32321

A DISPERSION MODELLING STUDY OF THE ARIES AND MURES RIVER SYSTEM

ANNEX TO WATER QUALITY MODELLING REPORT

By

Professor Paul Whitehead, University of Reading

and

Professor Steve Chapra, Tufts University

OCTOBER 2007

Executive summary

This Annex to the main report on water quality modelling by Whitehead (2007) has been written to answer the questions raised by the Hungarian team at the joint Hungarian-Romanian Meeting in July 2007. At that meeting a series of issues were raised concerning the water quality modelling and the Hungarian team showed some modelling results using their own dispersion model. In this report the set of questions raised have been answered illustrating the results of the earlier INCA modelling study. Also some analysis has been undertaken of the high concentrations predicted by the Hungarian modelling study, which was of concern to the Romanian team. The Hungarian team also requested a new water quality study using the dispersion model approach. This has been undertaken using a model developed by Professor Steve Chapra. This model solves the conventional dispersion equations using numerical integration routines but also builds in the dilution effects from incoming tributaries and the pollution degradation effects. Also, the Hungarian team were keen to explore the low flow conditions and the effects of different spill duration times on river pollution in order to fully understand and assess the worse case conditions.

In this report, the new dispersion model is described in considerable detail together with the numerical solution techniques used to solve the equations. The model has been set up for the Aries and Mures River System from the location of the proposed dam in the upstream Corna Catchment down to the border at Nadlac. The results indicate that in both high and low flow conditions a Baia Mare type rainfall or snowmelt event will not produce a severe pollution impact. This is primarily because of the dam spillway which is designed to release water in a controlled manner during high rainfall conditions.

More problematic is the effects of a catastrophic dam break which would release all the water and pollutant content of the dam storage reservoir. The risks of this happening are a question of geology and dam design and the risk assessment is beyond the scope of this report. However, the impact of such an eventuality occurring is considered in this report. The effect of the discharge under low flow conditions is to generate low concentrations under most releases and all are below the CN standards except for the very largest volumes. This is because of the reduced dilution downstream in low flow conditions as well as the significant dispersion and the large residence times. Dilution is also having a major effect in high flow conditions creating relatively low concentrations, although the reduced travel times suggests that year 17 high volume dam releases do create higher pollution levels.

The question of the duration of the spill release has produced some very interesting results. At first sight the effects of reducing the spill duration say from 24 hours to 3 hours should be significant because for the same mass of discharge the spill flow rates have to be much higher. Whilst this will have some effect in the upper reaches of the river, the additional impacts downstream at the border are minimal. This is because attenuation of the flood wave occurs and after about 100 km of river the pollution and flow pulse has mitigated so that it is indistinguishable from longer duration spills.

TABLE of CONTENTS

1.INTRODUCTION

Following the meeting with the Hungarian delegation in Bucharest in July, 2007, it was agreed to address the key areas of concern. These consisted of a set of questions posed by the Hungarian team at the time of the meeting and these are answered in section 2 below. The second area of concern was the very high values obtained by the Hungarian Dispersion modelling and this is addressed in section 3. Finally, in a new set of requests after the meeting, the Hungarian team requested a modelling exercise using a dispersion equation model. A new version of the dispersion equation model has now been developed by Professor Steve Chapra which incorporates dispersion behaviour, the dilution effects of incoming streams and also the chemical decay processes. This dispersion equation model is solved numerically and a set of scenarios undertaken to address the pollution issues.

2. ANSWERS TO HUNGARIAN QUESTIONS RAISED AT THE JULY MEETING

Question 1- lack of INCA model calibration

Answer
The INCA model has been calibrated against the flow and water quality data for the upper catchments and the main river. The observed water quality data were obtained from the long term sampling programme conducted by RMGC and from the local water authorities. Tables of model results compared to observed data are given in the report, as well as plots of detailed hydrological time series.

With regard to Cyanide there is, of course, no data to compare for the Mures River as there are no cyanide releases along the river. However, the model has been set up for the Baia Mare event and it is shown that the model can reproduce the cyanide concentrations that occurred during this pollution incident. This shows that the model does reproduce the mixing, dilution and decay processes in the river.

There is a large literature on Acid Rock Drainage (see list of papers in the modelling report) and there is worldwide consensus about the rapid loss of metals from streams downstream of mines as aeration processes rapidly oxidise the metals. The rate coefficients used in the model are consistent with this understanding.

It is very important to incorporate all the factors affecting water quality in rivers. These include mixing or dispersion processes, dilution effects from inflowing downstream water and also the chemical decay effects. It is the combination of all of these that determine downstream concentrations.

Question 2- The model is not fully described

Answer
The INCA model is fully described in the modelling report (see pages 10-21 and pages 37-53). The INCA papers referred to the main report by Whitehead et al (1998) and Wade et al (2002) give further descriptions of the model and the application to similar large catchment systems. The Dispersion Model used for the Monte Carlo analysis is fully described by in the report and is drawn from the standard water quality modelling textbook by Professor Steve Chapra (Chapra, 1997). The estimation of dispersion coefficients is difficult but it would be impossible to conduct an experiment along the Aries and Mures River system in order to

directly measure these. The size of the rivers and the residence times would mean that only a radioactive tracer would produce accurate results and this would be environmentally unacceptable. Further discussion of this issue is considered in section 5 of this Annex. Without direct measurements dispersion coefficients are always estimated using empirical formula. The empirical formula developed and tested by Professor Roger Falconer and his team at Cardiff University is probably the best relationship to use and is published in a refereed international journal. However, in section 4 of this annex an alternative method based on the classical method of Fisher is used. The inevitable uncertainty in estimating dispersion coefficients is taken into account by using the Monte Carlo approach and specifying a range of dispersion coefficients. The subsequent 5000 simulations enabled the complete range of likely dispersion coefficients to be explored. This is well beyond what most dispersion modelling studies undertake to reflect the difficulty of estimating dispersion coefficients.

Question 3- details of all models are required

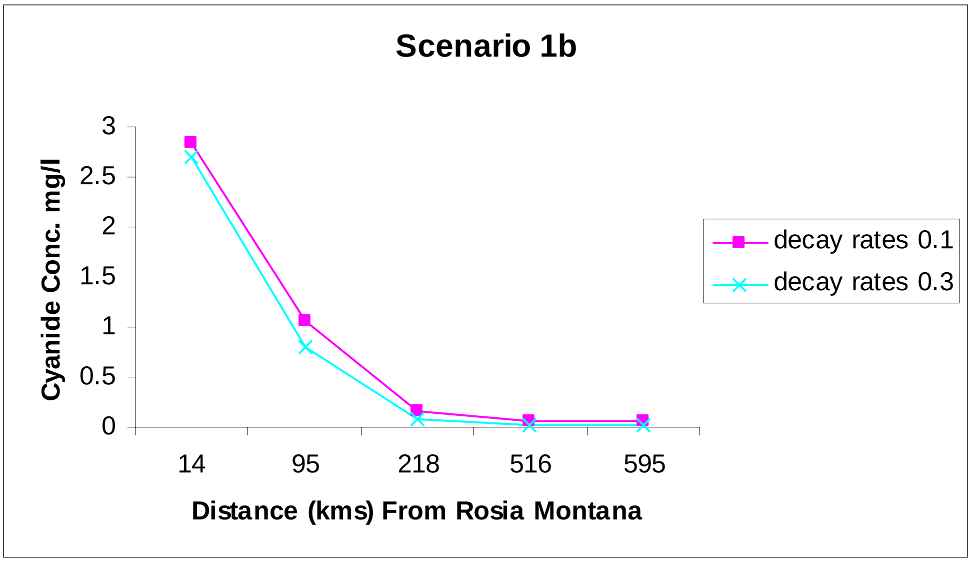
Answer
All these details are given in the report and attached papers. Further details of the dispersion modelling work is given in section 3 below

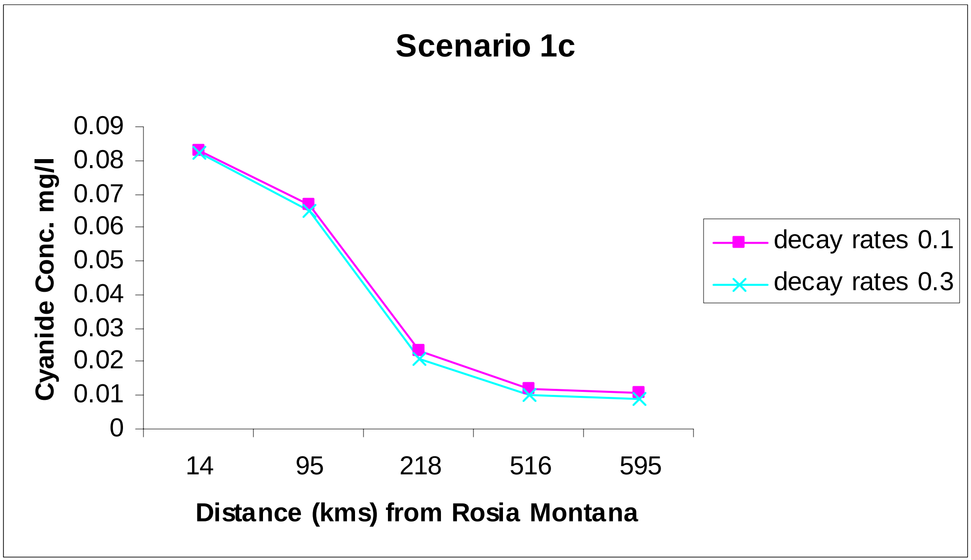
Question 4- details of each model scenario run is required

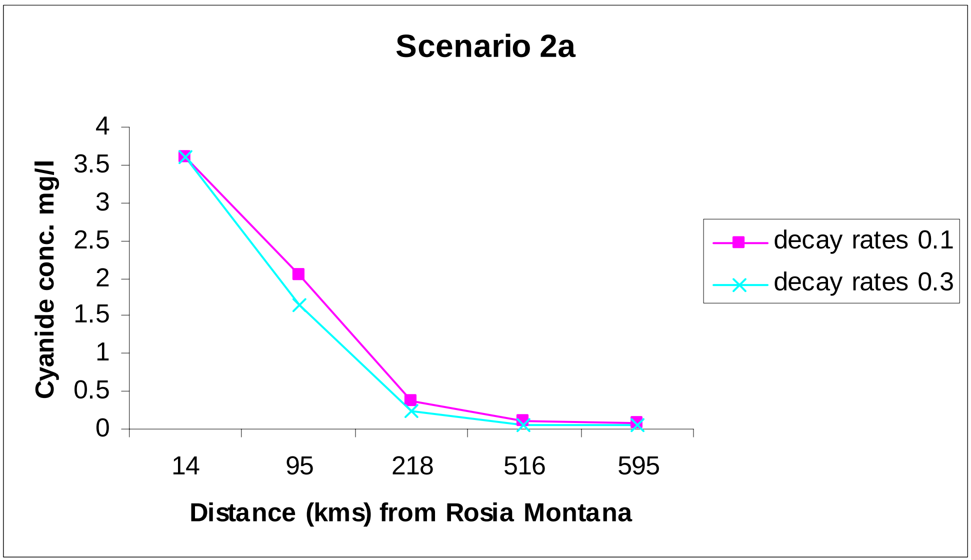
Answer
The report gives a list of the released flow under each scenario and this together with the dam concentrations defines the inputs to the models. Typically the m_p value in the dispersion equation Monte Carlo analysis is of the order of 800 g m^{-2} reflecting the high loads from the Dam failure. Whilst the loads may be high, the CN concentrations in the Dam are fairly low (see Tables in the main report). These low concentrations of cyanide from the dam are crucial to understanding the lower concentrations down the river, as the Dam concentrations are subject to a combination of dispersion processes down the river, then dilution from all the inflowing tributaries and, in addition, chemical decay. This issue is revisited in section 5 where further information on dispersion modelling is presented.

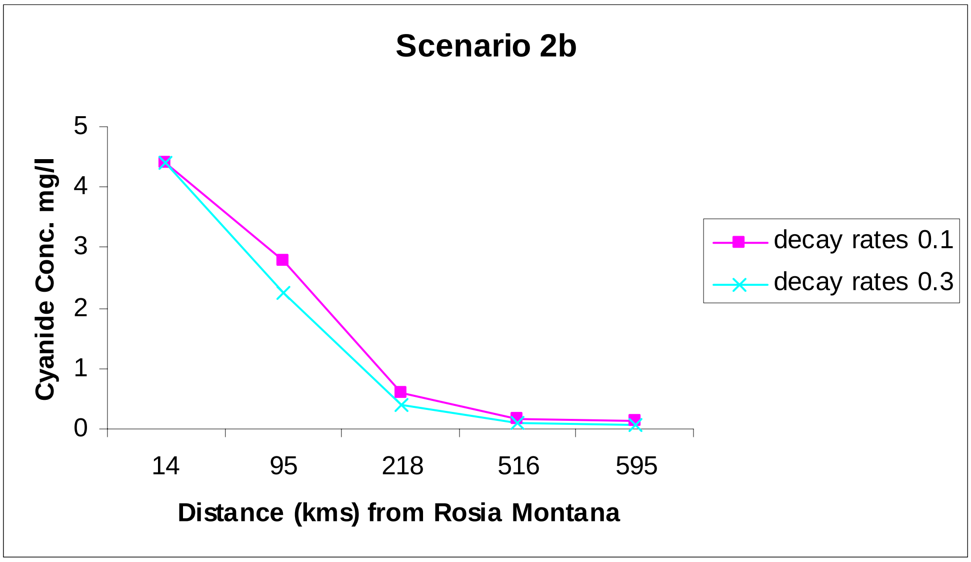
Question 5 – details of the longitudinal plots for all scenarios

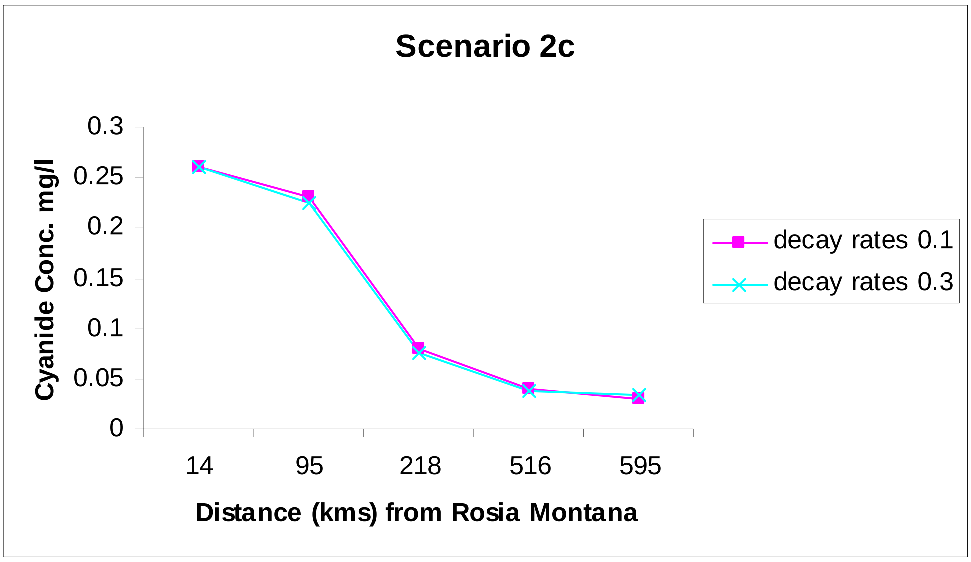
Answer
Here are the plots for the additional scenarios .


Scenario 1b
Cyanide Conc. mg/l
3
2.5
2
1.5
1
0.5
0
14
95
218
516
595
Distance (kms) From Rosia Montana
decay rates 0.1
decay rates 0.3


Scenario 1c
Cyanide Conc. mg/l
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
14
95
218
516
595
Distance (kms) from Rosia Montana
decay rates 0.1
decay rates 0.3


Scenario 2a
Cyanide conc. mg/l
4
3.5
3
2.5
2
1.5
1
0.5
0
14
95
218
516
595
Distance (kms) from Rosia Montana
decay rates 0.1
decay rates 0.3





Figures 1-6 Simulated CN concentration profiles along the river system using the INCA Model for a range of scenarios listed in the main report

Question 6 – undertake a low flow scenario results

The EIA considered the worst case scenarios. With a twice probable maximum rainfall event followed by a major flood event the river would not be in a low flow condition and hence this scenario was not considered. However given the concern, it has been agreed to undertake a low flow scenario and this is investigated using the dispersion model is section 5 below.

3 THE HUNGARIAN MODELLING STUDY

At the meeting in July the dispersion modelling work by the Hungarian team was presented. This indicated very high concentrations in the river immediately downstream of the Dam, of the order of 27mg/l and above (Jolánkai, 2007). These levels are far too high as the concentrations in the Dam as specified in the EIA report and the water quality modelling report by Whitehead (2007) are of the order of 4 mg/l. It is simply impossible for the concentrations to increase downstream of the Dam due to the basic laws of chemistry. However, incorrect high concentrations can be generated by dispersion models if it is assumed that the total mass enters the river instantaneously or over a very short space of time. This effect can be demonstrated using the basic dispersion model as specified in the water quality modelling report and as used by Professor Jolánkai. Figure 7 shows the CN concentrations along the river given a 29.4 tonne release of cyanide from the Dam. This is equivalent to scenario 2b with 5880800 cubic metres of water with a concentration of 5 mg/l. The velocity and dispersion coefficient are 1 m/sec and 82 m^2/sec, respectively, for the simulations and in Figure 7, three results are shown for decay rates of 0, 0.1 and 0.3 $days^{-1}$. The simulations show high initial concentrations, much higher than the Dam and this is an artefact of the model due to the assumption of instantaneous discharge.

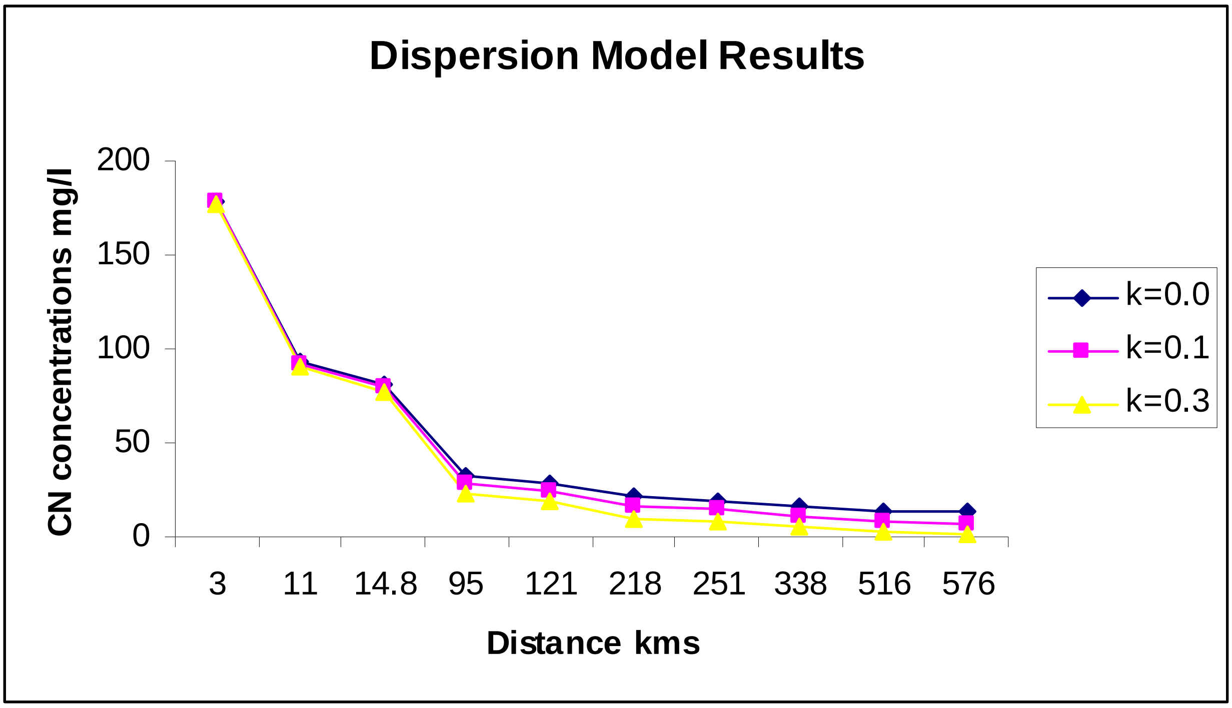


Figure 7 High Simulated CN Concentrations Erroneously Generated by the Dispersion Model using an instananeous discharge initial condition.

4 A NEW DISPERSION MODEL TO SIMULATE POLLUTANT SPILLS.

A new model has been developed to numerically simulate the transport and fate of a spill of a contaminant in a river. The model is based on the classical dispersion equation as requested by the Hungarian team but also incorporates the dilution effects of tributaries joining the main river as well as any chemical decay processes occuring in the river system. The model assumes that lateral and vertical gradients are minimal and that the contaminant can decay with first-order kinetics.

Segmentation

In order to derive a numerical solution, the river is divided into a series of reaches, as shown in Figure 8. These reaches represent river segments that have constant hydrogeometric characteristics. The reaches themselves are further divided into a series of equal-length computational elements. The elements represent the fundamental units for which water and mass balances are written and solved.

.

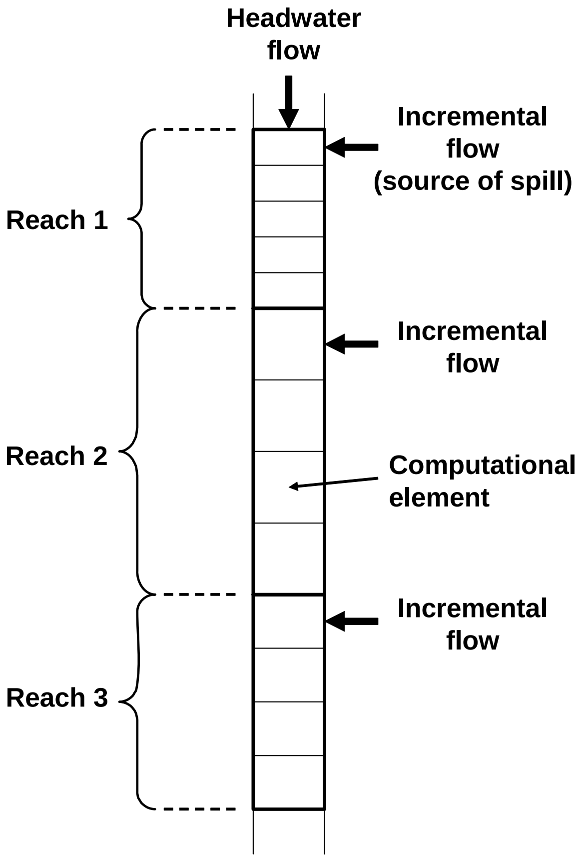


Figure 8 Spill model segmentation scheme showing reaches divided into equal-sized computational elements. In addition, the system's external flows are depicted.

In summary, the nomenclature used to describe the way in which the spill model organizes river topology is as follows:

- Reach. A length of river with constant hydraulic characteristics.
- Element. The model's fundamental computational unit which consists of an equal length subdivision of a reach.

Initial Flow Balance

A steady-state flow balance is implemented for each model element. For the first element in a reach, the budget is written as (see Figure 9)

$$Q_i = Q_{i-1} + Q_{\text{in},i} \quad \textbf{(1)}$$

where Q_i = outflow from element i into the downstream element i + 1 (m^3/s), Q_{i-1} = inflow from the upstream element i – 1 (m^3/s), and $Q_{\text{in},i}$ is the incremental inflow into the element from point and nonpoint sources along the reach's length (m^3/s). Thus, the downstream outflow of the first element is simply the sum of the inflow from upstream and the incremental flow. For the reach's other elements, $Q_{\text{in},i}$ = 0 and, therefore, outflow equals inflow: $Q_i = Q_{i-1}$.

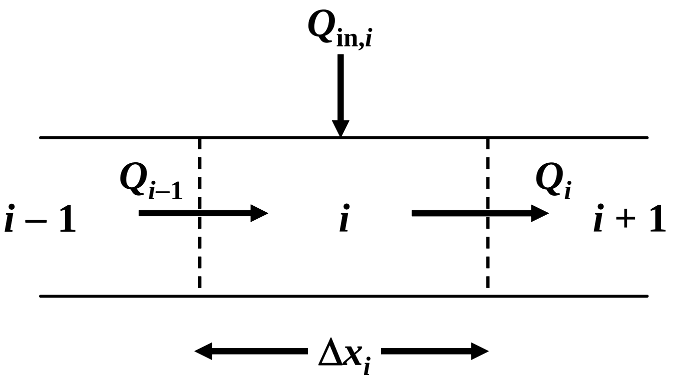


Figure 9 Flow balance for the first element in a reach.

Depth, Velocity and Other Hydraulic Parameters

Once the outflow for each element is computed, the depth, H_i (m), and velocity, U_i (m/s), are calculated in one of two ways: rating curves and the Manning equation.

Rating curves

Rating curves in the form of power equations are used to relate mean velocity and depth to flow for each element,

$$U_i = aQ_i^b \quad \textbf{(2)}$$

$$H_i = \alpha Q_i^\beta \quad \textbf{(3)}$$

where U_i = the mean velocity across the downstream interface of element i (m/s), H_i = the average depth of element i (m), and a, b, α and β are empirical coefficients that are determined from velocity-discharge and stage-discharge rating curves, respectively. Note that the sum of b and β must be less than or equal to 1. If this is not the case, the width will decrease with increasing flow. If their sum equals 1, the channel is rectangular.

After the velocity and depth of an element are computed with Eqs. (2) and (3), they can be used to compute other required hydrogeometric characteristics. For example, the velocity can be substituted into the continuity equation ($Q_i = U_i A_{c,i}$) to determine the element's cross-sectional area (m^2),

$$A_{c,i} = \frac{Q_i}{U_i} \quad \textbf{(4)}$$

The area can be directly related to flow by substituting Eq. (2) into Eq. (4) to give

$$A_{c,i} = \frac{Q_i}{aQ_i^b} = \frac{1}{a}Q_i^{1-b} \quad \textbf{(5)}$$

The mean width (m), wetted perimeter (m), and the volume (m³) follow

$$B_i = \frac{A_{c,i}}{H_i} \tag{6}$$

$$P_i = B_i + 2H_i \tag{7}$$

$$V_i = B_i H_i \Delta x_i \tag{8}$$

where Δx_i = the element length (m).

Besides computing the hydrogeometric characteristics as a function of flow, the rating curves can also be employed to perform the inverse calculation. That is, given volume, they can also be used to compute flow, depth, velocity, area, width, and wetted perimeter. Because Δx is known, we first determine the cross-sectional area as

$$A_{c,i} = \frac{V_i}{\Delta x_i} \tag{9}$$

Flow can then be evaluated by solving Eq. (5) for

$$Q_i = a^{1/(1-b)} A_c^{1/(1-b)} \tag{10}$$

Once flow is known, Eqs. (2), (3), (6) and (7) can be then employed to compute U_i, H_i, B_i, and P_i.

Manning Equation

Each element in a particular reach is idealized as a trapezoidal channel (Figure 10). For such channels, the Manning equation can be used to express the relationship between flow and depth as

$$Q_i = \frac{S_{0,i}^{1/2}}{n_i} \frac{A_{c,i}^{5/3}}{P_i^{2/3}} \tag{11}$$

where $S_{0,i}$ = bottom slope (m/m), n_i = the Manning roughness coefficient, $A_{c,i}$ = the cross-sectional area (m^2), and P_i = the wetted perimeter (m).

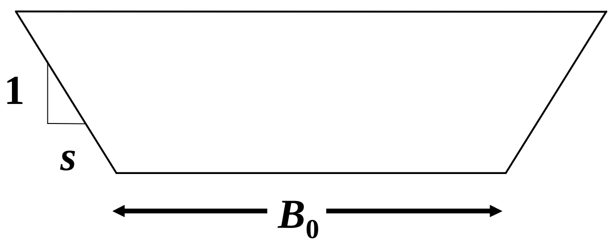


Figure 10 A cross section of a trapezoidal channel showing the parameters needed to uniquely define the geometry: B_0 = bottom width, s = side slope.

The cross-sectional area and wetted perimeter are computed as

$$A_{c,i} = \left(B_{0,i} + s_i H_i\right) H_i \tag{12}$$

$$P_i = B_{0,i} + 2H_i\sqrt{s_i^2 + 1} \tag{13}$$

where $B_{0,i}$ = bottom width (m), and s_i = the side slope as shown in Figure 10 (m/m). Substituting Eqs. (12) and (13) into (11) gives

$$Q_i = \frac{1}{n_i}\frac{\left[\left(B_{0,i} + s_i H_i\right)H_i\right]^{5/3}}{\left[B_{0,i} + 2H_i\sqrt{s_i^2 + 1}\right]^{2/3}} S_{0,i}^{1/2} \tag{14}$$

Given values for Q, B_0, S_0, n and s, Eq. (14) is a nonlinear equation with one unknown (H) which can be reformulated as

$$f(H_i) = \frac{1}{n_i}\frac{\left[\left(B_{0,i} + s_i H_i\right)H_i\right]^{5/3}}{\left[B_{0,i} + 2H_i\sqrt{s_i^2 + 1}\right]^{2/3}} S_{0,i}^{1/2} - Q_i \tag{15}$$

The root (i.e., the value of depth that makes this equation zero) is the reach depth. It can be shown that the root can be determined efficiently by successive substitution (Chapra and Canale 2006) using the following iterative formula,

$$H_{i,k} = \frac{(Q_i n_i)^{3/5}\left(B_{0,i} + 2H_{i,k-1}\sqrt{s_i^2 + 1}\right)^{2/5}}{S_{0,i}^{3/10}\left[B_{0,i} + s_i H_{i,k-1}\right]} \tag{16}$$

where k = 1, 2, …, n, where n = the number of iterations. If an initial guess of $H_{i,0}$ = 0 is employed, this approach is rapidly convergent for all natural channels (Chapra and Canale 2006). The method is terminated when the estimated error falls below a specified value of 0.001%. The estimated error is calculated as

$$\varepsilon_{a,i} = \left|\frac{H_{i,k+1} - H_{i,k}}{H_{i,k+1}}\right| \times 100\% \tag{17}$$

Once the depth is known, the cross-sectional area and wetted perimeter are computed with Eqs. (12) and (13), and the velocity can be determined from the continuity equation,

$$U_i = \frac{Q_i}{A_{c,i}} \tag{18}$$

The average element width, B_i (m), is then computed as

$$B_i = \frac{A_{c,i}}{H_i} \tag{19}$$

the top width, $B_{1,i}$ (m), as

$$B_{1,i} = B_{0,i} + 2s_i H_i \tag{20}$$

and the element volume as

$$V_i = B_i H_i \Delta x_i \tag{21}$$

As was the case with the rating curves, the Manning approach can also be employed to perform the inverse calculation. If the volume is given, the cross-sectional area can be generated with Eq. (9). The depth is determined by reformulating Eq. (12) as a quadratic,

$$s_i H_i^2 + B_{0,i} H_i - A_{c,i} = 0 \tag{22}$$

The positive root of this equation yields the depth[1]

$$H_i = \frac{2A_{c,i}}{B_{0,i} + \sqrt{B_{0,i}^2 + 4s_i A_{c,i}}} \tag{23}$$

The average width and flow are computed with Eqs. (19) and (14), respectively, and the velocity then follows from Eq. (18).

Dynamic Water Balance

After the initial volumes are determined, the software generates a numerical solution of the one-dimensional continuity equation,

$$\frac{\partial A_c}{\partial t} = -\frac{\partial Q}{\partial x} \tag{24}$$

Equation (24) can be expressed in numerical form by writing a water balance around each element to give

$$\frac{dV_i}{dt} = Q_{i-1} + Q_{i,\text{in}} - Q_i \tag{25}$$

where Q_i is the outflow which is computed as a function of volume as described in the previous section. Equation (25) is solved numerically as will be described in a subsequent section.

Dispersion

Dispersion can either be user-prescribed or computed. In the latter case, based on Rutherford's (1994) assessment, three empirically-derived equations are available to compute the longitudinal dispersion for the downstream boundary between two elements.

[1] This version of the quadratic formula prevents division by zero for rectangular channels (i.e., with $s_i = 0$).

Fischer et al. (1979):

$$E_{p,i} = 0.011 \frac{U_i^2 B_i^2}{H_i U_i^*} \quad \textbf{(26)}$$

where $E_{p,i}$ = the longitudinal dispersion between elements i and $i + 1$ (m^2/s), and U_i = mean velocity of element i (m/s), B_i = mean width (m), H_i = depth (m), and U_i^* = shear velocity (m/s), which is related to more fundamental characteristics by

$$U_i^* = \sqrt{g H_i S_{0,i}} \quad \textbf{(27)}$$

where g = acceleration due to gravity (= 9.81 m/s^2) and $S_{0,i}$ = bottom slope (m/m).

Liu (1979):

$$E_{p,i} = 0.18 \left(\frac{U_i^*}{U_i} \right)^{1.5} \frac{Q_i^2}{U_i^* R_h^3} \quad \textbf{(28)}$$

where R_h = the hydraulic radius (m), which is equal to the ratio of the cross-sectional area to the wetted perimeter.

McQuivey and Keefer (1974):

$$E_{p,i} = 0.058 \frac{Q_i}{S_i B_i} \quad \textbf{(29)}$$

This formulation is limited to systems with Froude numbers $(F = U / \sqrt{gH})$ less than 0.5. If this constraint is exceeded, the software automatically displays an error message and terminates.

Mass Balance

The software generates a numerical solution of the one-dimensional advection-dispersion-reaction equation,

$$\frac{\partial c}{\partial t} = -\frac{\partial Uc}{\partial x} + \frac{\partial}{\partial x}\left(E \frac{\partial c}{\partial x} \right) - kc \quad \textbf{(30)}$$

where c = concentration (mg/L), t = time (s), U = velocity (m/s), x = distance (m), E = dispersion (m^2/s), and k = first-order decay rate (d^{-1}).

Equation (30) can be expressed in numerical form by writing a mass balance around each element, as shown in Figure 11. In order to account for the nonuniformity, as well as to conserve mass, the fluxes between elements are specified at their upstream and downstream faces to give,

$$\frac{\partial M_i}{\partial t} = \left(UA_c c - EA_c \frac{\partial c}{\partial x}\right)_{i-1,i} - \left(UA_c c - EA_c \frac{\partial c}{\partial x}\right)_{i,i+1} - kV_i c_i \qquad \textbf{(31)}$$

where M_i = the mass of pollutant in element i (g) = $V_i c_i$.

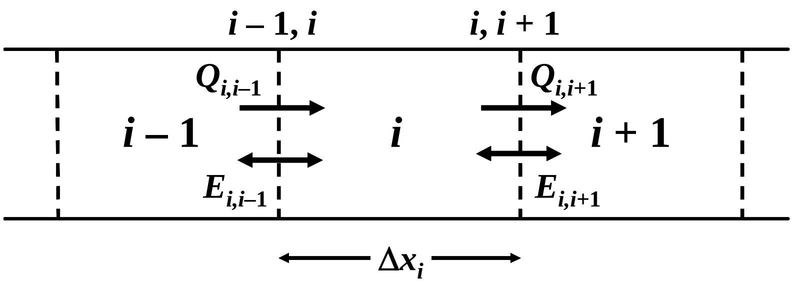


Figure 11 One-dimensional channel divided into a series of elements.

Assuming that the concentrations at each interface are equal to the upstream element (i.e., a backward or "upstream" difference), and using centered differences for the gradients yields,

$$\frac{dM_i}{dt} = W(t) + Q_{i-1,i}c_{i-1} - Q_{i,i+1}c_i + E_{i-1,i}A_{c,i-1,i}\frac{c_i - c_{i-1}}{\Delta x_{i-1,i}} - E_{i,i+1}A_{c,i,i+1}\frac{c_{i+1} - c_i}{\Delta x_{i,i+1}} - kV_i c_i \qquad \textbf{(32)}$$

where $W(t)$ = mass loading rate (g/s), $Q_{j,k}$ = the flow from element j into element k (m^3/s), $E_{j,k}$ = the dispersion between elements j and k (m^2/s), and $\Delta x_{j,k}$ = the length between the mid-points of elements j and k (m), where

$$\Delta x_{j,k} = \frac{\Delta x_j + \Delta x_k}{2} \qquad \textbf{(33)}$$

where Δx_i = the length of element i (m). This equation can then be written for all the elements and integrated numerically to obtain the solution.

Solution Method

Equation (25) and (32) are solved numerically with Euler's method as follows:

Step 1: Determine and save the initial values for all elements,
Step 2: Compute derivatives with Eqs. (25) and (33).
Step 3: Compute new volumes and masses with Euler's method:

$$V_i(t+\Delta t) = V_i(t) + \frac{dV_i(t)}{dt}\Delta t$$
$$M_i(t+\Delta t) = M_i(t) + \frac{dM_i(t)}{dt}\Delta t$$

Step 4: Compute new outflows for each element as a function of their new volumes.
Step 5: Compute other hydraulic parameters.
Step 6: Compute new concentrations: $c_i = M_i/V_i$

Step 6: Increment time: $t = t + \Delta t$.
Step 7: Save new values.
Step 8: If $t \geq$ final time, exit to step 10
Step 9: Loop back to Step 2
Step 10: Display results.

In the absence of numerical dispersion, the foregoing hydrualic solution is similar to the kinematic wave. However, because of the use of first-order forward time differencing and backward space differencing, it does exhibit numerical dispersion and hence is more akin to a diffusive wave solution. Techniques such as the Muskingum-Cunge method attempt to mitigate such effects by judiciously choosing the solution time step so that the numerical dispersion approximates the actual diffusion exhibited by waves subject to gravity effects.

In a similar fashion, the mass solution also generates additional numerical dispersion. As with the hydraulics, a time step can be chosen in an attempt to match the numerical dispersion to the actual dispersion.

Unfortunately, different time steps are needed for the hydraulic and mass solutions. Further, because the system being studied has a wide range of flows and velocities, the optimal time step will vary greatly spatially The following scheme attempts to minimize the impact of both effects while using a single time step.

For the mass solution, the total dispersion generated will consist of the model dispersion, E_i, along with some additional numerical dispersion, $E_{n,i}$. Thus, because we would like the solution to have the correct physical dispersion (i.e., either user specified or computed with Eqs. 26 through 29), $E_{p,i}$, we desire that

$$E_{p,i} = E_i + E_{n,i} \quad \textbf{(34)}$$

A Taylor series expansion (Chapra 1997) can be used to relate the numerical dispersion to the space and time steps as

$$E_{n,i} = 0.5U_i \Delta x_i - 0.5U_i^2 \Delta t \quad \textbf{(35)}$$

Substituting Eq. (35) into (34) and rearranging yields

$$E_i = E_{p,i} - 0.5U_i \Delta x_i + 0.5U_i^2 \Delta t \quad \textbf{(36)}$$

Therefore, to achieve accuracy, the dispersion used in the model, E_i, is automatically set equal to the desired dispersion. $E_{p,i}$, minus the numerical dispersion, $E_{n,i}$.

There are two stability constraints. First, a spatial positivity constraint can be formulated as

$$\Delta x_i < \frac{2E_i}{U_i} \quad \textbf{(37)}$$

This constraint guarantees positive solutions.

In addition, the time step is constrained according to

$$\Delta t < \frac{\Delta x_i^2}{U_i \Delta x_i + 2E_i + k\Delta x_i^2} \quad \textbf{(38)}$$

where the right-hand side is the element's residence time (s). This is the analog of the Courant condition for Eq. (32). These criteria can be used to develop a solution procedure that maximizes accuracy and guarantees stability as described next.

First, the user specifies the maximum desired size of the element length for each reach. Then, Eq. (37) is used to determine the maximum permissible size based on the velocity and the dispersion; i.e., using $E_i = E_{p,i}$. If the desired size is greater than the permissible size, the element length is set to the permissible size. Otherwise, the element length is set to the maximum desired size. The resulting element length is then divided into the reach length and the result rounded up in order to determine to number of elements for each reach.

Second, Eq. (38) is used to determine a maximum allowable time step for each reach. The minimum of these time steps is then taken as the computational time step for the entire system.

Finally, this time step along with the element size is substituted into Eq. (35) to compute the numerical dispersion. If it is less than the physical dispersion, Eq. (36) is used to compute the dispersion coefficient that should be input to the model.

5 APPLICATION OF THE DISPERSION MODEL TO THE ARIES-MURES RIVER SYSTEM

The dispersion model developed by Professor Chapra has been applied to simulate a range of worst case scenarios. There are 2 extremes of weather that can create worst case conditions, these being, firstly, an incident associated with high rainfall or snowmelt event in the mountains area of Rosia Montana during a low flow condition downstream or, secondly, a high rainfall or snow event when the rivers are in flood conditions. In the first low flow case, dilution is low but the travel time is relatively high due to the low water velocity, whereas in the second case the high flows generate significant dilution but velocities are fast with low travel times along the river. There are also two types of incident. Firstly there is a Baia Mare type event when a high rainfall significantly raises the water level in the dam and an overflow release occurs. The second type is accociated with a significant failure of the Dam wall with a sudden release of polluted water. Both these types of incident are considered here.

5.1 River Model Set Up

Much of the detailed characteristics of the river system have already been described in the main report by Whitehead (2007). Essentially the dispersion model has been set up in the same manner as the INCA model from the location of the proposed Dam in the Corna catchment above Abrud down to the Aries River and then into the Mures River down to the border at Nadlac. Information on elevations, slopes, locations of reaches etc are given in the main report. However, Table 1 gives additional information on velocity and depth conditions in the rivers under high, medium and low flow conditions and Table 2 summarises the low flow (95th percentile), high flow (5th percentile) conditions that have been used in the model runs.

RIVER	**Aries**	**Aries**	**Mures**	**Mures**	**Mures**
LOCATION	**Campeni**	**Baia de Aries**	**Ludus**	**Alba Iulia**	**Branisca**
Flow Condition	Depth m	Depth m	Depth m	Depth m	Depth m
high flows	2.72	3.53	5.36	3.00	4.70
med flows	0.50	0.76	1.13	1.25	1.55
low flows	0.23	0.38	0.37	0.60	0.43
	Velocity m/sec	Velocity m/sec	Velocity m/sec	Velocity m/sec	Velocity m/sec
high flows	2.69	2.75	1.20	1.65	1.86
med flows	1.72	1.58	0.98	1.20	1.25
low flows	0.37	0.21	0.18	0.37	0.48

Table 1 Depth and Velocity Information at a range of locations and under a range of flow conditions.

River	**Reach Location**	**Q m^3/sec 95th percentile**	**Q m^3/sec 5th percentile**
Abrud	Campeni	0.06	164
Aries	Campeni	0.96	320
Aries	Baia de Aries	1.44	455
Aries	Buru	1.64	610
Aries	Turda	1.66	640
Mures	Ludus	4.35	1020
Mures	Alba Iulia	9.5	1404
Mures	Gelmar	15.8	1436
Mures	Branisca	16.4	1458
Mures	Savarsin	17.1	1431
Mures	Nadlac	20.5	1404

Table 2 Low and High Flow conditions for the Abrud, Aries and Mures Rivers

Velocity calculation

A key issue of concern is whether to use the observed depth and velocity information provided in Table 1 to estimate travel times within the model or to use a formula such as the Manning equation. The problem with the observed data is that it is only available at a limited number of sites along the river. The river systems of the Abrud, the Aries and the Mures are complex natural rivers with highly variable geometry and highly variable flow conditions. The collection of field data to provide the necessary level of detail, over a range of flow conditions and at all sites of interest over 550 kilometres of river is a major task. However, we have used the available depth and velocity data to create rating equations and used these in the model to simulate pollutant transport. In addition we have built into the model the conventional Manning equation, such that it is can be applied at every location and under any flow condition. We have compared the concentrations simulated using the two different velocity estimation techniques. Table 3 shows the simulated concentrations of CN WAD (Weak Acid Dissociable) using the two approaches. The results are, in fact, very similar and it is considered that both approaches are acceptable for these river systems.

location	**Time days**	**Manning equation Peak CN concs mg/l**	**Rating equation Peak CN concs mg/l**
Abrud	0.11759	0.33725	0.33472
Campeni	1.00815	0.17961	0.17934
Baia de Aries	1.03241	0.12778	0.12771
Turda	1.15444	0.09160	0.09152
Ocna Mures	1.32426	0.05789	0.05786
Albalulia	1.82537	0.04225	0.04225
Deva	2.46593	0.04132	0.04132
Savirsin	3.39611	0.04070	0.04070
Arad	3.72407	0.04014	0.04014
Nadlac	4.00315	0.03961	0.03961

Table 3 A comparison of simulated CN concentration estimates using both Rating equations and Mannings equations for high flow conditions.

Dispersion Coefficient Calculation

Another similar issue is how to estimate the dispersion coefficients in the river systems. A series of tracer experiments to estimate these for all three rivers over a total of 30 reaches under a full range of flow conditions is a major undertaking, requiring considerable time to complete. Probably many years to cover the full range of flow conditions required. Also, there are significant practical difficulties. For example, the only tracers that could be used effectively to give adequate dilution is either Radioactive Tracers or flourescent tracers such as Rhodamine WT. These can be measured down to microgram/litre concentrations. However, in many European countries (eg UK) the Rhodamine tracer has been banned because of water colour problems (the tracer dyes the water bright red) and, also, there are potential health hazard problems associated with Rhodamine. Similarly there are significant health problems with radioactive tracers. Another tracer that is often used is Potassium Iodide (Whitehead et al, 1986), whch can also be measued to low concentrations. However this tracer is often adsorbed onto fine sediments. Hence, iodide can be lost to the bed sediments or suspended sediments and this tracer will give inaccurate results for long tracer experiments. Hence, on the Mures River System there is no practical alternative to using a mathematical approach to estimate dispersion coefficients. Essentially dispersion is changing all the time along a river as flows change, as slopes and geometry change and as velocity changes. Thus dispersion coefficients really need to be calculated at every time step. As described in section 4 above, the model developed by Chapra uses the Fisher Method to estimate Dispersion Coefficients (Chapra, 1997). This approach is used to provide the best estimate of dispersion coefficient at all times, all velocities and all locations.

5.2 SCENARIO ANALYSIS- SIMULATING THE BAIA MARE TYPE EVENT

The reasoning and set up of the Baia Mare Scenario is given on pages 62 and 63 of the main report. Essentially with a Baia Mare type rainfall event the spillway on the Corna Valley Dam would come into operation and a 1 in 10 year flood spillway flow of 2.3 m^3/sec flow would occur with the set of concentrations shown in Table 4 for summer and winter conditions. Note it is assumed in the table and graphs below that the release of water via the spillway continues for a 12 hour period, a reasonable estimate of the response time and duration of the flow to a rainfall event. However, the model is set up so that any duration of release event can be simulated.

Tables 5 and 6 show the simulation results for a high rainfall event assuming the worse case condition of a release in year 17 when the concentration behind the Dam is at its highest. Tables 5 and 6 show the low flow conditions for events occuring in the summer and winter respectively. In both cases the peak CN WAD concentrations at the border are low although the summer simulation is lower due to the loss of CN, due to volatilisation and degradation, as discussed in the main report. The decay coefficient is set to 0.1 $days^{-1}$, a relatively low value. Essentially the long travel time of over 20 days in summer conditions provides sufficent time for the loss of CN along the river system. Table 7 shows the simulation in a winter high flow condition. The peak concentrations are particularly low because of the large dilution effect of incoming streams and tributaries. The model simuation results for the low flow and high flow simulations are shown in Figures 12 and 13.

Scenario assuming 1 in 10 year flood spillway flow of 2.3 m^3/sec	CN Total Concentration behind the dam (mg/l)
Summer – initial dam condition year 1	0.36
Summer – final dam conditions year 17	1.09
Winter – initial dam condition year 1	1.09
Winter – final dam conditions year 17	3.27

Table 4 The CN concentrations in the Dam during a Baia Mare Type Event.

Reach	Travel Time days	CN conc. mg/l
Abrud	0.501	0.888
Campeni	0.604	0.637
Baia de Aries	1.022	0.396
Turda	3.186	0.105
Ocna Mures	5.737	0.029
Albalulia	10.173	0.009
Deva	13.971	0.004
Savirsin	18.582	0.002
Arad	20.152	0.002
Nadlac	21.483	0.001

Table 5 Simulated Peak CN WAD concentrations assuming a Baia Mare type event under low flow summer conditions assuming a final Dam condition.

Reach	Travel Time days	CN conc. mg/l

Abrud	0.501	2.659
Campeni	0.606	1.952
Baia de Aries	1.037	1.272
Turda	3.332	0.422
Ocna Mures	5.947	0.150
Albalulia	10.472	0.077
Deva	14.329	0.053
Savirsin	18.999	0.041
Arad	20.588	0.036
Nadlac	21.933	0.033

Table 6 Simulated Peak CN WAD concentrations assuming a Baia Mare type event under low flow winter conditions assuming a final TMF condition.

Reach	Travel Time days	CN conc. mg/l
Abrud	0.108	0.045
Campeni	0.122	0.045
Baia de Aries	0.509	0.023
Turda	0.533	0.016
Ocna Mures	0.692	0.012
Albalulia	0.976	0.007
Deva	1.684	0.005
Savirsin	2.350	0.005
Arad	3.237	0.005
Nadlac	3.815	0.005

Table 7 Simulated Peak CN WAD concentrations assuming a Baia Mare type event under high flow (Q5) winter conditions assuming a final dam condition.

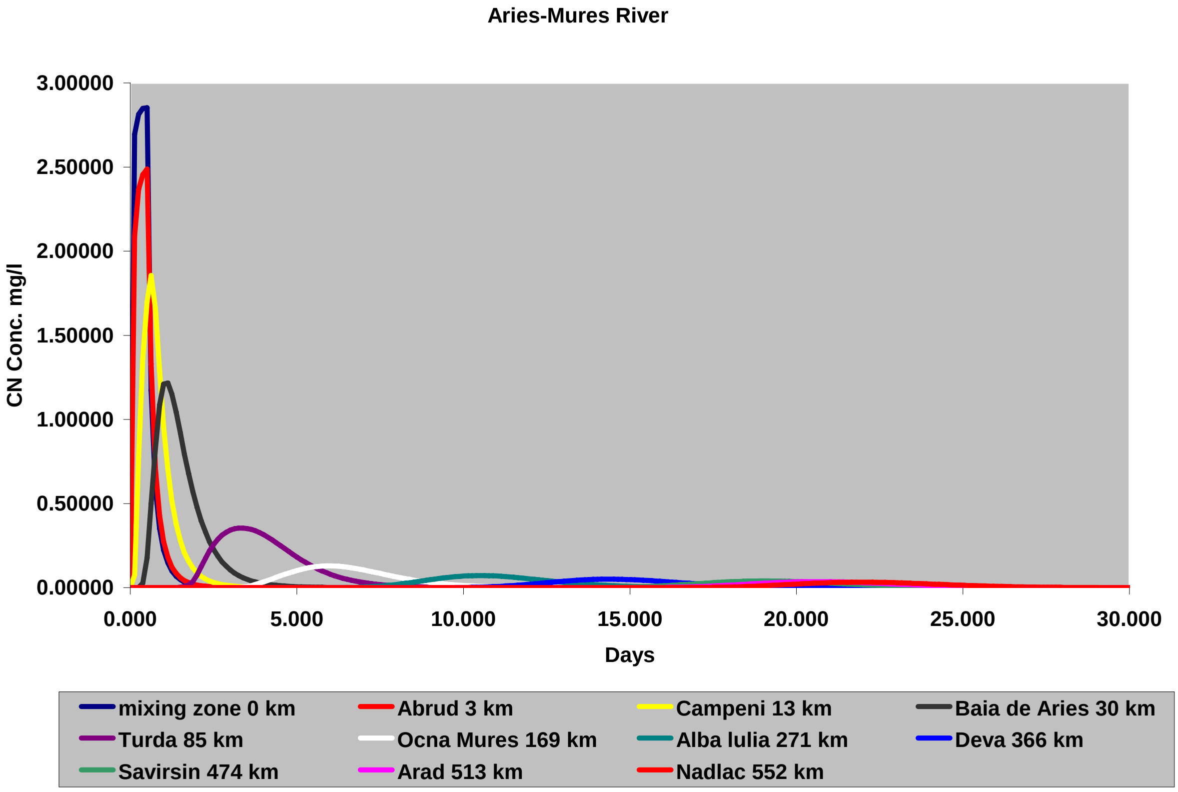


Figure 12 CN WAD Simulation of a Baia Mare Type Rainfall/Snow Event for the Mures River System under low flow winter conditions assuming a final TMF condition.

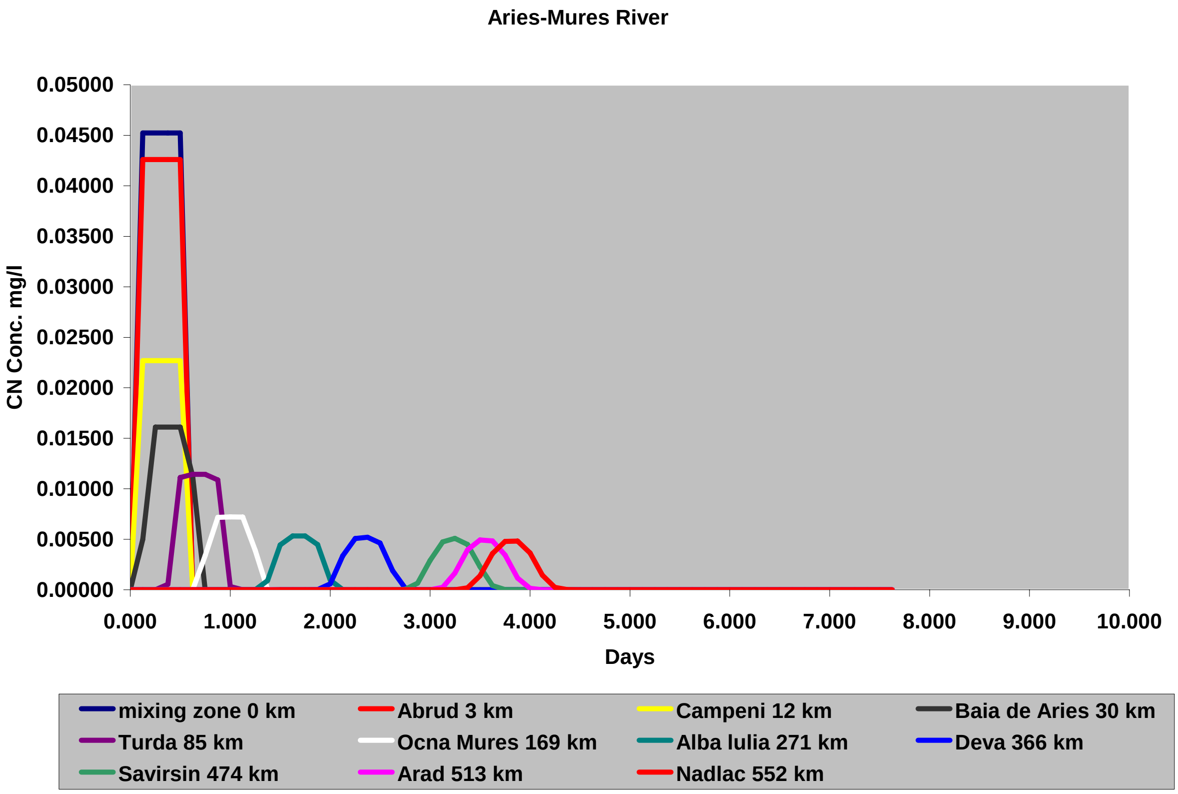


Figure 13 CN Simulation of a Baia Mare Type Rainfall/Snow Event for the Mures River System under high flow (Q5) winter conditions assuming a final dam condition.

It is clear from this analysis that a Baia Mare type rainfall and snow event will not produce the levels of pollution observed in the actual Baia Mare event. This is mainly because of the modern engineering design of a spillway that protects the Dam from a dangerous build up of water behind the Dam wall.

5.3 Dam Burst Scenarios

A key type of event to investigate in the impact study is that of a Dam Break which might occur releasing the water behind the Dam over a short period of time. This has been investigated using the new dispersion model. As in the case of the previous INCA study a range of flow and water quality condition has been investigated. Table 8 gives a summary of the scenarios investigated in this study. These have focused on the low flow conditions as requested by the Hungarian team, but some high flow runs have been undertaken to assess this type of incident. As part of the RMGC EIA, a study was undertaken to investigate the likelihood of a dam failure. These were considered by MWH, an environmental engineering consultancy, and are itemised in the EIA Report, Chapter 7, Risk Cases, Sections 6.4.3.1 and 6.4.3.2. MWH considered 2 sets of Dam Break scenarios with the first set of these representing start-up dam failure at the end of Year 1 and the second set assuming final dam failure at year 17. MWH also calculated the CN WAD concentration released in each scenario, and these are shown in Table 8 together with the maximum volumes of water released by the potential dam break. Again the scenarios considered in the study are both low and high flow conditions.

Scenario	**Pond CN WAD concentrations mg/l**	**Dam Failure Timing**	**Dam Release Volume m^3**	**River Discharge Conditions**
3a	4.1	Year 1	1078000	low
3b	4.1	Year 1	1078000	high
3c	4.4	Year 17	3811200	low
3d	4.4	Year 17	3811200	high
3e	5.0	Year 17	5880800	low
3f	5.0	Year 17	5880800	high

Table 8 The set of scenarios with a combination of Dam failures and river flow conditions

Scenarios 3a and 3b

The simulations for scenario 3a are illustrated in Figure 14 and Table 9 for a range of decay coefficients under low flow conditions. Thus under low flow conditions the peak CN concentrations at the border will be above the CN standard of 0.1 mg/l. Figure 14 shows the response along the river at key locations.

	Decay Coeff	**Decay Coeff**

	$0.1\ days^{-1}$	$0.2\ days^{-1}$
Reach	**Peak CN conc. mg/l**	**Peak CN conc. mg/**
Abrud	4.67	4.65
Campeni	4.34	4.26
Baia de Aries	4.07	3.88
Turda	2.96	2.48
Ocna Mures	1.35	0.94
Albalulia	0.49	0.23
Deva	0.23	0.07
Savirsin	0.11	0.02
Arad	0.09	0.02
Nadlac	0.07	0.01

Table 9 Simulation results assuming Scenario 3a under low flow conditions with a range of decay coefficients.

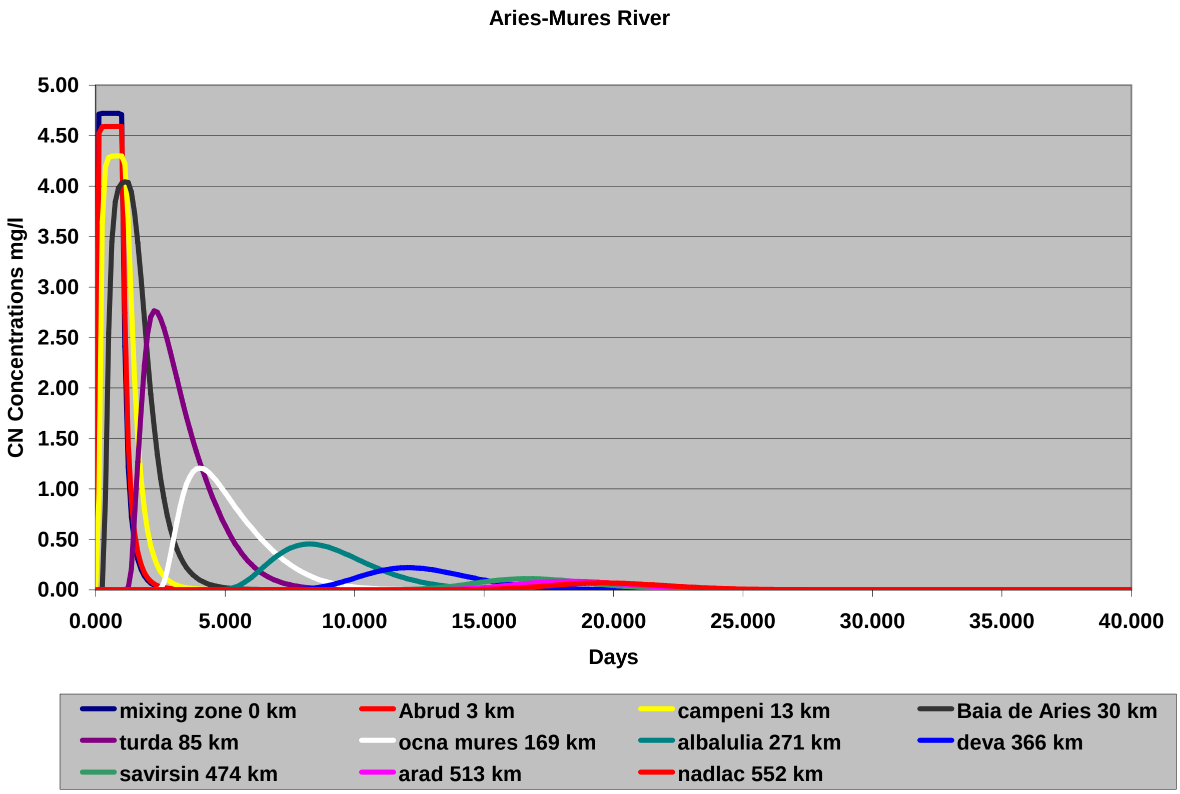


Figure 14 Simulation of Scenario 3a under low flow conditions assuming a decay of $0.1\ days^{-1}$

In the case of the 3b scenario, we are assuming that the river is in a high flow condition and hence dilution will be significant. This is illustrated in Table 10 and Figure 15.

Reach	Peak CN conc. mg/l
Abrud	0.87
Campeni	0.50
Baia de Aries	0.36
Turda	0.25
Ocna Mures	0.16
Albalulia	0.11
Deva	0.09
Savirsin	0.08
Arad	0.08
Nadlac	0.07

Table 10 Peak CN concentrations for Scenario 3b for high flow conditions.

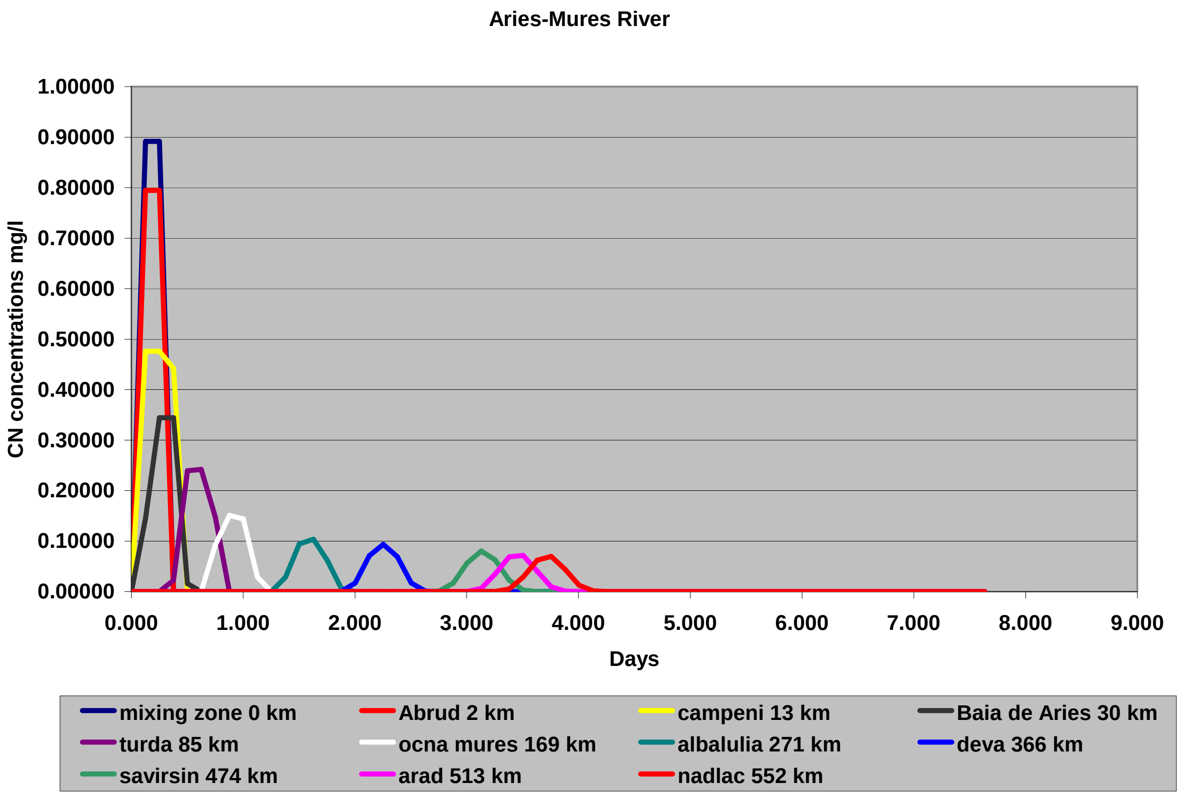


Figure 15 CN concentrations at a range of sites assuming the 3b Scenario with high flow conditions.

Scenario 3c and 3d

The scenarios 3c and 3d assume that the dam is in year 17 and has a water volume of 381100 cubic metres with a concentration of 4.4 mg/l of CN WAD. The results of the peak concentrations for scenario 3c with two decay rates are shown on Table 11. Figure 16 illustrates the CN simulation at a range of locations along the river. The results suggest that given the higher discharge from the Dam the concentrations will exceed the limit with the lower decay rate. However, higher decay rates suggest lower concentrations at the border.

Under high flow conditions the model suggests that dilution will be significant and hence concentrations stay below the standard at the border, as shown in Table 12.

Reach	Scenario 3c Decay Rate 0.1 days^{-} Peak CN Conc. mg/l	Scenario 3c Decay Rate 0.2 days^{-1} Peak CN Conc. mg/l
Abrud	4.37	4.37
Campeni	4.25	4.21
Baia de Aries	4.13	4.01
Turda	3.77	3.37
Ocna Mures	2.88	2.30
Albalulia	1.51	0.88
Deva	0.81	0.34
Savirsin	0.41	0.11
Arad	0.31	0.07
Nadlac	0.25	0.05

Table 11 Simulated CN under Scenario 3c for low flow conditions

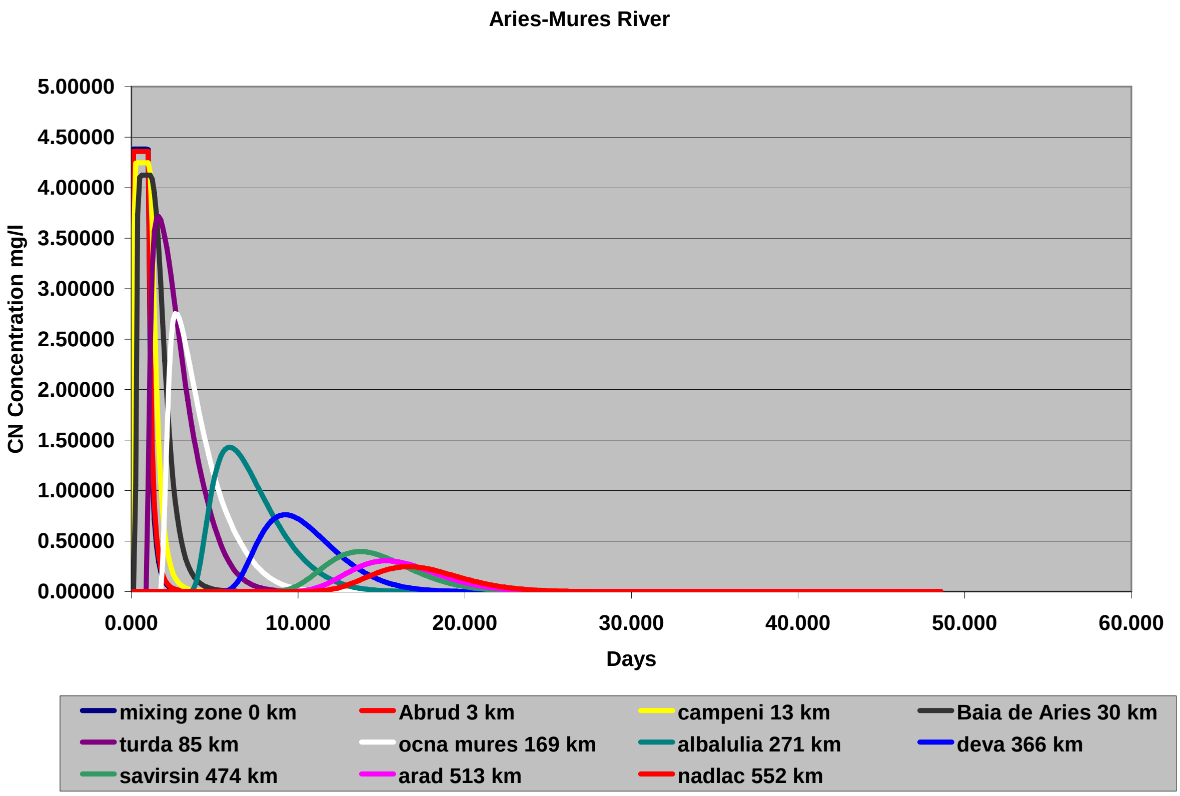


Figure 16 Low Flow Scenario 3c with a decay rate of 0.1 day^{-1}

Reach	Scenario 3d Peak CN Conc. mg/l
Abrud	0.991
Campeni	0.574

Baia de Aries	0.416
Turda	0.295
Ocna Mures	0.184
Albalulia	0.127
Deva	0.116
Savirsin	0.105
Arad	0.100
Nadlac	0.096

Table 12 Simulated CN concentrations for Scenario 3d under high flow conditions with decay coefficient at 0.1 days^{-1}

Scenario 3e and 3f

The scenarios 3e and 3f assume that the dam is in year 17 and has a water volume of 5880800 cubic metres with a concentration of 5.0 mg/l of CN WAD. The results of the peak concentrations for the low flow scenario 3e are shown on Table 13 and Figure 17 illustrate the CN simulation at a range of locations along the river. As might be expected with the high discharge rate under very low flow conditions, the CN concentrations are higher at the border and exceed the standards.

Reach	**Scenario 3e Decay Rate 0.2 days^{-1} Peak CN Conc. mg/l**	**Scenario 3e Decay Rate 0.1 days^{-1} Peak CN Conc. mg/l**
Abrud	4.98	4.98
Campeni	4.88	4.84
Baia de Aries	4.77	4.65
Turda	4.44	4.04
Ocna Mures	3.76	3.12
Albalulia	2.33	1.48
Deva	1.37	0.65
Savirsin	0.73	0.23
Arad	0.57	0.15
Nadlac	0.46	0.11

Table 13

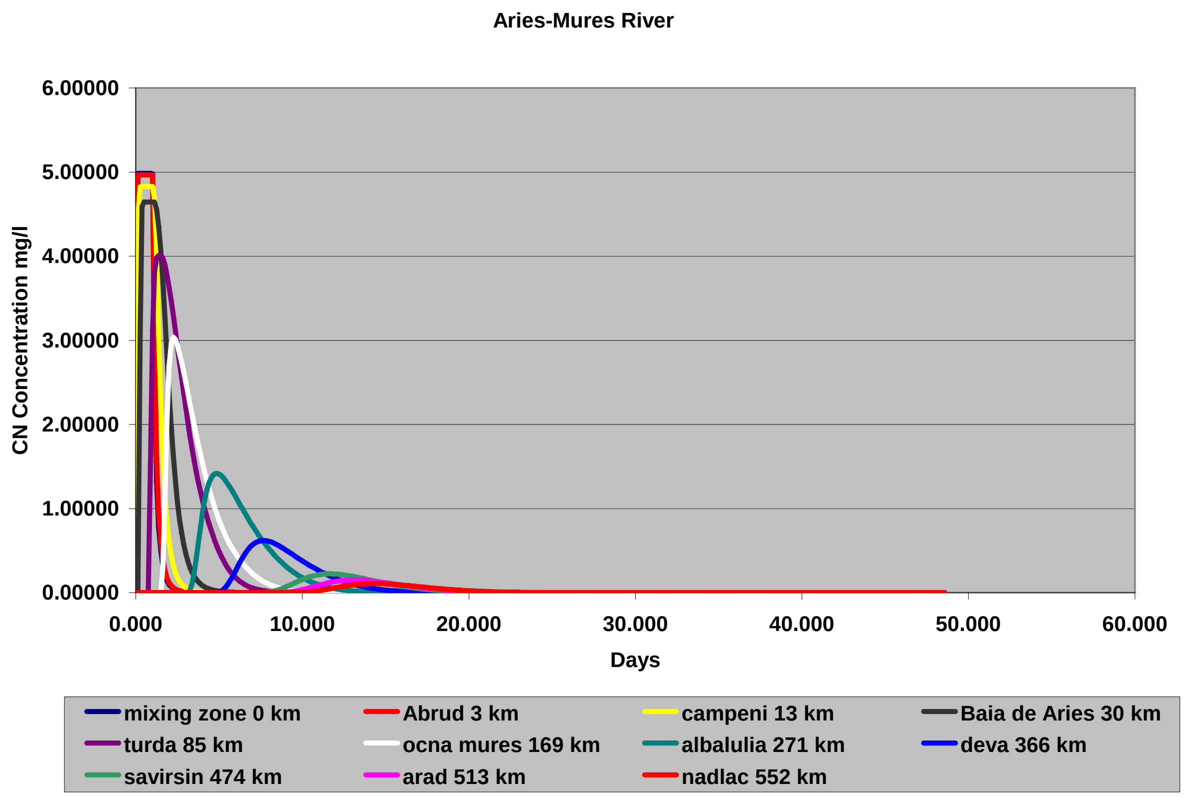


Figure 17 Scenario 3e with a decay of 0.2 days $^{-1}$ under low flow conditions.

Reach	Scenario 3f Peak CN Conc. mg/l Decay Coeff. 0.1 days^{-1}	Scenario 3f Peak CN Conc. mg/l Decay Coeff. 0.2 days^{-1}
Abrud	1.43	1.43
Campeni	0.87	0.86
Baia de Aries	0.64	0.63
Turda	0.46	0.44
Ocna Mures	0.29	0.27
Albalulia	0.20	0.17
Deva	0.18	0.15
Savirsin	0.17	0.12
Arad	0.16	0.11
Nadlac	0.15	0.10

Table 14 Simulated CN concentrations for scenario 3f under high flow conditions

Scenario Runs Assuming Shorter Discharge Times

All the above scenarios assume that the discharge occurs over a 24 hour period as specified in the original EIA. However, the Hungarian team were concerned about this assumption and were keen to see results assuming shorter release time. The release time could be a subject of considerable debate as the release will depend on the nature of any Dam break, the construction of the Dam etc. However, the model has been used to assess the impacts in the event of a shorter release time. Table 15 shows the effect of reducing the duration of the spill from 24 hours to a range of durations, down to 1.5 hours. At first sight, the results are counterintuitive, in that the reduced spill duration shows almost no differences at the lower reaches of the river. The reason for this is illustrated in Figures 18 and 19. In these figures are shown the simulated flow data for both the 24 hour and 1.5 hour spill duration. The figures show that although the flows are different in the top reaches of the river system due to the changed flows from the simulated dam break, by the time the flow gets some 100 km downstream the flows in the 2 runs are almost identical. Thus the velocities, dispersion and dilution effects will be identical in both runs downstream. Hence, it is not surprising that the concentrations are not significantly different 565 km downstream at the border.

	Spill duration 24hrs	**Spill duration 12hrs**	**Spill duration 6hrs**	**Spill duration 3hrs**	**Spill duration 1.5hrs**
Reach	**Scenario 3a Peak CN Conc. mg/l**	**Scenario 3a Peak CN Conc. mg/l**	**Scenario 3a Peak CN Conc. mg/l**	**Scenario 3a Peak CN Conc. mg/l**	**Scenario 3a Peak CN Conc. mg/l**
Abrud	4.668	4.745	4.776	4.787	4.792
Campeni	4.341	4.546	4.656	4.711	4.736
Baia de Aries	4.069	4.343	4.474	4.523	4.521
Turda	2.963	3.047	3.060	3.068	3.023
Ocna Mures	1.347	1.368	1.375	1.383	1.390
Albalulia	0.492	0.500	0.504	0.507	0.510
Deva	0.234	0.238	0.240	0.242	0.243
Savirsin	0.114	0.116	0.117	0.118	0.118
Arad	0.087	0.089	0.090	0.090	0.090
Nadlac	0.070	0.071	0.072	0.072	0.072

Table 14 Simulated CN Concentrations for a range of spill durations from 24 hours to 1.5 hours for Scenario 3a.

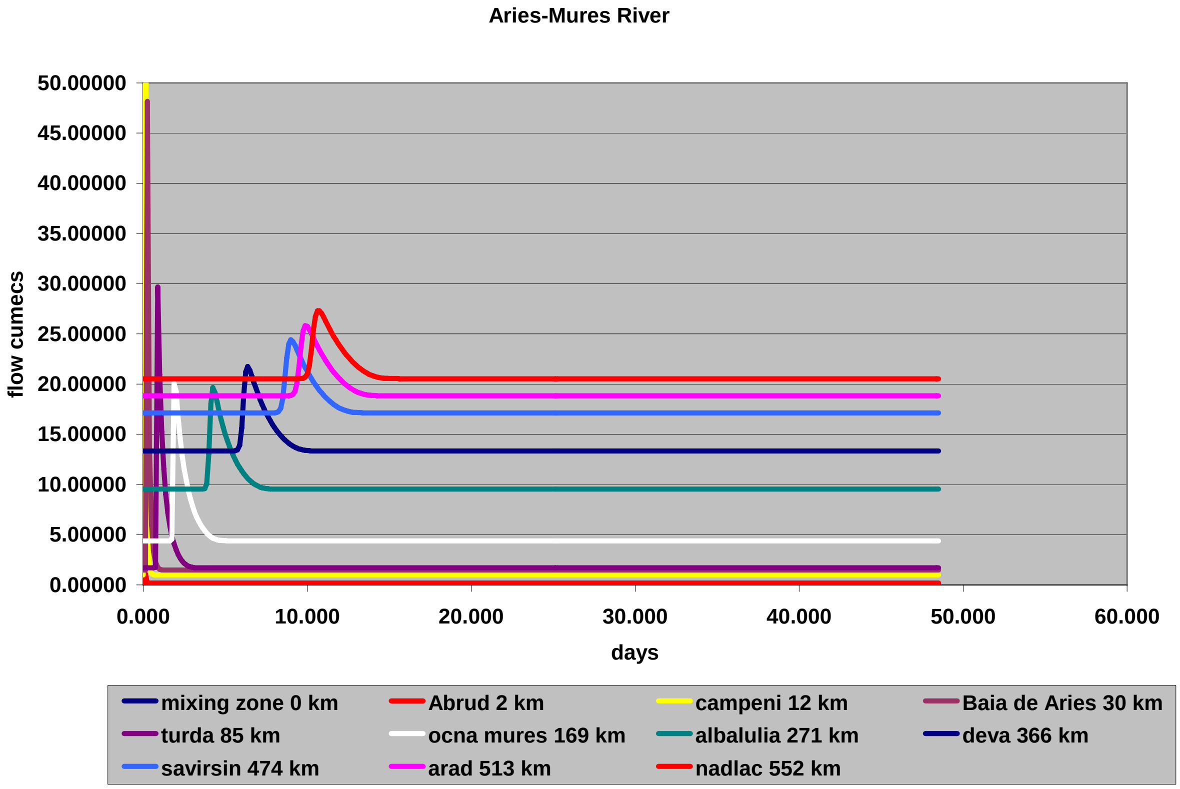


Figure 18 Flows simulated with a Spill Duration of 1.5 hrs

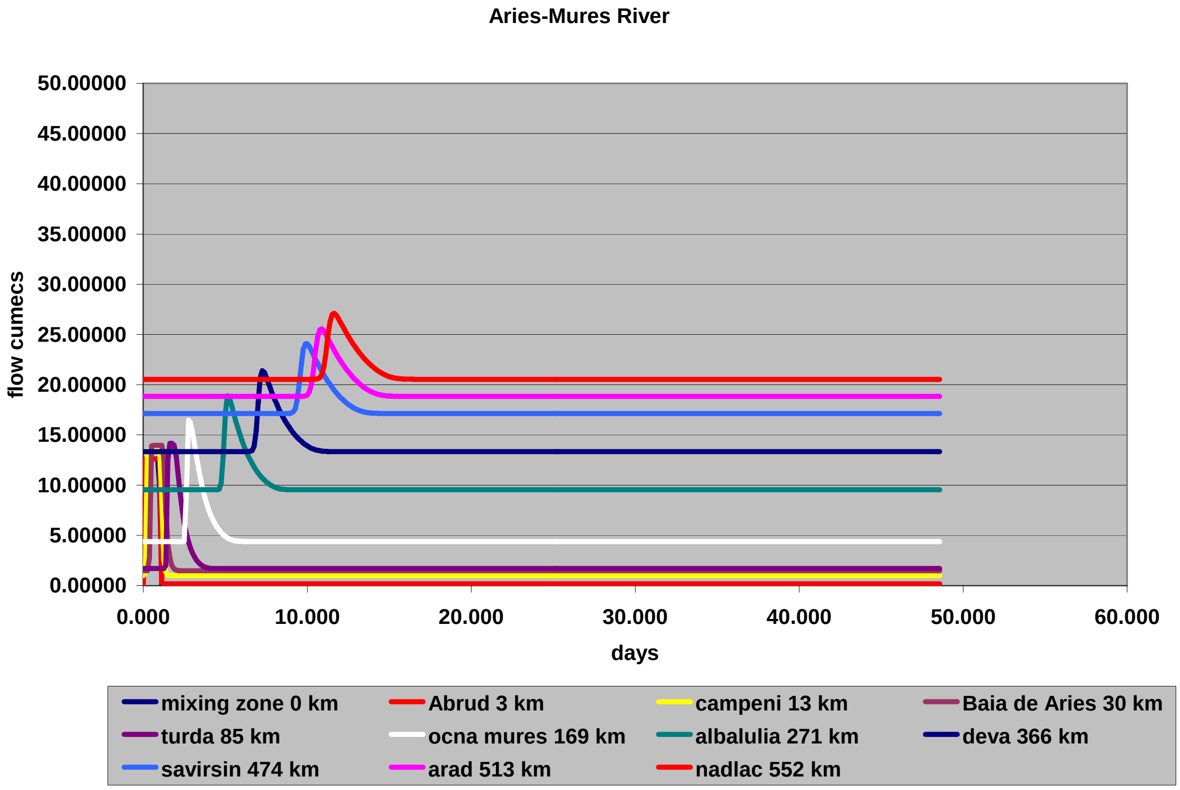


Figure 19 Flows simulated with a Spill Duration of 24 hrs

6 CONCLUSIONS

The results presented in this annex to the main report indicate the likely concentrations under a range on new scenarios. These new scenarios relate to low flow and high flow conditions and also to the duration of potential spills. In the previous report, the simulations were undertaken under average and high flow conditions, as specified in the EIA reports for the project. However, the Hungarian team were keen to assess the impacts using a dispersion modelling approach under low flow conditions and with reduced spill durations.

In this report, the new dispersion model is described in considerable detail together with the numerical solution techniques used to solve the equations. The new model has incorporated the dispersion and transport processes occurring in rivers and also accounts for the diluting effects of tributaries and runoff along the river. The model also allows for the degradation of pollutants along the river which in the case of CN is due to volatilization and chemical transformation to ammonia. This degradation depends on the residence time of the water in the river, temperature and the decay rate and the new model accounts for all these effects.

The model has been set up for the river system from the location of the proposed dam in the Corna Catchment down to the border at Nadlac. The results indicate that in both high and low flow conditions a Baia Mare type rainfall or snowmelt event will not produce a severe pollution impact. This is primarily because of the dam spillway which is designed to release water in a controlled manner during high rainfall conditions.

More problematic is the effects of a catastrophic dam break which would release all the water and pollutant content of the dam storage reservoir. The risks of this happening are more question of geology and dam design and is beyond the scope of this report. All that is considered in this report is the impact of such an eventuality occurring.

The impact of the releases under different years of dam development is considered in this report. The effect of the discharges under low flow conditions is to generate low concentrations under most releases and all are below the CN standards except for the very largest volumes. This is because of the reduced dilution downstream in low flow conditions as well as the significant dispersion and the large residence times. Dilution is also having a major effect in high flow conditions creating relatively low concentrations, although the reduced travel time suggests that the year 17 dam releases will create higher pollution levels.

The question of the duration of the spill release has produced some very interesting results. At first sight the effects of reducing the spill duration say from 24 hours to 3 hours should be significant because for the same mass of discharge the spill flow rates have to be much higher. Whilst this will have some effect in the upper reaches of the river, the impacts downstream at the border has almost no additional effect. This is because attenuation of the flood wave occurs and after about 100 km of river the pollution and flow pulse has mitigated so that it is indistinguishable from longer duration spills.

7 REFERENCES

Chapra, S.C. 1997. Surface Water-Quality Modeling. McGraw-Hill, New York.

Fischer, H. B. 1968. "Dispersion Predictions in Natural Streams." J. San. Engr. Div. ASCE, 94(SA5): 927-944.

Fischer, H. B., List, E. I., Koh, R. C. Y., Imberger, J., and Brooks, N. H. 1979. Mixing in Inland and Coastal Waters. Academic, New York.

Liu, H. 1977. Predicting Dispersion Coefficients in Streams. J. Envir. Engr. Div. ASCE, 103(EE1): 59-69.

McQuivey, R.S. and T.N. Keefer, 1974. Simple Method for Predicting Dispersion in Streams, J. Environ. Engr. Div. ASCE, 100(EE4):997-1011.

Rutherford, J.C. 1994. River Mixing. Wiley, New York.

Whitehead P.G. (2007) A Water Quality Modelling Study of Rosia Montana and the Abrud, Aries and Mures River Systems, Report to RMGC, pp 89

Whitehead, P.G., Williams, R., Hornberger, G.E. (1986), On the identification of pollutant or tracer sources using dispersion theory, J. of Hydrology, 84, 273-286.

STATEMENT OF LIABILITY

Note that the University of Reading and the University of Tufts has used all reasonable endeavours to ensure the accuracy of the work performed and any information given. The Universities makes no warranty, expressed or implied, as to the accuracy of the information used in the study. The Universities will not be held responsible for any consequence arising out of any inaccuracies or omissions. The parties agree that the obligations of the Universities and its agents shall cease upon delivery of the report and that no liability whatsoever either direct or indirect shall rest upon them for the effects of any product, process or action that may be produced, adopted and/or undertaken by any other party. Neither shall the university or its agents be liable for any death or injury sustained as a result of the research results presented in this report.