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PERFECTION OF OPERATION CONTROL FOR THE EMERGENCY RESERVOIRS IN THE KÖRÖS VALLEY

I. RÁTKY and L. SZLÁVIK*

Hydraulic and Water Resources Engineering
Budapest University of Technology and Economics
H–1521 Budapest, Hungary
Phone: +36 1 463 2248
Fax: +36 1 463 4111
E-mail: ratky@vpszk.bme.hu
*The Water Resources Research Centre Plc.,
Eötvös József College, Baja, Hungary
Telephone: +36 1 215 4158, Telefax: +36 1 216 1514,
Email: szlavik@vituki.hu

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Abstract

The state of art in this field is reviewed. Starting from a synthesis of the positive experiences gained in the operation of emergency reservoir specific proposals are submitted for improving particular steps in operation control and for possibilities of further perfection. The flood situations warranting emergency storage and the 'conventional' approach to reservoir dimensioning are described, pointing out the difficulties in determining the 'optimal' time of impoundment; the drawdown created by emergency storage in the river system is analysed (*Figs. 1* and *2*) and indices showing the effectiveness of impoundment are derived (*Table 1*). The key hydrologic-hydraulic parameters reflecting the effects of emergency storage events since 1966 in Hungary are summarised in *Table 2*. A simplified, approximate method is presented to estimate the drawdown curve from the maximum drawdown. The influence ranges estimated for the inundation of particular reservoirs are shown in *Fig. 4*. The approach by which the impact study on the parameters of typical flood waves can be simplified and thus made accessible to calculation is presented (sensitivity analysis). The results obtained for this complex phenomenon are compiled in a readily understandable form to support decision making in actual operation control situations. Summarising the ramified studies, the improvements recommended and the perfection options proposed, a refined version of the conceptual model of perfected operation control of emergency flood reservoirs is presented in *Fig. 6*.

Keywords: flood fighting, emergency reservoirs, emergency storage, flood peak reduction.

1. Introduction

The term '*emergency flood reservoir*' is understood as an *area made suited by engineering measures to temporary storage*. Such reservoirs are flooded in extraordinary situations alone, to avert impending failure of a main levee line and thus to control major losses and flood disasters. Under normal conditions the area serves the original purpose (agriculture or forestry). **The purpose of emergency storage** is to retain temporarily part of the flood volume and to reduce thereby the peak flood level, or the load on the flood embankments.

Before a decision on flooding an emergency reservoir is taken, the actual flood situation and a number of complex, ramified consequences must be taken into consideration. For this reason such decisions are reserved under the current legal provisions to the competent minister, in that detailed information on the hydrological situation of the entire river system and the catchment, *on the flood developments in a major region is essential, further besides the potential engineering impacts the costs and losses must be deliberated.*

2. The Key Methodological and Practical Issues in Emergency Flood Storage

There are no realistic chances of reproducing the complex phenomena and processes involved in a physical or mathematical model. Any model must necessarily be confined to some elements of the processes, or to some structures. The approach adopted so far consisted therefore mainly of observing, where possible measuring, analysing and assessing the events and phenomena related to the subject. The conclusions have been arrived at by generalising the experiences gained with the past levee failures, flood inundations and emergency operations in Hungary.

The 14 emergency storage events between 1966 and 1997 included both disaster decisions and measures planned in advance (SZLÁVIK 1998b; SZLÁVIK–RÁTKY 1999). In seven of these latter cases the early results of the present study have already been used to advantage (SZLÁVIK 1998a).

2.1. The Critical Situation Prompting Emergency Flood Storage

Emergency flood storage may be warranted in **four** substantially different **situations** (SZLÁVIK 1983, 1998a), for each of which examples can be quoted from Hungarian flood fighting practice:

(a) *To lower the peak* of the flood hydrograph at stages surpassing the design level for which the defences were built and which they are capable of safely withstanding;
(b) As an instrument of *controlling ice-jam floods*, to avert impending levee failure by overtopping;
(c) *To prevent flood disaster by loss of levee stability* owing to saturation in floods of extended duration or other defect;
(d) *To alleviate the consequences* of a levee failure.

From a detailed study of the circumstances which had prompted resort to emergency storage in Hungarian practice it has become evident that case (a), is the one from which the *design criteria* of emergency reservoirs should be derived. The purpose of emergency storage is then to prevent the development of stages, which would cause overtopping or jeopardise otherwise the stability of the levees and to avert an impending flood disaster in this way.

Dimensioning flood reservoirs specifically for the other three situations [(b), (c), (d)] is not justified, although such reservoirs dimensioned and built to retain part of the flood volume may be found effective in critical situations caused by ice-jam floods, or loss of levee stability, further of reducing the impacts of a levee failure propagating to other affected river sections. These considerations must be remembered in selecting the site and in formulating the design criteria of emergency flood reservoirs (SZLÁVIK 1980, 1997).

In designing, but especially in operation, distinction must be made between **isolated single** reservoirs and **co-operating reservoirs** connected parallel to each other.

In the general case of co-operating reservoirs two different hydrologic situations have been distinguished as regards emergency storage:

- The 'solitary', extremely high flood wave to be lowered, where the reservoir under consideration can only be counted upon along the river section influenced by the reservoir (the Rivers Körös 1974, 1981, 1995);
- Critical flood situations on several rivers, where flooding of all reservoirs in the river system may become necessary (the Rivers Körös 1970, 1980).

From the analysis of emergency storage experiences in Hungary it has been concluded further that an emergency reservoir situated at the confluence ('delta') of two rivers should be dimensioned on the one hand for the 'solitary' flood wave, on the other hand, for the one of the two tributaries which conveys the larger flood volume above the particular level (taking account at the same time also of the additional water volume resulting from lowering the water level on the other. **Parallel, or series connected reservoirs** ensure therefore effective level control in this latter hydrologic situation as well.

2.2. The Water Volume to be Stored

The total water volume W to be stored in a delta enclosed by two rivers is found as the sum of four part-volumes (SZLÁVIK 1980):

$$W = W_1 + W_2 + W_3 + W_4,$$

where W_1 = the water volume to be diverted in order to lower the water level sufficiently on the critical tributary;
W_2 = the additional water volume resulting from the drawdown on the other tributary;
W_3 = the 'opening correction';
W_4 = the storage space needed to accommodate a second flood wave travelling down the river while the emergency reservoir is still open.

There are several methods of producing the design flood hydrograph of emergency storage. **The key** issue in each concerns the **use of the flood loops** reflecting the characteristics of the flood wave. In calculations related to the dimensioning and operation of emergency flood reservoirs the use of the $Q - H$ curve described by a power function has been found expedient in the form corrected by the flood loop, determining the width of the latter on the basis of the flood loops actually observed (SZALAY 1975; SZLÁVIK 1975, 1978; SZLÁVIK–GALBÁTS–KISS 1996).

During the hydrological analysis of emergency flood storage, the water volume W_1 to be diverted for lowering the water level sufficiently on the critical tributary should be found using the $Q - H$ curve corrected by the flood loop. The additional water volume W_2 by which the desired drawdown can be achieved on the other tributary need not be taken into account, if the flood waves on the two branches do not coincide, or if the emergency reservoir is not situated in, or in the vicinity of the delta. In this case $W_2 = 0$.

An essential requirement of effective emergency storage is that the peak of the lowered flood wave on the river must not surpass the design flood level. To induce flow over the sill, or 'weir' of the opening, a certain weir head is needed. The top of the overfalling jet varies in time and must remain below the design level. For this reason diversion (impoundment of the reservoir) must be started at a lower water level. The part volume resulting from opening below the design flood level in the river is referred to as the 'opening correction' W_3.

The hydrologic characteristics of a particular river will provide information on the recurrence likelihood of flood waves. Considerations related to the rate and method of reservoir depletion will enable a sound engineering estimate on the part volume W_4 required to accommodate an additional flood wave travelling down the river while the emergency reservoir is still open.

Once the water volume to be stored has been determined, it will be possible to decide on the design water level in the emergency flood reservoir (taking into consideration also the magnitude and topography of the area available). The decision involves invariably particular deliberations in which non-hydrologic issues must also be taken into account.

2.3. Timing Reservoir Impoundment

Opening of an emergency reservoir may become necessary when the rising water level approximates the design water level of the defences and from an analysis of the hydrological situation the flood peak is found liable to attain, or surpass this level.

Evidently, the 'optimal' instant of flooding can only be determined in cases, where the reservoir is flooded in the interest of lowering the peak of 'normal' floods travelling down the river. In the event of ice-jam floods, for preventing levee failure, or for reducing further losses after a failure, flooding will usually allow no delay.

The water level below the design value at which impoundment flooding must be started will depend on the particular situation, including the violence of the flood, the method adopted for opening the levee along the reservoir, etc. During past floods the advisable instant was found to be that when the water level rose to **within 0.1–0.3 m of the design value**. One of the aims of the present study was precisely to improve the accuracy of timing which has been based so far on past experience.

2.4. The Impact of Emergency Storage on the River System, the Effect of Lowering the Flood Peak

The analysis of the hydrologic and fluvial hydraulic experiences gained with past levee failures and diversions to emergency storage has demonstrated the paramount importance of exploiting the *sudden drawdown* created along the river section influenced in maximising the effectiveness of the reservoir. Along the river section(s) upstream of the diversion, the *streamflow rate increases suddenly* within a few hours owing to the steeper slope of the water surface. This phenomenon is of crucial importance in the hydrology and hydraulics of emergency storage, in that *opening at the correctly chosen instant* will produce the desired effect while diverting a relatively small water volume.

By plotting the *water volume diverted* W against the *drawdown* Δh created for the various emergency storage operations in the form of $\Delta h = f(W)$ curves it can be demonstrated that **the diversion to storage of a relatively small water volume can produce a significant drawdown** already. The drawdown observed on the Remete gage following diversion to the Mályvád emergency reservoir at different instants is shown in *Fig. 1* (diversion at 8, 14 and 18 hours before the peak without diversion – RÁTKY 1998). Evidently, the importance thereof will be more pronounced in cases, where the aim is to reduce the waterload on the levees in the immediate vicinity of the reservoir.

By selecting correctly the *dimensions of the gap opened in the levee and timing the diversion correctly, a dynamic situation* can be induced in the early phases of diversion, which creates by exploiting the considerable weir head a major local drawdown and provides sudden relief of the critical river section. The opening functions subsequently as a conventional overflow with dimensions enabling it to divert the intended discharge.

By introducing the **indices** M_1 and M_2 it is possible to assess the particular emergency reservoirs and to compare the effectiveness in lowering the flood peaks:

- *The ratio of the highest discharges*

$$M_1 = \frac{Q_{\text{max to storage}}}{Q_{\text{max flood}}}.$$

- *The ratio of the volume stored to that conveyed in the river at stages above a*

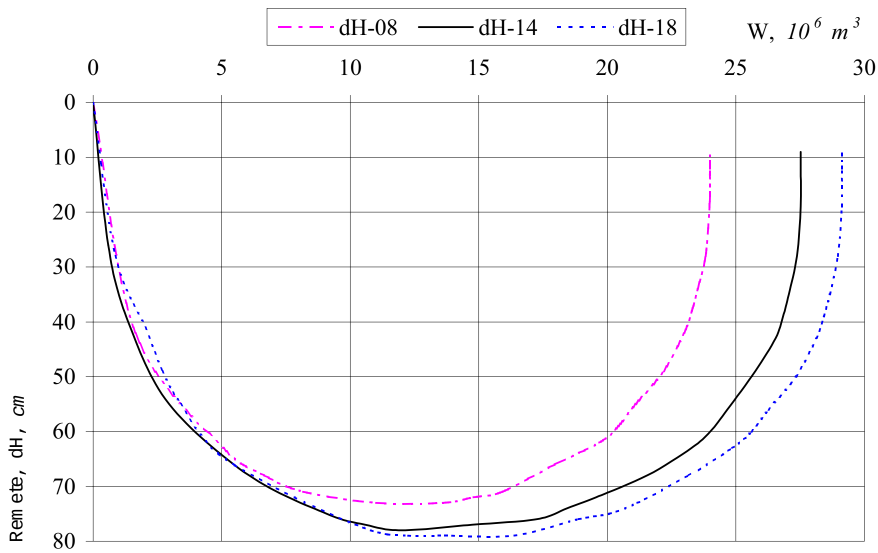

Fig. 1.

given level and without diversion to storage:

$$M_2 = \frac{V_{\text{stored}}}{V_{\text{flood wave}}}.$$

The indices characterising the *flood-peak capping effect* in six cases of emergency storage have been compiled in *Table 1*. As it will be perceived therefrom, the magnitude of M_1 may be higher than unity owing to the local drawdown which may reverse temporarily the direction of flow. M_2 **is a measure of the effectiveness** of emergency storage, the magnitude of which may approach unity at river stages above the warning level III.

The impact of emergency storage on the various sections of the river system will depend on the location of the river sections relative to the point of diversion. The hydrologic-hydraulic impacts of emergency storage may be classified into three groups:

- *Downstream of the diversion* the impact of emergency storage propagates along the successive sections of the particular branch of the river system;
- *Upstream of the diversion* the drawdown increases steeply the surface slope (on the main stem and any nearby tributary);
- *Along the tributaries* joining the main stem along the section influenced by the diversion or drawdown the surface slope is increased moderately.

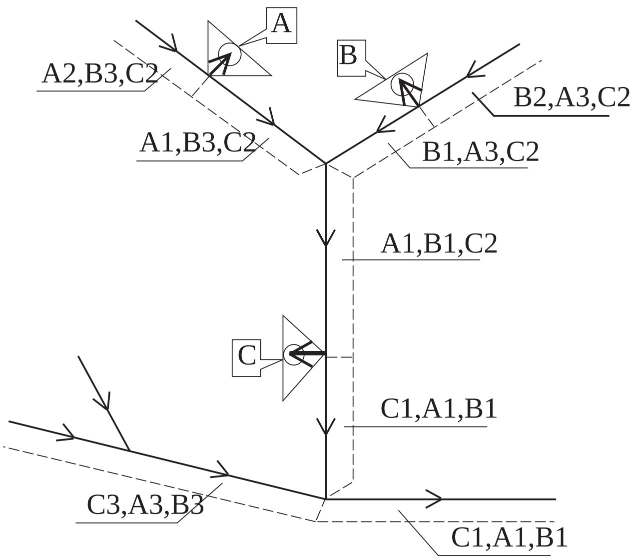

Fig. 2.

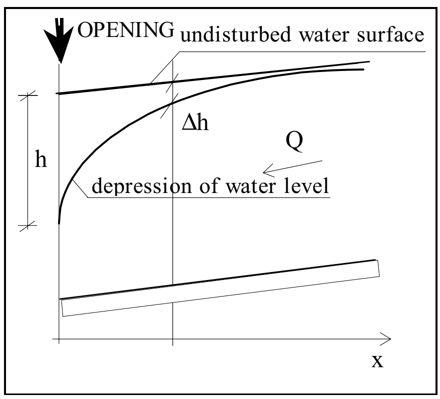

Fig. 3.

The hydraulic phenomenon on a tributary is the same one, regardless of whether it joins the main stem upstream, or downstream of the diversion.

The impacts of emergency storage in the river system are shown schematically in *Fig. 2*. The direct impact of diversions extends to a longer distance downstream, the flow diverted lowering the natural flood hydrograph by abstracting part of the flow for temporary storage. The impact of diversion (abstraction) increases with the value of M_1.

Table 1. Impacts of the emergency reservoirs in the Körös Basin on the lowering of the flood peaks

N^0	Parameter	Emergency storage in 1974 at the delta-opening in 3 cross-sections	The discharge through the levee failure in 1980 (Kettős-Körös)	Emergency storage in 1981 at Mályvád-opening in 2 cross-sections	Emergency storage in 1995 at Mályvád	Emergency storage in 1995 at Mérges
1.	Maximum discharge to the emergency reservoir [m^3/s]	780	750–850	910	150	200–250
2.	Natural maximum discharge of the river in time of opening of the emergency reservoir [m^3/s]	945	842	755	489	767
3.	$M_1 = (2)/(1)$	0.83	1.01**	1.21***	0.31	0.26–0.33
4.	Volume of the storage [million m^3]	118	200	75	7.4	38.8
5.	Flood volume over the emergency level I.	488*	550	168	37.0	198
6.	$M_2 = (4)/(5)$	0.24	0.36	0.45	0.20	0.20

Notes: * – Total flow of the Fekete- and Fehér-Körös [million m^3]
** – It is possible because of the local drawdown and the back-flow of the Kettős-Körös
*** – It is possible because of the local drawdown and the back-flow of the Fekete-Körös

Upstream of the diversion the drawdown is a marked one and decreases rapidly with distance therefrom (*Fig. 3*). (This will be considered more in detail in Section 3.1.)

The impact of diversion is an *indirect one on the tributaries* to the branch of the river system, from which water is diverted, in that the water level in the main stem if lowered, the slope and consequently the velocity of flow in the tributaries is increased so that these convey the arriving flow at a lower water level.

2.5. Operation Control and Flooding Strategy of Emergency Reservoirs

Flooding an emergency reservoir presumes special preparations including carefully planned activities. A **conceptual model** has been developed for controlling the operation of emergency reservoirs in any river system (SZLÁVIK 1983). The activities are comprised in three blocks:

(A) **Forecasting** the natural hydrological situation on the rivers;
(B) **Analysis** of emergency storage alternatives and assessment of their **impacts**;
(C) **Action plan** for implementing emergency storage.

An essential requirement is to couple *interactively the hydrologic forecasts and the flooding computations*.

The total time required for the necessary sequence of activities may be long relative to the lead time attainable on flashy streams. The train of preparatory activities must therefore be started at a time, when the complete set of information needed to decide on flooding the reservoir is not yet available. From an analysis of experiences gained with emergency storage in the past it has been concluded that the *preparatory activities* must be planned, ordered and carried out in a way to complete only the essential activities up to critical instants, in other words, the preparation of emergency storage must be realised with the necessary reliability but at the same time without expending superfluous efforts.

The activities involved in the preparation and implementation of emergency storage (Block C of the conceptual operations control model) must be comprised in an *action plan* to assist in selecting the '*optimal strategy*' to cope with the actual situation. The preparatory activities should be scheduled with the help of a 'critical path' diagram. The critical path determines the time needed to carry out the action plan and shows the instant prior to the envisaged (computed) opening of the emergency reservoir at which the sequence of preparatory activities must be started and executed successively to complete preparations by the time set for diversion. Implementation of the action plan must be harmonised continuously with the *hydrologic forecasts*. Starting the preparatory activities does not mean, evidently, that the emergency reservoir will necessarily be flooded, in that the preparations can be suspended, arrested any time, as long as an activity with irreversible consequences is started (e.g. placement and fusing the explosive charges). The final decision can be postponed to the commencement of such activities, to a few hours before flooding. It should be possible to assess the hydrological situation reliably and to complete technical preparations by that time. The activities executed according to the critical path represent therefore the 'optimal' strategy in preparing emergency storage (KHVM 1996; SZLÁVIK 1983; SZLÁVIK–GALBÁTS–KISS 1996).

The action plan of emergency storage should be revised and updated at regular intervals of time to allow for changes in the methods of opening, depletion, closure, etc.

2.6. The Characteristics of Past Storage Events

Although the phenomenon is understood well enough to describe it in words, or even in terms of analytical hydraulics, this does not imply that a general simulation method readily suited to practical application can be offered. For this purpose sufficiently in depth information is essential on the main characteristics and regularities of past storage events. Attention will be focused here on the issues, which are believed to assist in planning future uses of such reservoirs and in predicting the impacts of different opening methods on the river system. Drawing on the information published in the literature on the subject (NÉMETHY–BELEZNAY 1970; SZLÁVIK 1976, 1980, 1982; KÖVIZIG 1981; KHVM 1996) the key data of **six levee failures and/or emergency storage** events on the Hungarian sections of the Rivers Fekete Körös, Fehér Körös and Kettős Körös since 1966 are summarised in *Table 2* (RÁTKY 1997b).

> These will be referred to collectively as 'storage', regardless of whether the flood flow decreased by a failure of the defences, or deliberate emergency storage. In cases, where the latter occurred over a fixed weir, the impacts of storage on the river may differ slightly from that through a cut scoured to the base, or even deeper. Owing to these differences (shape, width, depth, rate of development of the opening, etc.) the cases must be distinguished in more detailed studies.

The hydrologic data compiled in *Table 2* on the 1966, 1970, 1974, 1980, 1981 and 1995 floods comprise:

- For the *storage events in Hungary the volume stored (million* m^3*)*, and the highest flow diverted (Q_{max}, m^3/s);
- **Lowering the flood peak** ($H_{max,\ reconstructed} - H_{max,\ actual}$, m, (subsequently ΔH_{max}), the difference between the calculated peak stage and the actual peak stage influenced by storage;
- **Maximum lowering** *of the water level* ($(H_{reconstructed} - H_{actual})_{max}$, m; subsequently Δ_{max}), the greatest drawdown created by storage, the widest difference between the calculated stage hydrograph and the actual one influenced by storage (which does not occur necessarily at H_{max}).

Table 2 offers a wealth of valuable information on the impacts of past storage events. Without any claim at completeness, the following are deemed to be of interest:

- The diverted flow Q_{max} provides information on the size of the opening. The actual size of the opening would be of little use in subsequent diversions, in that a failure would have caused excessive, or even deep scour which must be avoided in the future. A possible interpretation of Q_{max} is that the desired ΔH_{max}, or Δ_{max} can only be diverted through an opening of a size capable of conveying a discharge of approximately this magnitude.
- Besides Q_{max} the value of ΔH_{max} is influenced decisively by the length of the period between the time of opening and that of the predicted peak. Owing precisely to the decisive influence a given ΔH_{max} is difficult to use in planning the impact of future diversions (in that the opening were not always timed optimally in past storage events).
- Although the reduction of peak stages is mostly the main objective in flood fighting, the **maximum drawdown** Δ_{max} **is also of interest** when the head and duration of the hydraulic load on the defences must be reduced. These data are of particular relevance, in that they depend less than ΔH_{max} on the conditions of opening and can therefore be used for predicting the impacts more accurately. It should be underscored that they depend less, but they are also influenced by the shape of the flood hydrograph, the geometry of the opening and the time of diversion, etc.
- The data shown for ΔH_{max} and Δ_{max} represent always the combined impact, when several reservoirs were flooded. This is especially important to bear in mind when assessing the data of 1980.

3. Improvement of Operation Control for the Emergency Reservoirs in the Körös Valley

3.1. Hydraulic Phenomena Triggered by Flooding

The impacts on the river of opening a reservoir appear in the form of highly complex **hydraulic phenomena** any exact analytical description of which is possible at the cost of simplifying assumptions alone. No relations accessible to any practical solution, nor any numerical models covering every detail are known to exist. The following hydraulic phenomena are offered as an explanation of this fact:

- The main flow occurs in a natural bed subject to changes in space and time (composite, meandering flood bed with irregular vegetation and conveying capacity);
- They vary in three dimensions and in time;
- Diversion approximately perpendicular to the direction of flow in the river causes unsteady, suddenly varied flow;

- No analytically exact solution is available for flow over a side weir and the tail-water apron thereof even in the case of simplified geometry and steady flow;
- The opening in the levee is of irregular shape which changes in size and time alike;
- The jet entering over the sill may be drowned as the water level in the reservoir rises (no exact analytical solution is known);
- The hydraulic impacts on the river system studied (mathematically: the upstream and downstream boundary conditions) are extremely difficult to determine under field conditions.

For the calculations needed for planning the storage process **simplifying abstractions** and assumptions **must be adopted**. Several approximations have been published in the literature, some of those by Hungarian authors will only be mentioned here: CSOMA–KOZÁK–RÁTKY 1988; GODA–SZLÁVIK 1983; NÉMETHY–SZALAY 1977; RÁTKY 1988, 1997a; RÁTKY–CSOMA 1988, 1996; SZALAY 1975, 1976; SZALAY–BAKONYI 1976a, 1976b; SZLÁVIK 1975, 1980; VITUKI 1974; VITUKI Consult 1997. A common feature of the various publications – including also the unquoted foreign ones – is that instead of the entire storage process, they focus on *modelling selected component phenomena thereof with hydraulically widely differing approximation and accuracy*. (A combination of these results would not yield a model of uniform accuracy !)

No basic research has been conducted over the past decades on the hydraulics of the phenomenon. Mention must be made of the last detailed hydraulic studies in the mid-70s by Miklós SZALAY (SZALAY 1975, 1976; SZALAY–BAKONYI 1976a, 1976b; NÉMETHY–SZALAY 1977).

Advances in science and technology – computers, numerical models, accuracy of data, information flow – during the past 20 years would warrant basic research projects covering the entire runoff-storage process.

3.2. Approximation of the Drawdown Curve

In the absence of comprehensive scientific data, an approximate method will be presented for estimating a part-phenomenon of the process (RÁTKY 1997b). Diversion to storage is known to *create a drawdown lowering the water level over the river reach upstream of the diversion and in the tributaries*. The drawdown curve on this river reach can be approximated by a parabola of the second degree (*Fig.*3).

A drawdown of depth h in the diversion cross section or at the mouth of a tributary will produce at the distance x therefrom a drawdown, the magnitude of which is found approximately from the following expressions

$$\Delta h = \frac{S^2}{4h}x^2 - Sx + h \qquad \text{if} \qquad x \leq \frac{2h}{S},$$

where h – the drawdown in the diversion cross section, m
x – the distance upstream of the diversion, where Δh is to be found, m
h – the drawdown at the distance x upstream of the diversion, m
S – the original surface slope before diversion.

The root of the drawdown curve, i.e., the range influenced by the drawdown is found as

$$x_{\max} = \frac{2h}{S}.$$

(For instance, on the Fehér Körös, at a surface slope of 0.13 m/km, a drawdown of $h = 1$ m extends some $x_{\max} \approx 15$ km upstream of the diversion.)

Hydraulically similar phenomena take place over the river reach upstream of the diversion, further in the tributaries entering both upstream and downstream thereof. On the tributary upstream of the diversion the *drawdown* on the main stream at the confluence, on the tributary downstream of the diversion the *water level lowered by diversion* is effective. On both tributaries *drawdown curves rising opposite to the direction of flow* develop. The foregoing expression is therefore suited to estimating the effect of diversion on the water levels on the three river reaches.

3.3. Estimation of the Influence Range of Diversion

For predicting with any reasonable accuracy the range over which the influence of diversion to an emergency reservoir is felt in the river system, all available information must be collected. The information used in this case included:

- the ranges observed in the past (*Table 2*),
- the design criteria (SZLÁVIK 1980),
- the results of numerical simulation (RÁTKY 1988; RÁTKY–CSOMA 1996; RÁTKY 1997a), and
- the formula presented in the foregoing for the largest drawdown range.

It should be emphasised that any general method of predicting the influence range must involve necessarily a **high degree of inaccuracy** even if the flow is diverted over a fixed weir of known crest height and length. The key factors to be **remembered** are:

- the shape of the flood hydrograph, the rate of rise and fall,
- the duration above a particular level, the volume of water conveyed,
- the time of diversion relative to that of the peak,
- the initial conditions prior to the flood wave, bed fullness,
- the conveying capacity of the bed (vegetation, roughness), and
- the parameters of the flood waves arriving on the tributaries (peak stage, duration, time of peaking, etc.).

In the absence of information on these factors the influence range of emergency storage is impossible to predict accurately. An analysis of the data compiled in *Table 2* and of the results of numerical simulation will reveal that under the design conditions – when the reservoir must be flooded – the impacts and influence ranges scatter over a certain interval. The fact that in addition to the Mályvád Reservoir, the Mérges and Kisdelta reservoirs will also be flooded over fixed weirs, is expected to improve the accuracy of planning and operation control.

The influence ranges in the river system estimated for the case that the reservoirs are flooded at the 'optimal' instant are shown in *Fig. 4*. **The flooding of only one reservoir** was assumed in each case. The distances up to which the drawdown effect is considered significant has been indicated for each reservoir. The following drawdowns were adopted as significant:

at peak flood stage	$\Delta H_{\max}$	> 0.2 m and
at greatest drawdown	$\Delta_{\max}$	> 0.3 m.

The impacts on the River Hármas-Körös depend to a great extent on the actual regime of the recipient River Tisza, which may back up far into the tributary. In extreme situations the influence of the Tisza extends upstream as far as the mouth of the Kettős-Körös. Over the Hármas-Körös section downstream of Szarvas the drawdown may already be influenced strongly by the Tisza. This is implied by the influence lines in brackets in *Fig. 4*.

Owing to the reasons outlined in the foregoing, to the complexity of the hydraulic phenomena, as well as to the factors which are impossible to predict, great care and circumspection is advised in using *Fig. 4*. Notwithstanding the inevitable uncertainties, it is still a valuable tool **in showing the potential reservoir to be flooded when the need for lowering the water level arises on some river sections** (see subsequently as step 7 in the operation control model). The figure can be used also in the case of flooding several reservoirs, although the accuracy will even be poorer owing to the non-linear superimposement of the impacts.

3.4. Sensitivity Tests on Flood Parameters

One of the lessons learned from flooding the emergency reservoirs was that the potential impacts of each reservoir on the river sections within the influence range must be assessed carefully, quantified and taken into account in the scenarios compiled in advance and covering all possible circumstances of flooding. This assumes special importance in the ramified Körös river system, where it may become necessary to flood up to six emergency reservoirs simultaneously, or in different combinations.

From these scenarios situations must be identified, in which flooding of a reservoir would be ineffective (for instance a critical situation on an upstream reach cannot be influenced by flooding a reservoir far downstream, or to a limited extent only by flooding a reservoir on a nearby tributary).

Table 2. Key hydrological and hydraulic data of emergency reservoir impoundments in the Körös Valley on Hungarian territory

Emergency flood reservoir		1966	1970	1974	1980	1981	1995
Mályvád (Overflow Q_{max}	Mm^3 m^3/s)				19 (200)	75 (910*)	7.4 (150)
Nagydelta (Overflow Q_{max}	Mm^3 m^3/s)			118 (780)			
Hosszúfok (Overflow Q_{max}	Mm^3 m^3/s)				200 (800)		
Mérges (Overflow Q_{max}	Mm^3 m^3/s)				50		38.8 (250)
Halaspuszta (Overflow Q_{max}	Mm^3 m^3/s)	50			35 (200)		
Kutas (Overflow Q_{max}	Mm^3 m^3/s)		25 (300)				
Cutting flood peaks: $H_{max,\ reconstructed} - H_{max,\ observed}$, cm							
Fehér-Körös	Gyula	28	22	34		83	9
Fekete-Körös	Ant				7		
	Sarkad				8	38	
	Remete	7	22	32	7	84	16
Kettős-Körös	Doboz					69	10
	Békés	59		58		61	16
	Köröstarcsa					74	10
Berettyó	Szeghalom				38	0	0
Sebes-Körös	Fokihíd				23	0	0
	Körösladány				22	30	81
Hármas-Körös	Gyoma					50	64
	Szarvas					51	
	Kunszentmárton					24	
Max. relative difference of water levels: $H_{reconst} - H_{observed}$, cm							
Fehér-Körös	Gyula	150	100	220		250	13
Fekete-Körös	Ant					150	
	Sarkad					265	
	Remete	30	75	230		250	27
Kettős-Körös	Doboz						34
	Békés			274	120	150	73
	Mezőberény				160		0
	Köröstarcsa						50
Berettyó	Szeghalom		105				0
Sebes-Körös	Fokihíd		100				
	Körösladány		75				86
Hármas-Körös	Gyoma						110

* Total derived flow rate at two openings Q_{max}

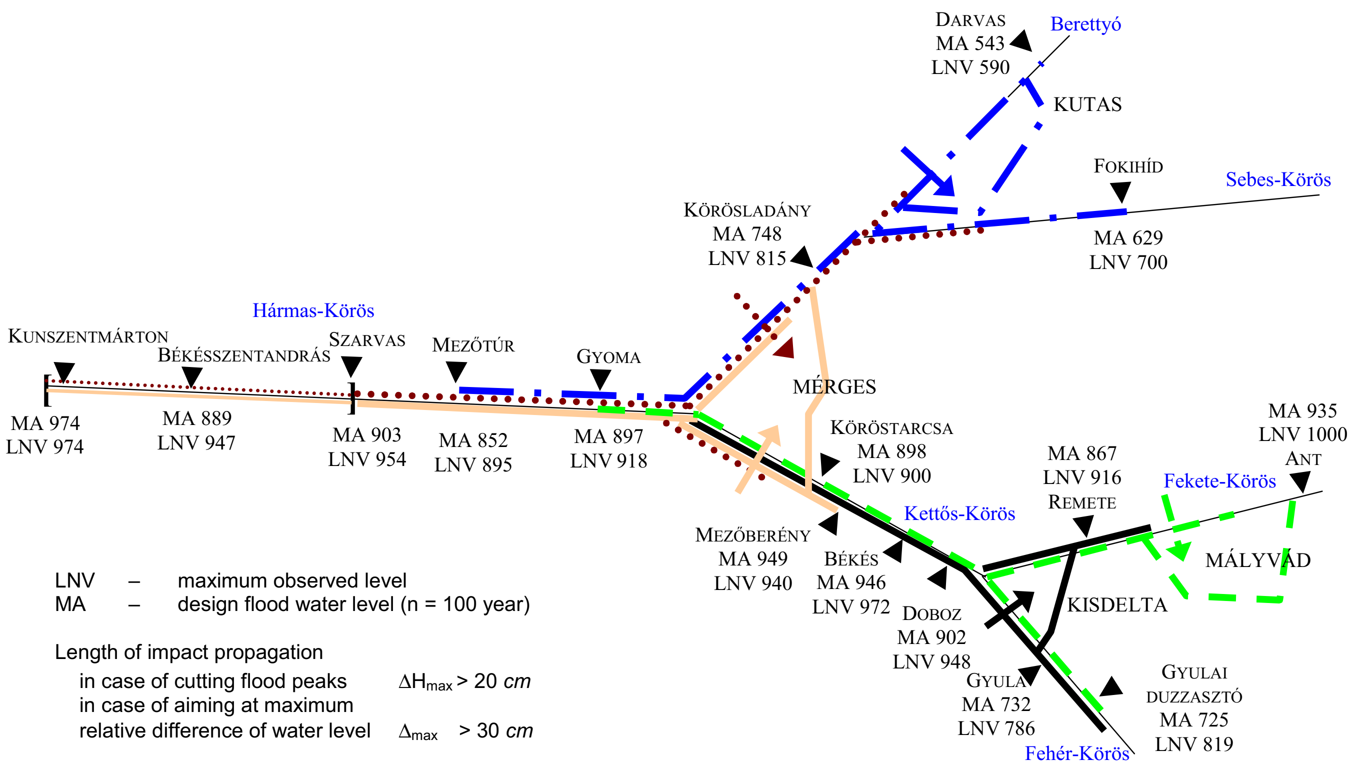
DARVAS
MA 543
LNV 590
Berettyó
KUTAS
FOKIHÍD
Sebes-Körös
MA 629
LNV 700
KÖRÖSLADÁNY
MA 748
LNV 815
Hármas-Körös
KUNSZENTMÁRTON
BÉKÉSSZENTANDRÁS
SZARVAS
MEZŐTÚR
GYOMA
MÉRGES
MA 974
LNV 974
MA 889
LNV 947
MA 903
LNV 954
MA 852
LNV 895
MA 897
LNV 918
KÖRÖSTARCSA
MA 898
LNV 900
MA 935
LNV 1000
ANT
MA 867
LNV 916
REMETE
Fekete-Körös
Kettős-Körös
MEZŐBERÉNY
MA 949
LNV 940
BÉKÉS
MA 946
LNV 972
MÁLYVÁD
DOBOZ
MA 902
LNV 948
KISDELTA
GYULA
MA 732
LNV 786
GYULAI DUZZASZTÓ
MA 725
LNV 819
Fehér-Körös
LNV – maximum observed level
MA – design flood water level (n = 100 year)
Length of impact propagation
in case of cutting flood peaks ΔHmax > 20 cm
in case of aiming at maximum
relative difference of water level Δmax > 30 cm

Fig. 4.

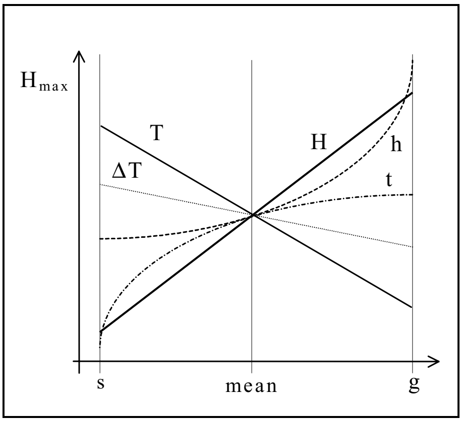

Fig. 5.

The improvement of controlling the operation of the emergency reservoirs in the Körös Valley is under way. The first essential step consists of formulating flooding scenarios for the four emergency reservoirs and studying the impacts of typical flood waves.

Analysis and assessment of the impacts of past emergency storage events are alone inadequate to draw up such scenarios. Owing to the complexity of the circumstances warranting emergency storage, the impacts of flooding, the complexity of, and the ramified interactions between, the hydraulic phenomena in the river network, simple situations must be studied at the outset. **Hydraulic situations** developing under simplified operating and boundary conditions (a single reservoir is only flooded, the volume and duration of the arriving flood wave are known, etc.) **have been simulated on a computer**, using a linear numerical model (RÁTKY 1988). **From the data thus obtained conclusions of general validity have been arrived at**, which were then **presented in simple tables and graphics** to assist in decision making under emergency conditions.

Great care has been devoted to abstracting and simplifying the aforementioned complicated phenomenon into a form accessible to numerical simulation without introducing an inadmissible error. The logic adopted in formulating the numerical model used for simulating the various storage alternatives will be described briefly with the intent of demonstrating the inevitable simplifications and providing guidance to assessing the reliability of the results. For the simulation studies the **following simplifying assumptions** have been introduced:

⇒ Emergency storage is resorted to because the flood predicted is higher than that which the defences can safely withstand, so that *lowering the peak is the primary aim*;

⇒ *A single reservoir is only flooded* at a time;

⇒ With guidance by the estimated influence range (*Fig.* 4), the *studied river system is demarcated*, so that

- when flooding the Mályvád, or the Kisdelta reservoir, the Sebes-Körös is studied down to the mouth of the Berettyó, without extending simulation to the Berettyó,
- when flooding the Mérges reservoir, the impacts of the Fekete Körös and Fehér Körös, further of the Berettyó and Sebes Körös are considered in combination,
- when studying the Kutas reservoir the impacts of the Fekete Körös and Fehér Körös are taken into account in combination.

As a result of these simplifications, even the most complicated river system consists of a main stem and two principal tributaries alone. The model is conceived so that the point of diversion is invariably on the main stem.

⇒ For the sake of comparison, the decision to flood a reservoir is based invariably on the assessment of *the same set of criteria.*

⇒ It is assumed that the flow enters the river system through the most upstream model cross section only and *it is described by the following parameters*:

1. **H** – the peak stage of the flood wave on the main stem (in the entrance cross section to the model)
2. **T** or **V** – the duration or volume conveyed above a given alert/stage (to characterise the shape of the flood hydrograph)
3. $\mathbf{h}_1$ – the peak stage of the flood wave arriving on ‘tributary 1’
4. $\mathbf{t}_1$ or $\mathbf{v}_1$ – the duration or volume conveyed above a given flood alert level in the most upstream cross section of ‘tributary 1’
5. $\Delta\mathbf{T}$ – the time lag between the peaks on the main stem and ‘tributary 1’
6. $\mathbf{h}_2$ – the peak stage of the flood wave arriving on ‘tributary 2’
7. $\mathbf{t}_2$ or $\mathbf{v}_2$ – the duration or volume conveyed above a given flood alert level in the most upstream cross section of ‘tributary 2’
8. $\Delta\mathbf{T}$ – the time lag between the peaks on the main stem and on ‘tributary 2’.

It is assumed that eight parameters are enough to describe a hydrologic situation sufficiently and that a change of a single one creates a new hydrologic situation. In a study aimed at exploring the influence of any parameter (sensitivity test), at least three parameter values (low, mean and high) must be entered. In this case the **number of situations** to be simulated for a single reservoir **would be over ten thousand**. For this reason further simplifications are inevitable:

> The flood wave arriving on the tributary farther away from the reservoir studied will be entered with a ‘critical’ value, the three parameters remaining constant while studying the reservoir.

As a result of this simplification ‘**only**’ **five parameters remain** to be tested for their effect.

Professionals with local experience must be relied upon for the hydrological inputs. These will take the form of simple standard flood hydrographs abstracted from the flood waves actually observed during past emergency storage events. These will be referred to subsequently as *design standard flood waves* for reservoir flooding. Evidently, no data (e.g. water levels) are available on the travel thereof down the river system. As a consequence thereof, for estimating (comparing) the impacts of flooding a reservoir, the hydraulic situation without flooding must also be simulated for these design standard flood waves. These simulations yield the data which are obtained in actual situation from the hydrologic forecasts.

Each parameter is entered with a *mean, high and low value*.

The resulting standards flood waves cover a band. A study of all potential alternatives formed by assigning three values to each of the five parameters would mean close to two thousand simulation runs. Further simplifications had to be introduced.

Using the mean value of each parameter a '*reference alternative*' was produced in which a single parameter was only varied at a time. In this way eleven independent alternatives were produced for each reservoir. Remembering, that the same hydrologic situations must be simulated also for the case of no-flooding, the *number of simulation runs becomes 22 for one and 88 for the four emergency flood storage reservoirs.*

The practical value of the results depends basically on the format in which they are presented, on their ready accessibility and fast use. In the case of a similarly large number of alternatives it is especially important to clear the main outlines of the format in which the results will be processed. **One of the potential formats** of a simulation study planned for a reservoir is presented in *Table3*. One line thereof shows for a critical hydrologic situation the highest stages at some key gauging stations on the river studied without and with flooding the reservoir, further the main values found therefrom:

$H_{\max}$ – the highest water level without flooding the reservoir,

$\Delta H_{\max}$ – the difference between the highest water levels without and with flooding the reservoir,

ΔH_{MA} – the level above the design flood level with the emergency reservoir flooded.

The lines, in groups of three, indicate the water levels corresponding to the three values of a particular parameter.

The only departure from the notations used so far consists of the omission of the subscripts $_1$ and $_2$, the changes on a single tributary being only considered, while the subscripts m, g and s refer to the middle, top and lower range of the band representing the hydrologic situation on which the flooding decision is based. The values found with the 'mean parameters' have been entered repeatedly to facilitate comparisons.

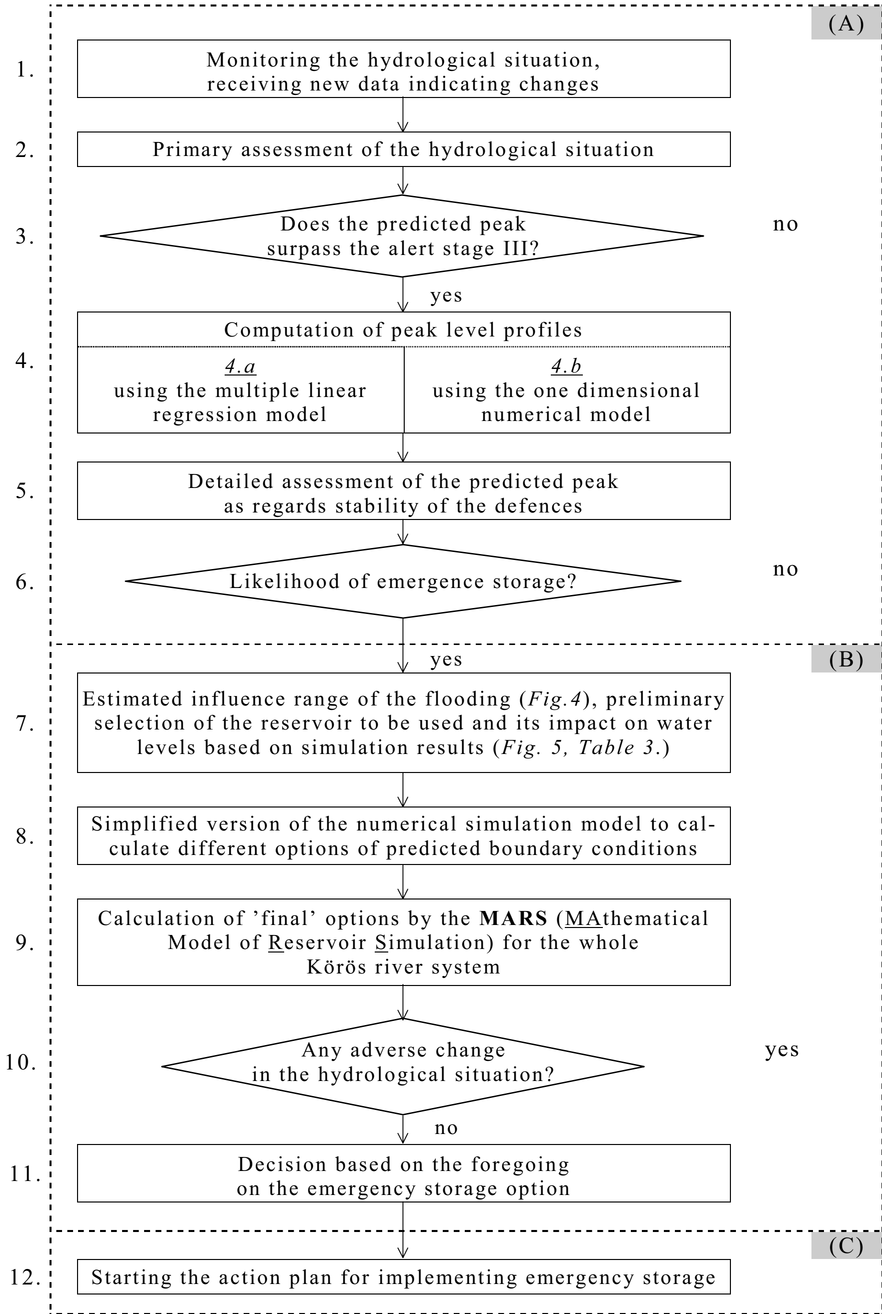
(A)
1.
Monitoring the hydrological situation,
receiving new data indicating changes
2.
Primary assessment of the hydrological situation
3.
Does the predicted peak
surpass the alert stage III?
no
yes
4.
Computation of peak level profiles
4.a
using the multiple linear
regression model
4.b
using the one dimensional
numerical model
5.
Detailed assessment of the predicted peak
as regards stability of the defences
6.
Likelihood of emergence storage?
no
yes
(B)
7.
Estimated influence range of the flooding (Fig.4), preliminary
selection of the reservoir to be used and its impact on water
levels based on simulation results (Fig. 5, Table 3.)
8.
Simplified version of the numerical simulation model to cal-
culate different options of predicted boundary conditions
9.
Calculation of 'final' options by the MARS (MAthematical
Model of Reservoir Simulation) for the whole
Körös river system
10.
Any adverse change
in the hydrological situation?
yes
no
11.
Decision based on the foregoing
on the emergency storage option
(C)
12.
Starting the action plan for implementing emergency storage

Fig. 6.

Table 3. Results of the sensitivity analysis (scheme)

					Ant		Remete		Varsánd		...		.. ..	...		...	
Flood parameters					H_{max} ΔH_{max}	ΔH_{max}	H_{max} ΔH_{max}	ΔH_{max}	H_{max} ΔH_{max}	ΔH_{max}	H_{max} ΔH_{max}	ΔH_{max}	.. ..	H_{max} ΔH_{max}	ΔH_{max}	H_{max} ΔH_{max}	ΔH_{max}
cm	h	cm	h	h	cm	cm	cm	cm	cm	cm	cm	cm	.. ..	cm	cm	cm	cm
$\mathbf{H}_g$					786 23	38							.. ..				
$\mathbf{H}_m$	T_m	h_m	t_m	ΔT_m									.. ..				
$\mathbf{H}_s$													.. ..				
	$\mathbf{T}_g$												.. ..				
H_m	$\mathbf{T}_m$	h_m	t_m	ΔT_m									.. ..				
	$\mathbf{T}_s$												.. ..				
		$\mathbf{h}_g$											.. ..				
H_m	T_m	$\mathbf{h}_m$	t_m	ΔT_m									.. ..				
		$\mathbf{h}_s$											.. ..				
			$\mathbf{t}_g$										.. ..				
H_m	T_m	h_m	$\mathbf{t}_m$	ΔT_m									.. ..				
			$\mathbf{t}_s$										.. ..				
				$\Delta \mathbf{T}_g$									.. ..				
H_m	T_m	h_m	t_m	$\Delta \mathbf{T}_m$									.. ..				
				$\Delta \mathbf{T}_s$									.. ..				

The influence of the hydrologic parameters H, T, h, ΔT and t on water levels has been plotted schematically, *Fig.* 5 illustrates the **influence on the highest stage**. Similar graphs can be plotted also for the estimated hydraulic parameters $\Delta H_{\max}$ and ΔH_{MA}.

A function plotted for a parameter from three values cannot be expected to describe the relationship within a broad range, though it illustrates well the trend of change and the relative influence on water levels of the various parameters. Similar graphs can be plotted for each of the computation cross sections, but evidently only the key stations used in deciding upon flooding are worth the effort.

3.5. Perfection of the Conceptual Model of Operation Control for the Emergency Reservoirs in the Körös Valley

The aforementioned conceptual model of emergency reservoir operation comprises the tasks needed to attain the set goal, such as the hydrologic and hydraulic computations, gathering the essential decision support data, establishment of the decisions criteria, the actual activities of flooding the reservoir, the organisational functions, the logical relations between these tasks and the optimal sequence of carrying out these (block diagrams, pert diagram, action plan). The term '*conceptual model*' is unavoidable, in that the activities preceding a flooding operation are extremely ramified, as a consequence of which no practical model of the desired accuracy and covering all details is yet available. The importance of the conceptual model, its correctness in the main issues have been demonstrated by the experiences gained over the recent decades. This approach was adopted in compiling the '*Maintenance and operating instructions for the flood emergency reservoirs in the Körös Valley*' (KÖVIZIG 1997) and in using several other reservoirs in Hungary for the purposes of emergency storage since 1997 (Mályvád 1980, 1981, 1995; Mérges 1980, 1995; Halaspuszta 1980; Lajta 1997).

The proposed improvements in operation control will be outlined subsequently and applied to the *Mályvád, Mérges and Kisdelta reservoirs*. The activities (computations) needed for operation control will be grouped into the three main blocks described in Section 1.5 and visualised in the **schematic block diagram** *of Fig.* 6.

3.5.1. Forecasting and Analysis of the Natural Hydrological Situation in the River System

The **multivariate linear regression model** used presently in emergency situations for predicting the levels and times of peaking yields generally a sufficiently successful forecast of the hydrologic events. The term '**sufficiently successful**' means that the method is accurate enough under abnormal conditions as well. Nevertheless, further studies into the theoretical and practical possibilities of perfection are continued.

The main **advantages** of the multivariate linear regression model are as follows:

- The widely known method has been *tested and* **found successful** *over decades*. (This is considered an advantage, in that the model exists not only on paper but functions - regardless of all theoretical objections – continuously on a computer.)
- It is *simple enough* to require no data, manpower and equipment beyond that available even under flood emergency conditions.

The **drawbacks** of the model may be listed as follows:

- The model relies on the statistics of past events – which may also be an advantage – though there are very few observation data in the *extreme range* and even these samples are often influenced by levee failures, or flooding of emergency reservoirs.
- Owing to the rare occurrence, the data may originate from past periods in which the runoff conditions were less affected by human activities in the catchment.
- The development opportunities are very limited, the *accuracy can hardly be improved*, especially in the range of greatest interest, in that of extreme flood waves. This is attributable primarily to the theoretical shortcomings of the model. It can be demonstrated by hydraulic methods that the relationship between the peak stages on two gauging stations is not necessarily a simple, linear one.

Methods resting on firmer hydraulic, theoretical foundations are known, which appear more advantageous in the longer perspective, but which are not yet in a form suited to operative applications. *Regression models will remain difficult* to replace and their refinement, improvement (e.g. revision of the exponents to allow for more recent data and for those of the gauging stations in Romania) are believed worthwhile and necessary, regardless of their theoretical shortcomings.

3.5.2. Hydraulic Analysis of Emergency Storage Alternatives and their Impacts

The hydraulic phenomena triggered by flooding an emergency reservoir may be classified broadly into three groups:

- the local hydraulic phenomena between the river and the reservoir (overfall, variations in the head and tailwater),
- the changes in flow conditions along the river system, and
- the flow phenomena within the reservoir.

The hydraulic phenomena *within the reservoir and between the reservoir and the river* were simulated first by Miklós SZALAY in a manner suited to practical use (SZALAY 1975; NÉMETHY–SZALAY 1977). The method is still applicable, minor refinements in response to recent advances in computer technology being alone considered necessary.

Improvements are essential in simulating the direct impact of emergency storage on the river, the propagation of the impacts along the river system and in modelling the runoff phenomena, based on **one-dimensional, non-steady, deterministic theory and numerical approach**. The correct theoretical foundations – on (1D) level – and successful practical applications of the method (RÁTKY 1988; CSOMA–KOZÁK–RÁTKY 1988; RÁTKY–CSOMA 1996; VITUKI Consult Co. 1997; RÁTKY 1997a) demonstrate its suitability for such applications. The main obstacle to its introduction in the control of emergency reservoir is that the model has not been adapted yet to operative conditions.

This model should be developed to a level allowing its application **to controlling** the operation of emergency reservoirs as **part of the daily routine functions**. For this end an accurate list of the activities needed to flood an emergency reservoir must be compiled, the necessary data and the methods of their acquisition must be identified, the suitable software must be selected, the formats of displaying and assessing the data must be adopted and the time of each activity must be estimated. The resulting schedule must be incorporated into the operating routine. In this way the computation and assessment processes would assist decision making on flooding an emergency reservoir. This in turn, would reduce the number of 'unforeseen situations' and of the improvised decisions prompted thereby, leaving more time for the assessment of inevitable, unpredictable events and important decisions on these.

The operation control model of emergency reservoirs referred to in Section 1.5 (SZLÁVIK 1983) **has been refined** along these lines. An updated, supplemented version of the operation control model is shown in *Fig. 6*. The steps expanded in their contents will only be detailed subsequently.

Step 4.a: The generation of the longitudinal profile of the probable peak stages by the multivariate, linear regression model.

Step 4.b: **Simulation using the 1D numerical model** *of the water levels developing without flooding any of the reservoirs*. The result displays the flood peaks in any computation cross section within the river system for the predicted hydrologic load arriving through the upstream boundary section.

Step 7: The four emergency reservoirs in the Körös Valley can be flooded in 15 different combinations. The number of potential combinations can be narrowed down on the basis of the *estimated influence range of flooding shown in Fig. 4* (RÁTKY 1997b). Using thereafter the results of the earlier studies on the emergency storage scenarios (*Fig. 5*, *Table 3*), the impacts of the potential flooding scenario on the river system can be predicted.

Step 8: Using an *operative version of the numerical simulation model* applied to determining the impacts of standard flood waves with the boundary conditions actually predicted, the results of the alternatives computed with the standard flood waves are refined. The computations on a particular reservoir can be performed for several flooding alternatives.

Step 9: The impacts of flooding one, or several emergency reservoirs on the influenced sections of the entire Körös system are found with the help of the *TÁMASZ (Hungarian mosaic for* **Ma***thematical* **R***eservoir* **S***imulation, in English* **MARS**) *model*. Thanks to the estimated influence ranges, scenarios and the results of preparatory computations with actual boundary conditions, a few alternatives remain to be analysed. The results include the balanced, or storage level, the volume to be impounded, the times of peaking at the key stations along the Körös sections influenced, the elevation of the peak levels with and without emergency storage, further relative to the design flood levels and the bridge clearances.

3.5.3. Action Plan of Emergency Storage

The 'Action Plan' prepared as part of the operating instructions in 1977 has proved successful on several occasions. The Mályvád reservoir was flooded in 1980, 1981 and 1995, the Mérges reservoir in 1995 according to this action plan (SZLÁVIK–GALBÁTS–KISS 1996). Thanks to the sound theoretical foundations (network plan, optimal strategy, critical path) on which the plan was developed for practical purposes, the duration of the critical path still applies. This is due also to the fact that as foreseen at the time of its formulation, the action plan has been reviewed regularly, at least every second or third year and modified if necessary to take account of new technologies (levee cutting, emptying, closure, etc.) (SZLÁVIK 1980). The experiences have been analysed after every flooding since 1980 (SZLÁVIK 1980; KHVM 1996). It is of interest to note, that the actual time required for preparations was practically the same as envisaged in the action plan.

4. Summary

As part of the methodical flood control development efforts, the method of emergency flood storage has received special attention. In the article '*Particular features of the floods on the Fekete Körös and Fehér Körös and flood control development options*' (SZLÁVIK–RÁTKY 1999) cases of actual, or contemplated emergency storage were described, demonstrating that these methods may be powerful tools in future flood emergency operations. Better understanding, forecasting and estimation of the hydraulic-hydrological processes involved in emergency storage are essential to the successful operation of these reservoirs. The progress made so far in this field has been reviewed here and by a synthesis of the past successful experiences specific proposals are made for improving some of the actions involved, indicating also the avenues of further development.

The flood situations in which emergency storage may be effective were described at the outset. The 'conventional' method of estimating the flood volume to be stored, i.e., the required storage space was outlined. The 'optimal' time of reservoir opening was defined and the difficulties encountered in determining it were dealt with. The reduction of flood peaks in the river system was analysed and indices were presented to quantify the effects, the effectiveness of impoundment (*Figs. 1, 2* and *3*) and to compare the storage alternatives (*Table 1*). A model of the action plan of controlling the operation and impoundment of emergency reservoirs was introduced as one which proved functioning well under actual emergency conditions already. The key hydrological-hydraulic parameters of the emergency storage events in Hungary since 1968 were compiled in *Table 2*.

With a view of taking account of the possibilities of perfecting operation control, the hydraulics of impoundment was analysed first. A simple approximation of the drawdown curve, based on the greatest drawdown was proposed. All available measurements, design and computation data were combined to determine the influence ranges shown in *Fig. 4* for the impoundment of a single reservoir. Besides the specific improvements which can be introduced already, further possibilities were outlined. A method consisting of a sequence of logical steps was outlined by which this highly complex phenomenon can be simplified enough to become accessible to analytical treatment and the results can be presented in a clear, readily overviewed form to assist decision making in emergency situations (*Fig. 5* and *Table 3*). The method is described in the section on the analysis of the parameters of typical flood waves (sensitivity analysis). As a summary of the studies, the improvements developed already and the proposed further refinements, a perfected version of the theoretical model of flood emergency reservoir operation was presented. The components of the model, their interrelations were illustrated in the block scheme of *Fig. 6*.

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