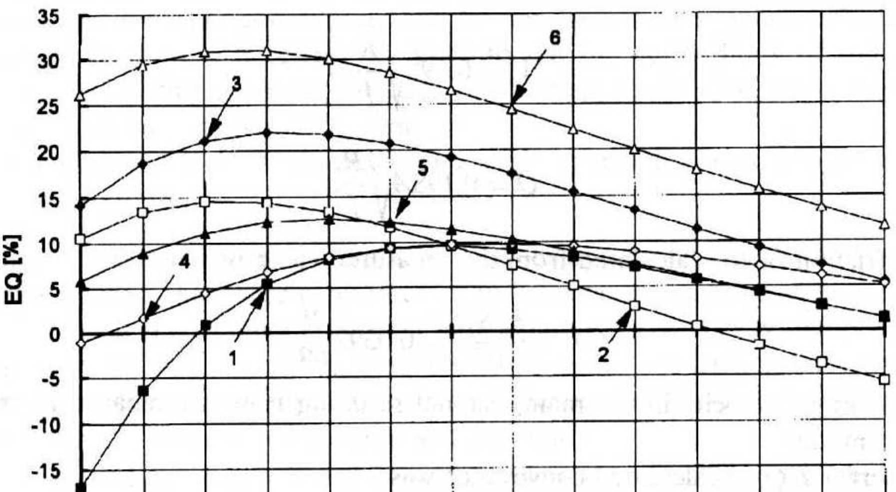

Fig. 8. Variation in total discharge error with depth
1 – for scheme 0; 2 – for scheme 1 but with apparent shear stresses at vertical interfaces; 3, 4, 5, 6 – for schemes 3, 4, 5, 6

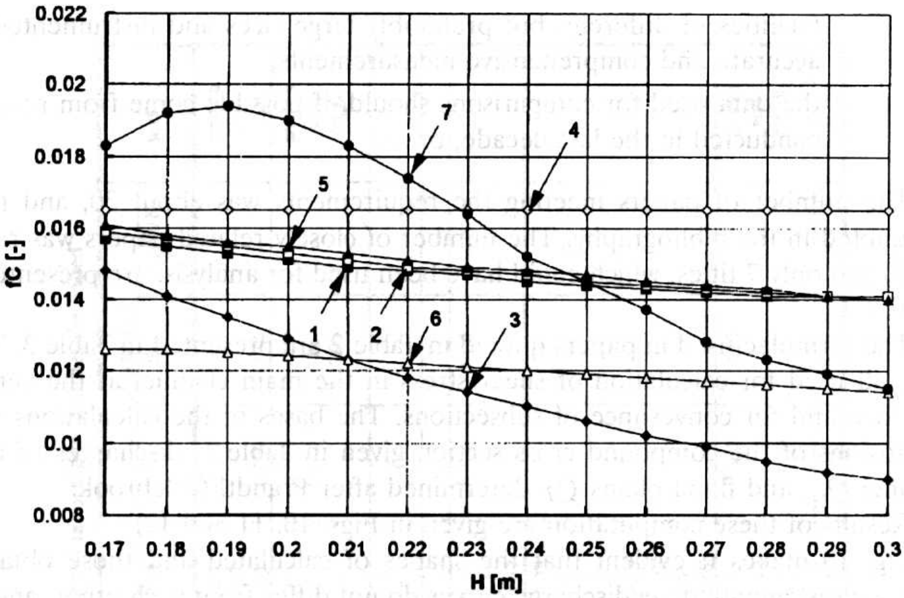

Fig. 9. Variation in friction factor with depth
1 – for scheme 0; 2 – for scheme 1 but with apparent shear stresses at vertical interfaces; 3, 4, 5, 6 – for schemes 3, 4, 5, 6; 7 – measured in laboratory

Fig. 7 shows that all stage/discharge curves (calculated and from measurement) have very similar shapes and are almost parallel; all calculated discharges are greater than the ones from measurements on the model.

Fig. 8 gives a better visual impression of relations between conveyances of compound channels obtained from different calculations and from measurements. The set of error curves shows clearly that after rejection, as hydraulically improper, of the approaches of schemes 0, 2 and 3, the results nearest the measured values are given by scheme 1, in which the wetted parameter of main channel is increased by adding two vertical interfaces.

3.3. Second Group

The comparisons were carried out between the values of discharges:

- measured in a laboratory of the Department of Hydraulic Structures,
- calculated after the equations given in a set of papers, selected from the sources mentioned in the references. These papers were chosen so that they have fulfilled the following requirements:
  - papers contained formulas or equations applicable for computation of discharge capacity of compound channel,
  - the formulas and relations would be constructed by respected scientists on the basis of results obtained in laboratories equipped with research

facilities of different, but preferably large sizes and instrumented for accurate and comprehensive measurements,
- the data used for comparisons should, if possible come from research conducted in the last decade, or so.

The number of papers meeting the requirements was about 30, and these are quoted in the bibliography. The number of closely related papers was rather small, and only 7 titles, which could have been used for analysis, are presented in Table 2.

The formulae used in papers quoted in Table 2 are presented in Table 3. They were all used for calculation of shear stress in the main channel at the vertical interface and for conveyance of subsections. The bases of the calculations were dimensions of the compound cross section given in Table 1, discharges of main channel Q_C, and flood plains Q_F determined after Prandtl-Colebrook.

Results of these computation are given in Figs. 10, 11 and 12.

Fig. 10 makes it evident that the shapes of calculated and those obtained from measurements stage/discharge curves do not differ from each other, and are almost parallel. The differences between calculated and measured discharges are smaller than discrepancies of the results in the assumption of subsections without interaction (Fig. 7).

The errors in Fig. 11 compared with those from Fig. 8 prove that all approaches assuming interaction between subsections give conveyances closer to the values based on laboratory investigations, then the discharges obtained from commonly used scheme 1 (three subsections vertical interfaces without interaction). Scheme 1, with wetted perimeter of main channel increased by adding two vertical interfaces, give conveyances of the same rank of proximity to measured values, as other schemes taking interaction into account.

The curves of Fig. 12 say the same as Fig. 10 and 11, except that in terms of friction factors.

4. Conclusion

1. The analysis of chosen methods and equations illustrates the applicability of the ones that are based on the supposition of interactin between subsections. Thesemay be easily used for estimating the discharge capacity of straight and smooth compound channels.
2. For the same purpose calculation based on scheme 1 can be used with wetted perimeter of main channel increased by the length of vetical interfaces.
3. It has been proved that hydraulic schemes 1 and 4 are more proper, and even though they do not take into account interaction, they give greater values of discharges than the equations of second group.

Table 2
Authors of selected papers and geometric characteristics of laboratory facilities described

No	Authors and year	shape of cross-sect.	Flume length [m]	slope	widths [m] main channel 2b	widths [m] flood plain B	widths [m] B/2b	depths [m] H	depths [m] H-h	depths [m] (H-h)/H	surface
1	Nicollet and Uan (1973)	trapezoidal rectangular	60.00	$1 \cdot 10^{-3}$ $0.5 \cdot 10^{-3}$	0.500	0.25 0.50 0.75	0.5 1.0 1.5	0.140 0.200	0.09 0.14	– –	smooth and rough
2	Wormleaton, Allen and Hadjipanos (1982)	rectangular	10.75		0.288 0.203 0.153 0.102	0.460 0.356 0.152	– – –	0.102 0.120	0.015 to 0.09	– – –	smooth and rough
3	Baird and Ervine (1984)	–	–	–	–	–	–	–	–	–	–
4	Rouve (1987)	–	–	–	–	–	–	–	–	–	–
5	Wormleaton and Merrett (1990)	trapezoidal	56.00 (46-60)	$1.027 \cdot 10^{-3}$	1.500	1.500 4.500 8.200	1 3 5.47	0.300	0.15	0.5	
6	Christodoulou (1992)	rectangular					1.75 2 3 4 4.17 5.33 6 6.67				
7	Ackers (1993)	trapezoidal	56.00 (46-60)	$1 \cdot 10^{-3}$	1.500	0.750 0.975 2.250 4.100	1 1.25 3 5.47	0.300	0.15	0.5	smooth and rough
8	Kubrak (1995)	trapezoidal	20.00	$0.5 \cdot 10^{-3}$	0.300	0.600	2	0.300	0.15	0.5	smooth

Table 3. Equations used in computations

Authors	Equations
Nicollet, Uan (1979)	$Q_{CI} = N_{QC}$; $Q_{FI} = Q_F\sqrt{1 + \frac{AC}{AF}(1 - N^2)}$ $N = N_0 = 0.9\left(\frac{KC}{KF}\right)^{1/6}$ if $K_C = K_N$ $N = N_0 = 0.9$ than for $r = \frac{RF}{RC} > 0.3$ $N = 0.9$ and for $0.03 \le r \le 0.3$ $N = \frac{1-A_0}{2}\cos\frac{\pi r}{0.3} + \frac{1+A_0}{2} = 0.05\frac{\pi r}{0.3} + 0.95$
Wormleaton, Allen, Hadjipanos (1982)	$\tau = 13.84(\Delta V)^{0.882}\left(\frac{H}{h}\right)^{-3.123}\left(\frac{B}{2b}\right)^{-0.727}$ $\Delta V = V_C - V_F$
Baird, Ervine (1984)	$\frac{\tau}{\rho g(H-h)s} = \left(\frac{H}{H-h} - \Psi\right)^{1.5}\left(\frac{0.5PC}{h}\right)\left[0.5 + 0.3\ln\frac{0.5PF}{h}\right]$ $\Psi = 1 + \left(\frac{h}{0.5PC}\right)^{1.25}$
Wormleaton, Merrett (1990)	$B_0 = 2(B+b) + H + h$ $\tau = 3.325(\Delta V)^{1.451}(H-h)^{-0.354}B^{0.519}$
Christodoulou (1992)	$Q_{CI} = Q_C(\varphi C)^{1/2}$ $Q_{FI} = Q_F(\varphi F)^{1/2}$ $\varphi C = 1 - \frac{\tau PJ}{\rho g ACS}$; $\varphi F = 1 + \frac{AC}{AF}(1 - \varphi C)$ $PJ = 2(H-h)$; $\tau = 1/2\rho\left(0.01\frac{B_0}{2b}\right)\Delta V^2$ $B_0 = 2(B+b) + H + h$; $\Delta V = V_C - V_F$
Ackers (1993)	Region 1: $(H-h)/H \le 0.2$ $\overline{Q_{T1} = Q_T - DISDEF}$ $DIDEF = \{Q \star 2C + Q \star IIF)(V_C - V_F)HhA_{RF}$ $Q \star IIC = -1.240 + 0.395(B+b+h)/2b + GH\star$ for $s_C \ge 1.0$ $G = 10.42 + 0.17f_F/f_C$ for $s_C < 1.0$ $G = 10.42 + 0.17s_C f_F/f_C + 0.341(1 - s_C)$ for aspect ratio $2b/h \le 20A_{RF} = 2b/10h$ $Q \star IIF = -H \star F_C/f_F$ $H\star \le (H-h)/H$ Region 2: $0.2 < (H-h)/H \le 0.4$ $\overline{Q_{T2} = Q_T DISADF2}$ $DISADF2(H_*) = C_{OH}(H_* + \text{shift})$ for $s_C \ge 1$ shift $= 0.15$ for $s_C < 1$ shift $= 0.09 + 0.06s_C$ Region 3: $(H-h)/H > 0.4$ $\overline{Q_{T3} = Q_T\ DISADF3}$ $DISADEF3 = 1.567 + 0.667C_{OH}$ Region 4: $Q_{R4} = Q_T\ DISADF4$: $\overline{DISADF4 = C_{OH}}$

Notes: P – wetted perimeter, R – hydraulic radius, K – Strickler factor, N, φ – coefficient, B, b, H, h – in Fig. 4, V – average velocity, Q – discharge rate, τ – apparent stress at vertical interfaces (N/m^2), S – hydraulic gradient of the channel, s – channel-flood plain side slope, $DISADF$ – factor taking into consideration the interaction between the channel subsections, $DISDEF$ – difference between zonal calculation of discharge and actual flow, ρ – density of fluid, g – gravitational acceleration, Re – Reynolds number

Subscripts: C – main channel, F – flood plain, T – total i.e. main channel and flood plains.

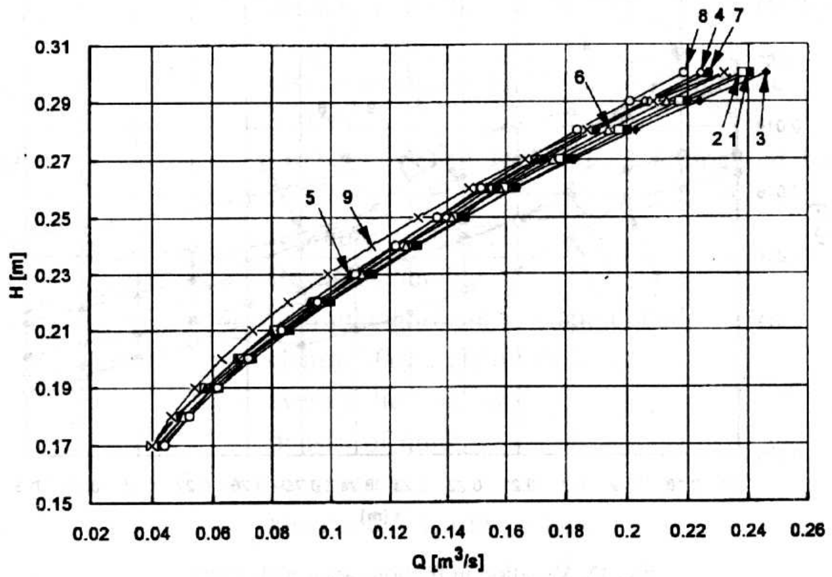

Fig. 10. Stage/discharges curves after
1 – Nicollet and Uan, 2 – Wormleaton, Allen and Hadjipanos, 3 – Baird and Ervine,
5 – Wormleaton and Merrett, 6 – Christodoulou, 7 – Ackers, 8 – scheme 1 but with apparent shear stresses at vertical interfaces, 9 – measured in laboratory

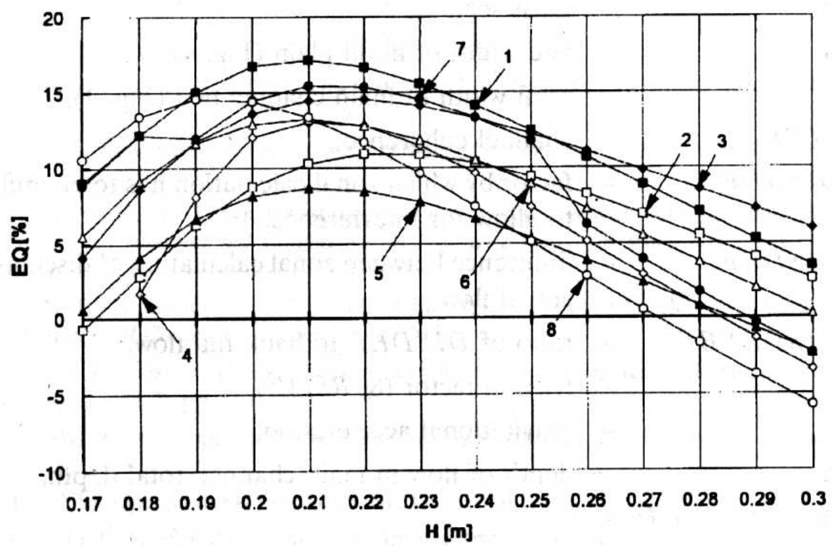

Fig. 11. Variation in total discharge error with depth
1 – Nicollet and Uan, 2 – Wormleaton, Allen and Hadjipanos, 3 – Baird and Ervine,
5 – Wormleaton and Merrett, 6 – Christodoulou, 7 – Ackers, 8 – scheme 1 but with apparent shear stresses at vertical interfaces

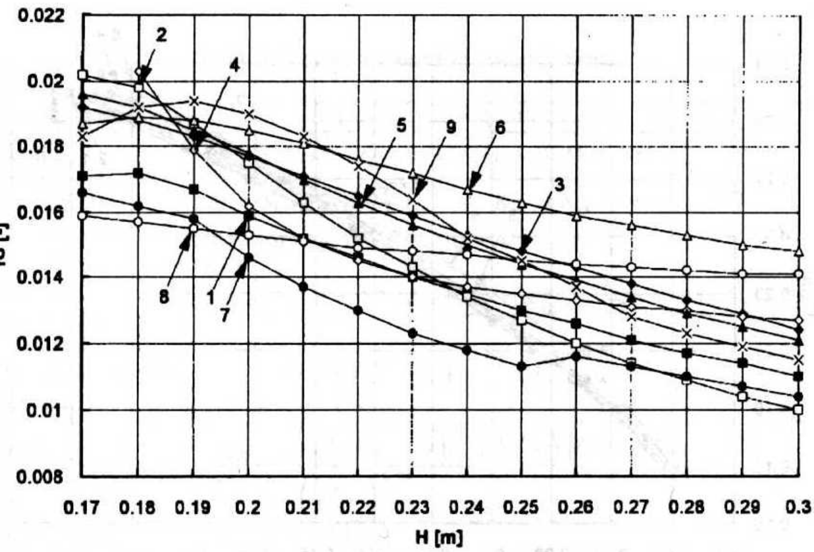

Fig. 12. Variation in friction factor with depth
1 – Nicollet and Uan, 2 – Wormleaton, Allen and Hadjipanos, 3 – Baird and Ervine, 5 – Wormleaton and Merrett, 6 – Christodoulou, 7 – Ackers, 8 – scheme 1 but with apparent shear stresses at vertical interfaces, 9 – measured in laboratory

Notes

A	–	cross-section,
B	–	bed width of flood plain (Fig. 4),
b	–	semi-width of main channel bed (Fig. 4),
COH	–	channel coherence,
$DISADF$	–	factor by which zonal calculation has to be multiplied to allow for interference,
$DISDEF$	–	difference between zonal calculation of discharge and actual flow,
$DISDEFBF$	–	ratio of $DISDEF$ to bank full flow,
f	–	friction factor ($8gRS/V^2$),
g	–	gravitational acceleration,
H	–	depth of flow in main channel, total depth,
$H_* = (H-h)/H$	–	
h	–	bank-full depth,
K	–	Strickler factor,
P	–	wetted perimeter,
Q	–	discharge, conveyance, discharge capacity, capacity,

Q_{*1} – discharge deficit ($DISDEF$) normalised by $(V_C - V_F)(H - h)h$,
Q_{*2} – discharge deficit ($DISDEF$) normalised by $(V_C - V_F)Hh$,
Q_{*3} – discharge deficit ($DISDEF$) normalised by $(V_C - V_F)2bH$,
R – hydraulic radius,
Re – Reynolds number,
S – hydraulic gradient of channel bed,
s – channel-flood plain side slope,
V – average flow velocity,
ρ – density of fluid,
τ – shear stress,
ν – kinematic viscosity of fluid.

Subscripts:

C – main channel,
F – flood plain,
I – values after allowing for interaction,
T – total i.e. main channel plus flood plains.

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