

## **MODELLING OF HYDRAULIC CONDITIONS IN THE UPPER NAREW RIVER SYSTEM**

*Dorota Mirosław- Świątek<sup>1</sup>, Janusz Kubrak<sup>2</sup>, Tomasz Okruszko<sup>3</sup>,  
Ignacy Kardel<sup>4</sup>*

**Abstract:** The aim of this work was to model the water management problem of the Upper Narew Valley (North East Poland). The analysis was carried out in the general form of the scenario study, which combined different hydrological conditions during the flood event with different releases polices from the reservoir Siemianówka. There are five basic components of this system, namely: rule based model of the Siemianówka Reservoir, hydrological model of the Narewka River, hydrological model of the Orlanka River, 1D unsteady state flow routing model and GIS ARCVIEW based platform for linking all models. The results of the calculation using developed modeling system led to the conclusions, that to cope the different hydraulic and hydrological models on the GIS platform, which serves not only for the preparation of the input data and presentation of the calculation results but also provides the data handling for each computational time step.

**Keywords:** recognition of hydraulic conditions in the river system, solution of water management problem

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<sup>1</sup> Ass. Professor, Warsaw Agricultural University, Faculty of Engineering and Environmental Science, 02-722 Warszawa, Nowoursynowska 166, dorotams@levis.sggw.waw.pl

<sup>2</sup> Professor, Warsaw Agricultural University, Faculty of Engineering and Environmental Science, 02-722 Warszawa, Nowoursynowska 166, kubrakj@alpha.sggw.waw.pl

<sup>3</sup> Ass. Professor, Warsaw Agricultural University, Faculty of Engineering and Environmental Science, 02-722 Warszawa, Nowoursynowska 166, t.okruszko@levis.sggw.waw.pl

<sup>4</sup> Ass. Professor, Warsaw Agricultural University, Faculty of Engineering and Environmental Science, 02-722 Warszawa, Nowoursynowska 166, i.kardel@levis.sggw.waw.pl

## **INTRODUCTION**

The aim of this work was to model the water management problem of the Upper Narew Valley (North East Poland). Hydraulic models seem to be the most appropriate tool for obtaining this type of characteristics for the riparian wetlands, as it is possible to derive the water stage characteristics for the whole floodplain area assuming the known inflow characteristic in the particular cross-section (e.g. gauge station data) for the particular flood (Demissie et al., 2000, Watanabe et al., 2002). The modeling efforts were concentrating in order to simulate the most important features of the Upper Narew System.

## **DESCRIPTION OF THE UPPER NAREW RIVER SYSTEM**

The area of the catchment of the Upper Narew covers approximately 7300 km<sup>2</sup>, the rivers' hydrological regime is typical of those for the lowland rivers of Central Europe. It is characterized by a single spring flood caused by snow melting, and quite a balanced summer runoff. Raised water stages in summer occur occasionally. Given the morphological conditions and economical development of the region, three reaches of the Upper Narew river valley can be distinguished (Fig. 1).

The first reach extends from the Siemianówka reservoir to the narrowing of the valley in the vicinity of the Suraż settlement. It is a relatively narrow valley, which is flooded for a short time every year. Its nature is typical of marshy meadows becoming dry-ground forests. The natural conditions favor this area for landscape protection, tourism and agriculture. However most of the natural values of this part of the valley strongly depend on water conditions, especially on the occurrence of the spring flood.

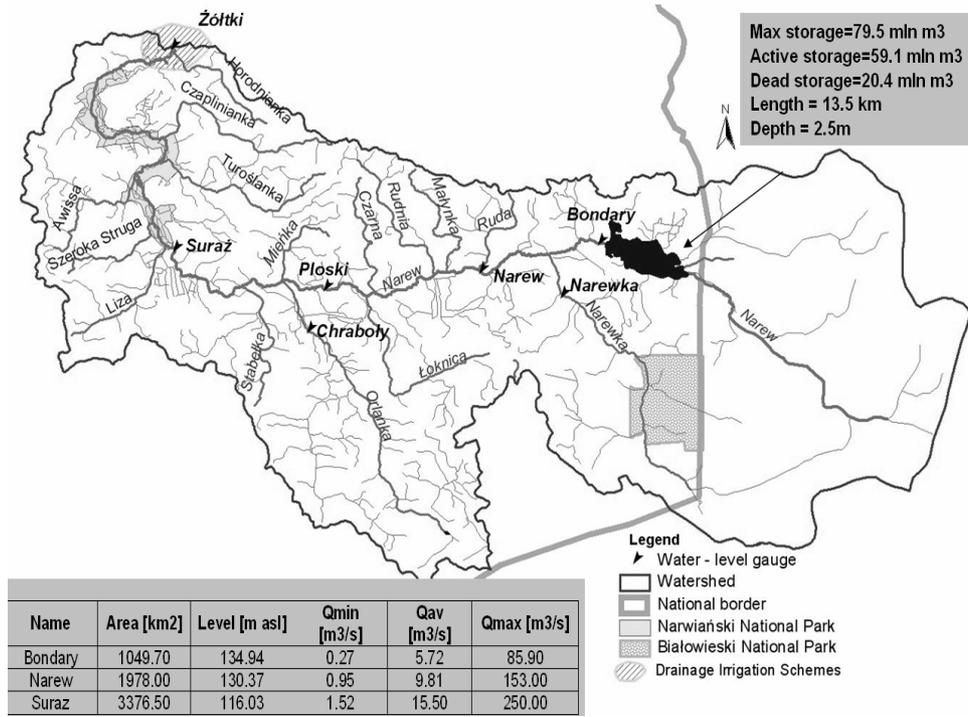


Fig. 1. The hydrographic system of the Upper Narew River

The second valley reach includes the wetlands between Suraż and the Żółtki weir. In 1994 the Narew National Park (NNP) was established to preserve the unique nature of this part of the valley. The basic objective of water management for this area is maintenance of flow rates in the river which ensure the appropriate soil moisture in the surrounding marshes.

The third reach extends downstream of the border of the NNP, in area of Żółtki. The river training works of the Narew River riverbed and the reclamation measures took place in the 1970s. Those activities dramatically changed the features of the valley. The numerous drainage-irrigation systems allowed for intensive agriculture. The basic water management objective in this part of the valley is to ensure sufficient water quantities for irrigation.

The Siemianówka Reservoir started its operation in 1995 after 17 years of construction works. The total capacity of the reservoir is equal to  $79 \cdot 10^6 \text{ m}^3$ . It was planned at early sixties, as a water reservoir supporting the huge investments in drainage of Narew valley wetlands. The wetlands downstream the reservoir are mostly river fed type of marshes. It means that for their sustainability the regular river water inundations are necessary. The natural, extensive meadows in the

upper part of the valley are used by local farmers for hay making and as a pasture. Prior building the reservoir spring floods were unavoidable, so farmers has adapted their practice to this phenomena, which resulted in bio-diversity rich, open spaces habitats of molinia meadows, lowland hay meadows and alkaline fens (all of them recognized by EU Habitat Directive). After constructing the reservoir, there is a constant pressure to go into the more intensive farming, which excludes the presence of the flood pulse. The spring inundation is vitally important for the downstream lying marshes of the Narew National Park.

Looking for the main features of the system it seems that crucial for the water management strategies identifications are: (1) inundation of the river valley between reservoir and Suraż gauge station in order to check, which areas in that part of the valley are sensitive to the water release policy on the reservoir and (2) modification of the inflow characteristics to the Narew National Park during the flooding period in order to check if there is a significant impact of the Siemianówka reservoir on the wetlands located downstream the Suraż gauge station. Modeling efforts were focused on those two phenomena's (Kubrak et al., 2004, Okruszko et al., 2004).

## **METHOD**

### **OVERALL APPROACH**

The analysis was carried out in the general form of the scenario study, which combined different hydrological conditions during the flood event with different releases police from the reservoir Siemianówka. The output contains the hydrological characteristics of inundated area and the inflow characteristics to the area of NNP. In order to perform this scenario analysis modeling system was constructed. There are five basic components of this system, namely: rule based model of the Siemianówka Reservoir, hydrological model of the Narewka River, hydrological model of the Orlanka River, 1D unsteady state flow routing model named hydraulic model in course of this paper and GIS ARCVIEW based platform for linking all models (see Fig. 2).

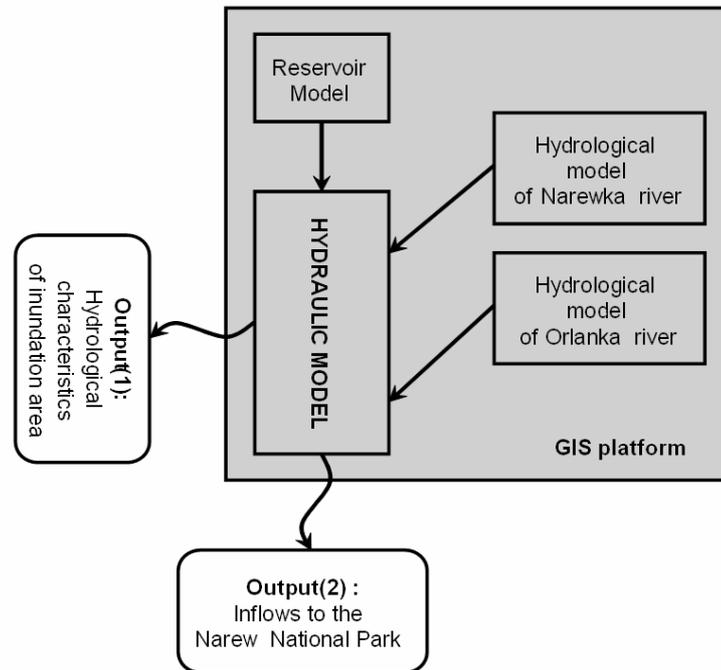


Fig. 2. The basic components of the system

Rule based model of the Siemianówka reservoir consists of two, the most popular in Poland release policies i.e.: standard policy and constant safe outflow policy. Both rules are linear, calculating the outflow from the reservoir as a function of inflow and actual state of the reservoir. The model of Siemianówka reservoir explores a mass balance method. The simple hydrological models of the Narewka and Orlanka Rivers (two main tributaries to the Upper Narew River) were based on the results of the gauge station observations in Narewka and Chraboły cross-sections. The gauge station results were transformed to tributaries outflow by adding the subcatchment flow based on aerial, time dependant specific discharges. Hydraulic model bases on a CCHE1D software package for 1-D unsteady state flow in open channel. The GIS platform consists of the three different programs, namely: NCCHE 1D, ArcView and PCRaster, which are responsible for data preparation for the hydraulic model and final calculation of the hydrological characteristics of the river valley.

### MODEL OF THE SIEMIANÓWKA RESERVOIR

Linear, water balance model was applied in order to simulate the release policy of the Siemianówka Reservoir. The storage area of the reservoir is described by the simple balance equation:

$$VP + Q = VK + QT \quad (1)$$

where:  $VP$  – water volume at the beginning of the time step,  $Q$  – inflow to the reservoir during the time step,  $VK$  – water volume at the end of the time step,  $QT$  – outflow from the reservoir during the time step.

It means that processes of the water evapotranspiration, rainfall on the reservoir surface, drainage, reservoir direct subcatchment inflow and dam filtration were not considered. These processes, which for Polish hydro-meteorological conditions are accounted for about 5 % of the balanced water during one year, were not taken into consideration due to the lack of the data.

So called “standard policy” was used in the Siemianówka reservoir model. It is one of the most common rule based polices used in Poland for reservoir control in normal and low flow conditions. The rules applied in this policy might be described as follows:

$$QT = \begin{cases} VP+Q & VP+Q < P1 \\ P1 & P1 \leq VP+Q \leq P1\beta VU \\ VP+Q-\beta VU & P1+\beta VU < VP+Q \leq P1+P2+\beta VU \\ P1+P2 & P1+P2+\beta VU < VP+Q \leq P1+P2+VU \\ VP+Q-VU & VP+Q > P1+P2+VU \end{cases} \quad (2)$$

where:  $P1$ - water demand of the 1<sup>st</sup> water user (higher in the hierarchy),  $P2$ - water demand of the 2<sup>nd</sup> water user (lower in the hierarchy),  $\beta$  – coefficient of the reservoir share used exclusively by the 1st user,  $VU$  – useful storage volume. In the case of the equal hierarchy of the both users their demands are added and  $\beta$  coefficient is set to 1.

For the high flood control other rule based policy is applied – namely constant safe outflow policy. This policy is described as follows:

$$QT_i = \begin{cases} Q_i & Q_i \leq Q_{doz} \\ Q_{doz} & Q_{doz} \text{ and } VP_i < V_{zb} \\ Q_i & Q_i > Q_{doz} \text{ and } VP_i \geq V_{zb} \end{cases} \quad (3)$$

where:  $Q_{doz}$  - safe outflow,  $V_{zb}$  – reservoir storage.

This rule is applied as long as there is an empty space in the reservoir in flood protection storage zone. The flood protection storage is created as an empty space between the useful volume of the reservoir and its total volume.

Both types of the rules are applied automatically. As long as the inflow to the reservoir is smaller than the safe flow the standard policy rules are used at the moment the inflows exceed the safe flow they are switched to the flood protection rules. The simulation model of the Siemianówka reservoir was used with the daily time step using as an inflow to the reservoir the historical data from the Bondary gauge station.

### **HYDRAULIC MODEL**

The CCHE1D software package was adopted to model the Narew River water flow. This tool was developed by the National Center for Computational Hydroscience and Engineering of The University of Mississippi. It is a software package intended for calculating water surface profiles in a full network of channels, a dendritic system, or single river reach (Weiming, W., Vieira, D.A., 2001). The CCHE1D model simulates unsteady flow in channel networks using the dynamic wave model, taking into account the difference between the flows in the main channel and flood plains of a compound channel. The governing equations of the dynamic wave model for open channel flows are the St. Venant equations:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \quad (4)$$

$$\frac{\partial}{\partial t} \left( \frac{Q}{A} \right) + \frac{\partial}{\partial x} \left( \frac{\beta Q^2}{2A^2} \right) + g \frac{\partial h}{\partial x} + g(S_f - S_o) = 0 \quad (5)$$

where:  $x$  and  $t$  are the spatial and temporal axes;  $A$  is the flow area;  $Q$  is the flow discharge;  $h$  is the flow depth;  $S_o$  is the bed slope;  $\beta$  is the correction coefficient for the momentum due to the nonuniformity of velocity distribution at the cross section;  $g$  is the gravitational acceleration and  $q$  is the side discharge per unit channel length;  $S_f$  is the friction slope defined as

$$S_f = \frac{Q|Q|}{K^2} \quad (6)$$

where  $K$  is the conveyance.

The momentum (5) and continuity (6) equations for water were solved simultaneously for unknowns  $h$  and  $Q$  using the weighted implicit finite difference scheme developed by Preissmann.

### **Cross Section Definition**

In model CCHE1D Version 3.0 the cross-sectional profile is defined by a finite set of nodes. At each node, a pair of the bed elevation and the distance to the left bank is specified. At each segment between two nodes, a value of Manning's roughness coefficient is specified. The CCHE1D model divides the channel cross section into three subsections: a main channel and optional right and left overbank channel subsections. The hydraulic properties are determined for each individual subsections. The conveyance for the entire cross section is determined as the sum of segment values.

### **Boundary and initial conditions**

The initial conditions specify the flow conditions for the river reach. The two initial conditions for the Saint Venant equations are the discharge  $Q(x,0)$  paired with the area,  $A(x,0)$  and stage  $h(x,0)$ , specified for the entire river length at the initial time  $t = 0$ .

In addition to the initial conditions, two boundary ones are required in order to yield a unique solution of Saint-Venant equations.

The boundary conditions for river flow are:

- for  $x = 0$ ,  $Q = f(t)$  – outflow hydrograph downstream of the Boundary gauge,
- for  $x = L$ ,  $Q = f(h)$  – rating curve at the Suraż - downstream end cross-section of the river reach.

At the junction of two rivers it is assumed that the water surface elevations of all branches are equal, and the flow discharge of downstream branch is equal to the sum of the discharges of two upstream branches.

### **GIS PLATFORM**

The GIS platform has been developed in order to generate the river valley cross-sections, which are used for hydraulic calculations and to generate the flood extent in the valley in each time step after the hydraulic model calculations. It is also used for calculation of the hydrological characteristics of the flood on the floodplain i.e. inundation area, depth of the inundation area, duration of the inundation, etc.

As it is presented in the Fig. 3, the exchange of the information between the Digital Elevation Model (DEM) and hydraulic model goes via valley cross-sections. Cross-sections are identified using the DEM (Werner et al., 2004), the actual distance between them is calculated using the flow path features. There are two distances between cross-sections: one for water in channel and second for flood plain. The first one is measured from a map. The second one is calculated using average inundation area and average distance between wet parts of neighbour's cross-

sections. In each time step, after the hydraulic calculations the water stages in the cross-sections are interpolated in the whole river valley.

The platform consists of the three different programs, namely: NCCHE 1D, ArcView and PCRaster. The ArcView is used in order to prepare the basic data for the NCCHE program as well as for the PCRaster. The latest is used for the hydrological characteristic calculations of the inundated area. Figure 4 presents the general calculation flow chart of the GIS platform. This diagram is divided on the two parts: first describe data preparation for hydraulic model and PCRaster software. Second part presents a way of performing the statistical calculations. Additionally detail calculations of this section are show in Fig. 5. For this calculation it was necessary to make own script (Kubrak et al. 2004) because PCRaster is only a programming software dedicated for surface analysis. The names used in Fig. 5 is describe in the PCRaster language script.

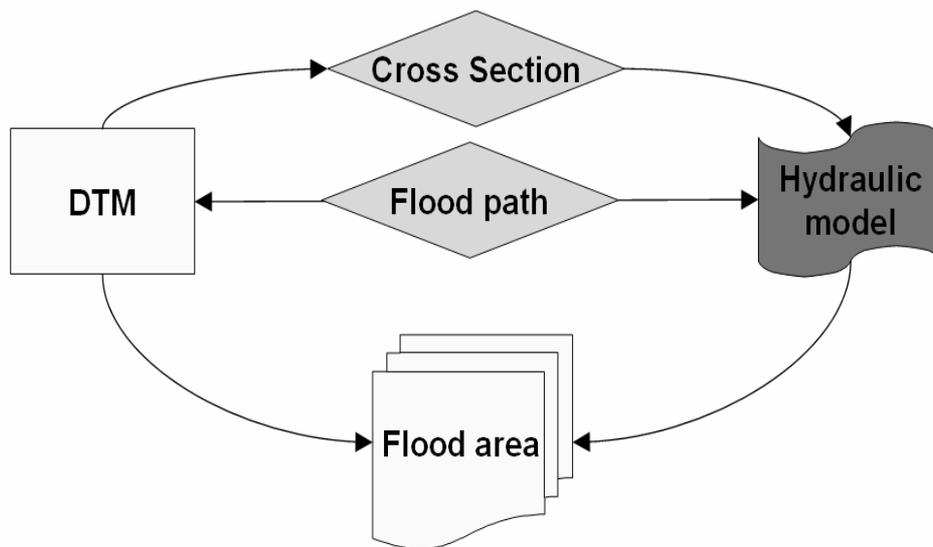


Fig. 3. Logical structure of the GIS platform

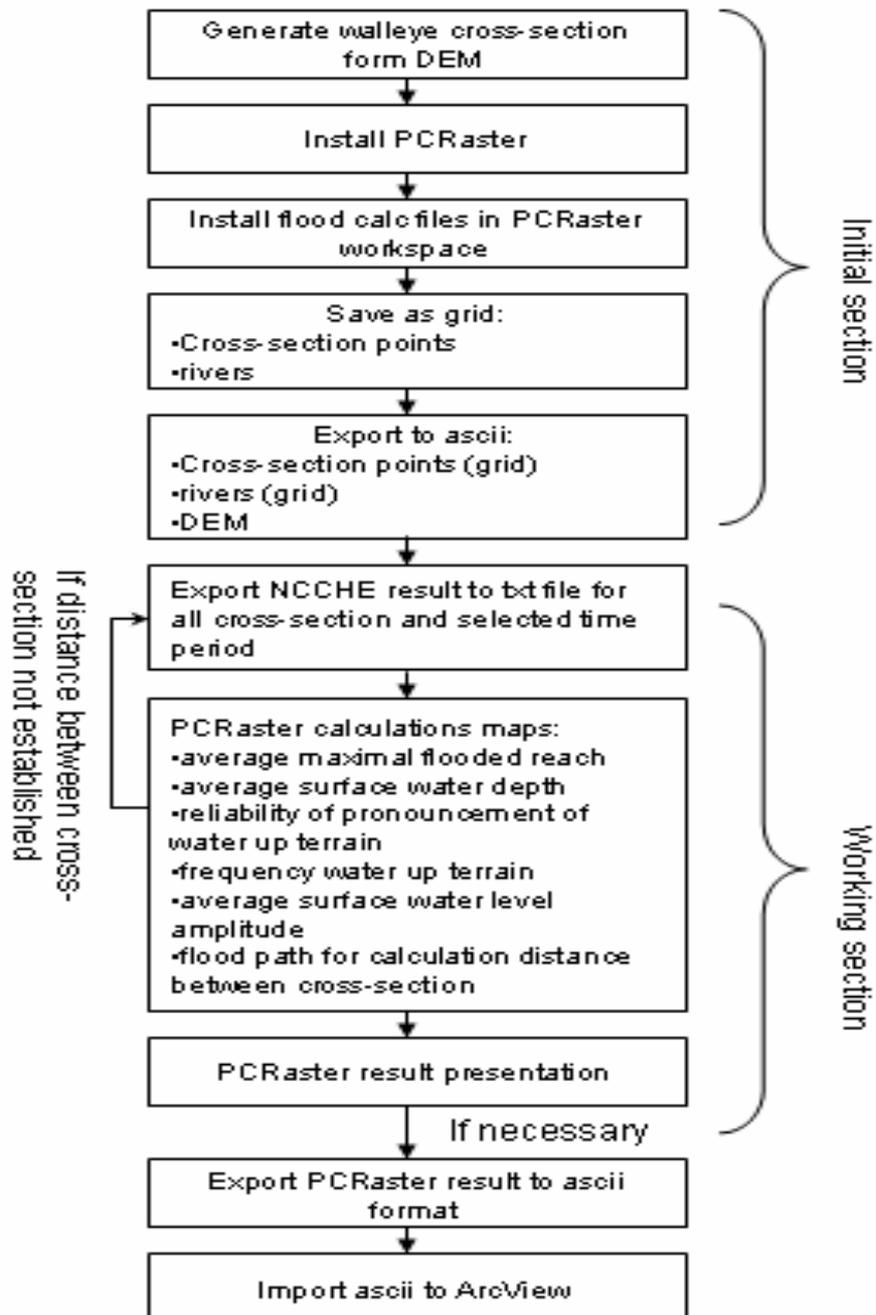


Fig. 4. General flow chart in GIS platform diagram

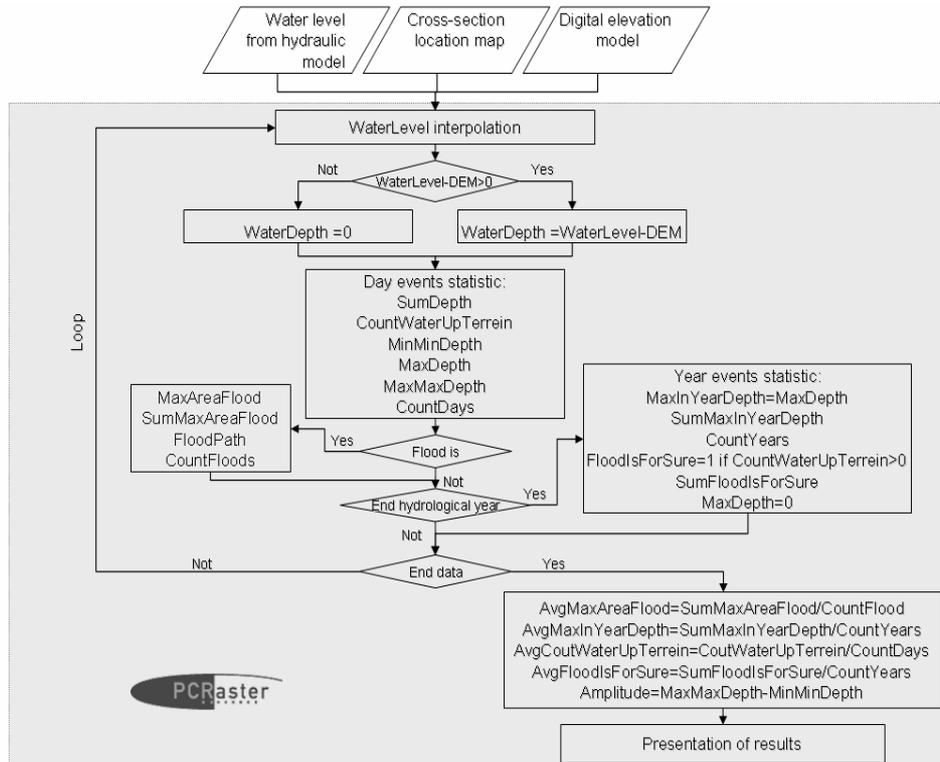


Fig. 5. Flood statistic calculation diagram (working section GIS platform)

## MODEL VERIFICATION

### DATA USED IN MODELING PROCESS

The CCHE1D model was used for the upper Narew River, that is considered as a single channel starting from Bondary gauge and terminating at Suraz gauge (Fig.6). The geometry of this river segment was described by 36 cross-sections (Kubrak et al., 2004). The average distance between cross-sections amount no more then 2 km. Drainage ditches network, old rivers-beds and spreads existing in this area were not taken into account, due to the lack of their geometric and topographic data. The geometry of cross-sections combines both: a river bed and a floodplain. The river-beds were measured by manual sounding, the neighborhood of river banks was leveled and coordinates of points situated on the floodplain were calculated with the use of the Digital Elevation Model. The floodplain cross-sections were extracted from DEM as a representative for variation of terrain topography between each two calculation points.

The upstream boundary condition was described by a flow hydrograph at Bondary (Fig.6). The Orlanka and Narewka Rivers were considered as a point lateral inflow

located at 18.9 km and 58.25 km respectively upstream of Suraż and are described by flow hydrographs in Chraboły and Narewka gauges. The single valued rating curve at Suraż cross-section was used as a downstream boundary condition.

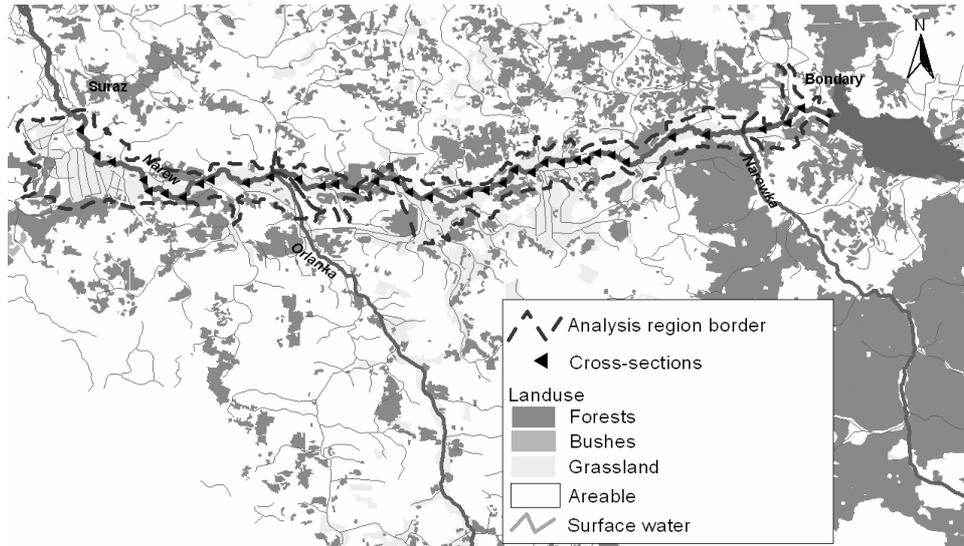


Fig. 6. The cross-sections localization in the Narew River

In analyzed segment of river valley there is a significant lateral inflow from subcatchment (Kubrak et al., 2004). Generally during flood event inflow from tributaries Narewka and Orlanka amount about 30% total volume outflow at Suraż gauge, like as inflow from subcatchment and upstream Bondary gauge. This inflow is modeled in the form of a flow hydrograph, which was described trough the following form:

$$Q_{Subcatchment}(t)=Q_{Suraż}(t+\Delta t)-Q_{inflow}(t) \quad (7)$$

where:

$$Q_{inflow}(t)=Q_{Bondary}(t)+Q_{Narewka}(t)+Q_{Chraboły}(t) \quad (8)$$

$t$  – time,  $\Delta t$  – time lag.

Value of  $Q_{Subcatchment}(t)$  is uniform distributed along the river between Bondary and Suraż. This solution is obviously a rough approximation of the actual situation, as it does not account for the temporal and spatial variation of the lateral inflow. A more realistic approach on this issue needs to be elaborated in further study and should be based at a hydrological model of subcatchment inflow.

In the topological discretization scheme of flow, the Narew River and floodplain was represented as a one-dimensional. Fig. 7 shows the scheme of topological discretization and boundary conditions.

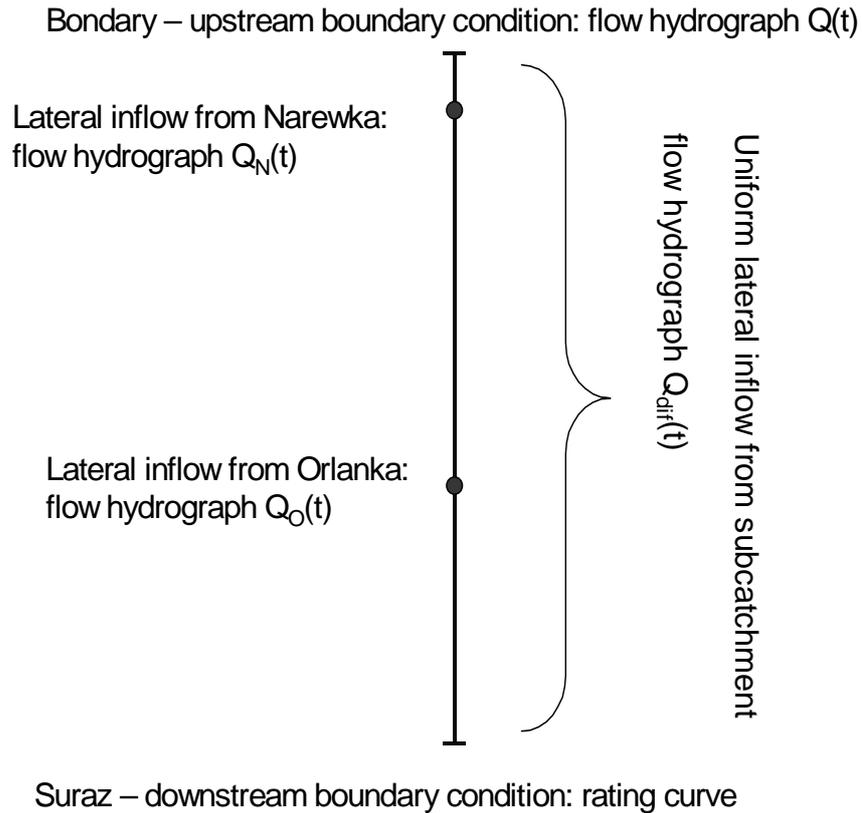


Fig. 7. Topological scheme and boundary conditions

The DEM of the Narew Upper Basin was generated by the ArcInfo TOPOGRID method during an interpolation process (Fig. 8). The TOPOGRID is an interpolation method specifically designed for the creation of a hydrologically correct digital elevation model. From the structure point of view the output from TOPOGRID is a DEM (Digital Elevation Model), which is represented in a form of a raster matrix of cells.

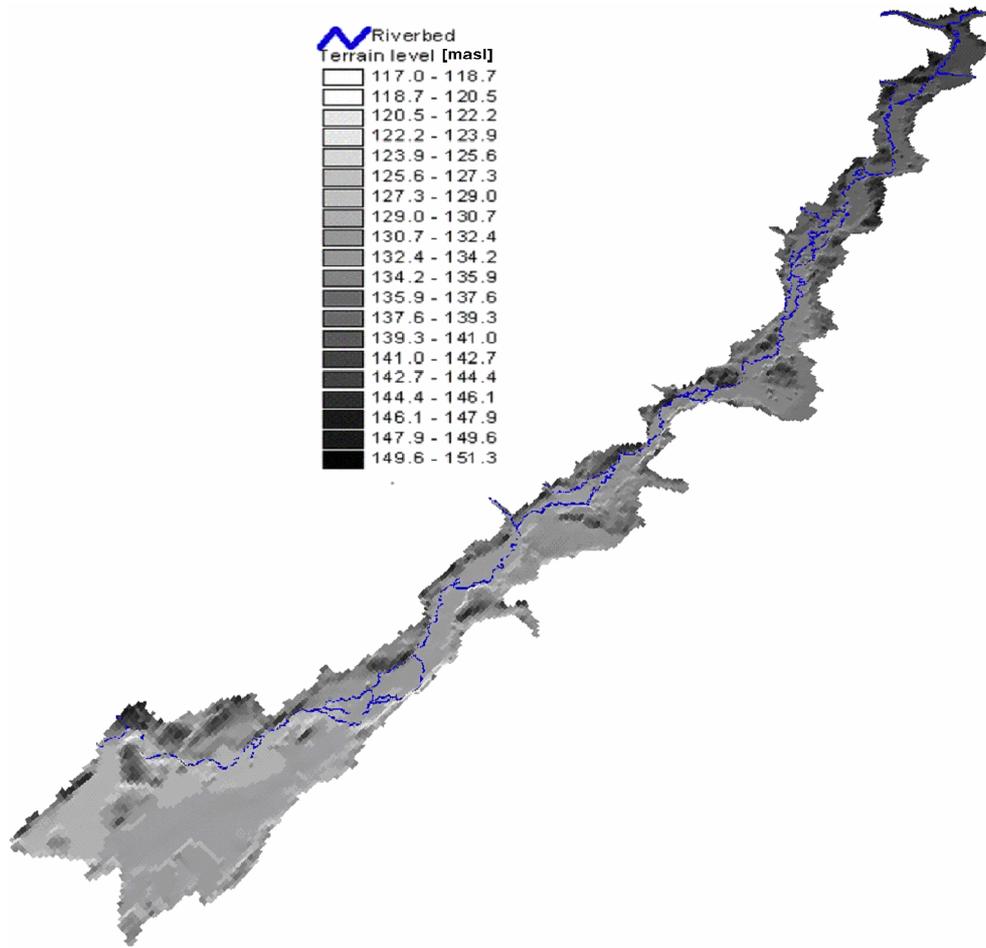


Fig. 8. Digital Elevation Model of Narew Valley between Bondary and Suraz

The DEM of the Narew Upper Basin was created as a raster network with a cell resolution of 20 meters. The main data source for model generation was a set of the contour lines. This set was obtained by “on screen” digitizing of contour lines from topographic maps in the scale of 1:25000 and 1:10000, which had been earlier scanned and registered. The contours on the map mostly were drawn with interval of 1.25 meter, which gave elevation accuracy about 1/3 of the contour interval. The improvement was done by additional measurements, cross-section constructing, streams depth and enforcement drainage parameter for lake and stream in TOPOGRID.

DEM was used in the GIS platform for input, output data exchange with the hydraulic model. Additionally GIS platform was used during the calculation for the data exchange between hydrological model, Siemianówka reservoir model and hydraulic component of the system.

## CALIBRATION AND MODEL VALIDATION

The historical data of flow discharges were used for the model parameters calibration and model performance validation. The ever highest recorded flood of 1979 was used for the model calibration, 24 other flood events of years 1967 - 1988 were used in validation process. Following parameters: Manning's  $n$  factor, time lag  $\Delta t$  in term (8), distance between cross-sections in floodplain and their effective flow area were adapted by calibration for unsteady flow so that observed and calculated values became congruent. In order to evaluate the model performance, the most commonly employed error measures, such as the root mean square error ( $RMSE$ ) calculated for Narew and Suraz gauges. The  $RMSE$  is defined as:

$$RMSE = \sqrt{\frac{\sum_i^N (Q_{m_i} - Q_{p_i})^2}{N}} \quad (9)$$

where:  $Q_m$  - the measured water discharge,  $Q_p$  - the predicted water discharge,  $N$  - the number of observations.

Comparison of the model prediction results with the measured data is given in Table 1 and Fig. 9 for Suraz gauge station. The numerical model satisfactorily simulated the measured data. The model is in very well congruent with observed discharge hydrograph at Suraz gauge in 1979 year. The  $RMSE$  of the flood event amounts  $3.48 \text{ m}^3/\text{s}$  next to maximum discharge  $250 \text{ m}^3/\text{s}$  and the maximum value of  $RMSE$  is 9%. Fact that rating curve at Suraz gauge approximates downstream boundary condition is reason very well accuracy of the model. The discharge and stage  $RMSE$  were calculated for Narew gauge, which is located 55 km upstream Suraz and works out of influence downstream boundary condition.

Table 1. Hydraulic model calibration and validation (Suraż gauge)

Date of flood event	Suraż gauge			
	RMSE Q [m <sup>3</sup> /s]	Q <sub>min</sub> [m <sup>3</sup> /s]	Q <sub>max</sub> [m <sup>3</sup> /s]	RMSE Q <sub>max</sub> - Q <sub>min</sub>
28.04.67 - 25.05.67	0.78	32.5	44.5	6%
18.03.68 - 27.04.68	3.05	27.3	120	3%
02.04.69 - 12.05.69	0.63	26.8	38.5	5%
20.03.70 - 01.06.70	3.02	11.7	176	2%
12.08.70 - 05.09.70	0.26	18.0	25.5	3%
29.10.71 - 23.02.71	1.29	14.0	71.7	2%
19.03.71 - 12.05.71	1.48	27.7	96.8	2%
30.03.72 - 29.04.72	0.25	15.4	24.6	3%
21.03.73 - 18.04.73	0.42	24.5	38.7	3%
29.07.73 - 11.08.73	0.23	18.0	20.7	9%
10.02.74 - 05.03.74	1.82	25.2	56.0	6%
04.10.74 - 02.02.75	2.39	19.6	123	2%
28.03.76 - 05.05.76	0.93	24.8	62.7	2%
22.02.76 - 04.05.77	0.88	11.8	57.2	2%
06.03.78 - 01.05.78	1.72	23.3	81.6	3%
19.03.79 - 24.05.79	3.48	30.3	250	2%
31.03.80 - 19.05.80	2.28	24.1	101	3%
08.03.81 - 24.04.81	2.33	14.1	111	2%
29.12.81 - 21.01.81	1.80	20.2	46.2	7%
03.03.82 - 22.04.82	1.61	24.4	77.4	3%
10.03.83 - 19.04.83	1.42	24.1	68.0	3%
20.03.85 - 17.05.85	0.62	16.3	50.3	2%
21.03.86 - 30.04.86	1.60	24.8	101	2%
28.03.87 - 05.05.87	1.38	18.9	70.4	3%
25.03.88 - 10.06.88	5.48	7.40	93.8	6%

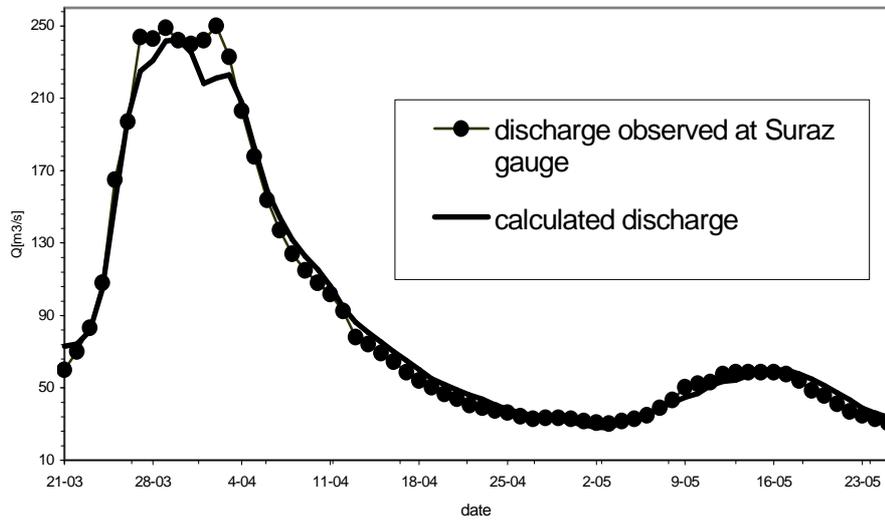


Fig. 9. Calculated and observed discharges at Suraz gauge for flood event in 1979

Table 2. Hydraulic model calibration and validation (Narewka gauge)

Date of flood event	Narew gauge							
	RMSE Q [m <sup>3</sup> /s]	Q <sub>min</sub> [m <sup>3</sup> /s]	Q <sub>max</sub> [m <sup>3</sup> /s]	RMSE $\frac{Q_{max} - Q_{min}}{Q_{max} - Q_{min}}$	RMSE H [m asl.]	H <sub>min</sub> [m asl.]	H <sub>max</sub> [m asl.]	RMS $\frac{H_{max} - H_{min}}{H_{max} - H_{min}}$
28.04.67 - 25.05.67	2.64	15.2	29.5	18%				
18.03.68 - 27.04.68	4.71	14.0	90.9	6%				
02.04.69 - 12.05.69	3.15	10.3	30.0	16%				
20.03.70 - 01.06.70	12.51	7.18	97.5	14%				
12.08.70 - 05.09.70	2.73	11.8	19.7	35%				
29.10.71 - 23.02.71	4.19	8.10	50.2	10%				
19.03.71 - 12.05.71	3.24	13.6	58.5	7%				
30.03.72 - 29.04.72	1.40	11.4	21.0	15%				
21.03.73 - 18.04.73	1.11	14.4	32.5	6%				
29.07.73 - 11.08.73	2.54	10.7	17.6	37%				
10.02.74 - 05.03.74	2.33	14.1	37.4	10%				
04.10.74 - 02.02.75	3.88	12.2	86.0	5%				
28.03.76 - 05.05.76	1.84	13.3	51.2	5%	0.14	131.23	132.08	16%
22.02.76 - 04.05.77	1.92	7.92	32.0	8%	0.15	131.33	131.79	33%
06.03.78 - 01.05.78	3.68	15.8	73.8	6%				

19.03.79 - 24.05.79	5.34	15.3	149	4%	0.18	131.33	132.58	14%
31.03.80 - 19.05.80	3.78	13.1	62.5	8%	0.11	131.25	132.17	12%
08.03.81 - 24.04.81	3.23	9.00	50.0	8%	0.14	131.25	132.05	18%
29.12.81 - 21.01.81	5.19	16.0	34.4	28%				
03.03.82 - 22.04.82	6.14	15.6	53.9	16%				
10.03.83 - 19.04.83	2.46	16.0	39.9	10%				
20.03.85 - 17.05.85	2.48	10.5	38.6	9%				
21.03.86 - 30.04.86	3.69	14.1	85.9	5%				
28.03.87 - 05.05.87	4.41	11.7	53.7	11%				
25.03.88 - 10.06.88	9.46	4.72	87.0	11%				

The maximum value of discharge RMSE, related to difference between maximum and minimum of flood event discharge, amounts 37% for the summer flood. Similar errors were calculated for the others summer and winter flood events (Table 2). These significant errors appear because hydraulic model does not take into account ice phenomena and is not able to right reproduce growth of vegetation in summer. The discharge and stage maximum errors for historical spring flood waves not limited 16% and 18% respectively. This error is caused most probably due to inaccuracy in calculation of the lateral inflow originated in subcatchment.

## **CONCLUSIONS**

The results of the calculation using developed modeling system led to the following conclusions:

1. It was feasible to cope the different hydraulic and hydrological models on the GIS platform, which serves not only for the preparation of the input data and presentation of the calculation results but also provides the data handling for each computational time step.
2. The hydraulic model calibration, however successful in the terms of the simulation of discharges from the modeled system, needs the improvement in subcatchment lateral inflow identification as well as in verification of the spatial extend of the flood against the RS data for the historical floods.
3. Due to high variation of each component of the flood wave in Suraz it is necessary to performed the detailed hydrological study of flood formation in the Upper Narew rivers system. In this study hydrological monitoring system should be developed, containing hydrological data recording in main river and floodplains during flood event by network of automatically gauges. This data will be use in calibration process of model and can allow to recognize of water flow regime in overbanks. Furthermore this knowledge will apply for decision witch one 1D or 2D models is necessary for representing flooding in this area.

4. The numerical tool described in the paper should be used for the delimitation of the inundated areas, which appears during the average flood in the area between the reservoir and NNP.

## **ACKNOWLEDGEMENTS**

The research described in this paper was founded by the *Technology Transfer Program Project TTP07* National Center for Computational Hydrosience and Engineering of the University of Mississippi provided the computational program and technical advice which is highly acknowledged.

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