at shallow depth. It was taken into account by Bolotovskii. Therefore in our calculations there are some differences in the amplitude of the waves and in the velocity of their propagation. Figure 5 shows the chronograms of elevation of the free surface at the points indicated in Figure 3 by numbers 1,2,3, respectively.

We earlier simulated the process of draining and filling this basin without considering the bottom friction. At the instant the total depth H is relatively large, taking into account the friction does not lead to any substantial refinement of the calculation results. If the value of H is small and the friction is not taken into account, the amplitudes of incoming and reflected waves increase by about 0.3–0.5 m. This testifies that it is advisable to take into account the friction in the first approximation of the shallow-water model.

We also simulated the wave processes that appear when the temporal dam built across the basin was destroyed. In this case the whole boundary of the basin was considered as an impermeable vertical wall.

Note that when the dam is destroyed, two groups of waves form, they propagate along the opposite banks. Then these groups of waves interact with each other, and a wave object of significant amplitude results, which is well seen in Figure 6. If the bottom friction is taken into account, the amplitude of this object decreases by about 7 per cent as compared to the amplitude calculated without taking into account the friction.

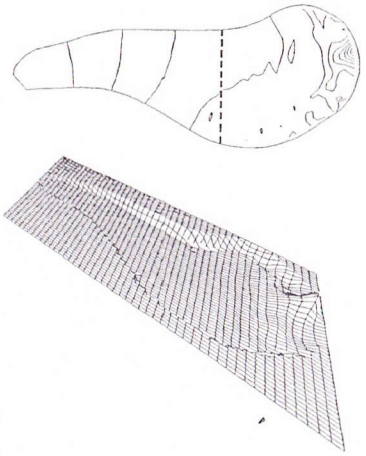

Figure 6. Free surface and its isolines in the problen on the dam distruction.

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System identification on open-channels

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ABSTRACT: This paper is on identification of open-channel characteristics. It describes a method to obtain an approximate model of water movement using linear regression techniques known as system identification. The derived model is to be used for the design of a real time control system. The principles of identification and design of experiments on open-channels are expounded. The method was applied at the Arizona Canal.

1 INTRODUCTION

Water is an increasingly scarce resource. Due to this the need for efficient water use is increasing. In the field of irrigation the performance of the channel system could be improved by better control of the regelators. Automation of the control system will improve efficiency and ease the complex task of the water managers (Ankum 1995). Along with other institutions, the Water Management Section at the Faculty of Civil Engineering of Delft University of Technology has rationalised the design process for automated control of open-channels.

2 MODELLING OF OPEN-CHANNEL FLOW

To obtain simple, reliable control systems, efficient design methods have been developed in linear control theory (e.g. Åström 1984). Such methods require a linear model of the water movement. Schuurmans (1997) simplified the St. Venant equations (e.g. Chow 1959) and derived an approximate model for open-channel flow. This divides a reach into two parts: a part with uniform flow and a backwater part (fig.1).

In the part of the channel in which the flow can be described as uniform flow, it is assumed that waves travel in downstream direction only. Furthermore, it is assumed that flow in the uniform part can be described by a time delay only and that the deformation of waves can be neglected:

$$q_{in2}(t) = q_{in}(t-\tau) \tag{2.1}$$

where q_{in} = inflow (m^3s^{-1}); q_{in2} = discharge into backwater part (m^3s^{-1}); τ = time delay (s); t = continuous time variable (s).

In the backwater part the water surface is assumed to be horizontal. It can be modelled using a reservoir approach:

$$A\frac{dh(t)}{dt} = q_{in2}(t) - q_{out}(t) \tag{2.2}$$

where A = backwater surface (m^2); h = water level (m); q_{off} = offtake discharge (m^3s^{-1}); q_{out} = outflow representing the sum of the outflow to the next reach and the offtake discharge (m^3s^{-1}).

The approximate model for a reach consisting of a part with uniform flow (eq. 2.1) and a backwater part (eq. 2.2) becomes:

$$A\frac{dh(t)}{dt} = q_{in}(t-\tau) - q_{out}(t) \tag{2.3}$$

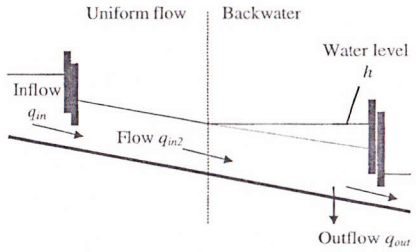

Figure 1. Open-Channel divided in a part with uniform flow and a backwater part.

The control design computations require a discrete description of the water movement in the frequency domain. Assuming that all signals are measured at a sample time T_s and that τ/T_s is an integer, the discrete frequency domain variable z can be defined as the backward shift operator, with the property (e.g. Ljung 1994):

$$z^{-1}h(t) = h(t - T_s) \tag{2.4}$$

When defining the backwater coefficient b (sm^{-2}) and the delay n_k (-):

$$b = \frac{T_s}{A} \tag{2.5}$$

$$n_k = \frac{\tau}{T_s} \tag{2.6}$$

the discrete frequency domain description for an open-channel reach becomes:

$$h(t) = \frac{b}{1 - z^{-1}}\left(z^{-n_k} q_{in}(t) - q_{out}(t)\right) \tag{2.7}$$

This model is reasonably accurate under the condition that the inflow variations are small around steady state. For design of a control system two models are defined each describing an extreme operational condition: one model at the lowest flow rate and one at the highest. The backwater area and the time delay of both models are used to calculate the settings of a PI-controller (e.g. Åström 1984). It has been shown that this leads to stable and reliable controllers (Schuurmans 1997).

The backwater area and the time delay can be defined using knowledge on physical parameters of the system, such as geometry and friction. For irregular channels though, the accuracy of the model may be insufficient because these parameters are not known precisely, and may change in time. Applying experimental modelling techniques known as system identification help to overcome these problems.

3 IDENTIFICATION

When using identification techniques, a linear model of the open-channel flow is obtained by parametrising a model set using the system's externally measurable signals. The inflow q_{in} and the outflow q_{out} affect the system and are defined as its inputs. The water level h shows the state of the system and is called output.

3.1 *System and disturbances*

The approximate model of open-channel flow (eq. 2.7) is of course an abstraction from reality. The open-channel will not satisfy the property of linearity, but will exhibit deviations from this framework. Additionally, measurements that are taken from the system will always be contaminated with disturbance signals due to e.g. wind waves or noise induced by measurement devices. In order to deal with these effects the disturbance term $v(t)$ is added to equation 2.7:

$$h(t) = \frac{b}{1 - z^{-1}}\left(z^{-n_k} q_{in}(t) - q_{out}(t)\right) + v(t) \tag{3.1}$$

This equation shows that the observed water level depends on inflow, outflow, and disturbance. However, the outflow on its turn depends momentarily on the water level: e.g. a rise of the water level will result in an increase of the outflow. This mechanism is knowns as feedback. The relation between the water level and the outflow is the transfer function C, which can physically be intepreted as the discharge equation of the control structure at the downstream end. The open-channel system is schematically shown in figure 2.

The disturbance $v(t)$ can be modelled as a sequence of zero-mean identically distributed, independent random variables. This means that we can write:

$$v(t) = H_0(z)w(t) \tag{3.2}$$

where $H_0(z)$ is a proper rational transfer function that is stable, and $w(t)$ is a sequence of zero mean, identically distributed, independent random variables known as white noise.

If the transfer function:

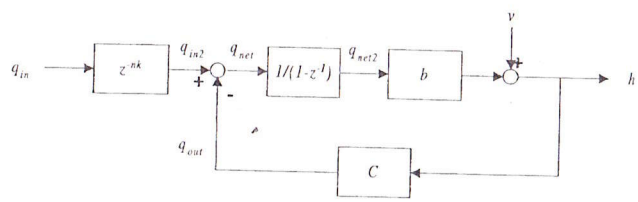

Figure 2. Block diagram of open-channel system.

$$G_0(z) = \frac{bz^{-n_k}}{1 - z^{-1}} \tag{3.3}$$

is introduced and it is assumed that the outflow is fixed, equation 3.1 is reduced to:

$$h(t) = G_0(z)q_{in}(t) + H_0(z)w(t) \tag{3.4}$$

This equation will be used to explain the applied identification method.

3.2 *Identification method*

The transfer functions $G_0(z)$ and $H_0(z)$ can be identified using the ARX model structure (Ljung 1987):

$$G(z,\theta) = \frac{B(z)}{A(z)} = \frac{b_1 + b_2 z^{-1} + \ldots + b_{n_b} z^{-n_b+1}}{1 + a_1 z^{-1} + \ldots + a_{n_a} z^{-n_a}} z^{-n_k} \tag{3.5}$$

$$H(z,\theta) = \frac{1}{A(z)} = \frac{1}{1 + a_1 z^{-1} + \ldots + a_{n_a} z^{-n_a}} \tag{3.6}$$

where n_k is the number of delays from input to output and n_a and n_b are the orders of the polynomials $A(z)$ and $B(z)$ respectively. The parameter vector θ thus contains the coefficients a_i and b_i of the transfer functions and is defined as:

$$\theta^T = \begin{bmatrix} a_1 & a_2 & \cdots & a_{n_a} & b_1 & b_2 & \cdots & b_{n_b} \end{bmatrix} \tag{3.7}$$

For each value of the parameter set θ the model provides us with a prediction of the water level. The signal w(t) cannot be predicted, since it is independent of everything that has happened before. The prediction is denoted by:

$$\hat{h}(t,\theta) = \varphi^T(t)\theta \tag{3.8}$$

where $\varphi(t)$ is the regression vector, which in the ARX case satisfies:

$$\varphi(t) = \begin{bmatrix} -h(t-1) \\ -h(t-2) \\ \vdots \\ -h(t-n_a) \\ q_{in}(t) \\ q_{in}(t-1) \\ \vdots \\ q_{in}(t-n_b) \end{bmatrix} \tag{3.9}$$

At time t it is evaluated how good this prediction is by calculating the prediction error:

$$\varepsilon(t,\theta) = h(t) - \varphi^T(t)\theta \tag{3.10}$$

When having collected data sequences of inflow $q_{in}(t)$ and water level $h(t)$ over a period $t = 1,2,\ldots,N$, it is possible to evaluate how well the model with the parameter θ can describe the performance of the open-channel system. The criterion function is defined:

$$V_N(\theta) = \frac{1}{N}\sum_{t=1}^{N}\left(h(t) - \varphi^T(t)\theta\right)^2 \tag{3.11}$$

Since the model is a linear regression, this function is quadratic in the prediction error $\varepsilon(t)$ and the parameter set θ. This implies that the optimal parameter set θ can be calculated when the first derivative of the criterion function $V_N(\theta)$ is set to zero. The result is:

$$\hat{\theta} = \left[\frac{1}{N}\sum_{t=1}^{N}\left(\varphi(t)\varphi^T(t)\right)\right]^{-1}\left[\frac{1}{N}\sum_{t=1}^{N}\left(\varphi(t)h(t)\right)\right] \tag{3.12}$$

3.3 *Design of identification experiments*

A successful identification application requires that the collected measurement data contain significant information about the system. This means that the data aquisition has to be well planned.

To capture the characteristics of an open-channel reach, it is necessary to execute an experiment using an inflow signal q_{in} covering a wide frequency range. One single pure sinusoid with frequency omega, for example, only gives information on the value of the transfer function at frequency omega. There are infinitely many systems that give the same output with this input. A Pseudo Random Binary Signal (e.g. Van den Hof 1996) meets the requirement. It shifts randomly between two levels at discrete points in time and can be defined by determining amplitude and basic time interval between two consecutive step changes. These parameters should be designed such that the inflow q_{in} results in measurable and safe variation of the water level h. They can be found by using historical data and knowledge of canal operators on the effect of stepwise inflow changes on the downstream water level.

Furthermore, it is important to define a sample time T_s for the identification procedure. It is coupled to the time constants of the open-channel system. Sampling that is considerably faster than the system dynamics will lead to data redundancy. The result is that the model fit may be concentrated to the high-frequency range, and thus not describe the essential dynamics. Sampling that is considerably slower than the system dynamics will lead to serious difficulties in determining the parameters that describe the dy-

namics. The time delay of a reach is used as a measure of these dynamics: the longer the time delay, the more the dynamics will influence the deformation of waves. Operators will be able to give an estimate of the time delay. It is advised to place about 10 sample points over the estimated time delay. However, it is wise to sample faster during data aquisition. The decision which sample time is going to be used for identification can be deferred to computer sessions.

To obtain sufficient data for a proper identification, experiments needs to be executed for duration of roughly 8 times the estimated time delay.

It should be noticed that the parameters, which set up an experiment, depend on the characteristics of a channel reach. These are the characteristics we would like to identify. Thus, the design of an experiment will remain a process of trial and error. The rules of thumb presented above will have to be used as a starting point only.

3.4 *Treatment of data*

Prior to identification of the parameters, it is necessary to modify the set of measured data in order to obtain consistent identification results.

First, data points that are obviously incorrect are removed by linear interpolation between good measurements that neighbour these. Afterwards, a slowly varying trend, which results in a drifting output signal, is removed by the best straight line fit. Next, the average of both input and output signals is made zero mean by subtracting the mean from the original data.

3.5 *Identification of delay and backwater*

After having applied the pre-processing steps the data is ready for identification. First, the time delay n_k is estimated using the inflow q_{in} as input and the water level h as output. As recommended by Ljung (1994), the quadratic criterion function (eq. 3.12) is evaluated for all reasonable delays in a fourth order ARX model (n_a = 4 and n_b = 4). The time delay leading to the smallest value of the criterion function is chosen and used to shift the inflow q_{in} in time leading to a desciption of the discharge into the backwater part q_{in2} (fig.2):

$$q_{in2}(t) = z^{-n_k} q_{in}(t) \tag{3.13}$$

Next, the net discharge into the backwater area q_{net} is calculated by subtracting the measured outflow q_{out} from the discharge into the backwater part q_{in2} (fig.2):

$$q_{net}(t) = q_{in2}(t) - q_{out}(t) \tag{3.14}$$

The transfer function from the water level h to the net discharge q_{net} is the approximate model of the backwater part. It contains a pure integrator and the backwater coefficient b. In order to be able to identify a value of b, the knowledge of the integrator is used to construct the integrated net discharge:

$$q_{net2}(t) = \frac{1}{1-z^{-1}} q_{net}(t) \tag{3.15}$$

Now, the backwater coefficient b is identified using q_{tot2} as input and the water level h as output:

$$h(t) = b q_{net2}(t) + H_0(z) w(t) \tag{3.16}$$

by applying an ARX model structure with $n_a = 0$ and $n_b = 1$ analogue to the previous identification of the time delay n_k.

4 APPLICATION ON THE ARIZONA CANAL

4.1 *Description of the Arizona Canal*

The identification method was applied on the Arizona Canal, which is one of the main canals delivering water to the city of Phoenix (USA) and its surroundings. The canal is owned and operated by Salt River Project (SRP). Experiments were prepared in cooperation with the United States Water Conservation Laboratory and executed on five reaches (Silvis 1998). The results of the experiment on the first and the fifth reach are worked out, which will lead to a good understanding of possibilities and limitations of identification. Table 1 summarizes the average dimensions of these reaches (Clemmens 1997).

The Arizona Canal is monitored and controlled by one water master in SRP's central water control room. A supervisory control and data aquisition (SCADA) system is used. It includes electronic equipment in the field and a computer housed at a central location. The equipment receives telemetered data from control structures, such as water levels and gate positions. Data is stored at a one-minute time interval. The central computer automatically calculates flow rates for all gates at any given station. This information is used by the water master to schedule flows through the canal.

4.2 *Experiments on the Arizona Canal*

SRP imposed the constraint on the experiments that normal operation of the canal should not be influenced: the water level variations at the downstream end had to remain within a fixed range, and the duration of each experiment had to be limited to one night.

The inflow q_{in} and the sample time T_s were designed with the help of the rules of thumb presented previously (tab.2). The experiments were executed by remotely operating the upstream control structure in order to create the stepwise varying inflow. The gate opening of the control structure at the downstream end was fixed. Resulting data was stored by SCADA and could be read from computer screens during the experiment. Measured water level, inflow and outflow, which is the sum of the offtake discharge and the outflow to the next reach, are plotted in figures 3-8.

Table 1. Dimensions of reaches of Arizona Canal.

	Reach #1	Reach #5	
Flow capacity	53	45	m^3s^{-1}
Length	12.7	2.6	km
Bottom width	18	18.9	m
Side-slope (H/V)	1.4	1.4	-
Bottom-slope	4.7e-4	2.0e-4	-

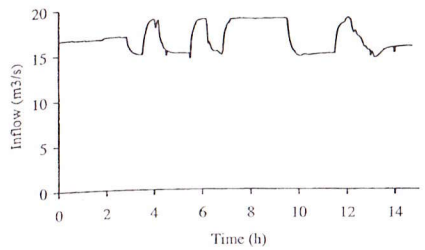

Figure 3. Inflow reach #1.

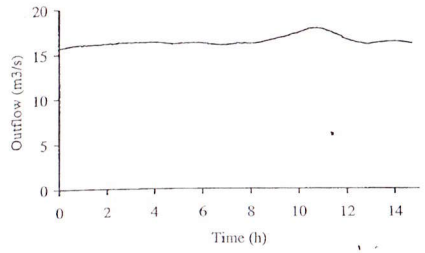

Figure 4. Outflow reach #1.

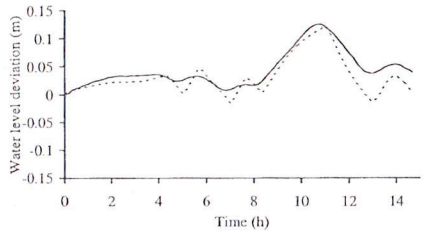

Figure 5. Measured (solid) and simulated (dashed) water level deviation reach #1.

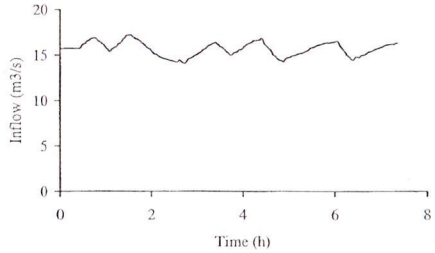

Figure 6. Inflow reach #5

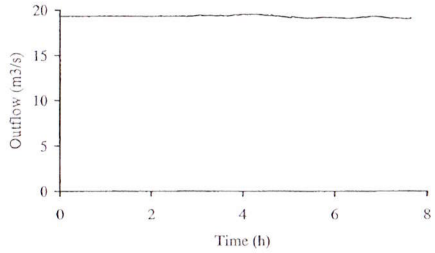

Figure 7. Outflow reach #5.

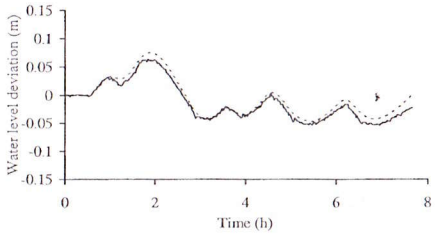

Figure 8. Measured (solid) and simulated (dashed) water level deviation reach #5.

Table 2: Parameters describing experiments.

		Reach #1	Reach #5	
Nominal flow		17	16.5	m^3s^{-1}
Estimation of delay		120	15	min
Inflow:	Basic interval	40	20	min
	Amplitude	2	1.4	m^3s^{-1}

4.3 *Identification*

The measurements of inflow, outflow and water level were used to identify the characteristics of the channel reaches. First, reach #1 was identified using 60 samples at a sample time T_s of 10 minutes. The identification method as described previously was applied, and led to estimates of the backwater coefficient b and the delay n_k setting up the model:

$$G_{reach\#1}(z) = \frac{bz^{-n_k}}{1-z^{-1}} = \frac{0.0058z^{-8}}{1-z^{-1}} \qquad (4.1)$$

Analogue to the identification of reach #1 the parameters characterizing the fifth reach were identified. Using 100 samples at a sampling interval T_s of 1 minute the following model was identified:

$$G_{reach\#5}(z) = \frac{bz^{-n_k}}{1-z^{-1}} = \frac{0.0016z^{-1}}{1-z^{-1}} \qquad (4.2)$$

Both models were used to simulate the water movement using measurements of the inflow q_{in} and the outflow q_{out} as input signals. The outputs of simulation of the first and the fifth reach should be in agreement with the measured water level h.

It can be seen that the identified model of reach #1 overestimates the water level variation in the beginning, but is very well able to catch the peak at 11 hours (fig. 5). This indicates that the discharge variations were forced out of the range in which linearity is valid. Due to the long delay time of the first reach, dispersion of waves will make identification of the water movement difficult. The results of the experiment on reach #5 (fig.8) show that identification of a reach having a shorter delay has led to very accurate simulation results.

Finally, the identified backwater and delay coefficient were translated into physically interpretable backwater areas and time delays using the sample time T_s and equation 2.5 and 2.6 respectively (tab.3). These values can be used for the design of a control system.

5 CONCLUSION

The aim of identification of open-channel characteristics is to estimate models that can be used for controller design. The estimation of the time delay can be used for the design of a feedforward controller, and a feedback controller can be tuned using both the time delay and the identified backwater area. It is shown that identification leads to an approximate model of the water movement in the Arizona Canal that is able to give a good description of its characteristics. Although the actual controller design has not been a subject of the study, the structure of the identified model will enable the design of stable and reliable controllers.

When applying identification on open-channels having a long delay time, dispersion of waves will make identification of water movement more difficult since this dynamic effect is not taken into account by the approximate model. It will be hard to design an inflow signal, which results in a measurable water level variation at the downstream end of a reach without forcing the system out of the operational range in which linearity is valid. It might be useful to develop a model which is able to give a more accurate description of the uniform flow part.

A problem of models in general is that the system's properties may change in time, so that the model may become invalid for the new situation. However, the advantage of identification is that it is relatively easy to execute experiments. A new, low cost model can be found for the changed situation, so that the control mechanism can easily be updated with the help of identification.

Table 3. Results of identification.

	Reach #1	Reach #5	
Sample time T_s	10	1	min
Backwater coeff. b	0.0058	0.0016	sm^{-2}
Delay coeff. n_k	8	2	-
Backwater area A	10.3	3.8	ha
Time delay τ	80	2	min

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Increasing accuracy of the box model

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ABSTRACT: This paper presents non-full mixed box model with a retarded parameter. The model assumes the transport time of water and contamination through the reservoir. It gives the opportunity to describe the exchange process more exactly. As a result, the model UNDBE increase the accuracy of prediction of a pollutant concentration in the outflow of reservoir particularly in the case of a short time forecast. This approach not requires more detailed field measures than the full mixed camera model. The mathematician and soft complexity of the model increased not so strongly. The block of parameters identification was include in the model's soft. It makes possibly to choose the best model parameters and therefore to calibrate the model for a concrete reservoir and pollutant. There are results of modeling for concentration ^{90}Sr in the Kiev Reservoir after the Chernobyl accident in the article.

1 INTRODUCTION

The Chernobyl accident heavily contaminated the Dnieper river-reservoir system. The Chernobyl Nuclear Power Plant(NPP) is situated on the bank of the Pripyat river 30 km from its inflow into the Kiev Reservoir at the Dnieper river. The floodplain territories near Chernobyl NPP and surrounding watersheds are heavily contaminated by ^{137}Cs and ^{90}Sr. Radionuclides are washing out from the watersheds under the impact of snow-melting and rain storm runoff. Significant part of radionuclides fallout took place on the watershed of the Dnieper's right tributary -- the Pripyat River. The highest level of radioactivity in the water of the Pripyat river was observed during the initial period after the accident. Radionuclides contents in the river flow are permanently decreased during the time from 1986 to 1997. But during each high water period its concentration in the water temporary increased under the wash-off and erosion phenomena, creating a secondary contamination of the water masses. The largest part of an annual runoff routs through the reservoirs during spring flood from March till June. The spots of radionuclides exist also in the upper Dnieper watershed on Russian and Belorussian territories. Radionuclides washed out from April 1986 transported down flow by the Dnieper water to the Black Sea through the set of six reservoirs. From the Pripyat River mouth downstream there are Kiev Reservoir, Kanev Reservoir, Kremenchug Reservoir, Dneprodzerzhinsk Reservoir, Dneprovskoje Reservoir and Kakhovskoje Reservoir. Since 1986 the level of ^{137}Cs concentration in the Dnieper reservoirs decrease (compare with the pre-accident values). It due to low magnitude of the spring floods and as a result of the diminishing of ^{137}Cs wash-out coefficient. The coefficient of ^{90}Sr washes out does not diminish in the same manner. Therefore the ^{90}Sr contamination remains most significant problem for high spring floods in the Pripyat River - Dnieper River system. The population of Dnieper basin regions is 32.5 million people. Dnieper water is consumed through the municipal tap water supply more than 8.1 million people in 11 regions. The Dnieper reservoirs are also used for commercial fishing. More than 1.8 million hectares in Ukraine are irrigated by Dnieper water. Therefore this problem is very important even nowadays ten years after the accident.

There are many approaches and models for contamination transport modeling in surface waters (IAEA, Safety Series N50-SG-S6 1985). The urgency to increase the predictability of models in post-Chernobyl situation requires development the methods' pollution migration modeling especially radionuclides. From another side there is possibility to compare mathematical models and field measures. The set of models of different complexity(from three dimensional to box) was developed to simulate radionuclide transport in the Dnieper reservoirs in the Cybernetics Center, Kiev (Morozov et al. 1996). One of them WATOX is a box model which variables are averaged over compartments that as usual