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APPLICATION OF THE RAINFALL-RUNOFF MODELS TO DESIGN FLOOD COMPUTATION

INTRODUCTION

The main task was to select and test rainfall — runoff hydrological models which could be used for transformation of the design hyetographs into design hydrographs. For this purpose 6 models have been considered. They are relatively simple with a small number of parameters, all are the linear ones based on IUH characteristics: (1) original Wackermann model, (2) Wackermann model modified by Agriculture University in Warsaw (SGGW), (3) GIUH in version of Institute of Meteorology and Water Management (IMGW) version, (4 and 5) GIUH in versions of University of Warsaw and (6) IUH in Lutz version. Model calibration has been performed in 14 catchments (mainly mountainous) using measured rainfall events and flood hydrographs.

STRUCTURE OF SELECTED MODELS

WACKERMANN MODEL

Conceptual model by Wackermann is the simplified version of Diskin (1964) model. It has very simple structure and only three parameters. The natural catchment is described by the two parallel routes of linear reservoirs. The first is describing transformation of effective rainfall into the direct runoff, and the second into subsurface flow. Every cascade consists of two reservoirs with constant storage coefficients (K_1, K_2). The third parameter (B) is distributing the effective rainfall impulse into the two cascades. The unit hydrograph is described by the equation:

$$IUH = B\, t\, K_1^{-2} \exp(-tK_1^{-1}) + (1 - B)\, t\, K_2^{-2} \exp(-tK_2^{-1}) \qquad (1)$$

In the ungauged catchments the parameters may be determined from empirical formulas given by Thiele and Euler (1981):

$$K_1 = 0.7308(L_{cc}S^{-1/2})^{0.2175} \tag{2}$$

$$K_2 = 2.0246(L_{cc}S^{-1/2})^{0.2814} \tag{3}$$

$$B = 2.0188(L_{cc}S^{-1/2})^{0.5078} \tag{4}$$

where: L_{cc} — the maximum catchment length (along the main channel), S — dimensionless river slope.

Use of Wackermann model in Polish conditions has been studied by Ignar (1993). The optimum values of model parameters have been estimated giving similar to Thiele and Euler ones regression equations:

$$K_1 = 0.893(L_{cc}S^{-1/2})^{0.374} \tag{5}$$

$$K_2 = 1.283(L_{cc}S^{-1/2})^{0.159} \tag{6}$$

$$B = 1.038(L_{cc}S^{-1/2})^{0.403} \tag{7}$$

In both versions the instantaneous unit hydrograph does not change in time, and is related to two simple characteristics of the main channel.

GEOMORPHOLOGICAL INSTANTANEOUS UNIT HYDROGRAPH MODEL (GIUH)

The GIUH model has been defined as a probability density function of the travel times for a drop of water landing on the surface of the catchment. In this interpretation the catchment response to rainfall is dependent on channel network structure described by Strahler (1953) ordering scheme, and travel time within it.

The most important characteristics of *IUH* are the peak flow (q_p) and the time to peak (t_p). As long as these two measures are correct the actual form of IUH is not very important. These two parameters depend on catchment geomorphological features and flow velocity is expressed by the relationships after Rodriguez-Iturbe and Valdes (1979):

$$t_p = 0.44\, L_\Omega\, v^{-1}\, (R_B R_A^{-1})\, 0.55\, R_L^{-0.38} \tag{8}$$

$$\mathrm{q_p} = 1.31\, \mathrm{L}_\Omega^{-1}\, \mathrm{R}_\mathrm{L}^{0.43}\, \mathrm{v} \tag{9}$$

where: L_Ω — stream length of highest order Ω, R_B, R_L, R_A — ratios of Horton and Schumm laws, v — flood velocity.

The geomorphological characteristics may be determined from maps. More difficult problem is estimation of the flow velocity (v). In original works it is interpreted as maximum celerity of flood propagation. It is assumed that flood is caused by rainfall of duration exceeding the runoff concentration time in the catchment. For its determination kinematic wave approach has been used

$$v = 0.665\,\alpha^{0.6}\,(I_E A)^{0.4} \tag{10}$$

where: 0.665 — units conversion coefficient, I_E — effective rainfall intensity (cm h^{-1}), A — the catchment area (km^2), v — kinematic wave celerity (ms^{-1}), α — kinematic wave parameter in catchment cross-section ($s^{-1}m^{-1/3}$) dependent on stream characteristics, calculated as

$$\alpha = S_k^{1/2}\, n^{-1}{}_N^{-2/3} \tag{11}$$

where: S_k — dimensionless channel slope, B — channel width (m), n — Manning roughness coefficient.

Particularly meaningful is relationship of GIUH to effective rainfall intensity, which is a non-stationary parameter. In literature one may meet many methods for flood velocity interpretation. Within this, three versions of the GIUH model have been tested. Two of them have been worked out at University of Warsaw (UW) and the third one at the Institute of Meteorology and Water Management (IMGW). In all versions the two-parameters gamma density function (Nash, 1960) to describe the GIUH have been used:

$$IUH = [K\,\Gamma(N)]^{-1}\,(t\,K^{-1})^{N-1}\exp(-t\,K^{-1}) \tag{12}$$

IUH parameters N and K may be estimated in relationship with q_p and t_p characteristics, as

$$t_p = K\,(N-1) \tag{13}$$

$$q_p = /K\,\Gamma(N)/^{-1}\exp(1-N) \tag{14}$$

Comparing the above relations with equation (8 and 9) the classic Nash conception can be combined with GIUH theory.

WARSAW UNIVERSITY GIUH VERSION (UW1)

The first version of GIUH model is similar to the original. Parameters of Nash model have been estimated from equations (13,14), and relationship given by Rodriguez Iturbe and Valdes (eq. 8 and 9). The dynamic parameter expressed by flood wave celerity was calculated after equation (10), taking into account the initial flow velocity v_0:

$$V = v_0 + 0.665\,\alpha^{0.6}\,(I_E A)^{0.4} \tag{15}$$

GIUH form is changing also with the effective storm intensity (I_E), more heavy is the storm the catchment response is faster and more violent.

WARSAW UNIVERSITY GIUH VERSION (UW2)

In the second GIUH version the dynamic parameter (v) is interpreted in another way. In Polish conditions floods are caused very often by short-term storms of duration (t_r) not exceeding the catchment concentration time (t_r). For this reason have been considered two phases of flood forming — flow rising and its equilibrium state.

During the first phase, parallely to active catchment area increase, flow is rising, as well as its velocity. Duration of this phase is equal to the concentration time (t_c), being the effective storm travel time from the farest point of catchment to its mouth.

During the equilibrium phase flow as well as its velocity will be constant in case when effective storm intensity (I_E) is not changing. This phase can be reached when storm duration (t_r) is longer or equal to the concentration time (t_c).

Flow velocity within the both phases has been estimated by solving the kinematic wave hydrodynamic equations. Taking into account the initial conditions (v_0) equations become:

$$v_1 = v_0 + 1.17\,\alpha\,(I_E\, t_r\, A\, L_c^{-1})^{0.67} \qquad t_r < t_c \tag{16}$$

$$v_2 = v_0 + 0.665\,\alpha^{0.6}\,(I_E\, A)^{0.4} \qquad t_r \geq t_c \tag{17}$$

where: L — main stream length from sources to the outlet (km), t_r — effective storm duration (h), t_c — concentration time (h), which may be calculated as

$$t_c = 0.28\, L_c\, v_2^{-1} \tag{18}$$

Application of the above mentioned relationships allows modelling of the catchment response description by the family of GIUHs. During the first phase GIUH is changing due to storm intensity (I_E) and storm duration (t_r), after reaching the second phase ($t_r \geq t_c$) GIUH is dependent only on effective storm intensity (Fig. 1).

INSTITUTE OF METEOROLOGY AND WATER MANAGEMENT (IMGW) GIUH VERSION

In this version gamma function parameters (N, K) are estimated from empirical relationship established by Ostrowski (1994). The relations have been worked out using data from 50 small catchments in Poland, giving following formulas:

$$N = 3.329\,(R_A\, R_B^{-1})^{0.744}\, R_L^{0.072} \tag{19}$$

$$K = T_L\, N^{-1} \tag{20}$$

where: T_L — lag time (h) playing role of the dynamic parameter, usually considered as flood velocity (v). Lag time is a difference between the geometric centers of the effective hyetograph and direct hydrograph. In ungauged catchments this parameter may be estimated from the regional equation established for Polish conditions:

$$T_L = 2.652 + 0.079A - 0.046P_E + 0.149\,t_r \tag{21}$$

where: A — catchment area (km^2), P_E — effective rainfall depth (mm), t_r — duration of the effective rainfall.

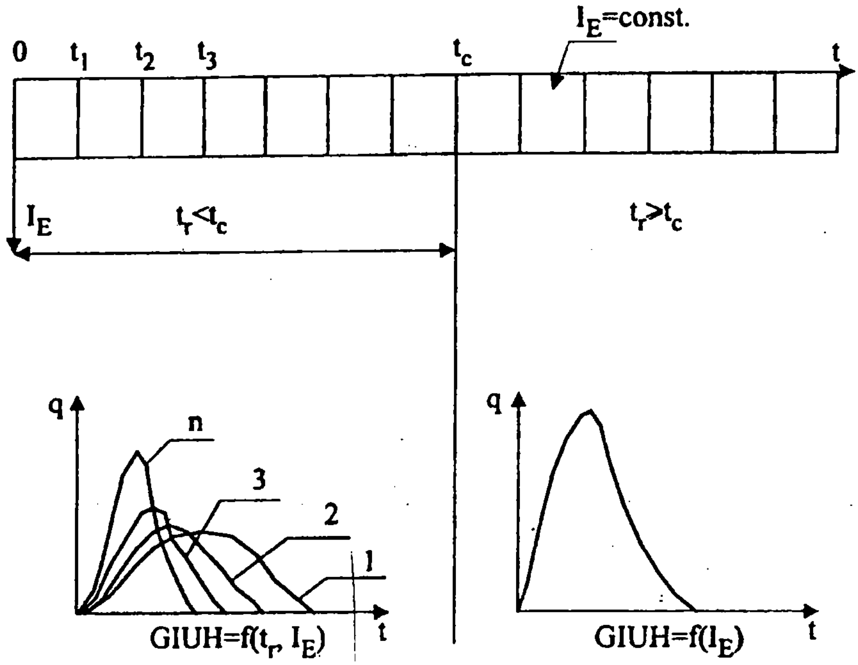

Fig. 1. The family of GIUHs.

Advantage of this formula is that it can be easily calculated from standard hydrological data. Catchment response is described by specific IUH which depends on effective rainfall depth and its duration.

INSTANTANEOUS UNIT HYDROGRAPH (IUH) MODEL IN LUTZ VERSION

One of the newest methods of IUH characteristics determination is one elaborated in Germany (Lutz, 1984; *Ermittlung...*, 1989). It is based on analysis of more than 900 floods hydrographs in 75 catchments with the area up to 500 km^2. The characteristics of IUH are estimated from the following formulas:

$$t_p = P_1\,(L_{cc}\,L_g/S_g^{1.5})^{0.26} \cdot e^{-0.016U} \cdot e^{0.004W} \qquad (22)$$

$$q_p = 0.464\,t_p^{-0.824} \quad \text{for } \Delta t = 1h \qquad (23)$$

where: t_p(h) — time to peak of IUH, $P_{1(-)}$ — catchment parameter dependent on stream bed roughness, L_{cc} (km) — maximum length of the catchment, L_g(km) — channel length from the gravity center to the outlet of the catchment, S_g(–) — straight channel slope, U(%) — part of urban areas, W(%) — part of forest areas, q_p — peak of IUH.

From equations (13) and (14) the following relationship may be established:

$$q_p t_p = (N-1)^N \, [\Gamma(N)]^{-1} \exp(1-N) \tag{24}$$

From formula (24) was estimated parameter N by iteration, while parameter K may be established from equation (13). Then, ordinates of IUH (q_i) for itteration step (i) for the time interval Δt, may be calculated from the formula:

$$q_i = [(i-0.5)\Delta t]^{N-1} [K_N \, \Gamma(N)]^{-1} \exp - (i-0.5)\, \Delta t K^{-1} \tag{25}$$

VALIDATION OF SELECTED MODELS

Fourteen basins from the territory of Poland were selected for the models verification. All basins have relatively small area which does not exceed 300 km^2 and are located in mountainous regions.

Models calibration in the basins required estimation of great number of different characteristics. There are 13 physiographic parameters, 3 of them describe the area of the basin, and its landuse (forest and urban index). Characteristics of main river describe its lenght L and slope S. Methods of its determination depend on the model being calibrated. Other parameters are indexes calculated in accordance with Horton Laws and Strahler ordering method determining spatial structure of drainage system.

Parameters were identified with the use of the Integrated Land and Water Information system (ILWIS) elaborated by International Institute of Aerospace Survey and Earth Science (ITC), Enschede, The Netherlands.

Analysed parameters vary with the time, and they depend on the hydrometeorologic conditions. Therefore it was assumed that they should be estimated for every considered rainfall-runoff event. The exception was the river slope assumed constant. The river channel width and Manning's coefficient have been determined for two different phases of every rainfall-runoff event, namely initial conditions of flood forming, and the peak flow. Lutz parameter ($P1$) has been estimated on the basis of Manning's coefficient, and an average value of it applied. Rainfall characteristics are automatically calculated from hydrometeorological data by computer programs created for the verification of the models.

For the purpose of runoff modelling and models calibration computer program 'FALA' in TURBO-PASCAL programming language has been written. It is interactive, and has a graphics functions.

The effective rainfall was calculated by the SCS method or constant runoff coefficient. In the both cases effective rainfall was calculated from the volume of direct runoff, and measured rainfall values. Total rainfall (P), effective rainfall (P_E), runoff coefficient (α), volume of direct runoff (V_p), parameters of SCS method, and lag time are displayed on the screen. In the next step of the calculations user may choose one of the six rainfall-runoff models described.

Verification of the results is done automatically by the program which compares calculated and observed flood hydrographs. Structure of the computation programme is shown at the Fig. 2.

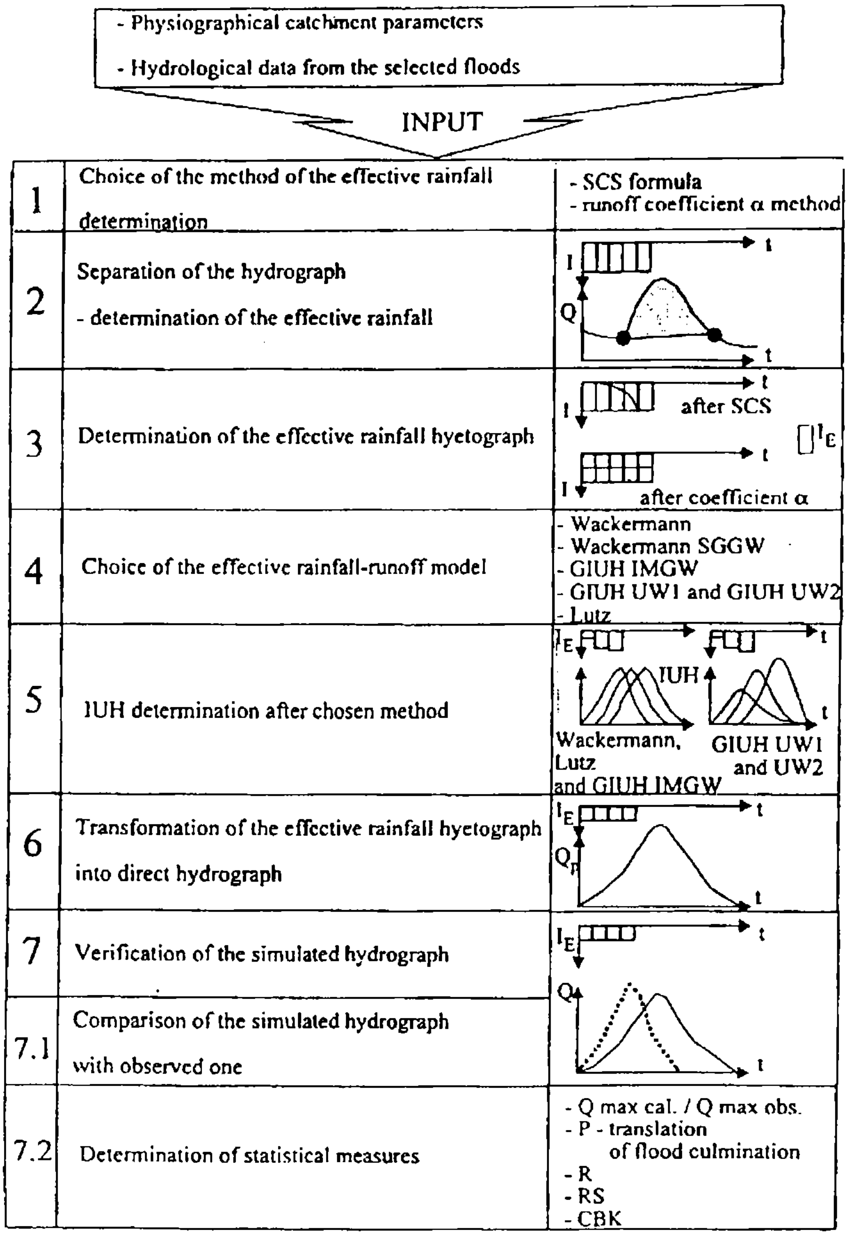

Fig. 2. Structure of computation programme.

On the basis of obtained results the following models have been chosen for prediction of the design flood runoff hydrographs: two Wackermann models, GIUH IMGW model, and Lutz model.

PREDICTION OF THE DESIGN FLOOD FREQUENCY

The main aim of the proposed method was prediction at the small basins maximum annual peak flows on the basis of design storms of the stated duration and return period. Transformation of design storms into flood hydrographs has been done assuming that maximum discharge corresponds to the storms of the same frequency. Basins selected for verification of hydrological models were also used for the prediction of the design floods.

The peak flows frequency has been calculated by empirical method. It was approximated by a probability distribution, which parameters can be estimated from the empirical sample of annual maximum discharges time series. Peak flows frequency has been calculated by Pearson III probability distribution and kvantiles method of parameter estimation. Peak flows cal-

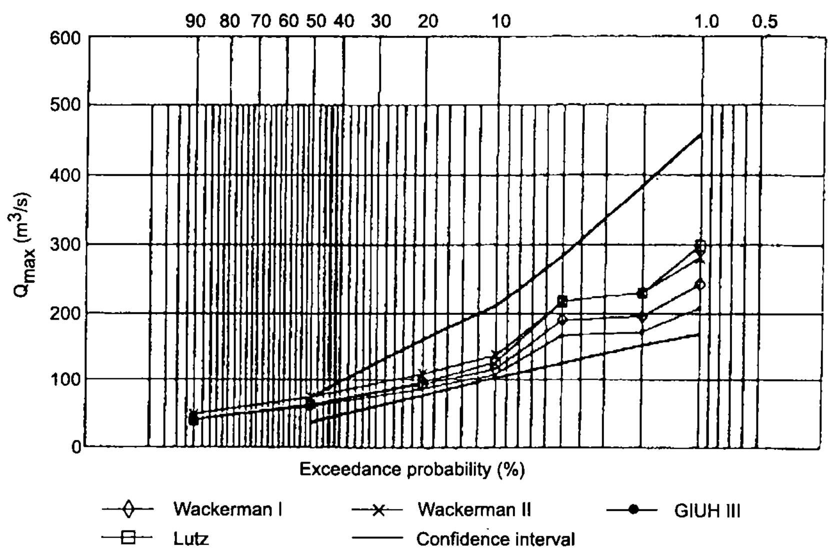

Fig. 3. Comparison of the simulated probability curves with the theoretical one calculated after Pearson III distribution model (Skawa River basin at Osielec).

culated by empirical method, have been then compared to the output obtained from rainfall-runoff models.

The design storm transformation has been made at the following assumptions:

— input to the models was storm intensity of specified time duration and probability of exceedance $I(t_r, p)$;

— effective design storm was not taken into account which is equivalent to CN = 100 or runoff coefficient $\alpha = 1$;

— the base flow (QB) was not added to the probable direct outflow;

— design storm duration was assumed $t_r > 8.5$ hours (Kupczyk, Suligowski, 1997);

— verification has been made in 11 catchments using probablility density distributions (estimated directly from observations). The results of verification are shown in Figures 3 and 4 respectively.

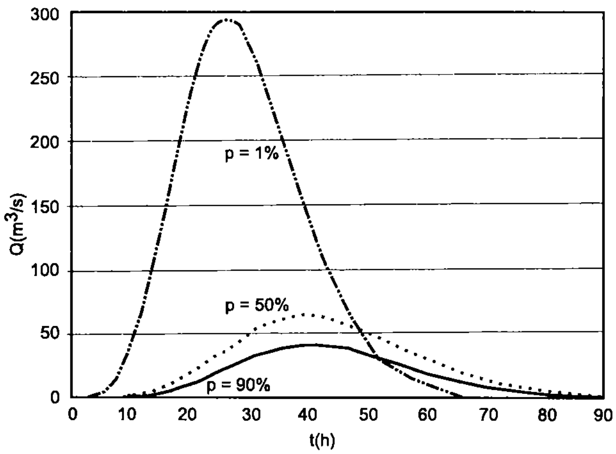

Fig. 4. Design flood hydrographs for the Skawa River basin at Osielec for probabilities p = 1%, p = 50% and p = 90%.

CONCLUSIONS

Achieved results have been found promissing. The method may be recommended for the estimation of peak flow of the flood frequency p > 10% in mountainous catchments. At lower probabilities the results usually have been underestimated. There is a need of similar investigations in lowland catchments, where we encounter problem of design effective rainfall estimation. The difficulty stems from the fact that in lowland basins we cannot assume the equality of effective and design rainfall.

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