INTERNATIONAL INSTITUTE FOR HYDRAULIC AND ENVIRONMENTAL ENGINEERING

DELFT NETHERLANDS

On optimisation of sediment exclusion measures at intakes

Arie Setiadi Moerwanto

M.Sc. Thesis H.H. 44

1990

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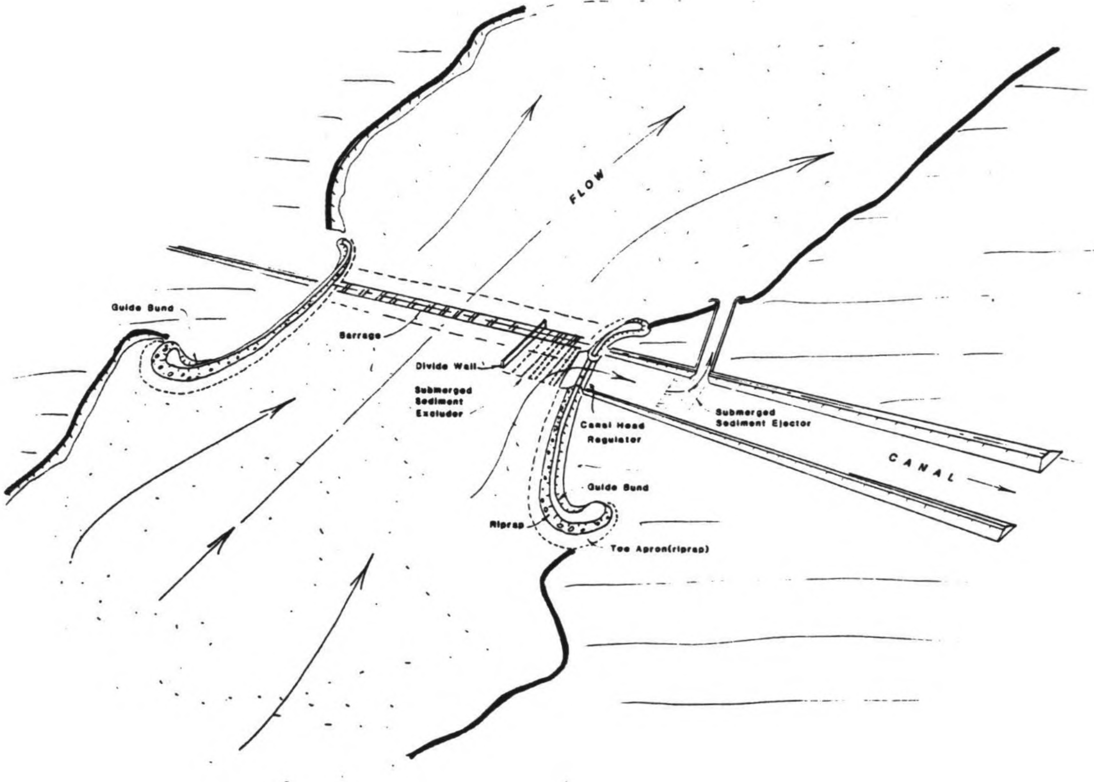

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LIST OF MAIN SYMBOLS

Symbol	Definition	Dimension
a	dimensionless bed layer thickness	[-]
	reference level above the bed ($\approx k_s$)	[m]
A	coefficient in Ackers and White formula	[-]
	cross section area	[m²]
A_1	dimensionless variable in Coles´ flow velocity profile	[-]
A_2	dimensionless variable in Coles´ flow velocity profile	[-]
B	width of a structure	[m]
c	sediment concentration by mass.	[ppm]
	coefficient in Ackers and White formula	[-]
c_j	concentration by mass of j-th fraction of sediment	[ppm]
c_{ja}	concentration by mass of j-th fraction of sediment at $z = ah$	[ppm]
C	Chézy coefficient	[$m^{1/2}/s$]
D_n	grain size at which n% of the sediment is smaller	[m]
D_{gr}	dimensionless grain size	[-]
$D*$	particle size parameter $D_{50} \cdot [\frac{g}{\nu^2}]^{1/3}$	[-]
Fr	Froude number	[-]
F_{gr}	particle mobility	[-]
g	acceleration of gravity	[m/s^2]
h	water depth	[m]
h'	defined by $\frac{u*'^2}{g\ I}$	[m]
I	energy slope	[-]
I_o	bottom slope	[-]
k_s	effective roughness height	[m]
k_N	Nikuradse roughness	[m]
l	lenght of vortex tube	[m]
L	lenght of a structure	[m]
m	coefficient in Ackers and White formula	[-]
n	coefficient in Ackers and White formula	[-]
p	overall trapping efficiency	[-]
p_i	trapping efficiency for j-th fraction of sediment	[-]
q	unit discharge	[m^3/ms]
Q	discharge	[m^3/s]
R	extraction ratio	[-]
	hydraulic radius	[m]

LIST OF MAIN SYMBOLS (continued)

Symbol	Definition	Dimension
s	ratio of settling velocity of sediment to shear velocity	[-]
	sediment transport in a unit of width (grain)	[m^3/ms]
S	sediment transport (volume of grain)	[m^3/s]
T	bed-shear stress parameter	[-]
u	mean velocity	[m/s]
u*	shear velocity	[m/s]
u*'	shear velocity due to grain	[m/s]
w_s	fall velocity of the particle	[m/s]
w	local vertical flow velocity	[m/s]
Y	Shield's entrainment function defined by $\frac{u*^2}{[g\ D\ (\Delta - 1)]}$	[-]
Y'	entrainment function defined by $\frac{u*'^2}{[gD\ (\Delta - 1)]}$	[-]
z	bed level	[m]
	height above bed of channel	[m]
α	dimensionless height above bed	[-]
Δ	relative density under water	[-]
ε	momentum transfer coefficient	[m^2/s]
ε_s	sediment mixing coefficient	[m^2/s]
κ	Von Karman's constant	[-]
λ	inverse friction factor	[-]
ν	kinematic viscosity	[m^2/s]
ρ_s	density of water and sediment mixture	[kg/m^3]
ρ	density of water	[kg/m^3]
τ_w	critical bed-shear stress	[N/m^2]
τ_o	effective bed-shear stress	[N/m^2]

Suffices:

0	condition upstream of a structure
1	condition downstream of a structure
j	an integer defining sediment fraction or grid position
n	an integer defining time level

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Summary

Sediment transported by natural stream flow has often presented the irrigation and hydropower plant designer with major problems, and measures for solving the sediment problems have been proposed. Most of the proposed sediment exclusion measures have been developed based on certain basic hydraulic principles and experiences gained from field observations of typical structures. This qualitative approach together with hydraulic scale model studies have formed the basis of the design for several of the sediment exclusion devices.

In recent years, several quantitative methods of design based on experimental and analytical approach have been proposed for some of the sediment exclusion devices. The availability of these quantitative design methods allows for setting up a comprehensive model on optimatisation of sediment exclusion devices. Such a model should allow to select an optimal combination of devices and operational rules to reduce sediment intake, considering also other aspects like irrigation and other system requirements.

The development of the comprehensive model is the main objective of this study. Four commonly used sediment exclusion devices, namely tunnel excluder, tunnel extractor, vortex tube and sandtrap (for which quantitative methods are available) are included. The evaluation is based on the trapping efficiency of sediment exclusion devices and the quantity of used water.

The performance of a designed model to predict the efficiency of each sediment exclusion devices was evaluated. For this purpose, field data and an existing (more advanced) model were used for verifying. The evaluations were carried out by considering a range of parameters which are representative for the conditions met in most irrigation schemes and hydropower plants. The results of the evaluation show that the designed model gives a good result as far the trapping efficiency is considered.

To test the developed model for its ability, it was used in optimising the design and operation of the proposed sediment exclusion devices of the Ye-U Irrigation Project in Burma. The test results show that the model is handy and capable for this purpose. The results can be included in water management.

Finally some proposals for a further development of the model are given.

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1. Introduction

1.1 General

Sediment transported by natural stream flow has often presented the irrigation system and power plant designer with major problems. The entering of sediment through an intake structure for an irrigation system or for a hydropower plant has given rise to the following problems:

(i) The deposition of sediment in the system which results in the partial blocking of canals. It will come to a reduction in flow capacity and requires costly maintenance operations.
(ii) The abrasive damages caused by the continuous rapid moving sediment in water to pumps and turbines.
(iii) The heightening of pady field elevation and reducing in fertility of the soil, due to the deposition of sand on the irrigated areas.

The sediment must be removed from the water at the canal intake or transported through the canal system with a minimum of accumulation within the canal prism and structures. Complete elimination of the sediment at the diversion point is generally impractical and would be too costly. A combination of its control at headworks and design of the canal hydraulics to minimise deposition through its length, can be used to provide practical solutions to the problems.

In general, three approaches can be taken in solving these problems (Vanoni, 1977 and Avery, 1981):

(i) Only direct water into the canal intake and return the sediment to the stream.
(ii) Design the canal system hydraulically so that the water with its sediment will be transported out onto the land with a minimum of sediment deposited in the canal.
(iii) Design the canal headworks to direct as little sediment as practicable into the system and remove the sediment deposited by the most inexpensive method available.

Most of the proposed sediment exclusion measures have been developed based on certain basic hydraulic principles and experiences gained from field observations of typical structures. This qualitative approach, together with hydraulic scale model studies, have formed the basis of design for several of the sediment exclusion devices.

In recent years several quantitative methods of design based on experimental and analytical approaches, have been proposed for some of the sediment exclusion structures, for example tunnel excluders, tunnel extractors, vortex tubes and sand traps. The availability of quantitative design method allows for an analytical assessment of the most appropriate device or a combination of devices at the preliminary design stage. It is also possible to establish an optimal operational rule for each of the devices when used singly or in series with other devices.

1.2 Problem Definition

The optimisation of sediment exclusion measures should take into account five interrelated aspects such as the trapping efficiency of sediment, the river and transported sediment characteristics, the available hydraulic head and water discharge (energy) for the system, the simplicity of operation and financial considerations.

Whereas quantitative design methods have been developed for some of the devices, a decision on an optimal solution is still tedious and time-consuming because of the numerous options and possibilities that have to be considered. It will be useful to have a mathematical model that can be used for optimising the selection of at least some sediment-exclusion devices.

1.3 Objective of Study

Nowadays, an extensive river-basin simulation model has been set up. The model is used in relation to typical water resources planning and operation problems in the basin. It is useful to have a simple model concerning the optimisation of a solution for the sediment-exclusion problem that can give some detailed data which can also be used within the frame work of river-basin planning. The model should allow the decision maker to consider a detailed operation concerning the water and sediment management at the intake, because this operation would influence the whole river basin planning. Otherwise the water resources planning cannot be applied in the field due to a sedimentation problem. A general overview of how the resulted data of the designed model can be used further, will be described in Chapter 5.

The development of a comprehensive mathematical model that allows the optimisation of solutions for a sediment-exclusion problem at a particular site, is the main objective of this study.
In order to achieve the main objective, an evaluation of the performance of each sediment-exclusion device was carried out. In this study only the four commonly used sediment-exclusion devices, namely tunnel excluder, tunnel extractor, vortex tube and sandtrap are used. In addition, it must be said that only sandtrap-sub model, canal-capacity model and optimisation model with its supporting model, are newly designed in the present study. The basic theory and verification of trapping efficiency of vortex tubes and tunnel extractors has been developed by the Wallingford Hydraulics Research (Sanmuganathan, 1976 and Atkinson, 1984). In this study we will only consider some additional schematisation and assumptions that should be taken for a tunnel excluder, tunnel extractors and vortex tubes, so as to allow the use of the theory in the optimisation model with a certain accuracy.

In this stage, the evaluation will only be based on the trapping efficiency of sediment-exclusion devices and the quantity of used water. The designed comprehensive mathematical model should be used mainly in the design stage.

1.4 Structure of the Report

To achieve a comprehensible report, it will be systematised in the following way:

(i) In chapter two, a review of earlier work concerning sediment-exclusion measures will be described. The descriptions are written in a way to give an insight into basic principles, design criteria and trapping efficiency of each type of sediment-exclusion devices.

(ii) In chapter three the basic idea, scope and organisation of the designed model on optimisation of sediment-exclusion devices will be described. Detailed descriptions of mathematical modelling of each device, assumptions, model verification with field data, data needed and boundary condition will also be given.

(iii) Use of the designed model for analysing a problem of sedimentation in an irrigation system are described in chapter four.

(iv) Other aspects that should be considered as well in the water and sediment management at intakes, are described in a qualitative way in Chapter five. Some proposals of the use of output data will be described also.

(v) The conclusion and proposed study on model development, based on the present study, are described in Chapter six.

2. Sediment Exclusion Measures

2.1 General

Complete elimination of the sediment at the diversion point is generally impractical and not considered because it would be too costly. Part of the sediment carried in irrigation water, is beneficial in sealing the canal and also helpful in improving the texture and fertility of the irrigated land. Considering the field experiences, general techniques that are commonly used to overcome the entering sediment problems (Vanoni, 1977 and Avery, 1981) are:

(i) Positioning of the intake.
Selecting a point of diversion and angle of diversion are the most important aspect of positioning.

(ii) Controlling the approach flow to the intake.
Some measures that are commonly used in this case are river-training works, guiding walls and gate operation.

(iii) Using sediment-exclusion devices before the canal intake. Tunnel excluder and skimming-wall are commonly used.

(iv) Removing sediment after it has entered the canal system. Tunnel extractors, vortex tube extractors and sandtrap should be used in these measures.

(v) Designing the canal system hydraulically so that the water with its sediment will be transported onto land with a minimum of sediment deposited in the canal.

Detailed explanation and principles involved in each technique are described in the following sections.

2.2 Selection of Point of Diversion

Careful selection of the point where the water is to be diverted from a stream, is an important factor in the reduction of the quantity of sediment taken into canal. In general, the outside or concave side of the curvature has proven to be the best location (Jogklekar, et al., 1950 and Vanoni, 1977). Due to the spiral flow, the heavy bed load is swept towards the inside of the curve and the sediment concentration at this point is lower than at other points in the stream. A schematisation of this flow type is shown in figure 2.1. This measure of sediment exclusion has been effective universally especially for rivers where the sediment transport is predominantly bed load.

Mosonyi (1987) reviewed the work of Habermaas, Grishin and Tison who have carried out a series of experiments to get a better understanding of the selection of the site on a concave curve. In the experiments, a constant discharge was maintained in the diversion canal but they were varied in the original course. Later, Mosonyi formulated ten principles governing the selection of the intake site. The four most important principles are as follows (John, 1989):

(i) The intake should be located, wherever possible, on the concave side of a curved strech of a river which is well-established and has stable banks.

(ii) Efficiency of the intake in preventing sedimentation increases with the sharpness of the bend.

(iii) The amount of the bed load transported into the canal decreases as the ratio of the total discharge to the amount diverted increases.

(iv) Intakes are most favourably located along the downstream reach of the curve near the end. In the absence of a scale model (Vanoni, 1977), the point of diversion should be located in the region from 2/3 to 3/4 of the length of the curve measured from the beginning of the curvature. This is to ensure that the spiral motion of the flow is fully developed.

The experiments conducted in the Irrigation Research Institute of Pakistan (Mushtaq Ahmad et al., 1960) for sediment exclusion from the right and left bank off-taking canals at Kotri, Taunsa and Gudu headworks have shown that sediment entering the canals off-taking from outside of the bend is 1/3 to 1/10 of that passing into the canals located on the inside of the curve.

The location of intake will also vary due to several other factors such as flow condition, characteristics of sediment transport, geometry of the river section, location of the proposed irrigation system and field conditions. Due to the varying conditions, no quantitative method has been developed for selecting the best site of the intake. The most appropriate location of the intake can be determined by a scale model study.

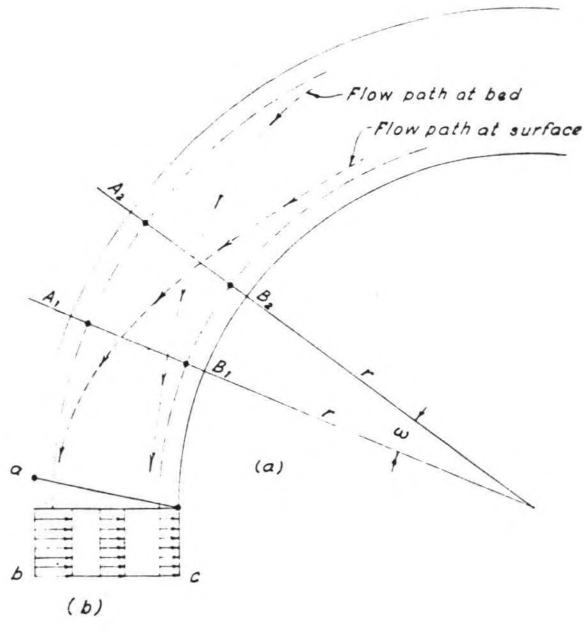
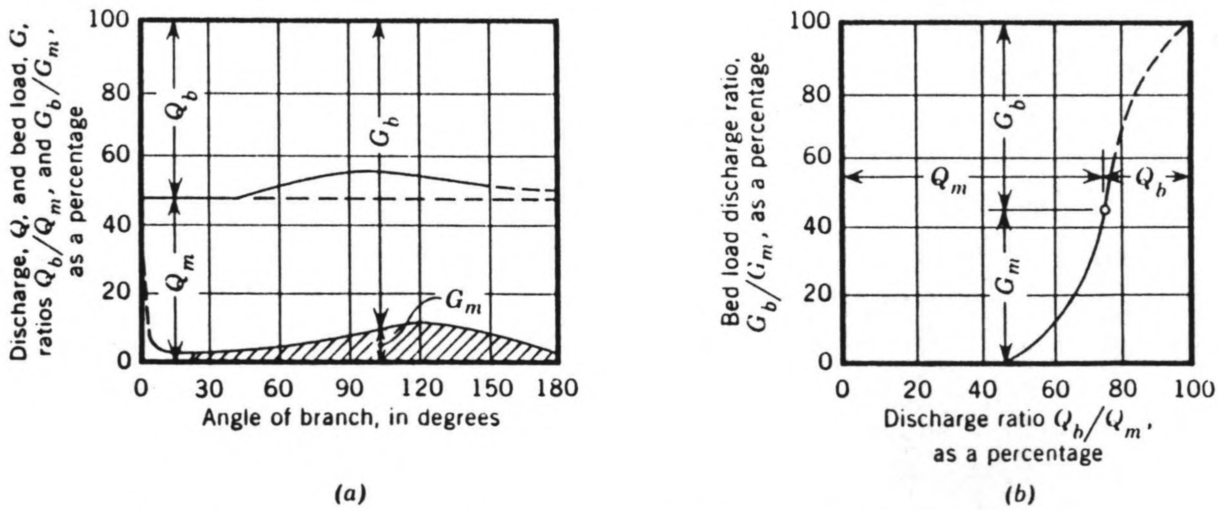


Figure 2.1 Schematic Diagram of Flow in Curved Channel.



(a) Relation between division of bed load discharge and angle of branch. Subscripts *b* and *m* apply to branch and main channel, respectively (Schoklitsch, 1937).

(b) Relation between divisions of bed load and division of discharge. Plotted example for angle of branch of 30° as determined by H. Bulle (1926) and reported by Schoklitsch (1937).

Figure 2.2 Relation between Division of bedload discharge, Division of discharge and Angle of branch.

(After Vanoni, 1977)

2.3 **Angle of Diversion**

The angle of deflection between the direction of flow in the stream and the direction of flow in the intake canal is generally called the angle of diversion. Any diversion at an angle with the flow in the parent stream channel becomes, in effect a curve with a curvature opposite to that of the parent channel. The higher-velocity surface water requires a greater force to turn it than the slower moving water near the bed. The surface water tends, by its higher momentum, to continue with the parent stream. The slower moving water near the bed carrying the greater concentration of sediment, tends to flow to the intake.

Figure 2.2. shows the result of model studies conducted by Bulle (1926). These results and those reached by A. Schoklitsch (1937) indepedently, attempt to give some parameters through which the optimum angle of diversion could be determined. For an angle of diversion of 30°, and when the brach canal is drawing in 25% of the flow, there is an equal amount of distribution of bed load into the branch and main canal. When the proportion of flow into the branch canal is increased to 50%, then the bed load diversion into the branch increases to 90 % (Mosonyi, 1987).

From the above example it should be realised that there is no such thing as an optimum angle of diversion, because the optimum angle of diversion will also vary depending on the next two factors (Vanoni, 1977):

(i) The ratio of discharge in the diversion to that in the main stream (diversion ratio).

(ii) The position of the intake in a bend.

The best solution to the problem is probably to select the diversion angle by a scale model study for the dominant diversion ratio or for the condition that produced the maximum bed-load discharge.

2.4 **Controlling the Approach Flow to the Intake**

2.4.1 Background

Controlling the rate of bed-load transport to the intake can be achieved by creating the flow condition in which the sediment will be deposited, rather than entering the canal headworks or by hindering the direction of bed-load flow towards the canal headworks. Some structures or measures that can be used for this purpose are a divider wall and a sediment pocket, guide vanes and gates regulation.

2.4.2 Sediment Pocket, Divider Wall and Gates Regulation

This structure arrangement to reduce the amount of sediment taken into a canal headgate (Vanoni, 1977) is produced by constructing a divider wall upstream from a diversion dam, so as to form a pocket in front of the canal intake. The wall divides the flow as it approaches the diversion dam, so that one part of the flow is directed to the sluiceway and the other part through the spillway.

Where the diversion dam is gated, the amount of opening of the dam gates can greatly affect the amount of sediment taken into the canal headworks. The gates most distant from the point of canal diversion, should be opened widest. The gates nearest to the pocket holding the sediment should be opened only slightly. The increased discharge through the far gates creates a curvature effect that pulls the sediment away from the canal head gates. If there is a canal headworks at each end of the diversion dam, the gates in the middle of the dam should be opened widest.

The amount of sediment excluded from the canal headworks is increased to some extent by keeping a favourable ratio of velocity on each side of the divider wall that form the sluicing pocket. This can be done by regulation of openings of the sluice gates and the dam gates adjacent to the divider wall.

If the diversion dam is an overflow weir, the main function of the divider wall is to form a sluicing pocket upstream from the sluice gate to produce a pond area of low velocity in which the sediment will deposit rather than enter the canal headworks.

The sluicing of sediment out of the pocket can be accomplished by the still-pond method, which is intermittenty sluicing, or by semi-open pond which is continuously sluicing if an adequate amount of streamflow is available.

(i) Still-pond condition.

A still pond is created in front of the intake by closing the undersluice gate in the pocket. With the still pond, the water in the pocket flows with sufficiently low velocity, resulting in deposition of sediment in the pocket. Figure 2.3. shows the flow pattern during still-pond regulation.

(ii) Semi-open pond condition.

Under this condition, the undersluice gates are left open partially while the canal is flowing. Higher flow velocity will occur in the pocket area and less sediment will deposit. The advantage of semi-open pond condition is that there is a continuous flushing of the pocket, thus preventing it from silting up.

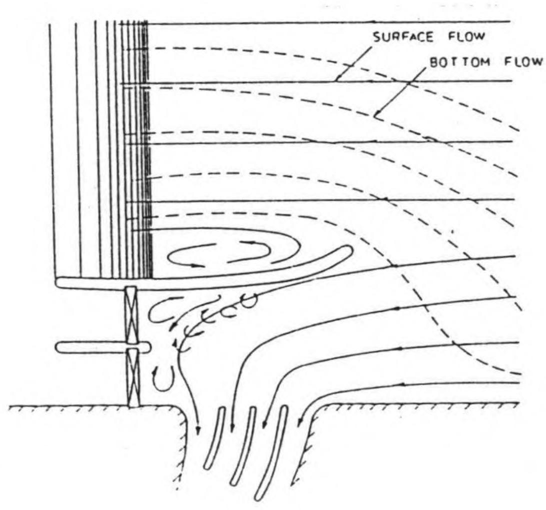


Figure 2.3 Schematic Diagram of Flow in A Still Pond Condition.

(After Avery, 1981)

2.4.3 Guide Vanes

Diversion structures with guide vanes are shown diagrammatically by figures 2.4. and 2.5. The guide vanes produce localised helicoidal flow patterns that are similar to those generated naturally in a flow around a curve. The two types of known guide vanes:

(i) Bottom guide vanes are so placed on that the bottom edge of the vanes are located at or near the stream bed. The vanes then direct the flow in the lower part of the stream prism. Because most of the heavy bed load is concentrated in this area of the flow, it is also deflected away from the canal intake.

(ii) Surface vanes are generally supported by a raft arrangement. The surface vanes direct the surface water, which contains relatively little sediment toward the canal headgates. This induced a transverse flow of the bottom-area water with its bed load away from the canal headgates.

For proper functioning of silt vanes it is essential that the velocity of water passing over the vanes should not be high enough to suck inbetween the vanes. The vanes are generally not considered suitable in cases in which discharge of the off-taking channel is more than 1/3rd of the parent channel, or where the parent channel is not wide enough to provide sufficient room for aligning vanes of desired radius.

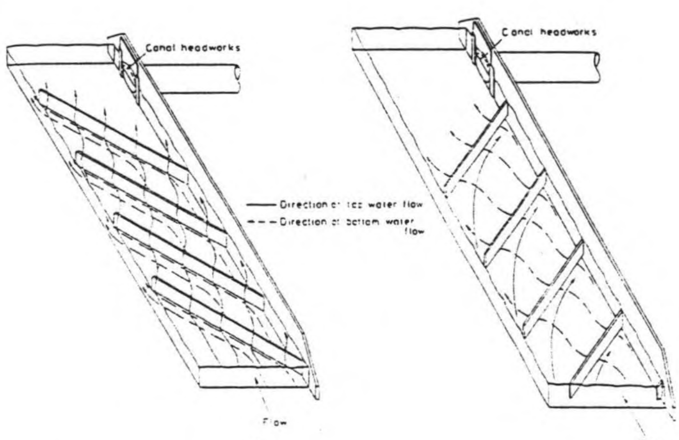


Figure 2.4 Bottom Guide Vanes   Figure 2.5 Surface Guide Vanes

(After Vanoni, 1977)

2.4.4 River-Training Works

River-training works are often necessary near intakes, not only to stabilise the river so as to prevent the formation of undesirable cut offs, but also to achieve a more favourable stream pattern. This holds especially when the training works are used to create a helicoidal flow to sweep the bed load away from the intake side. Training the river to a definite course is possible by providing training works at suitables places.

When there is only one parent channel and a canal intake is located on one side, the sediment can be excluded by creating a curvature so that the intake is located on the outside of the curve. This can be achieved with spurs alone or in conjunction with an island, where necessary (Mushtaq Ahmad et al., 1960).

When the canal intakes are located on both sides of the weir or barrage, the utilisation of the principle of flow curvature needs special corrective measures by considering the local condition.

2.5 Tunnel Excluder

2.5.1 Basic Principle

The distribution and size of sediment particles in the vertical are important. Most sediment diverters are designed to take advantage of the concentration of the havier particles in the lower part of the water prism. A tunnel excluder which is usually constructed upstream of the canal intake structure, on the side part of diversion dam, uses that advantage. A tunnel excluder is an exclusion device of the preventive-measure type.

2.5.2 Trapping Efficiency

Based on Einsteins sediment-transport formulation, Sanmuganathan (1976) developed a trapping efficiency model for a vortex-tube sediment extractor. Later, using a constant water-extraction ratio accross the width of the device, Attkinson (1984) initiated to use this theory for analysing the trapping efficiency of a tunnel type sediment extractor.

The sediment is kept in suspension by the balance between gravitational forces and turbulence fluctuations. The balance can be expressed by the equation:

$$w_s\, c + \varepsilon_s \frac{\delta c}{\delta z} = 0 \qquad (2\text{-}1)$$

where:

c = sediment concentration by mass.

w_s = fall velocity of the particle [m/s]

ε_s = sediment mixing coefficient [m²/s]

The sediment mixing coefficient generally is taken to be equal to the momentum-transfer coefficient (ε). The momentum-transfer coefficient is a function of depth and therefore a function of z. Taking a logarithmic velocity distribution:

$$\varepsilon = \kappa\, h\, u{*} \left(\frac{z}{h} - \frac{z^2}{h^2} \right) \qquad (2\text{-}2)$$

Equation (2-2) may be simplified by assuming that ε_s is a constant over the depth and is equal to its average value (Lane and Kalinske, 1942) h u*/15. Einstein considers the case where ε varies with the depth. The solution of equation (2-1) then becomes:

$$\frac{c}{c_a} = \mathrm{Exp}\left\{ \frac{-15w}{u{*}} \left(\frac{z}{h} - a \right) \right\} \qquad (2\text{-}3)$$

where:

c = c_a at z = ah.

ah = height of the bed layer which is given by the larger of twice the grain size or the heigth at which u=u*.

Einstein assumed a continuity of sediment concentration between the suspended load in the main bulk of the fluid and the bed load in a thin layer close to the bed. He assumed a constant concentration in the bed layer of

thickness twice the grain diameter. The flow velocity in this region was assumed to be directly proportional to grain-shear velocity $u*'$. Within the bed layer can be summarised: $c = c_a$ and $u = u*'$.
With the aid of flume experiments to determine the constant of proportionality, he obtained the concentration at the edge of the bed layer as (neglecting wash load):

$$c_{ja} = \frac{1.}{11.6} \; \frac{j_B \; q_B}{2D \; u*'} \tag{2-4}$$

where:

c_a = the reference concentration at $z = ah = 2D$.
q_B = bed load rate in weight per unit of time and width.
j_B = fraction of bed load in a given grain size.

Considering the previous explaination, the sediment concentration and velocity fields can be visualised in the following form:

$$\left.\begin{aligned} c &= \mathrm{Exp}\left\{ \frac{-15w}{u*} (\alpha - a) \right\} \\ \frac{u}{u*} &= \frac{u}{u*} + \frac{1}{\kappa} (\ln \alpha + 1) \end{aligned}\right] \text{ for } a < \alpha < 1. \tag{2-5}$$

$$\frac{c_j}{c_{ja}} = 1. \quad \text{and} \quad \frac{u}{u*'} = 1. \qquad \text{for } 0. \leq \alpha \leq a \tag{2-6}$$

α = z / h.
κ = Von Karman's constant.

In order to carry out computation using equation (2-5) two parameters $u*$ and $u*'/u*$ need to be evaluated beforehand. Engelund's experiments with varying sand sizes between 0.19 mm and 0.93 mm showed that there exists a relation between:

$$Y = \frac{u*^2}{g(\Delta - 1) \; D_{35}} \quad \text{and} \quad Y' = \frac{u*'^2}{g(\Delta - 1) \; D_{35}}$$

The relation shown in figure 2.6. The $u*'$ was obtained from the relation:

$$\frac{u}{u*'} = 2.5 \ln (h'/ (2 \; D_{65})) + 6 \tag{2-7}$$

where: $h' = (u*'/u*)^2 \; . \; h$
Δ = relative density under water.

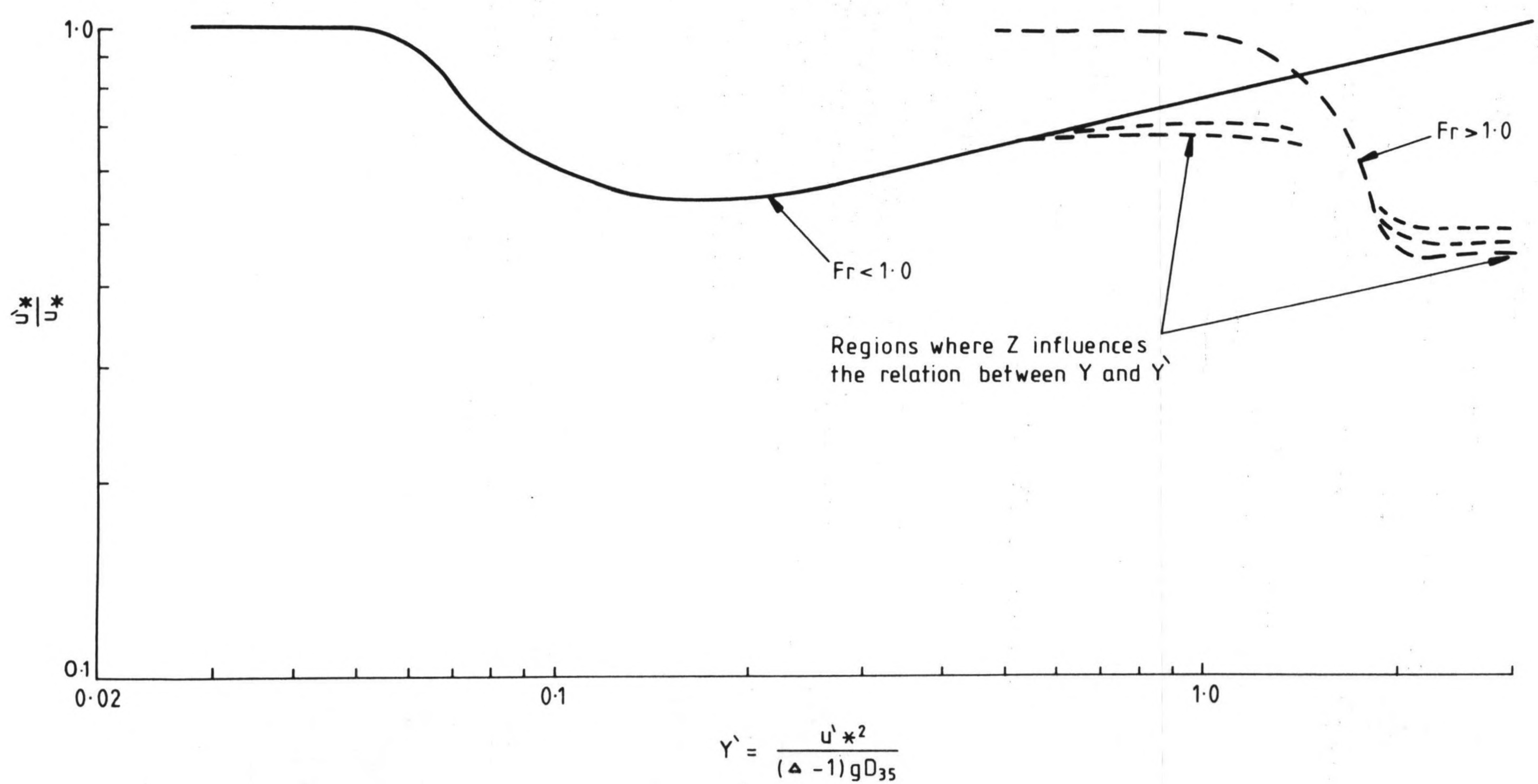


Figure 2.6 Variation of $\frac{U'}{U_*}$ with $Y' = \frac{U'^2}{(\Delta - 1)\, g\, D_{35}}$

(After Sanmuganathan, 1976)

The distribution of sediment concentration and velocity given in equation (2-5), provides the basic model for estimating the trapping efficiency. The amount of sediment transported in the region $0 \le z \le \alpha h$ can be expressed in the form:

$$\int_0^{\alpha h} c \; u \; dz \tag{2-8}$$

In a similar manner, the quantity of water passing through the region $0 \le z \le \alpha h$ is:

$$\int_0^{\alpha h} u \; dz \; = \; R \; \bar{u} \; h \tag{2-9}$$

As the quantity of water extracted is $R\bar{u}h$ per unit width, it can be assumed that this water comes from the region $0 \le z \le \alpha h$ in the channel. Therefore, R in equation (2-9) is taken as the water extraction ratio and αh is taken as the height of the dividing stream line. It can further be assumed that this water, coming from the region $0 \le z \le \alpha h$, carries with it all the sediment moving in that region. When the velocity distribution is logarithmic, equation (2-9) becomes:

$$R = \alpha + \frac{\lambda}{\kappa} \alpha \ln \alpha \tag{2-10}$$

where: $\frac{1}{\lambda} = \frac{u}{u*}$, friction factor.

Based on this model, the trapping efficiency, p_j, for the sediment fraction j is given by:

$$p_j = \frac{\int_0^{\alpha h} c \; u \; dz}{\int_0^{h} c \; u \; dz} \tag{2-11}$$

The overall trapping efficiency is given by:

$$p = \frac{1}{m} \sum_{j=1}^{m} p_j \tag{2-12}$$

2.6 **Tunnel Extractor**

Tunnel extractor is a curative measure which is usually located a short distance downstream of the canal intake structure. The tunnel extractor consists of parallel tunnels placed up on the canal bed across the whole cross-section of the canal. A typical layout of a tunnel extractor is shown in Figure 2.7.

2.6.1 Basic Principle

The tunnel extractor has also been developed on the principle that the bed material sediment is concentrated in the lowest third of the flow prism. Water is extracted from the upper portion of the flow prism, so clearer water is obtained. The portion of the flow prism which is concentrated with sediment flows through the tunnel and is diverted back to the river.

In order to increase the effectivity of the tunnel extractor, it should be constructed after widening the canal section so that suspended sediment can settle to the bottom and be removed through the tunnel. Rows of baffled blocks of various sizes should be provided at the entrance of the canal expansion.

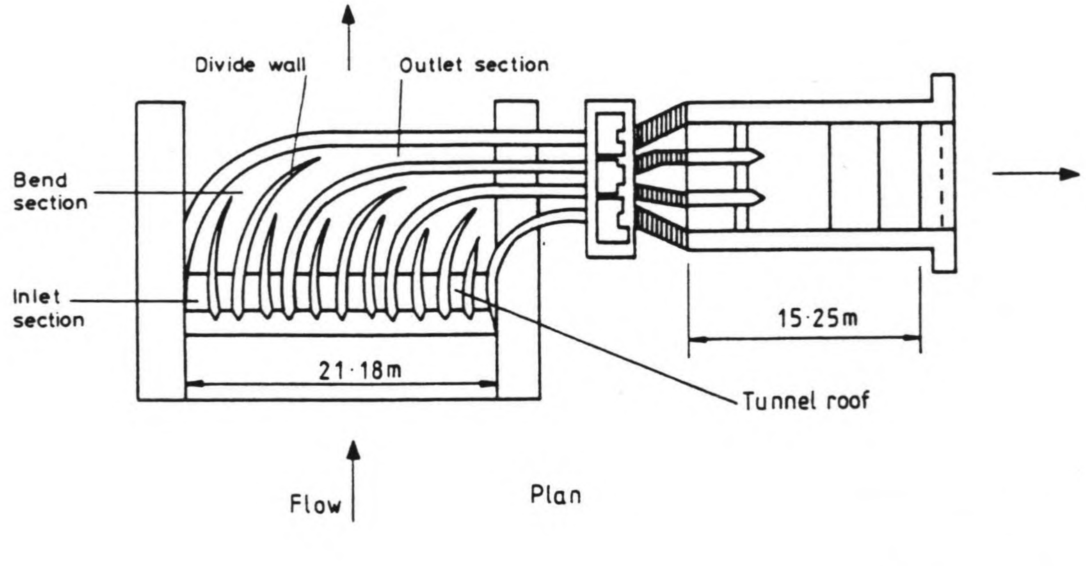


Figure 2.7 Tunnel Extractor on Upper Bari Doab hydel Channel, Punjab, India.

(After Atkinson, 1984)

2.6.2 Design Criteria

Exact design criteria for the design of extractors are difficult to establish. The design should be carried out using a scale model study (Vanoni, 1977). On the other hand, the problem of simulating sediment transport phenomena in a model has often lead to a wide disparity between sediment-trapping efficiency predicted using a model and subsequently realised with the prototype structure (Dhillon, 1980).

Atkinson (1974), based on Sanmuganathan's (1976) trapping efficiency for vortex tube, proposed a quantitative design method for the tunnel extractor. The steps that were proposed in the design procedure may be separated in two phases:

(i) In the first phase, the applicablility of a tunnel extractor for the specific application under consideration is assessed with the aid of the trapping efficiency program that has been proposed.

(ii) When the preliminary study shows that a tunnel extractor is feasible, the design can proceed to the second phase with the determination of the detail dimensions of the extractor. The two main criteria used for designing the extractor are:

   a. the required extractor discharge must be obtained with the available operation head.
   b. the velocity in the tunnel must be large enough to hinder sediment deposition by considering the maximum permissible velocity of the material of the structure.

The design procedure is schematised in figure 2.8. , as given by Lawrence and Atkinson (1984). Detail-design criteria are described by Atkinson (1984).

2.6.3 Trapping Efficiency

Since the basic principles of the tunnel extractor are similar to the tunnel excluder, the trapping efficiency theory for vortex tube proposed by Sanmuganathan (1976) should also be used for analysing the trapping efficiency of a tunnel extractor.

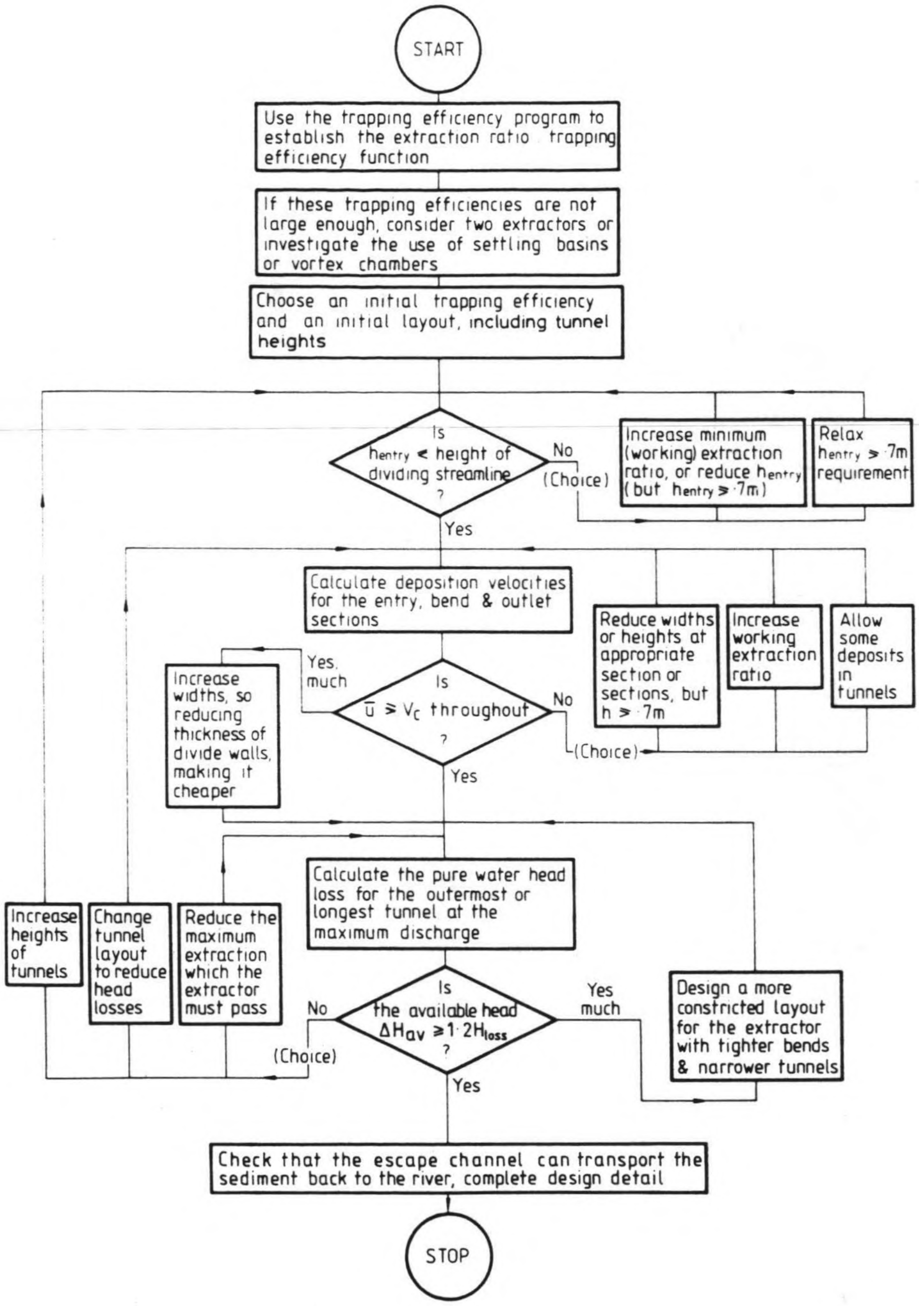


Figure 2.8 Flow chart for the design procedure of tunnel type sediment extractor.

(After Atkinson, 1984)

2.7 **Vortex Tube Extractor**

The vortex-tube extractor consists of a slotted tube or pipe embedded in the bed of the canal in such a way that the slot lies flat upon the canal bed. In plan view the tube is placed at an angle of between 45° and 90° to the direction of flow as shown in figure 2.9. The flow entering the vortex tube has to be discharged at one end of the tube itself. In this case the discharge end acts as a sluiceway, whereas tube is clogged at the other end. It has been found that the efficiency and the performance of the vortex tube increase considerably if the vortex tube also discharges into another exit pipe off-taking from the centre of the tube (Dhillon, 1980). If the site conditions permit, the sediment-laden discharge may be transported by pipeline and be allowed to discharge into the river beyond the point of intake.

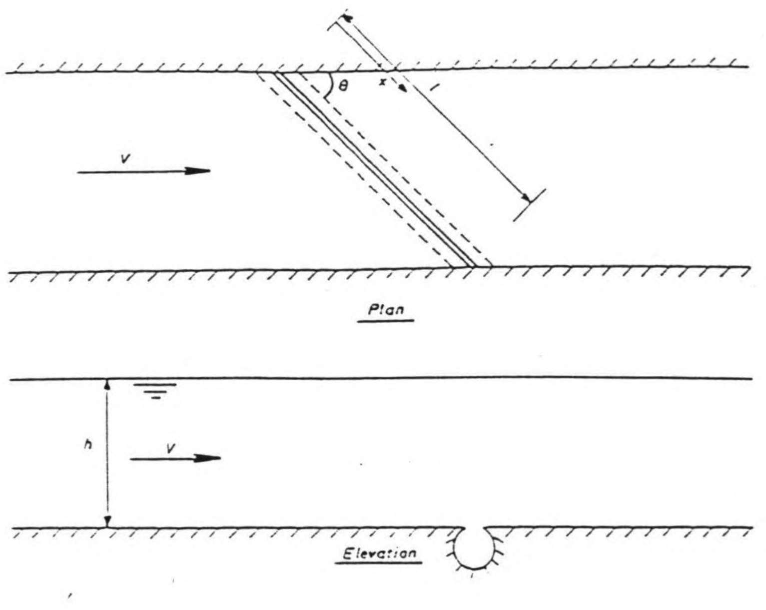


Figure 2.9 Layout of Vortex Tube Extractor.

(After Sanmuganathan, 1976)

2.7.1 Basic Principle

The vortex-tube extractor is designed based on the principle of horizontal separation of flow. The device draws the sediment-laden water near the canal bed by suction induced in the tube. Experience shows (Sanmuganathan, 1976) the structure is more efficient with large size sediment and relatively less efficient with smaller sediment. Its advantage is in the simplicity of the structure and in the insignificant obstruction it causes to the flow in the canal. The disadvantages of the vortex-tube device often mention are:

(i) The limitation imposed on the length. Robinson, as mentioned by Sanmuganathan and Lawrence (1980), suggests an upper limit of 4.6 m.

(ii) The trapping efficiency appears to be a function of the Froude number in the canal. A Froude number between 0.7 and 1.0 seems to be recommended by most researchers. Since most alluvial channels in the field operated at a lower Froude number, the flow should be forced toward the critical stage by some measures. These measures would most probably cause problems (Vanoni, 1977).

2.7.2 Design Criteria

Much research has been undertaken to determine a design criterion for a vortex tube extractor. Some general aspects that need to be considered, has been summarised (Sanmuganathan, 1976):

(i) The trapped material should not be thrown back into the channel.

(ii) The trapped material should not be allowed to settle down in the tube.

(iii) The extracted water should be kept to a minimum.

(iv) The ratio of sediment concentration in the tube to the canal should be maximum.

To meet that requirement, a clear design procedure has been provided by Sanmuganathan (1976).

2.7.3 Trapping Efficiency

Sanmuganathan has developed a theory to predict the trapping efficiency of vortex-tube sediment extractor. A simplified version of the theory considering a constant rate of extraction accros the channel width, is described in Section 2.5.

In a vortex-tube extractor the local extraction ratio varies along the length of the tube. To facilitate the consideration of this variation, equation (2-11) may be generalised to

$$p_{ij} = p_{\alpha i,j} / p_{1,j} \tag{2-13}$$

where αi is given by

$$r_i = \alpha_i + \frac{\lambda}{0.4} \alpha_i \ln \alpha_i \tag{2-14}$$

and r_i is the extraction ratio at the i-th section of the tube.

The trapping efficiency for the j-th fraction of sediment then becomes:

$$p_j = \frac{1}{N} \sum_{i=1}^{N} p_{ij} \tag{2-15}$$

and the overall trapping efficiency is given by

$$p = \frac{1}{M} \sum_{j=1}^{M} p_j \tag{2-16}$$

In order to obtain αi from equation (2-14) r_i needs to be determined. This can be achieved by using the following equation.

$$r_i = y R \cosh y_i / \sinh y \tag{2-17}$$

where:

$y = y_i x_i / 1$

x_i = the distance of the section from the closed end of the tube [m].

y = constant x tl / A.

1 = the length of vortex tube [m].

A = the cross-sectional area of vortex tube [m²].

t = width of slit of vortex tube [m].

Comparing the seventy-two sets of data, obtained by Robinson, with the trapping efficiencies calculated by assuming a constant extraction ratio along the tube, equal to the overall extraction ratio, gives a correlation between p_R and p in the form

$$p_R = 1.295\, p + 33.551 \qquad (2\text{-}18)$$

with a correlation coefficient of 0.747 and a standard error of 9.564 was obtained. In equation (2-18) p_R is the percentage trapping efficiency with constant extraction ratio R and p is the calculated value with relevant parameters. This relation cannot be used to estimate p from p_R directly, because it imposed an upper limit p of 51.31 corresponding to $p_R = 100$. Considering the limitation imposed, the trapping efficiency estimated with constant extraction ratio will be a 30% overestimate. Assuming that $p_R \approx 1.3\, p$ the trapping efficiency of a vortex-tube extractor

will be: $$p = p_R / 1.3 \qquad (2\text{-}19)$$

At the design stage this approach is accurate enough.

2.8 Settling Basin (Sand trap)

2.8.1 Basic Principle

The settling basin consists of an oversized section of the canal, or an other arrangement in which the velocity is low enough to permit the suspended particle to settle down. Only relatively clear water will then enter the canal and the sediment is removed from the settling basin by sluicing or by other mechanical means.

The settling basin can be designed to control the amount of suspended sediment removed by varying the dimensions of the structure and thus the time that the water is retained in the basin. A typical lay-out of a settling basin is shown in figure 2.10.

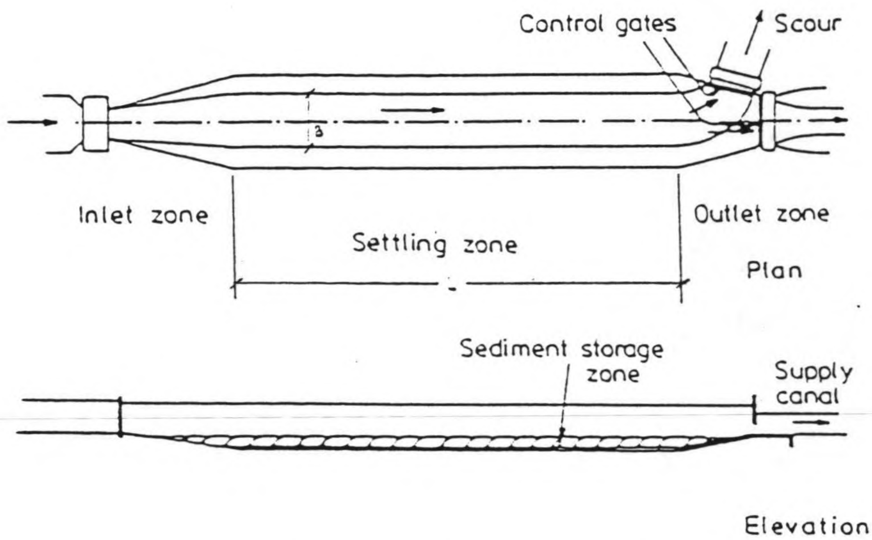


Figure 2.10 General Layout of Sandtrap

(After Avery, 1989)

2.8.2 Design Criteria

The following parts of the settling basin have to be considered in the design stage (Avery, 1989):

(i) The inlet should be designed to distribute the inflow and suspended sediment uniformly over the cross sectional area of the settling zone.

(ii) For the settling zone, Camp (1946) showed that the hydraulic behaviour of a long narrow basin is superior to that of a low velocity basin. Basins with not too-low values of the Froude number give better flow patterns and less dispersion.

(iii) To avoid meandering of the flow in the basin, a length-to-width ratio of 8 to 10 is generally adopted for irrigation systems. Where it is not possible to achieve this ratio, the basin can be sub-divided by providing a dividing wall.

(iv) When the outlet is narrower than the basin, a well-designed transition is required to avoid short-circuiting and to maintain an even flow distribution.

(v) The design method of removing sediment from the basin will influence the needed head and the lay-out of the settling basin. The hydraulic flushing is prefered because it is relatively inexpensive.

To control the water level in the settling basin, a spillway should be provided at the outlet. The spillway can also be used as a discharge-measuring device.

2.8.3 Trapping Efficiency

2.8.3.1 The Concept of the Ideal Horizontal Basin

The basic theory of sedimentation in a settling basin was developed by Hazen in 1904 and used in the concept of "ideal horizontal basin". The following relationship was obtained (see figure 2.10).

$$w_s = \frac{Q}{B.L} \qquad (2\text{-}20)$$

Other basic relations for the design of settling basin were established by the USBR (Vanoni, 1977):

$$S = S_0 \text{ Exp. } [- \frac{w_s \, . \, L}{q}] \qquad (2\text{-}21)$$

where:

S = weight of sediment leaving the basin.

S_0 = weight of sediment entering the basin.

w_s = settling velocity of particle.

L = length of settling basin.

q = discharge per unit width of settling basin.

B = width of settling basin.

One of the more popular approaches to settling basin design, the work done by Camp (1946). The fluid velocity and turbulent mixing coefficient are assumed constant throughout the fluid. Then a sediment-removal efficiency graph was established, as given in Figure. 2.11. The graph gives the efficiency as a function of the parameter w/w_o and w/u where w is the fall velocity of particle with a size different than the size for which the trap was designed, w_0 is the design fall velocity and u_0 is the average velocity of the flow in the settling basin. Using this graph the efficiency of different fractions of sediment particles can be checked.

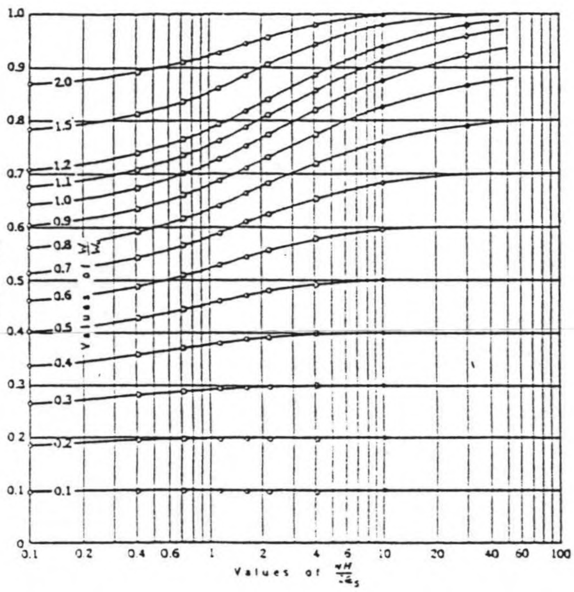


Figure 2.11 Camp's Graph for Sediment Removal Function.

(After Camp, 1946)

2.8.3.2 Two-dimensional Vertical Model

The trapping of sediment in the settling basin can be described using a two-dimensional vertical model. In this model the flow condition and bed level are averaged across the width. The sediment transport is not assumed to depend on the local hydraulic condition. A two-dimensional vertical model is particularly of interest if suspended load is concerned as in the case of sedimentation in a settling basin. Setting up and simplification of this vertical model is described in detail by de Vries (1987).

Kerssens et al (1977 and 1979) presented a two-dimensional vertical model for gradually varying flows neglecting vertical convection and horizontal diffusion. Logaritmic velocity profiles were used to represent the flow-velocity field. The vertical sediment mixing coefficient were represented by a parabolic-constant distribution. A concentration type boundary condition was applied at the bed, assuming instantaneous adjustment to equilibrium conditions close to the bed. A six-point implicit finite-difference method was used to solve the basic convection-diffusion equation.

Van Rijn (1980 and 1987) also developed a two-dimensional vertical model for the computation of bed-load and suspended load transport and bed-level changes under steady and unsteady flow conditions. The model is based on sediment transport in gradually converging or diverging channels (stream tube). The flow is assumed to have a rectangular cross-section while neglecting the horizontal diffusive transport component. The basic equation used in the SUTRENCH - 2D model:

$$\frac{\delta}{\delta x} (Buc) + \frac{\delta}{\delta z} B(w-w_s)\, c - \frac{\delta}{\delta z} (B\varepsilon_s \frac{\delta c}{\delta z}) = 0 \qquad (2\text{-}21)$$

where:

c	= local sediment mass concentration	[kg/m³]
u	= local horizontal flow velocity	[m/s]
w	= local vertical flow velocity	[m/s]
w_s	= fall velocity of sediment particle	[m/s]
ε_s	= sediment mixing coefficient	[m²/s]

The equation can be solved numerically when the fluid velocities (u,w), the sediment mixing coefficient (ε_s), the fall velocity (w_s) and geometrical and physical boudary conditions are known.
The flow fields in horizontal and vertical directions are derived with the profile-method based on flow-velocity profiles according to Coles. The horizontal flow-velocity profile can be described by a combination of a logaritmic and a perturbation profile:

$$u = A_1 u_h \ln (\frac{z}{z0}) + A_2\, u_h\, F(\frac{z}{h}) \qquad (2\text{-}23)$$

where:

A_1, A_2	= dimensionless variables	[-]
$F(z/h)$	= perturbation profile	[-]
h	= local water depth	[m]
u_h	= flow velocity at the surface	[m/s]
z_0	= zero-velocity reference level (0.03 k_s)	[m]
k_s	= effective roughness height	[m]

The vertical flow velocity component is derived via the continuity equation of water:

$$\frac{1}{B}\frac{\delta(Bu)}{\delta x} + \frac{\delta w}{\delta z} = 0 \qquad (2\text{-}24)$$

where B = the width of stream tube.

The sediment mixing coefficients are based on a semi-empirical formulation, and are represented by a parabolic-constant distribution. The concentration close to the bed is used as a physical boundary condition. In this case an empirical formulation is applied:

$$c_a = 0.015 \frac{D50}{\alpha} \frac{T^{1.5}}{D*^{0.3}} \qquad (2\text{-}25)$$

where:

a	=	reference level above the bed (= ks)	[m]
T	=	bed-shear stress parameter parameter	[-]
	=	$(\tau_o - \tau_{cr}) / \tau_{cr}$	
τ_o	=	effective bed-shear stress	[N/m²]
τ_{cr}	=	critical bed-shear stress (Shields)	[N/m²]
D*	=	particle size parameter $D_{50}\{g/\nu^2\}$	[-]
Δ	=	relative density	[-]
ν	=	kinematic viscosity	[m²/s]

The water level is approximated by a rigid lid. At the upstream boundary, the flow velocity and concentration profile are pre-described, and at the downstream boundary weak boundary conditions are applied (second derivatives of u and c are zero). The SUTRENCH model computes sediment transports for one characteristic grain size only. Computation for bed material, consisting of widely range of grain sizes, has to be carried out fraction by fraction using each characteristic of grain size.

Delft Hydraulics has derived a formula for sedimentation in a dredged channels based on disturbance of the natural conditions and relative changes in the vertical diffusive sediment transport (Eysink and Vermaas, 1983).
For $(w / u*) \leq 0.3 - 0.4$ the following formula was proposed to use:

$$s_{(x)} = \frac{B1}{B0} s_0 - \{ \frac{B1}{B0} s_0 - s_1 \}\{ 1. - \mathrm{Exp}^{-\frac{\alpha . x}{h_1}} \} \qquad (2\text{-}26)$$

where:

$$\alpha = 0.015 \frac{2w_s}{u*_1} \{ 1 + \frac{w_s}{u*_1} \} [\{ 1. + 4.1 \{ \frac{k_n}{h_1}\}^{0.25}$$

Subscripts 0 and 1 denote the condition in the upstream and downstream of the settling basin respectively.

s_1 = the sediment transport in the equilibrium condition.
s_0 = sediment flux entering the system in unit volume per unit time.
B = width of settling basin [m].
k_n = Nikuradse roughness coefficient [m].
x = distance measured from the entering point [m].

By considering the Strickler formula, can be derived:

$$C = 25 \left(\frac{h}{k_n}\right)^{1/6}$$

$$\frac{u}{u*} = \frac{C}{\sqrt{g}}$$

$$\alpha = 0.24 \{\frac{h_1}{k_n}\}^{1/6} . \frac{w_s}{u} .[1 + 8 \{\frac{h_1}{k_n}\}^{1/6} \frac{w_s}{u}][1 + 4.1 \{\frac{h_1}{k_n}\}^{-.25}] \qquad (2\text{-}27)$$

2.9 Summary of Sediment Exclusion Measures

From the previous description, a summary of well-known sediment-exclusion measures can be made by considering their principles involved, principal type of sediment excluded, present basis of design, quantitative approach available and a possibility of quantitative approach (John, 1989). The summary is shown in tabel 2.1.

	Sediment exclusion method	Principles involved	Principal type of sediment	Present basis of design excluded	Quant. approach available	Quant. approach possible
	<u>Preventive</u>					
1	Siting of intake	CF	BL	FE & SM	No	?
2	Alignment of intake	VS & HF	BL	FE & SM	No	?
3	Divide wall and pocket	SS & VS	BL & SL	FE & SM	No	?
4	Barrage regulation	CF	BL	FE & SM	Yes	Yes
5	Guide vanes	HS & CF	BL	FE & SM	No	Yes
6	High level sill	HS	BL	FE & SM	No	?
7	Tunnel excluder	HS	BL	FE & SM & QA	Yes	-
	<u>Curative</u>					
8	Slits	HS	BL	FE & SM	No	Yes
9	Circulation chamber	CF	BL	FE & SM	No	Yes
10	Settling Basin	SS	SL & BL	FE & SM & QA	Yes	-
11	Tunnel extractor	HS	BL	FE & SM & QA	Yes	-
12	Vortex tube extractor	HS & CF	BL	FE & SM & QA	Yes	-

CF - Curvilinear Flow

VS - Vertical separation

HS - Horizontal separation

SS - Settling of sediment

FE - Field experience

SM - Scale model study

BL - Bed Load

SL - suspended Load

QA - Quantitative approach

Table 2.1 Summary of sediment-exclusion measures (after John, 1989)

3. Comprehensive Model on Optimisation of Sediment Exclusion Devices

The optimisation of sediment-exclusion divices should take into account at least the following interrelated aspects:

(i) The river and transported sediment characteristics.

(ii) The available discharge and hydraulic head for the system.

(iii) The efficiency and effectiveness of the system especially in relation to the trapping efficiency of sediment.

(iv) The energy used for the operation of the system, and financial consideration.

The availability of quantitative design methods for estimating the efficiency of several well-applied sediment-exclusion divices allow for setting up a comprehensive model on optimisation of sediment-exclusion devices. Hopefully, the designed model can be used to choose the most appropriate device, or combination of devices, at the preliminary design stage. The model should also be used to establish the optimal operational of devices for the system of sediment exclusion.

In this chapter will be described the setting up of a model by mainly considering the first three aspect that should be taken into account. Since the design of some sediment-exclusion device is still based on hydraulic scale models, only tunnel excluder, tunnel extractor, vortex tube and sandtrap will be discussed in the comprehesive model.

For any system where the problem of entering sediment has to be tackled, one type of the four devices or combination of them, can be used in series. Since the tunnel excluder, tunnel extractor and vortex tube are used to essentially remove the bed load, and since they also work on the same principle, it is highly unlikely that they used in series. Nevertheless, the model allows for the flexibility of using any of the four types of device. Considering the design criteria of each device, when they are used in combination only the following lay-out can be used:

Tunnel excluder - tunnel extractor - Vortex tube and sandtrap.

In a system of sediment-exclusion devices, only tunnel extractors and vortex tubes are possible to be applied more than once.

Based on those criteria, the designed model will be schematised in a form as shown in Figure 3.1.

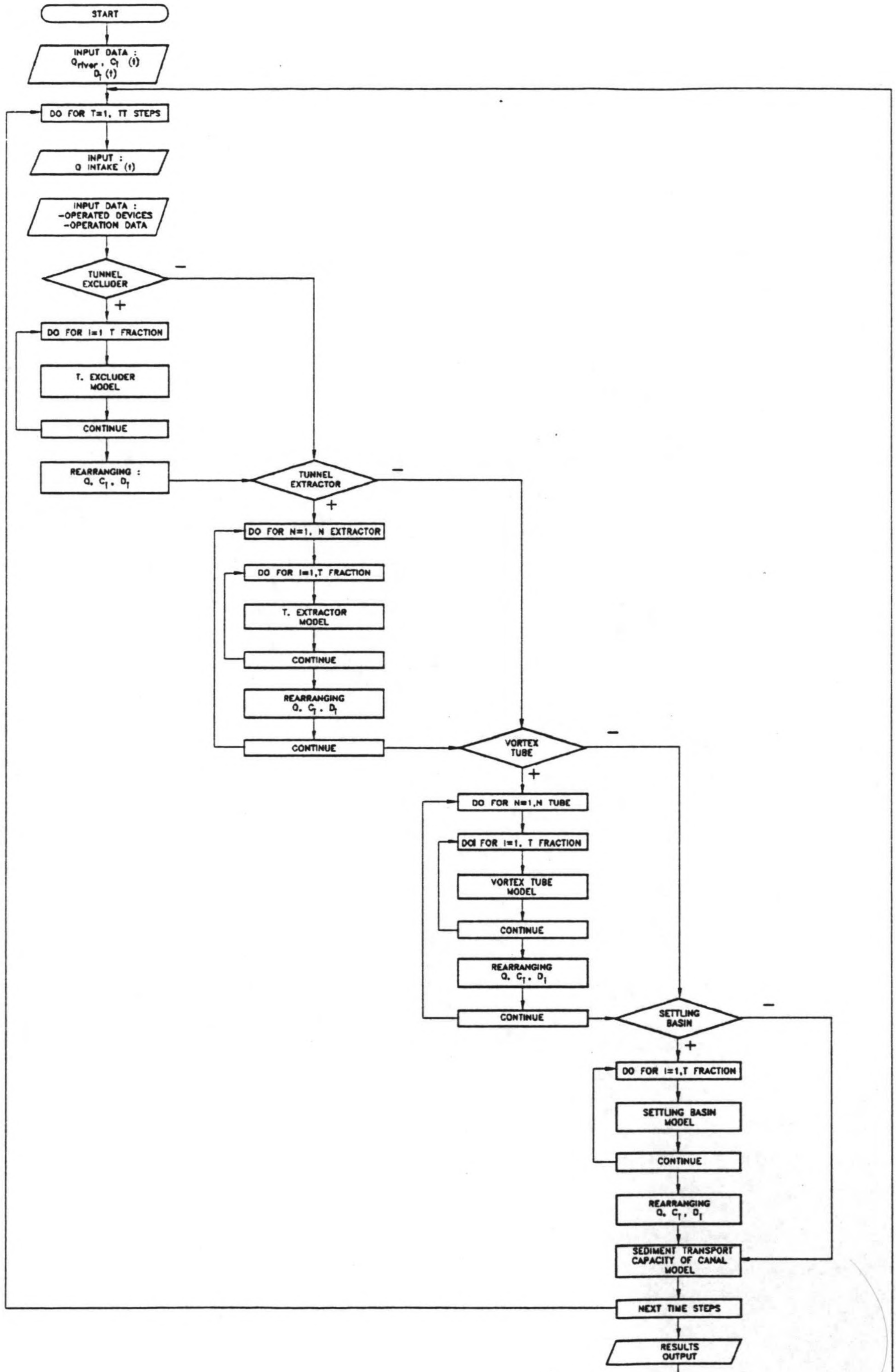


Figure 3.1 Schematisation of the designed model

The present designed model there has not yet been completed with a sub-model for simulating the flushing of a sandtrap. To solve this ristriction, in the case of simulating a system of sediment-exclusion devices in which a sandtrap in provided, some input data from a hydraulic scale model are needed. Typical required input data are:

(i) The flushing window of the designed sandtrap

(ii) The effectiveness of a flushing operation as a function used discharge, the available hydraulics head and duration of the flushing operation.

Examples of those required data are shown in Figure 3.2.

3.1 **Trapping Efficiency Model of Tunnel Excluder, Tunnel Extractor and Vortex Tube**

Since the principle of sediment exclusion of Tunnel Excluder, Tunnel Extractor and Vortex Tube are similar, and also the same basic theory used for analysing the trapping efficiency, only one sub-mathematical model will be designed. The distiction of treatment for each device can be described by the following schematisation.

3.1.1 Schematisation of Tunnel Excluder

The flow and its carried sediment in the river upstream of the tunnel excluder is schematised as follow:

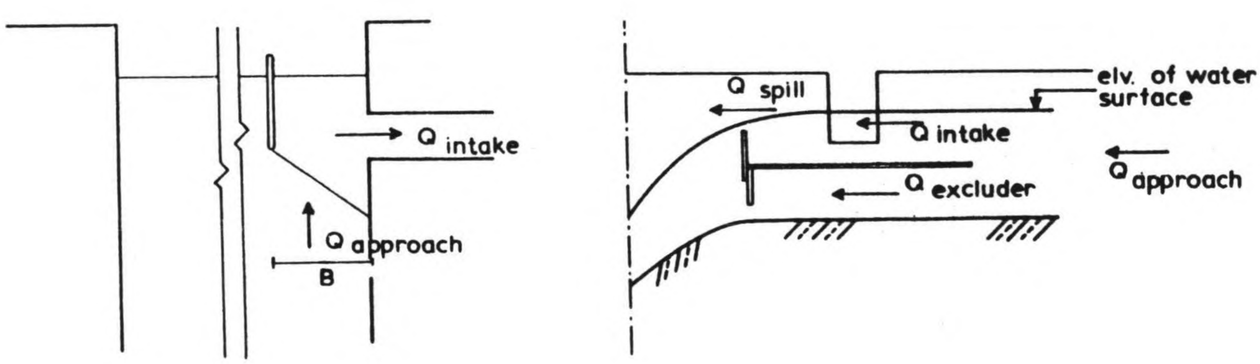


Figure 3.3 Schematised tunnel excluder

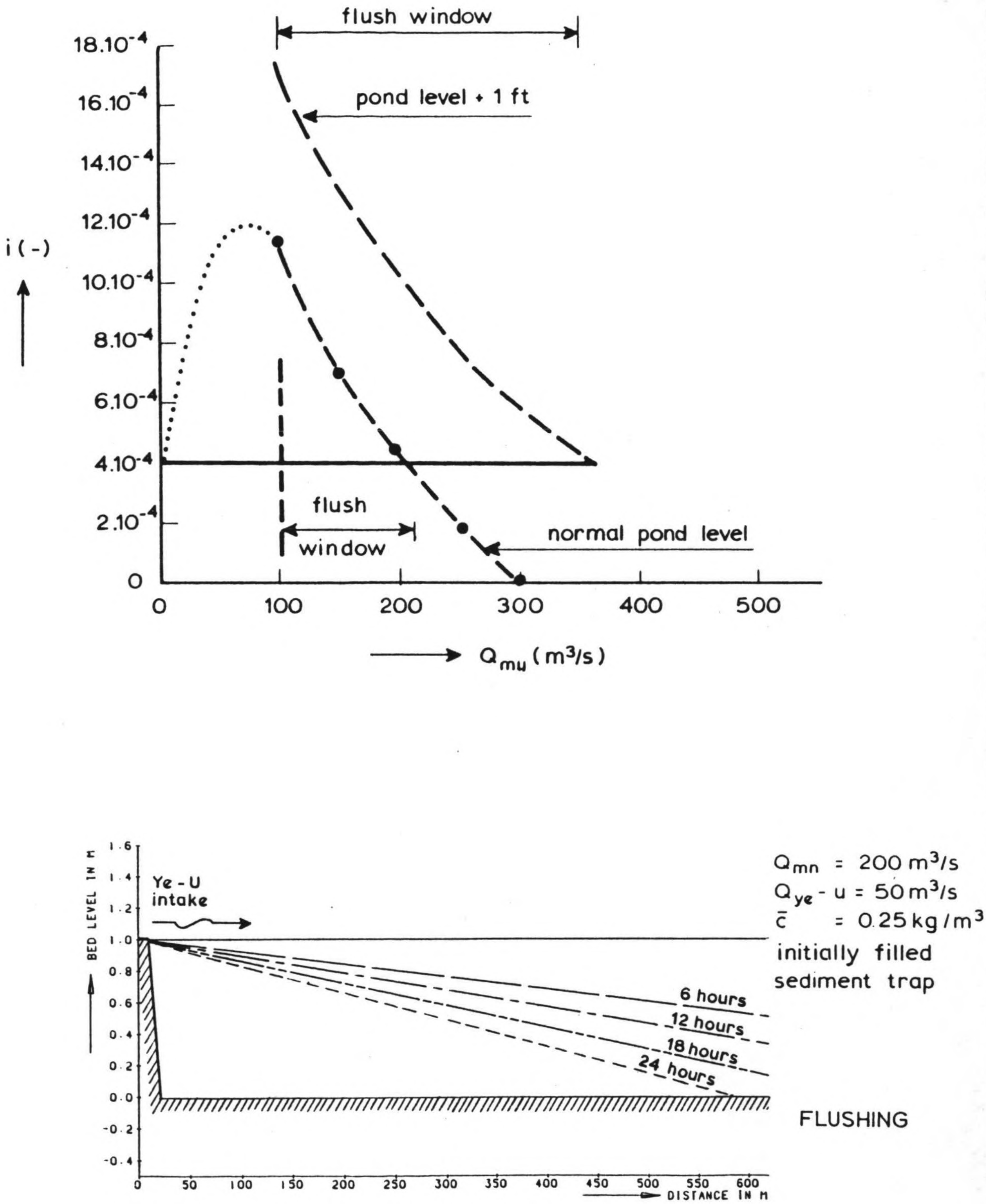


Figure 3.2 Needed supporting data from a hydraulic scale model.

(After Perdijk, 1989)

$$Q_{approach} = Q_{intake} + Q_{spill} + Q_{excluder} \tag{3-1}$$

Q_{intake} = set by considering the available river discharge, water requirement and needed discharge for other operation

Q_{spill} = calculated by considering upstream water level, setting-up of gate crest. Estimating spill discharge is carried out by assuming the gate as a sharp crest spillway.

$Q_{excluder}$ = set by considering the available river discharge and required trap efficiency

Later the water extraction ratio is assumed constant along the width and defined as:

$$R = \frac{Q_{excluder}}{Q_{approach}} \tag{3-2}$$

For simplicity, the width of the exclusion device is taken as the total cross width of the tunnel excluder.

The concentration of sediment entering the system is assumed not influenced by the ratio of approach discharge to the river discharge. The entering concentration is equal to the concentration in the river.

3.1.2 Schematisation of Tunnel Extractor

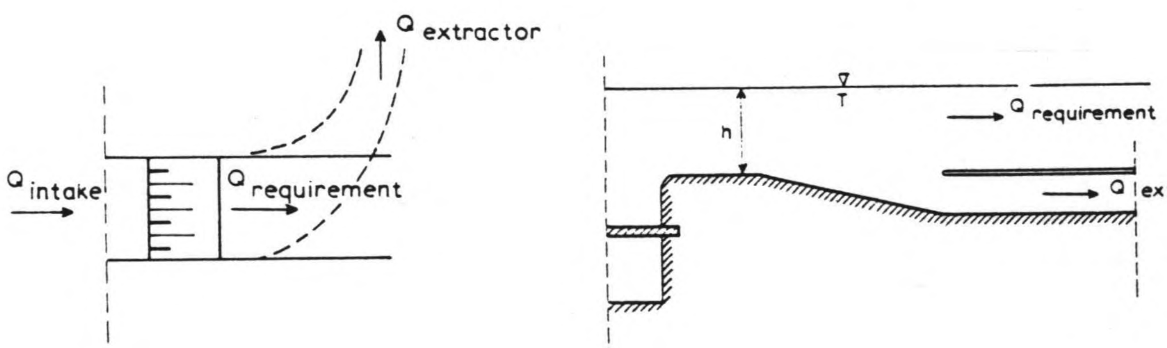


Figure 3.4 Schemitised tunnel extractor

For a tunnel extractor there is no special distinction. For analysis, the following schematisations are carried out:

$$Q_{intake} = Q_{requirement} + Q_{extractor} \tag{3-3}$$

$$R = \frac{Q_{intake}}{Q_{entractor}} \tag{3-4}$$

The water-extraction ratio along the width of a tunnel extractor is constant. Elevation of the water level of the tunnel extractor near the intake structure is assumed equal to the elevation of surface water in the river. If there are other tunnel extractors, the water depth for analysis should be taken the same as water depth of the canal in that section.

3.1.3 Schematisation of Vortex tube extractor

Actually, the water-extraction ratio along the width of the vortex tube is not constant. In the designed model it will be assumed that the water extraction is constant. To compensate for this assumption, a reduction factor will be applied in the trapping efficiency analysis. Considering this weak point, the length of the vortex tube should be restricted. It can be achieved by designing a vortex tube with two end openings.

3.1.4 Mathematical Modelling of trapping efficiency

Based on the theory of Sanmuganathan and Einstein's sediment transport mentioned in Chapter 2, a model for predicting the trapping efficiency of sediment-extractor devices was developed by Wallingford Hydraulic Research. A basic principle of the model in summarised in a form of the following basic steps:

(i) To evaluate the value of u^* and $k_u = u^{*'}/u^*$.
The evaluation was carried out basically using Engelund's curve shown in Figure 2.6. Calculation is started by assuming $h' = h$ and corrected later to get a certain accuracy.

(ii) Calculate the dimensionless height α_u where $u = u^*$

(iii) Calculate dimensionless bed layer (a) where in the region a.h a constant concentration is assumed. The calculation is carried out by evaluating which value is higher between α_u and $2 * \frac{D_i}{h}$.

(iv) Calculate dimensionless height of the dividing streamline α by considering the water extraction ratio R.

(v) Compute the trapping efficiency of each fraction.

$$p_j = \frac{p_{\alpha_j}}{p_{1j}}$$

where p_α = volume of sediment transported in the region below the dividing streamline

p_1 = total volume of transported sediment fraction

The computations were carried out using equations (2-5), (2-6), (2-8) and (2-11) by considering the dimensionless bed layer (a). In evaluating p_{α_j} and p_{1_j} it has been assumed that in the range $0 \leq z \leq a.h$, $\mathrm{Exp}[-15 \frac{sz}{h}]$ is a constant equal to one. This has been made to facilitate computation when 'a' is small.

A more complete structured diagram is shown in Appendix A-4.

3.1.5 Model verification

The accuracy verification of the proposed model has been carried out by Wallingford Hydraulic Research. A set of data collected by Robinson was used. The result of model verification shown in Figure 3.5. From the result can be concluded that in most cases the calculated trapping efficiency is higher than the field data with a range of discrepancy ratio between 0.7 - 1.2. Subject to the complexity of sediment transport phenomena and the aim of the overall model, this accuracy is acceptable.

3.2 Mathematical Modelling of a sandtrap

3.2.1 Basic Equation and Assumption

Subject to the aim, application, accuracy and time needed for computation, a one-dimensional mathematical model will be designed for analysing trapping efficiency and simulating sedimentation in a sandtrap. Eysink and Vermaas equation [1983] which can represent a two-dimensional phenomenon of sedimentation in a settling basin, into a form of one dimensional phenomena, proposed to be used.

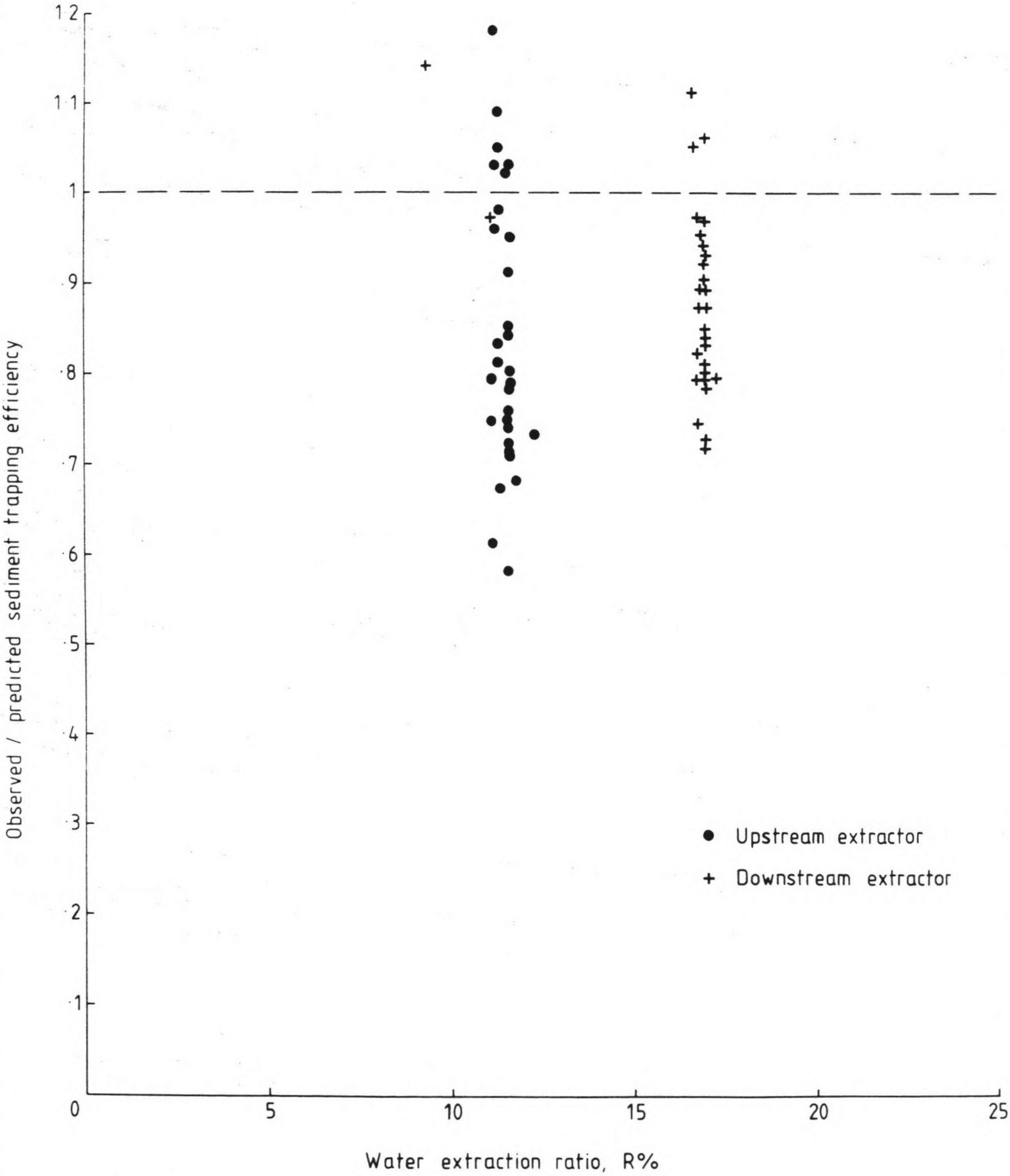


Fig. 3.5 Sediment trapping efficiency discrepancy ratio as a function of water extraction ratio.

(After Atkinson, 1984)

The designed model should be used in a range of moderate Froude numbers ($F_r \leq 0.6$ to 0.8). For this range, de Vries [1987] analysed the celerity of bed disturbance compared to the celerity of water flow and he came to the conclusion that the time scale of liquid wave propagation is much shorter than the time scale of a longitudinal bed profile. Other, the flow can be considered as a quasy-steady flow. This fact leads to the following solution:

(i) During the time interval, important for the computation of the water movement, it can be assumed that the bed is fixed. If the time dependence of bed-level changes is analysed, the flow can then be considered as quasi-steady.

(ii) The model can be decoupled to a hydrodynamic sub model and a sediment (bed-level changes) sub-model.

The general description of the solution algorithm can be schematised as follows:

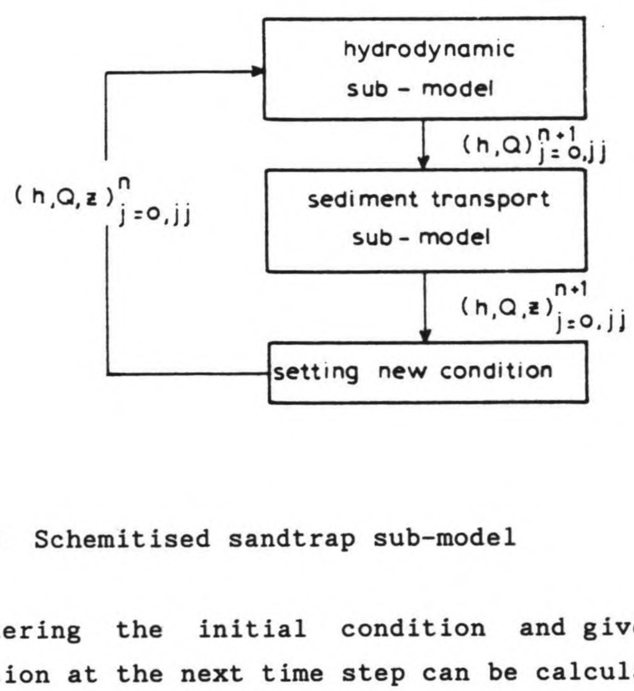


Figure 3.6 Schemitised sandtrap sub-model

1. Considering the initial condition and given boundary data, the flow condition at the next time step can be calculated. Since a quasi-steady flow is considered, a relatively simple method for calculation back-water curve should be used.
2. The calculated flow condition at (n+1) time step: $(h, Q)^{n+1}_{j=1, jj}$ and bed level condition at initial condition $(z)^{n}_{j=1,jj}$ are sent to the sediment model as the input.

3. The changes of bed level and efficiency of the sandtrap are predicted on the sediment sub-model by considering the conservation of sediment in a control volume:

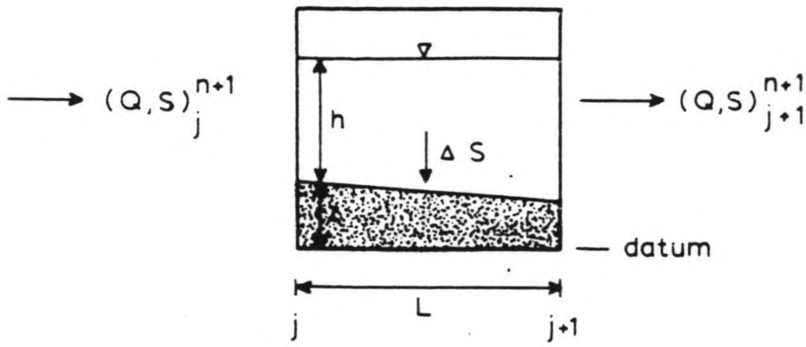


Figure 3.7 Schemitised conservation of mass

In this case the grid size of the model is small compared to the adaptation length of the concentration. The sediment deposition is computed using the actual concentration, the equilibrium concentration and the adaptation concentration proposed by Eysink and Vermaas.

4. Before entering the hydrodynamic model to calculate the next time-step condition, the new initial condition must be set by maintaining the water level. The depth will change due to the change of bed elevation, but surface-water elevation will be kept as resulted from former hydrodynamic computation.

Based on the former analysis, the following basic equations will be used:

(i) Continuity of water:

$$\frac{\partial Q}{\partial x} + \frac{\partial h}{\partial t} = 0 \qquad (3\text{-}5)$$

(ii) Diffusive wave approximation of the momentum equation:

$$\frac{\partial h}{\partial x} + \frac{\partial z}{\partial x} + \frac{|Q|Q}{C^2A^2R} = 0 \qquad (3\text{-}6)$$

(iii) The continuity of sediment:

$$\frac{\partial z}{\partial t} + \frac{\partial s}{\partial x} = 0 \qquad (3\text{-}7)$$

(iv) The equation of adaptation of concentration:

$$s_{(x)} = \frac{B_0}{B_1} s_0 - \{\frac{B_0}{B_1} s_0 - s_1\} \{1 - \mathrm{Exp} \frac{-\alpha x}{h_1}\} \qquad (2\text{-}26)$$

3.2.2 Hydrodynamic Model

A hydrodynamic model is part of the sandtrap sub-model designed for analysing the flow-field condition. Two types of flow-field condition can be chosen. The first is a rigid lid condition and the second is the back-water condition. The first condition is intended for simplicity and the second is aimed for analysing the impact of sedimentation in the sandtrap and approach canal to the change of water level in the intake. Later, by adding some data, it can be used for analysing the change of intake capacity.

Considering the hydraulic design of a sandtrap, is can be assumed that there is no lateral withdrawal or injecting water along the sandtrap. This condition makes the hydrodynamic model simpler because only diffusive wave approximation of the momentum equation has to be treated:

$$\frac{\partial h}{\partial x} + \frac{\partial z}{\partial x} + \frac{|Q|Q}{C^2A^2R} = 0, \qquad \text{say } \frac{\partial z}{\partial x} = I_o$$

The equation can be discretised to a finite difference form:

$$\frac{\partial h}{\partial x} \approx \frac{h_{j+1} - h_j}{\Delta x}$$

$$h_{j+1} = h_j + \Delta x \{\frac{Q^2}{C^2A^2R} - I_0\}_j \qquad (3\text{-}9)$$

To get a higher accuracy, an improved Euler method is used to solve equation (3-9) in the following steps:

(i) Predicton steps:
In this step, the friction term is calculated by using the flow condition at grid point j th. The result of this step is a prediction of the flow condition at grid point (j+1)-th.

(ii) Correction steps:

In this step the friction term is calculated by using the average value of the flow condition at grid point j-th, and the predicted flow condition at grid point (j+1) th.

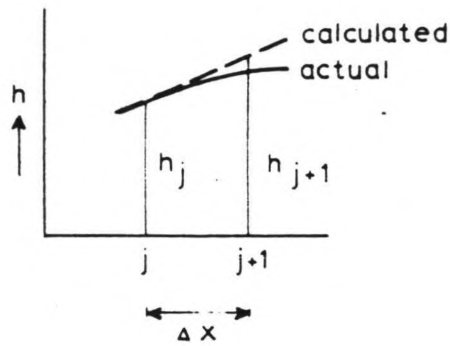


Figure 3.8 Discretised water surface

The computation can be carried out if the geometry condition, upstream boundary condition, $Q_{(t)}$, and downstream boundary condition, $h_{(t)}$ are known. The water level at the downstream boundary condition is computed by considering an overflow spillway type provided at the downstream boundary point.

In case a rigid lid condition is considered, the water level in the sand-trap will only be determined by input discharge and hydraulic design of the provided spillway at the downstream boundary point. The water level will be the same along the sandtrap.

3.2.3 Sediment Model

The sediment model is a part of a sandtrap submodel aimed for simulating the change of concentration in a sandtrap and simulating the bed-level changes due to the settling of sediment particle. In this model two basic equations are used:

(i) Equation of adapting concentration

$$s_{(x)} = \frac{B_0}{B_1} s_0 - \{\frac{B_0}{B_1} s_0 - s_1\} (1 - \text{Exp} \frac{-\alpha x}{h_1}\} \qquad (2-26)$$

where:

$$\alpha = 0.24 \, \{\frac{h_1}{k_n}\}^{1/6} \, . \, \frac{w_s}{u} \, . \, [1+8 \, \{\frac{h_1}{k_n}\}^{1/6} \, \frac{w_s}{u}] \, [1+4.1 \, \{\frac{h_1}{k_u}\}^{-0.25}] \qquad (2\text{-}27)$$

(ii) Continuity of sediment

$$\frac{\partial z}{\partial t} + \frac{\partial s}{\partial x} = 0$$

The first equation can be calculated if the flow field condition (u, h), equivalent Nikuradse roughness height (k_n), and the equilibrium sediment transport are known. The equilibrium sediment transport can be analysed, for example, using the following formulas:

(i) Engelund and Hansen
(ii) Ackers and White
(iii) Einstein and Brown
(iv) van Rijn.

For calculating the change of bed level, the continuity of sediment is used in the integration form:

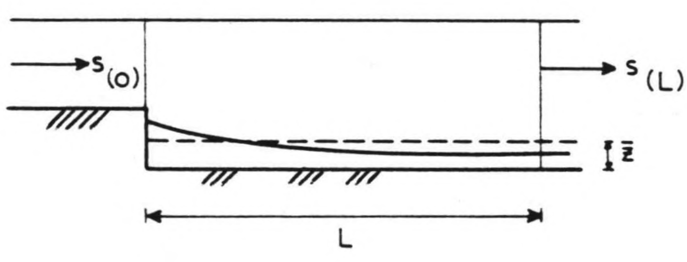


Figure 3.9 Bed level changes definition sketch

$$\int_0^L \frac{\partial z}{\partial t} \, . \, dx + \int_0^L \frac{\partial s}{\partial x} \, . \, dx = 0 \qquad (3\text{-}10)$$

$$\frac{\partial}{\partial t} \int_0^L z \, dx = s_{(0)} - s_{(L)}$$

$$\frac{\partial \bar{z}}{\partial t} = \frac{s_{(0)} - s_{(L)}}{L} \qquad (3\text{-}11)$$

Since the bed level is changing in the time under consideration and to get a stable solution, the equation (3-11) is solved by a predictor-corrector method.

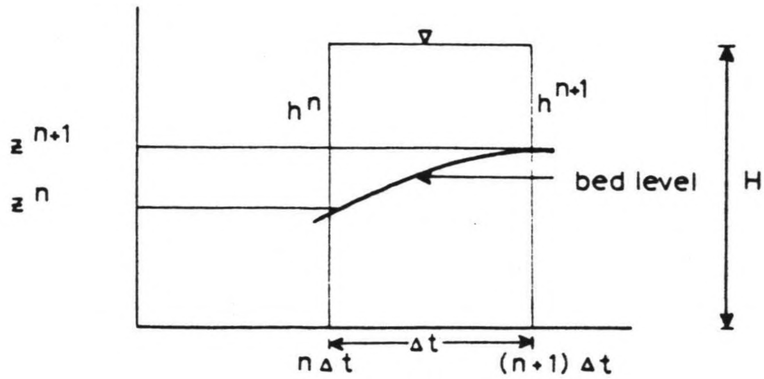


Figure 3.10 Discretised bed-level changes

Prediction steps:

$$z^* = z^n + \{ s_{(0)} - s_{(L)} \} \cdot \frac{\Delta t}{L} \tag{3-12}$$

Correction steps:

$$h^* = H^n - [\frac{z^* + z^n}{2}] \tag{3-13}$$

$$s^*_{(L)} = f\ [h^*, \text{ parameters}]$$

$$z^{n+1} = z^n + \{s_{(0)} = s^*_{(L)}\} \cdot \frac{\Delta t}{L} \tag{3-14}$$

Controlling step:

$\Delta = |z^* - z^{n+1}|$, if Δ is not acceptable, the same procedures should be repeated

where: * = denote predicted value at the new time level.

The generated truncation error by this numerical solution can be analysed using Taylor expansion. Complete analysis of the truncation error is presented in Appendix A-1.

3.2.4 Initial and boundary condition

Considering the former derivation, the following needed required initial and boundary conditions can be summarised:

(i) Geometry of the sandtrap: width ($B_{(x,o)}$, bottom elevation ($z_{(x,o)}$ and hydraulic design of an overflow spillway type at the end part of the sandtrap, are used as initial boundary condition.

(ii) Intake discharge, $Q_{(o,t)}$ and the entering sediment concentration, $c_{(o,t)}$ are used as upstream boundary condition.

(iii) Computed water level at the end part of the sandtrap is used as downstream boundary condition.

(iv) The internal boundary conditions applied in the model are:

$$z_{(x,t)} \geq z_{(x,o)}$$

$$s_{(x + \Delta x,\ t)} \leq s_{(x,t)}$$

3.2.5 Model verification

To know the characteristics and accuracy of the designed sandtrap model, two kinds of tests were carried out:

(i) Comparing the designed model with more advanced models for simulating the sedimentation and trap efficiency of a sandtrap. In this case SUTRENCH-2D model was used as comparison.

(ii) To use the model to re-simulate the sedimentation of a constructed and operated sandtrap in the field. For this purpose, field data of the Sapon sandtrap in Indonesia, collected by Wallingford Hydraulic Research, was used.

3.2.5.1 Comparison with SUTRENCH-2D model

The aims of comparison with SUTRENCH-2D model are not only to know the accuracy of the designed model to predict the trap efficiency and sedimentation in a sandtrap, but also to get a guidance for defining the required number of division of sub sandtrap (ΔL) to achieve an acceptable accuracy. In order to achieve the aims of tests, most applicable field parameters of transported sediment to an irrigation system, and ditto dimensions of sandtrap, were used.

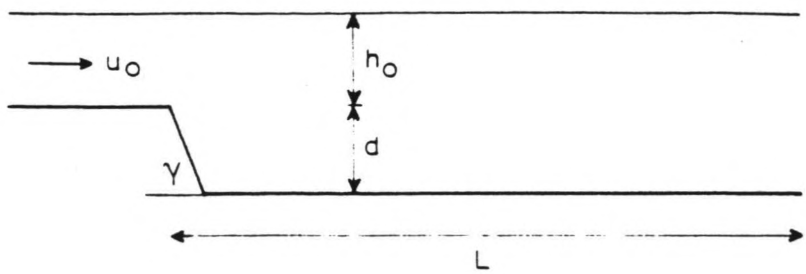


Figure 3.11 Sandtrap-parameter description

The efficiency of a sandtrap is a function of the following parameters (van Rijn, 1987):

$$e = F\ [\bar{u}_o;\ \frac{w_s}{u_{*,o}};\ \frac{k_n}{h_o};\ \frac{d}{h_o};\ \frac{L}{h_o};\ \gamma]$$

Considering the field condition of irrigaton and hydro-power plant purpose, the following parameters should be taken:

$\bar{u}_o$ = 1.0 m/s
D = 0.063 mm and 0.125 mm.
k_n = 0.15 m
h_o = 1.5; 2.0 and 2.5 m
d = 1.5 m

Using those values of parameters, the following tests were carried out.

Series	h_o [m]	D [mm]	w_s [m/s]	u_o^* [m/s]	C [$m^{1/2}$/s]	d/h_o	w_s/u_*
T1	1.5	0.063	0.0033	0.08	37.0	1.0	0.0412
T2	1.5	0.125	0.0133	0.08	37.0	1.0	0.141
T3	2.0	0.063	0.0033	0.079	39.7	0.75	0.0418
T4	2.0	0.125	0.0133	0.079	39.7	0.75	0.143
T5	2.5	0.063	0.0033	0.076	41.0	0.60	0.043
T6	2.5	0.125	0.0133	0.076	41.0	0.60	0.149

Table 3.1 Specificaton of verifying tests

For each series of test, the computation of sedimentation in the sandtrap and evaluation of trapping efficiency were carried out for the total time of 10 days of simulation. The upstream boundary, incoming discharge and concentration were kept constant. The downstream, water level, were also kept constant. Other computational data can be described as follows:

(i) Sutrench-model: $\Delta x = 5m$

$\Delta t = 3600$ seconds

$k_n = 0.15$ m

(ii) Designed model: $\Delta t = 0.25$ days

Δx = varied

Used sediment transport formula = varied

The results of the computations using SUTRENCH and Designed model on t = 10 days for simulation of sedimentation in the sandtrap and on t = 4 days and t = 8 days for analysis of trapping efficiency, are shown in the Figures 3.12 - 3.23.

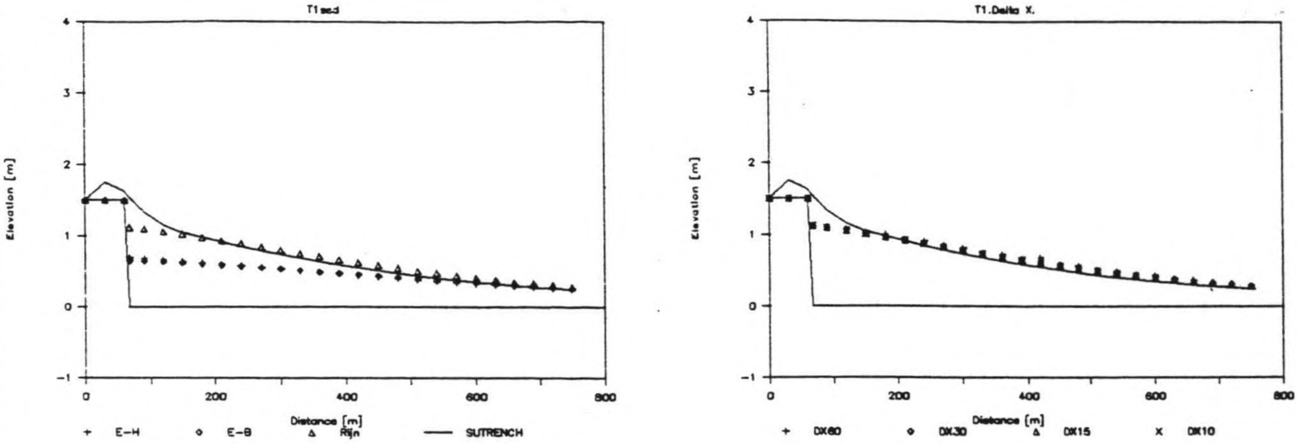


Figure 3.12 T1 Sedimentation test results

(a) Varying used sediment transport formula.
(b) Varying delta x.

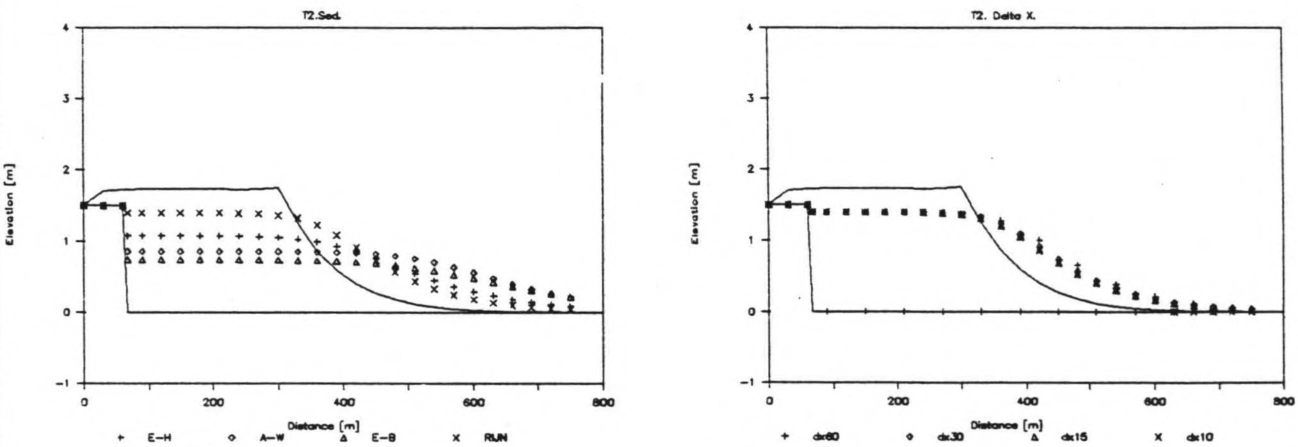


Figure 3.13 T2 Sedimentation test results

(a) Varying used sediment transport formula.
(b) Varying delta x.

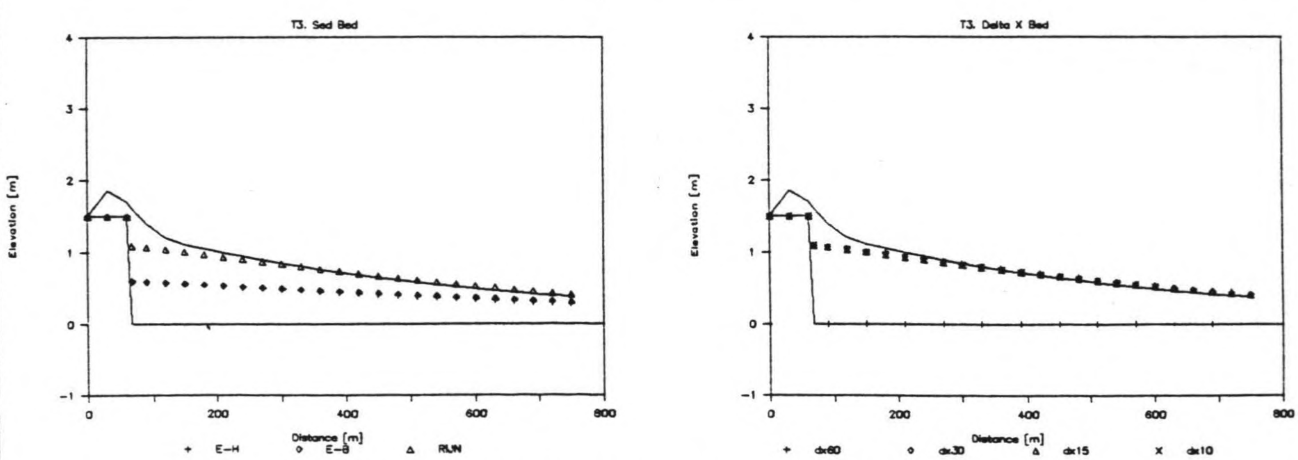


Figure 3.14 T3 Sedimentation test results

(a) Varying used sediment transport formula.
(b) Varying delta x.

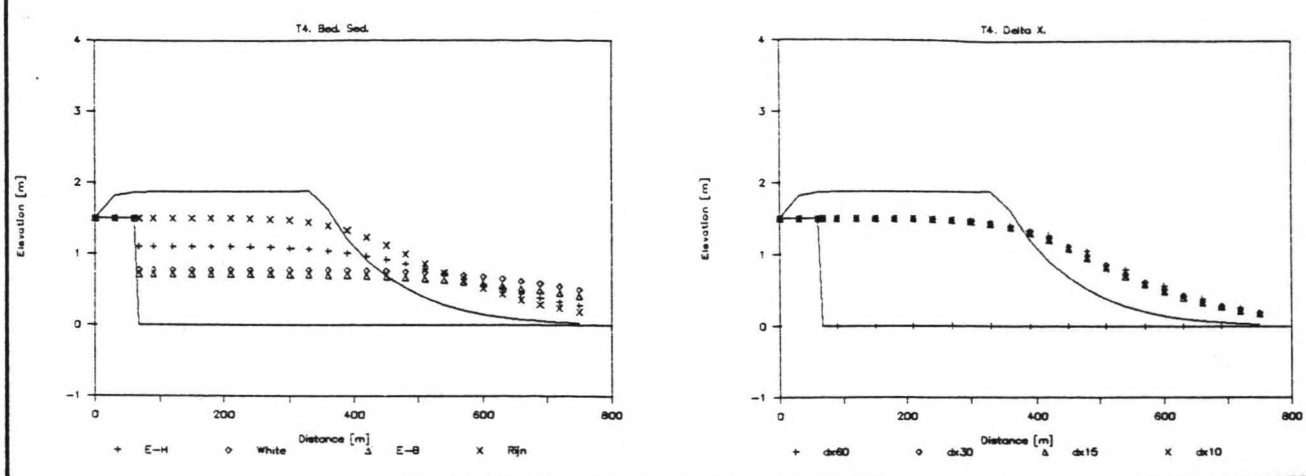


Figure 3.15 T4 Sedimentation test results

(a) Varying used sediment transport formula.
(b) Varying delta x.

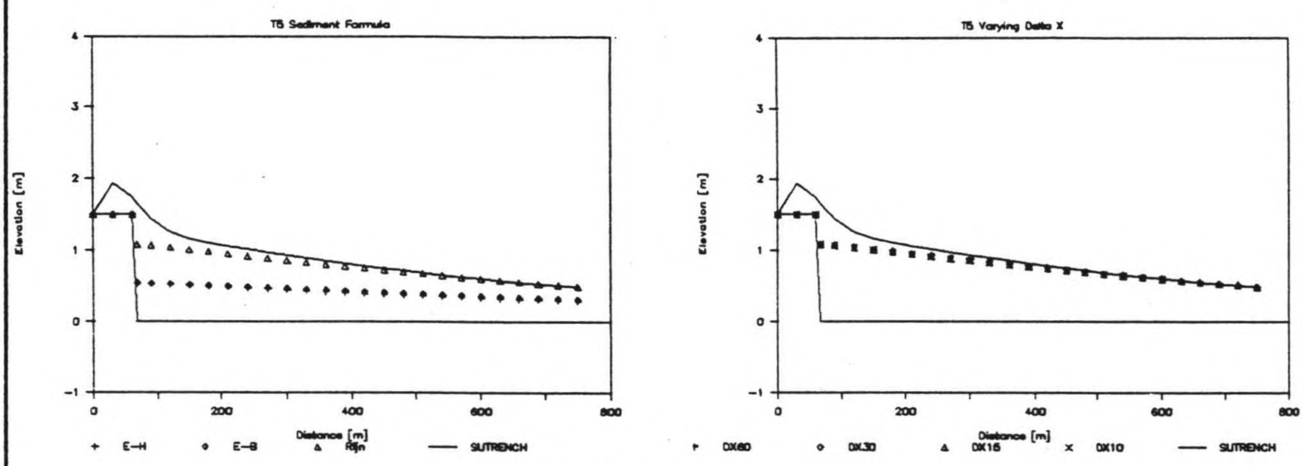


Figure 3.16 T5 Sedimentation test results

(a) Varying used sediment transport formula.
(b) Varying delta x.

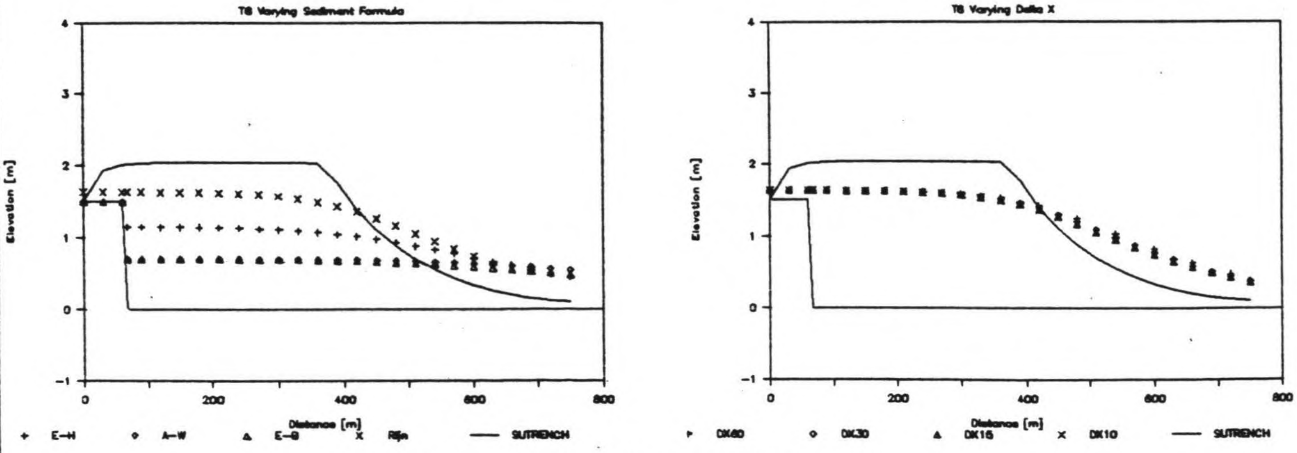


Figure 3.17 T6 Sedimentation test results

(a) Varying used sediment transport formula.
(b) Varying delta x.

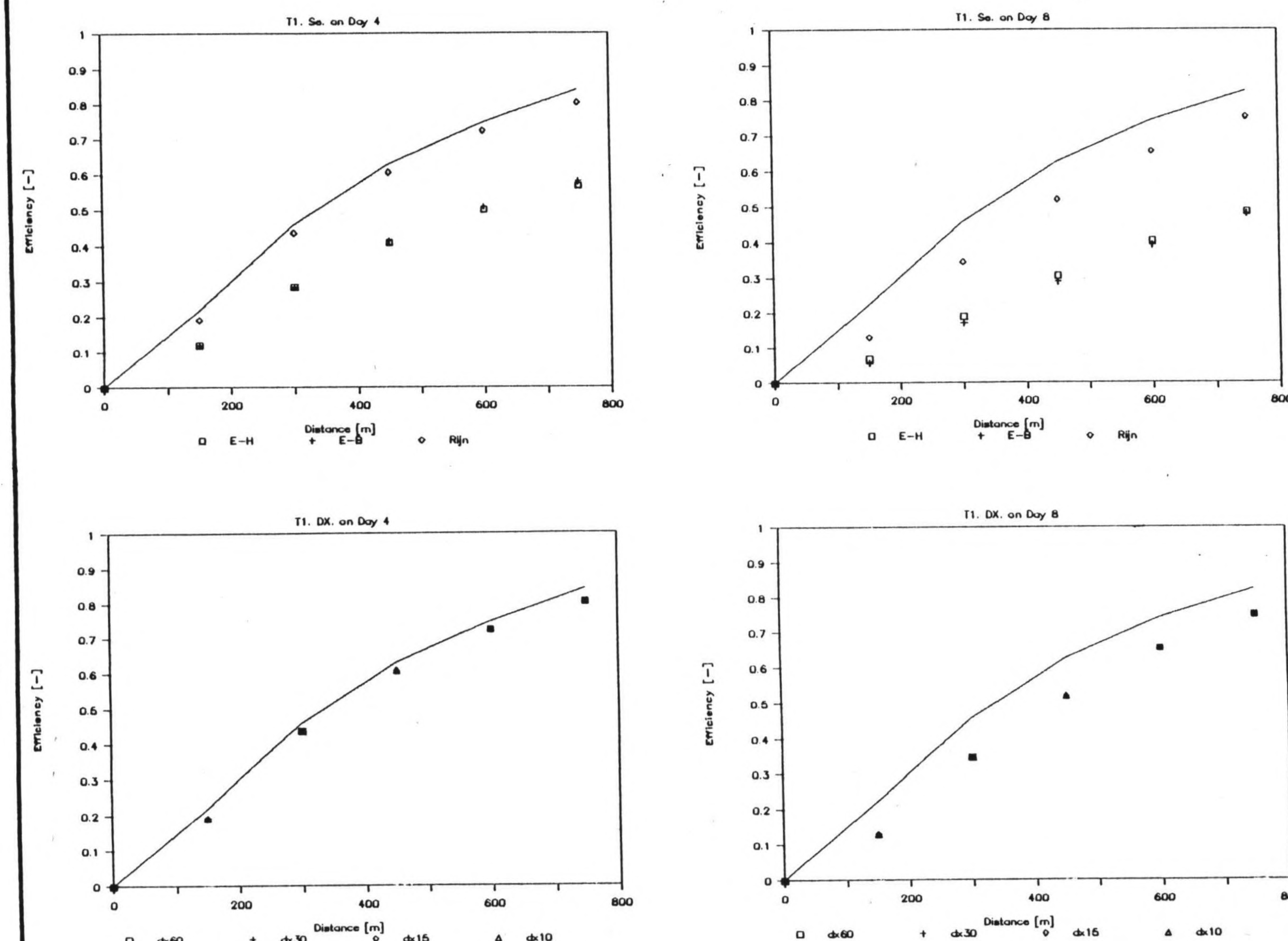


Figure 3.18 T1 Effeciency test results.

(a) & (b) Varying used sediment transport formula.
(c) & (d) Varying delta x.

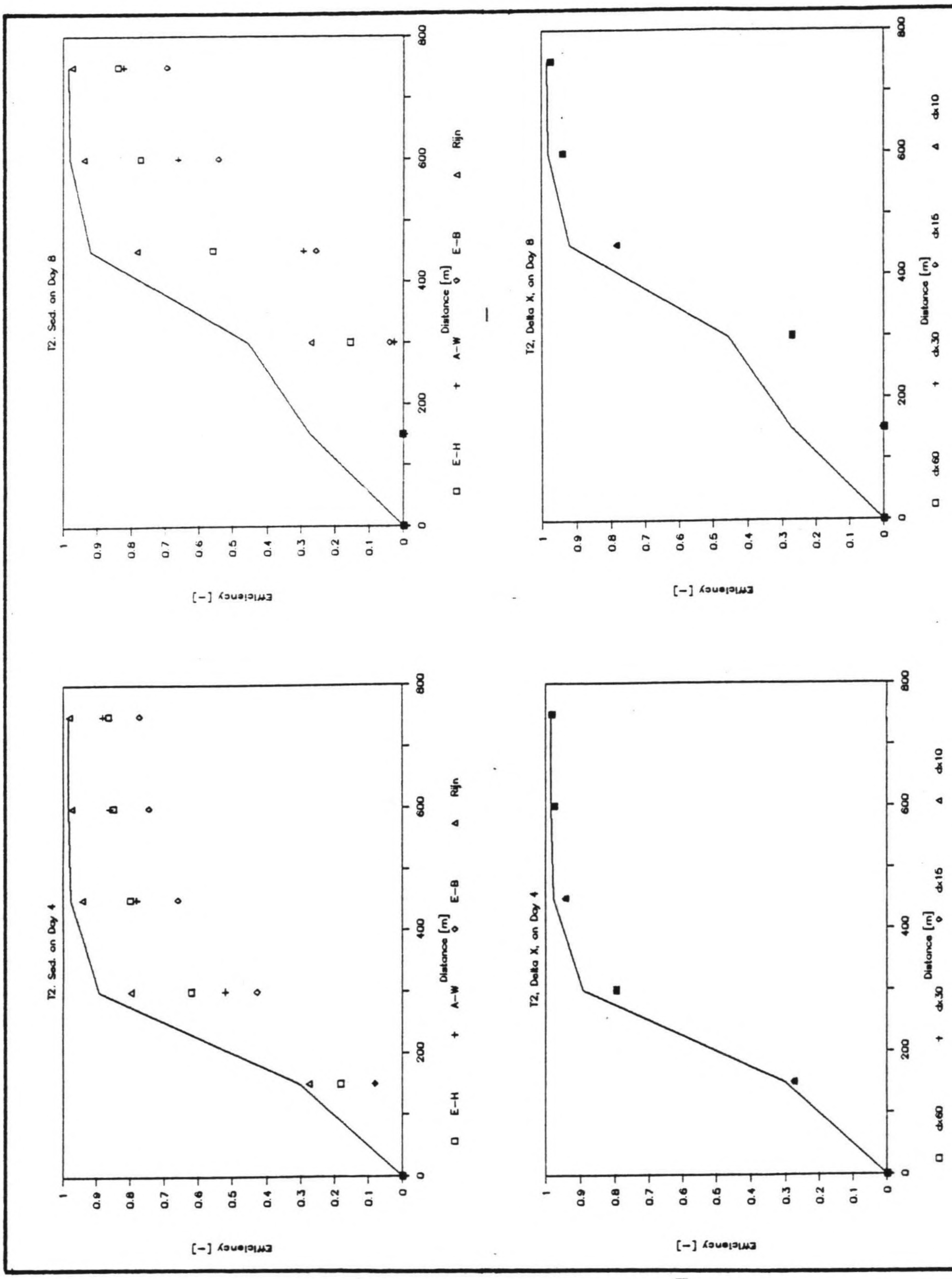


Figure 3.19 T2 Effeciency test results.
(a) & (b) Varying used sediment transport formula.
(c) & (d) Varying delta x.

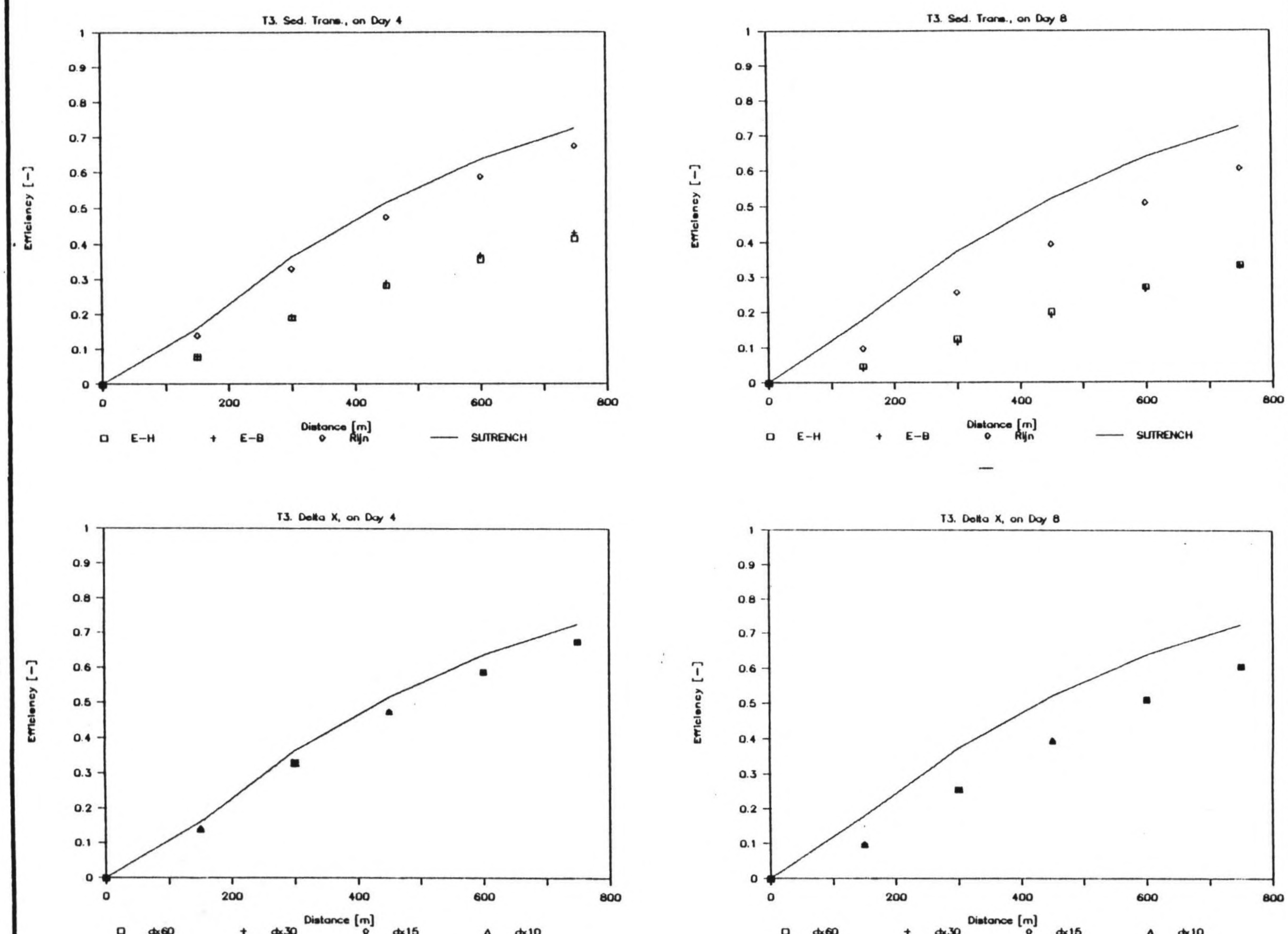


Figure 3.20 T3 Effeciency test results.

(a) & (b) Varying used sediment transport formula.
(c) & (d) Varying delta x.

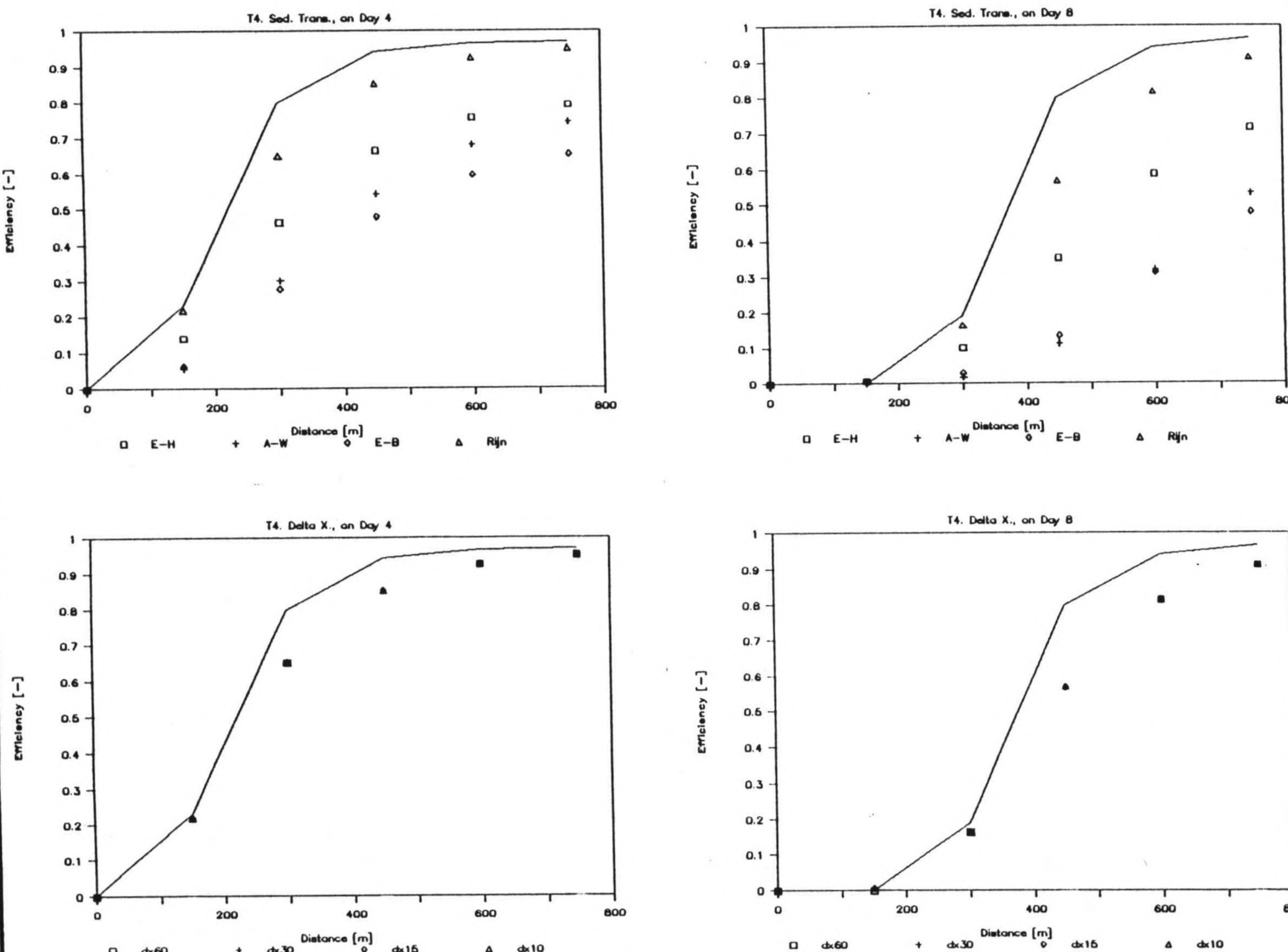


Figure 3.21 T4 Effeciency test results.

(a) & (b) Varying used sediment transport formula.
(c) & (d) Varying delta x.

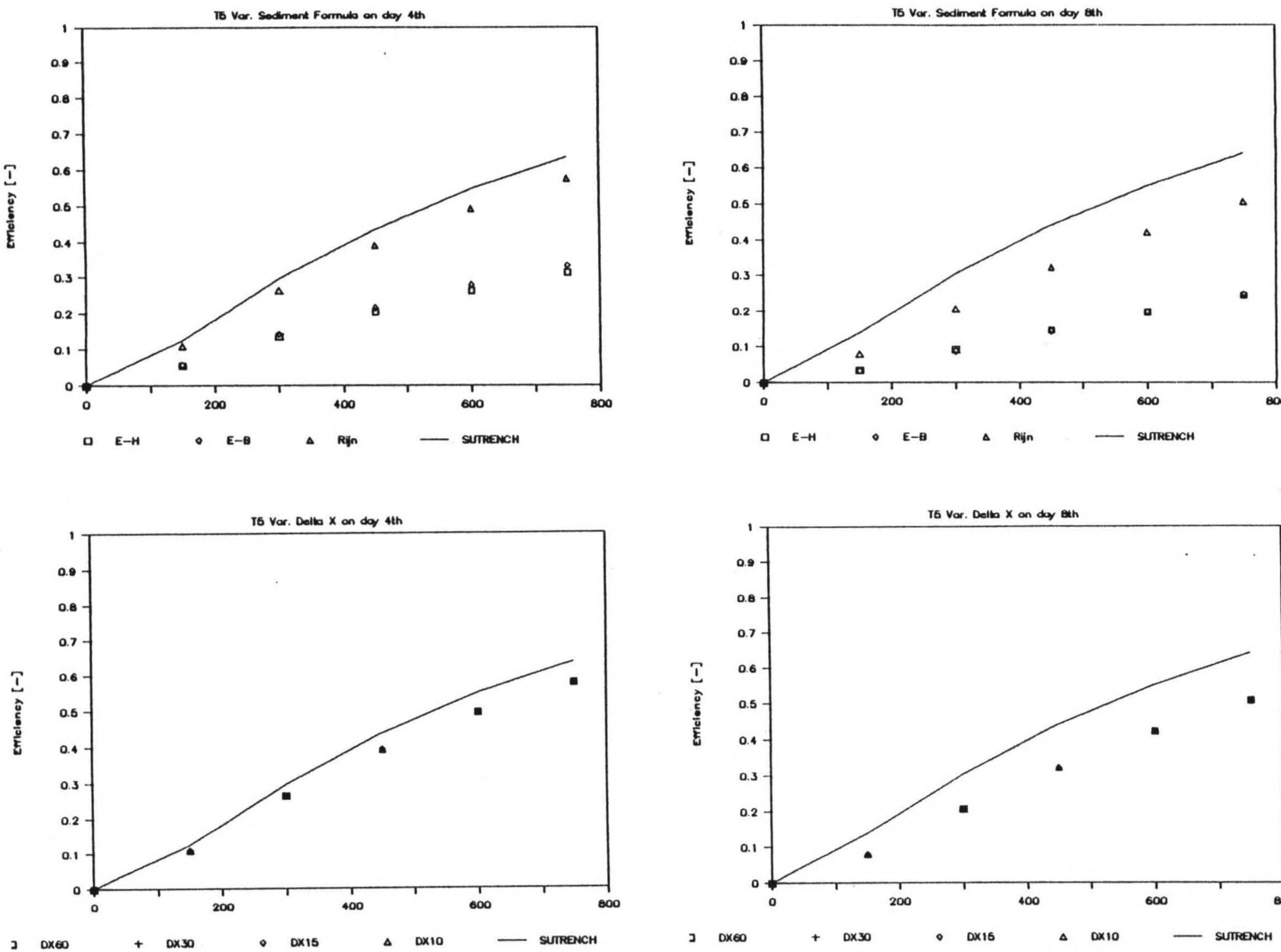


Figure 3.22 T5 Effeciency test results.

(a) & (b) Varying used sediment transport formula.
(c) & (d) Varying delta x.

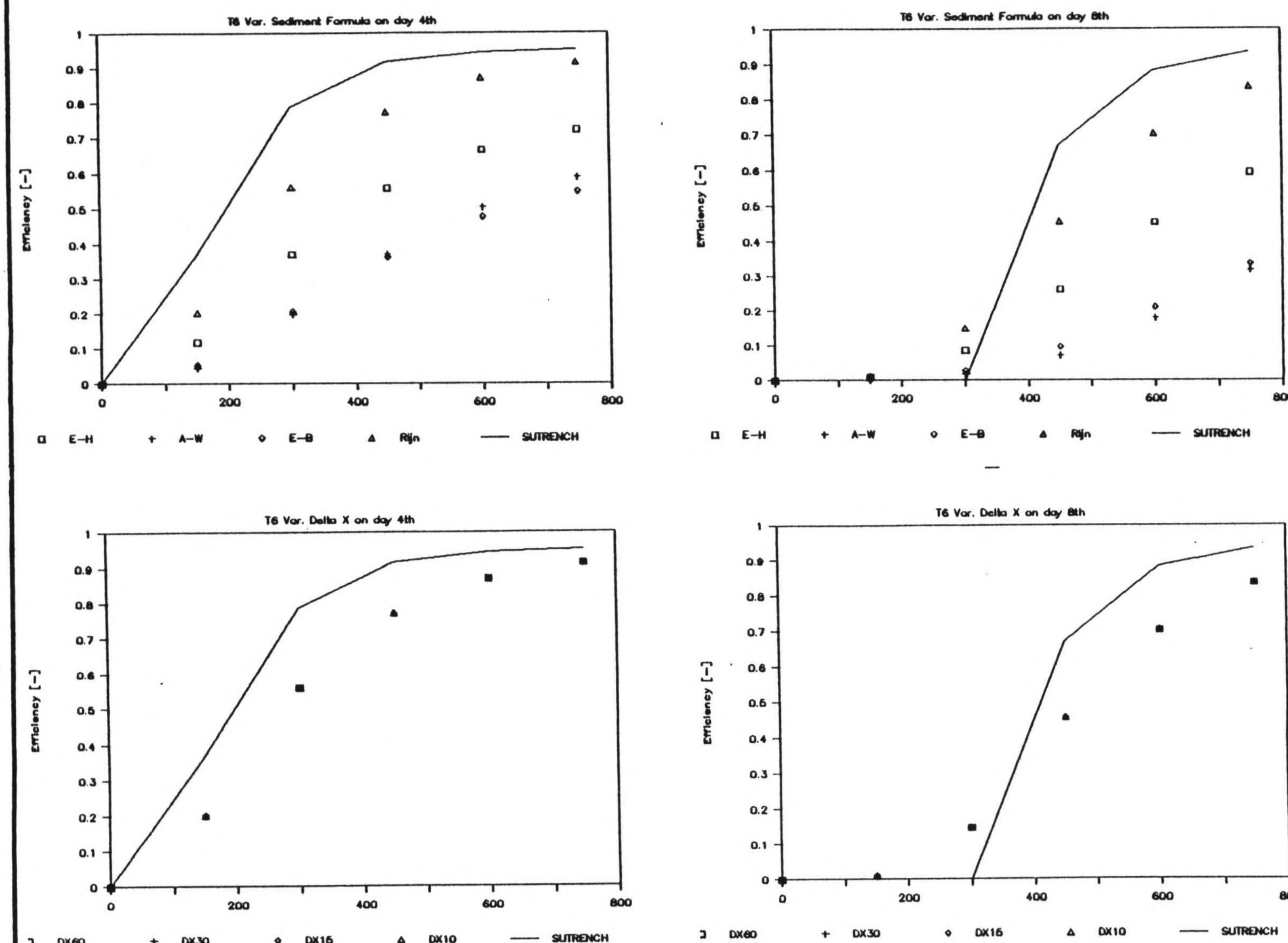


Figure 3.23 T6 Effeciency test results.

(a) & (b) Varying used sediment transport formula.
(c) & (d) Varying delta x.

From the comparison with SUTRENCH model, the following conclusion can be drawn:

(i) The model is very sensitive to the used sediment-transport formula for calculating the equilibruim sediment-transport capacity. In this test the Van Rijn formula gives the closest computaton result to the SUTRENCH model, which is understandable because this formula is also used in the SUTRENCH model.

(ii) Varying the Δx or numbers of division of sandtrap do not give considerably different results. Considering this result, in order to use this model, Δx can be used in a range between 25-50 m.

(iii) The phasing of sedimentaton in the sandtrap are different between SUTRENCH model and designed model. The result of the SUTRENCH model show that more sedimentation occurs in the upstream part of the sandtrap and then moving downstream-ward. The present model shows that sedimentation is distributed along the sandtrap. The total amount of sedimentation is said to be the same. It can also be concluded by considering the comparison of trap efficiency of the sandtrap.

(iv) In order to simulate the trap efficiency of a sandtrap, the present model gives an error range from 5-10% during the whole simulation time, compared to the result of SUTRENCH model. The model gives more accurate results in the first part of simulation time and start to deviate later due to some differences in the sedimentation processes.

3.2.6 Comparison with field data

In order to know the capability of the present model for simulating the sedimentation processes in a sandtrap, analysing the trap efficiency and how the model can be calibrated with field data, the model was tested with the Sapon sandtrap data. The existing data collected by Wallingford Hydraulics Research are:

(i) General hydraulic design data of the Sapon sandtrap in Indonesia

(ii) Flow velocity data of some sections in the sandtrap with 3 points in the vertical of each section.

(iii) The grain-size distribution of sedimented material along the sandtrap.

The existing data are shown in Figure 3.24 - 3.27.

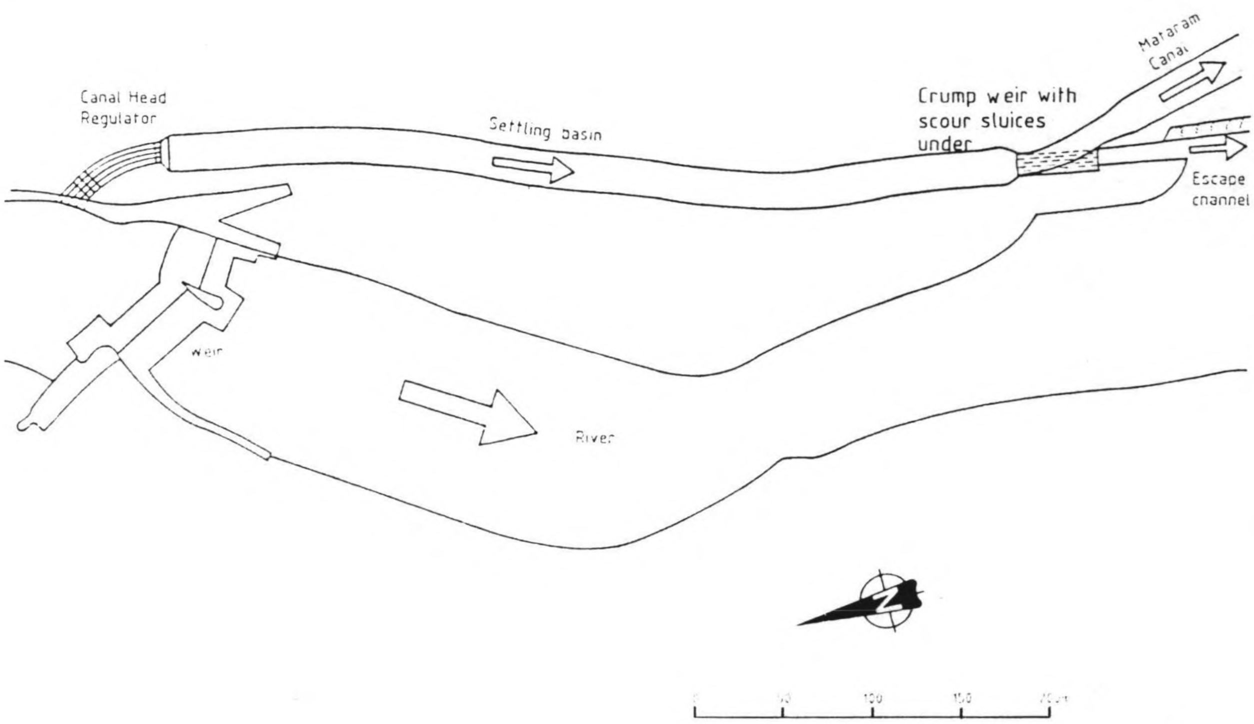


Fig. 3.24 Sapon sandtrap in the Mataram canal headworks

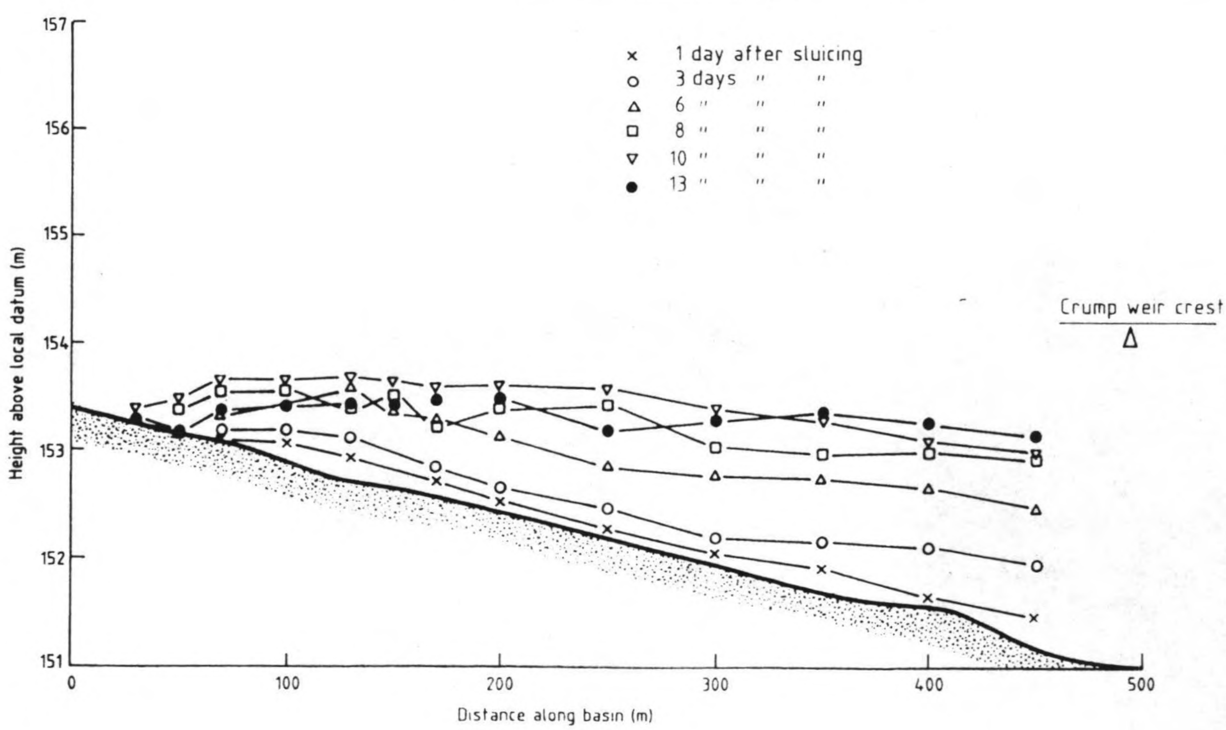


Fig. 3.25 Mean bed level changes.

(After Fish, 1987)

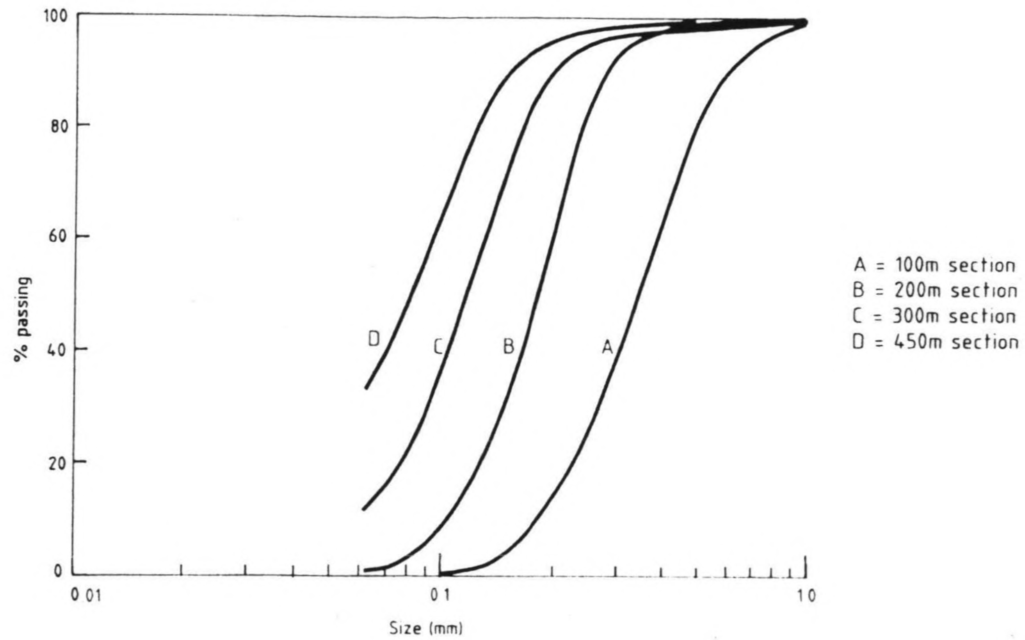


Fig. 3.26 Size grading of the bed samples collected 3 days after sluicing.

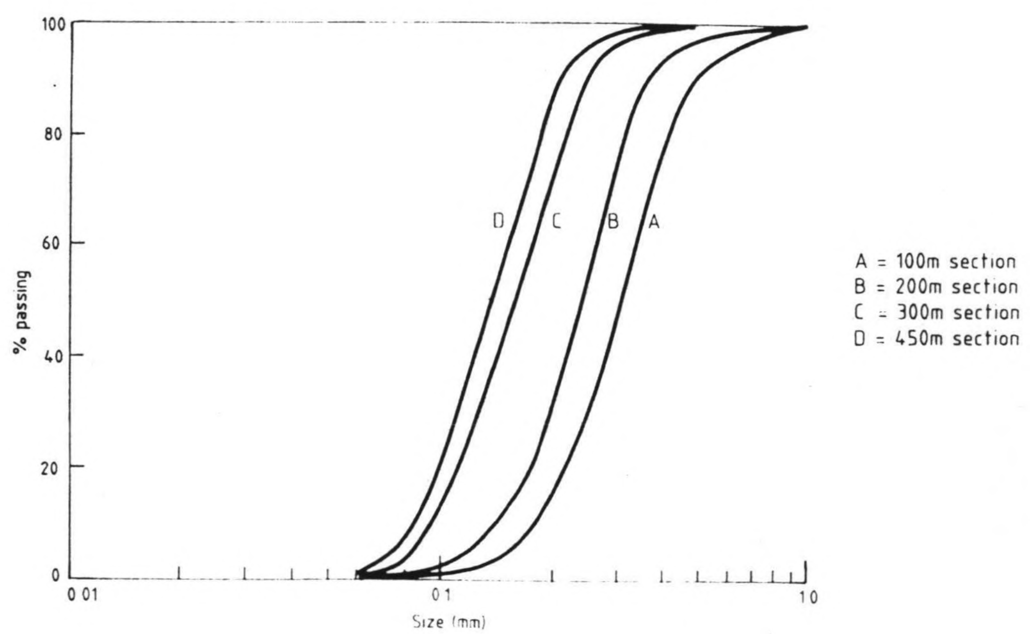


Fig. 3.27 Size grading of the bed samples collected 13 days after sluicing.

(After Fish, 1987)

The diverted discharge from the river to the sandtrap was described about 20 m^3/s during the normal operation. The sediment concentration entering to the sandtrap was not measured. The mean concentration of bed material entering the sandtrap over the duration of the field measurement was obtained from the concervation of mass of sediment. This was 476 ppm for total load and 215 ppm when the wash load is included.

The schematisation taken for simulating the sedimentation and trapping processes are as follows:

(i) The sandtrap dimensions are: L = 500 m, bed slope = 4.72×10^{-3}, bottom width = 12.5 m and side slope = 1:1.

(ii) The discharge entering the sandtrap is 17.5 m^3/s constant. This value is obtained by analysing the measured flow velocity and the wetted area.

(iii) Assuming that the several grain-size distributions of settled material along the sandtrap can represent the grain-size distribution of transport sediment, the average of those grain-size distribution can be taken as the starting value for defining the representative diameter used in the calculation. This value will be corrected by analysing the phasing of sedimentation in the sandtrap.

(iv) The sedimentation condition on t = 3 days is taken for calibrating and the other data used for checking the capability of the model.

Others computation data used in the model are:

Time steps : 0.25 days
Δx : 25 m
Used transport formula: Engelund-Hansen and van Rijn

From the simulation results shown in Figure 3.28, the following conclusions can be drawn:

(i) The representative diameter d = 0.085 mm should be taken for simulating the phasing of sedimentation on t = 3 days

(ii) The concentration of sediment entering the sandtrap should be higher than c = 215 ppm. Average concentration c = 450 ppm matches the field data better, as was concluded by considering the sedimentation in the sandtrap and the concentration entering the irrigation canal.

(iii) The sedimentation in the end part of the sandtrap is difficult to be simulated. The uncertainty of some data generates more possibilities in simulation, on the other hand will cause some difficulties.

(iv) To simulate the sedimentation in a whole part of the sandtrap, the characteristics of the entering sediment, for example representative diameter, should be varied in the time under consideration.

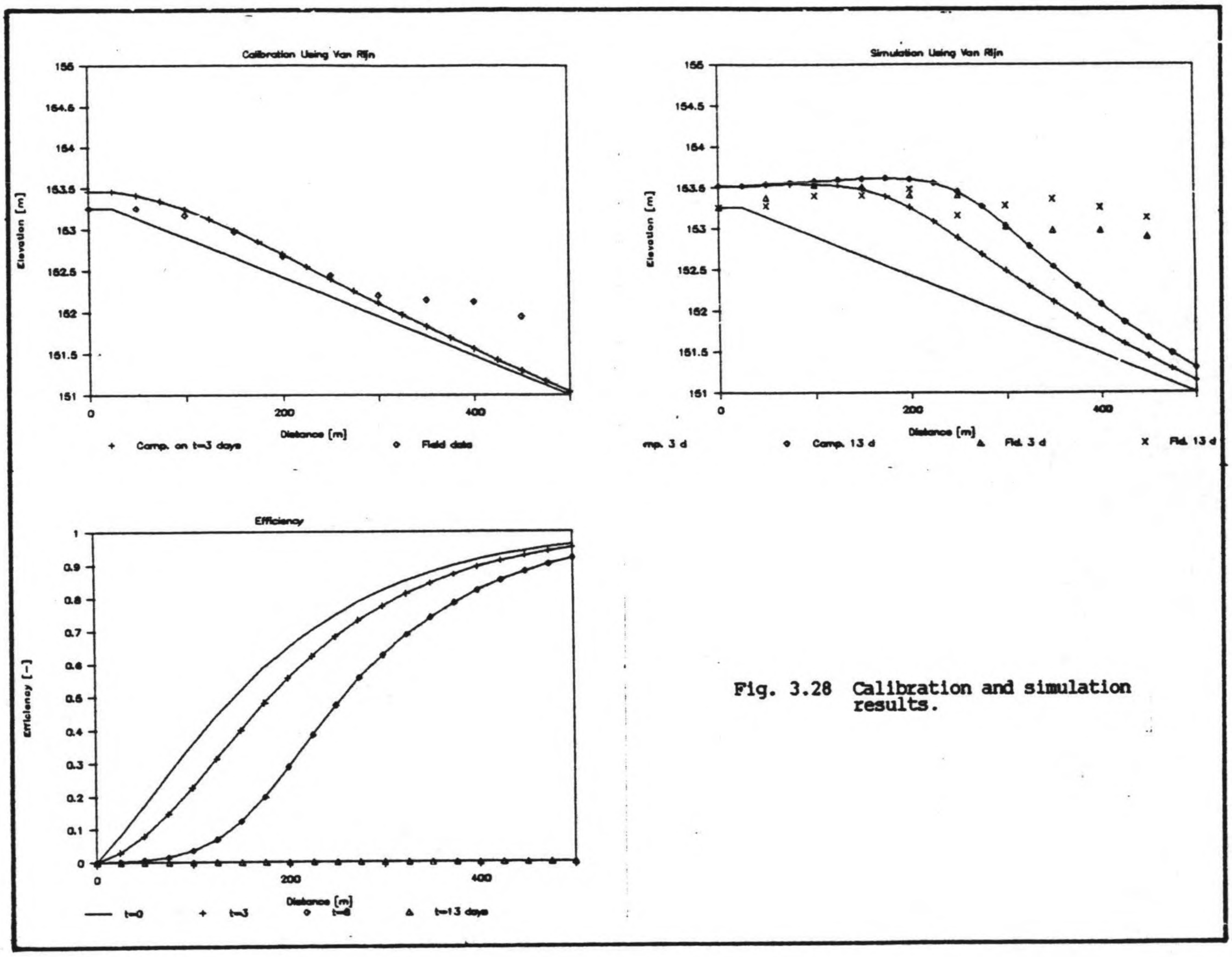


Fig. 3.28 Calibration and simulation results.

3.3 Subroutines for supporting the model

To carry out the whole trapping analysis of sediment exclusion devices, listed subroutine are needed:

(i) Subroutine for analysing the settling velocity of a sediment particle in different condition. For this purpose, a formula given by Gibbs et al [1971] is applied.

(ii) Subroutine for analysing the sediment transport capacity of a flow field in a equilibrium condition to enable some possibilities for calibrating the sandtrap model with field data, the following sediment transport formula are provided in the model:

a Engelund-Hansen formula

b Ackers-White formula

c Einstein-Brown formula

d Van Rijn formula

A description of those formula are listed in Appendix A-3

(iii) Model for analysing the transport capacity of an irrigation canal. The model was designed for predicting the problem that will happen in the main canal, based on the amount and characteristic of the entering sediment, canal discharge and hydraulic design of the canal.

(iv) Subroutine for rearranging the flow condition and the characteristics of transported sediment after passing a sediment exclusion device.

The routine was designed based on the continuity of water and the plotted grain size distribution of transported sediment in a log probability paper.

3.4 Configuration of input-output data

The input and output data are handled in a configuraton shown in Figure 3.29

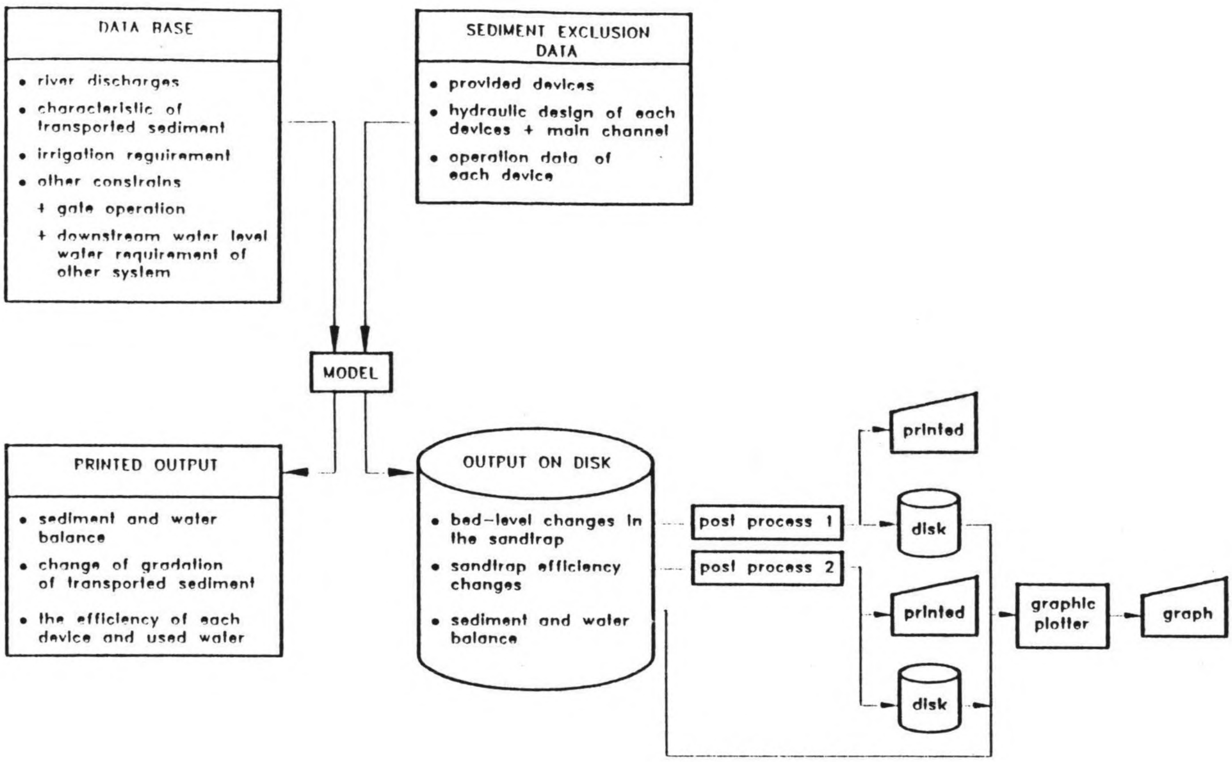


Figure 3.29 Configuration of handling the files

Note:

Post processing 1 model is designed for presenting the bed-level changes in the sandtrap on the specified days

Post processing 2 model is designed for presenting the sandtrap efficiency changes on the specified days.

Constrains in the data base must be considered by the user when deciding an operation strategy of the sediment exclusion devices.

4. Application of the Designed Model on a Field Problem

It is useful to test the designed model for its ability in optimising the operation of sediment exclusion devices for an existing or a designed system. The field and design data of the Ye U Irrigation Project in Burma will be used for this purpose.

4.1 Problem Description

The Kabo Weir System is a part of the existing irrigation scheme in the Northern part of Burma, irrigating about 93,000 hectares of rice field on its left bank via the Shwebo irrigation scheme and 51,800 hectares of rice field on its right bank via the Ye U irrigation scheme. The Ye U intake which is located at the inner bend of the Mu river is facing a sediment problem.

In order to minimise the amount of sediment entering to the Ye U canal system, a sediment ejector or another more appropriate structure is proposed to be constructed upstream of the intake in combination with a sand trap in the Ye U main canal.

The existing Kabo weir has on its crest 91 shutters which can only lowered in a group of 10 to 15 shutters. The shutters can be pushed down by the upstream water pressure. When the river discharges are very low (after the monsoon). The vertical position of the shutters can only be achieved by manual operation. Later, this constrain influences the undersluice gate operation in order to prevent unnecesserally lowered down of the shutters.

Based on the collected field data and an intensive investigation using a hydraulic scale model and mathematical models, Delft Hydraulics proposed the following items for overcoming the sediment problem at the Ye U intake:

(i) The hydraulic design of a tunnel-excluder type of sediment extractor, which should be constructed upstream of the Ye U intake.
(ii) The hydraulic design of a sandtrap, completed with its characteristics and a guide line for the operation and maintenance, which should be constructed downstream of the Ye U intake.
(iii) A guide line of the gate operation of the Kabo weir and a prediction of the morphology changes of the Mu river.

Based on the above data, a complete simulation of design steps for optimising the proposed sediment exclusion devices will be set up.

4.2 Model Schematisation

In order to simulate the proposed devices, the following schematisations and assumptions will be used:

(i) The gate operation of the Kabo weir and the setting upstream water level will be taken from the proposed guide line as shown in table 4.1.

(ii) Considering the structure of the Kabo weir, the tunnel excluder should also be used as a sluice way to regulate the upstream water level.

(iii) The sediment concentration entering the system as a function of river discharge, shown in figure 4.5., will be used. These data were analysed, based on the field data.

(iv) The grain size distribution of river bed material is shown in figure 4.3. The grain size distribution of the transported sediment will be analysed through a total sediment transport formula. For this analysis, the relation of water depth, river width and flow velocity in the river as a function of river discharge shown in figure 4.4. will be used.

(v) If there is enough water in the river, the maximum requirement discharge of the Ye U irrigation scheme will be diverted. Otherwise the Shwebo irrigation scheme has a priority or alternating supply.

(vi) Since there is no sub model for simulating the flushing operation of a sandtrap, the resulted data from the hydraulic scale model concerning this operation will be used. The flushing can only be done in the flushing window 100 $m^3/s \leq Q$ river ≤ 200 m^3/s. The needed flushing discharge is 50 m^3/s and it will need about one or two days of flushing operation.

(vii) Considering the available field data and variety of river discharges, the time interval between September 1, 1987 up to November 15, 1987 will be simulated.

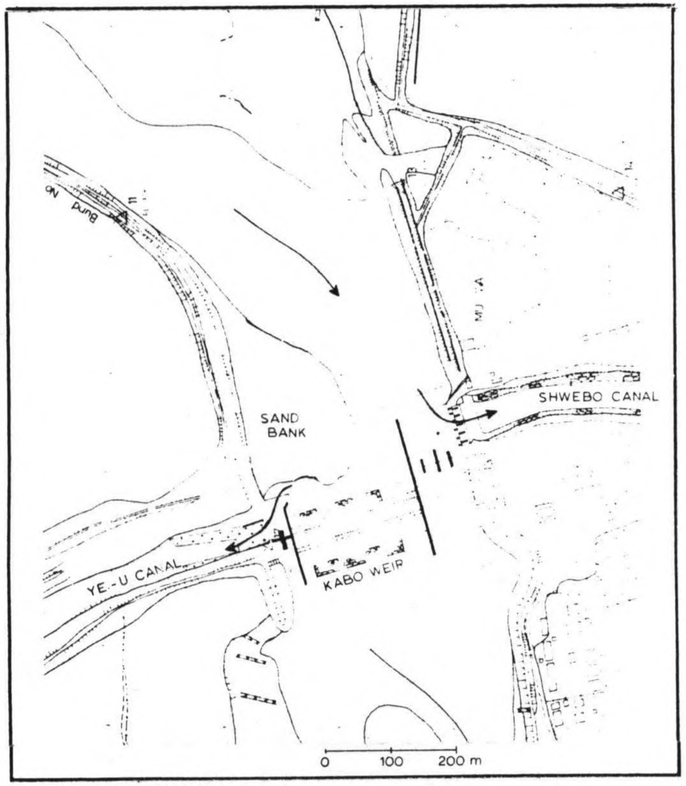


Fig. 4.1 Layout of the Kabo system.

Discharge Mu River (m^3/s)	water level u/s (m)	Shwebo intake	Ye-U intake	Kabo Weir shutters	Shwebo sluices	Ye-U * sluices/ ejector
0-140	< 110.5	open	open	up	closed	half open
> 140	110.5-110.6	open	open	up	closed	open
156-207	109.9-110.6	open	open	10/91 down	closed	open
207-269	110.2-110.6	open	open	23/91 down	closed	open
269-350	110.3-110.6	open	open	40/91 down	closed	open
350-595	110.1-110.6	open	open	down	closed	open
595-929	110.6-111.4	open	open	down	closed	open
929-1014	111.2-111.4	closed	closed	down	1/4 open	open
1014-1085	111.2-111.4	closed	closed	down	2/4 open	open
1085-1110	111.3-111.4	closed	closed	down	3/4 open	open
1110-1150	111.3-111.4	closed	closed	down	open	open
> 1150	> 111.4	closed	closed	down	open	open

Table 4.1 Proposed gate operation.

(After Perdijk, 1989)

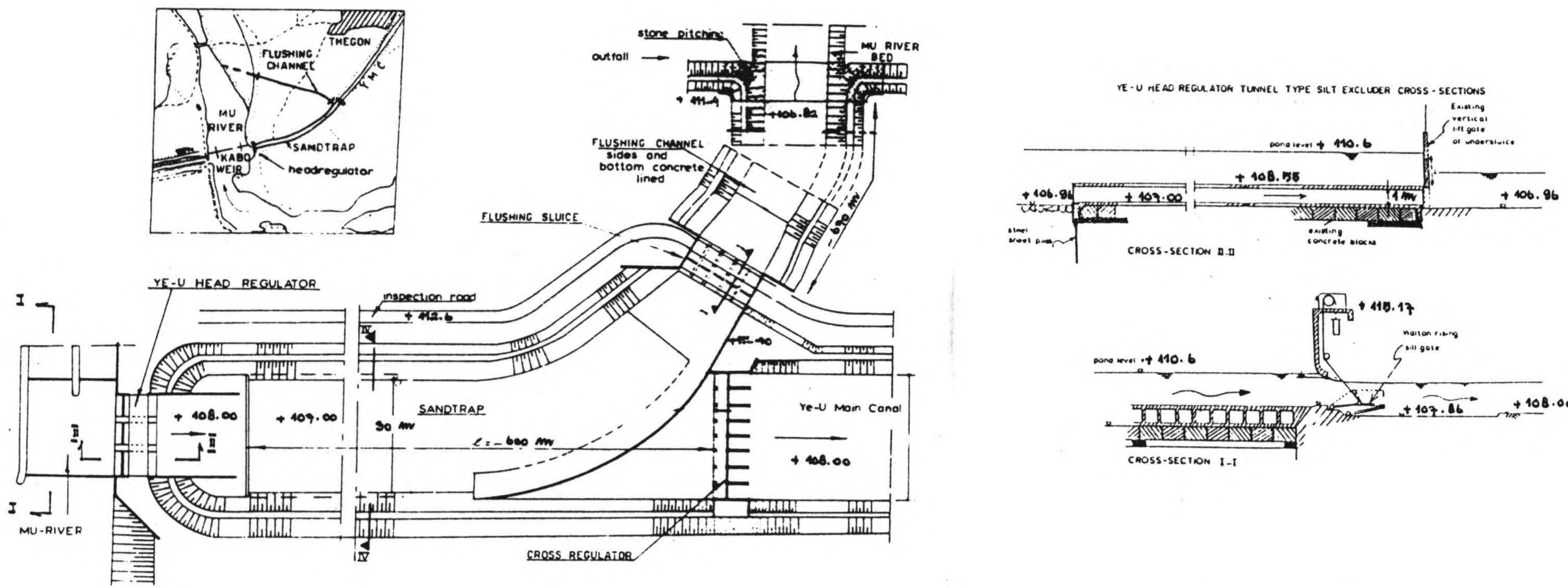
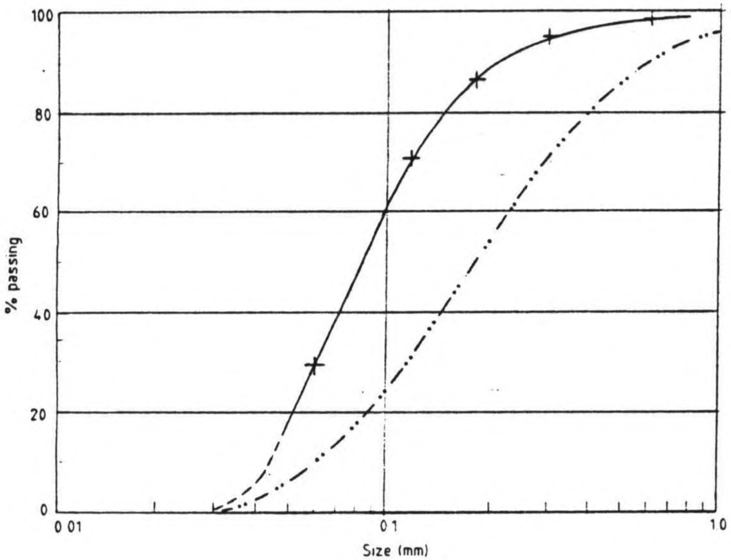


Fig. 4.2 Design hydraulic of the proposed sediment exclusion devices.

(After Perdijk, 1989)



—··—: bed material
——— : transported material

Fig. 4.3 Grain size distribution.

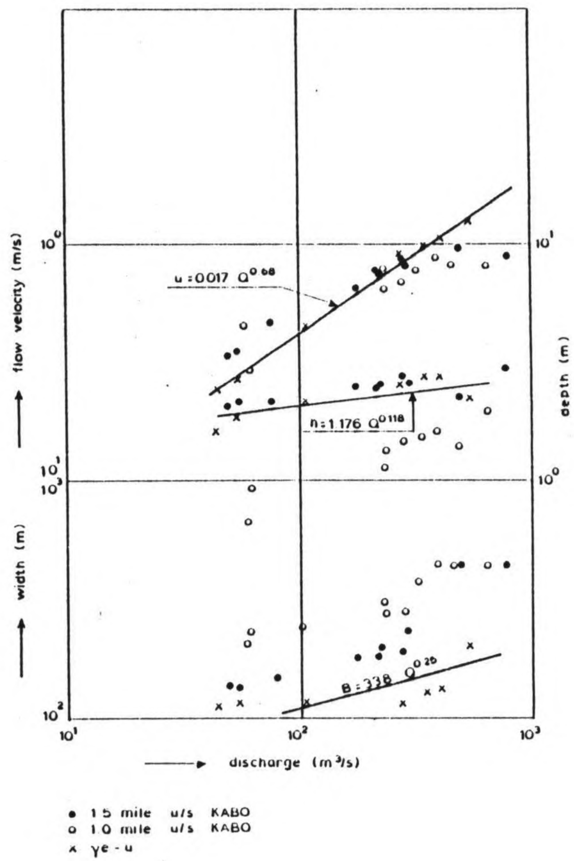


Fig. 4.4 Regime equation for Mu river.

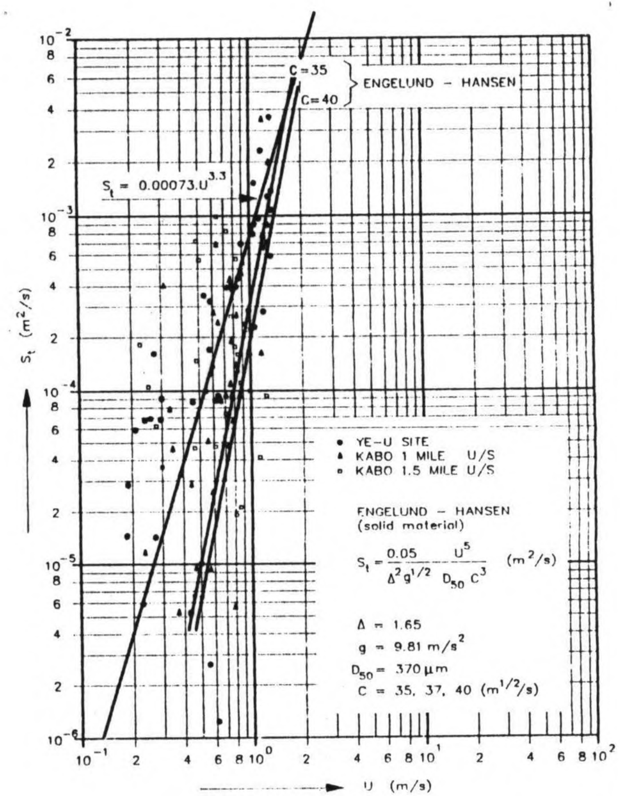


Fig. 4.5 Measured total bed material transport in Mu river.

(After Perdijk, 1989)

The following strategies with its specification as described below will be simulated using the designed model :

(i) Strategy 0.
The existing situation without an appropriate sediment exclusion device. The simulation is aimed at getting an insight prediction of the facing sediment problem in the field. The simulation will also be used to calibrate the sediment transport capacity of the main canal.

(ii) Strategy 1.
A tunnel excluder is provided upstream of the Ye U intake. The hydraulic design of the tunnel excluder is shown in Figure 4.2. The tunnel excluder will be operated by considering the available discharge in the river. In this case the irrigation will not be disturbed by the operation of a tunnel excluder.

(iii) Strategy 2.
A sandtrap is provided downstream of the Ye U intake. The hydraulic design of the sandtrap is shown in figure 4.2. The sandtrap can only be flushed in the flushing window 100 $m^3/s \leq Q$ river $\leq 200\ m^3/s$. During the sandtrap being flushed, the irrigation will be disturbed.

(iv) Strategy 3.
The former two sediment exclusion devices are provided in the Ye U Headworks and will be operated in series.

(v) Strategy 4.
In the strategy 4 test, the provided sediment exclusion devices in the strategy 3 will be operated in combination with an intake gate regulation.

(vi) Strategy 5.
Considering the results of the hydraulic scale model, the flushing window of the proposed sandtrap can be widened by heightening the crest of the Kabo weir by 0.30 meter. In this case the sandtrap can be flushed in the interval of river discharges $100\ m^3/s \leq Q_{river} \leq 300\ m^3/s$. The advantages of the widening of the sandtrap flushing window will be tested in the strategy 5 test.

(vii) Strategy 6.

In the strategy 6 test, the strategy 5 condition will be combined with an intake gate regulation.

(viii) Strategy 7.

This strategy should be carried out considering the former test results. The test results show that the efficiency of the tunnel excluder is very high. The additional strategy 7 test were carried out to prevent an unpredicted problem caused by the following possibilities:

- The actual trapping efficiency of the tunnel excluder is lower than was predicted.
- The utilisation of the tunnel excluder as a sluice-way generates a higher concentration approaching the intake. To prevent this disadvantage, the tunnel excluder should be operated with a small water extraction ratio.

To simulate this condition, a smaller opening of the gate of the tunnel excluder will be set.

The specifications of each series of tests can be summarised and presented in table 4.2.

Strategy	Provided Sediment Exclusion Devices		Intake Gate Regulation	Heightening the weir crest
	T. Excluder	Sandtrap		
0	-	-	+	-
1	+	-	-	-
2	-	+	-	-
3	+	+	-	-
4	+	+	+	-
5	+	+	-	+
6	+	+	+	+
7	+	+	+	+

Table 4.2. Summary of The Test Specifications.

where : + denotes the device is provided or the effort is done.

4.3 **Test Results and Analysis**

The strategy 0 test results are presented in figures 4.6. The river discharges, irrigation requirement and supplied discharge to the irrigation system as a function of time are shown in figure 4.6.a. In order to minimise the volume of sediment entering the irrigation system, in the period of river discharges 500 $m^3/s \leq Q$ river $\leq 900 \ m^3/s$, the diverted discharge to the intake is lowered from 50 m^3/s to 30 m^3/s. The intake gate is closed in the period of river discharges higher than 900 m^3/s.

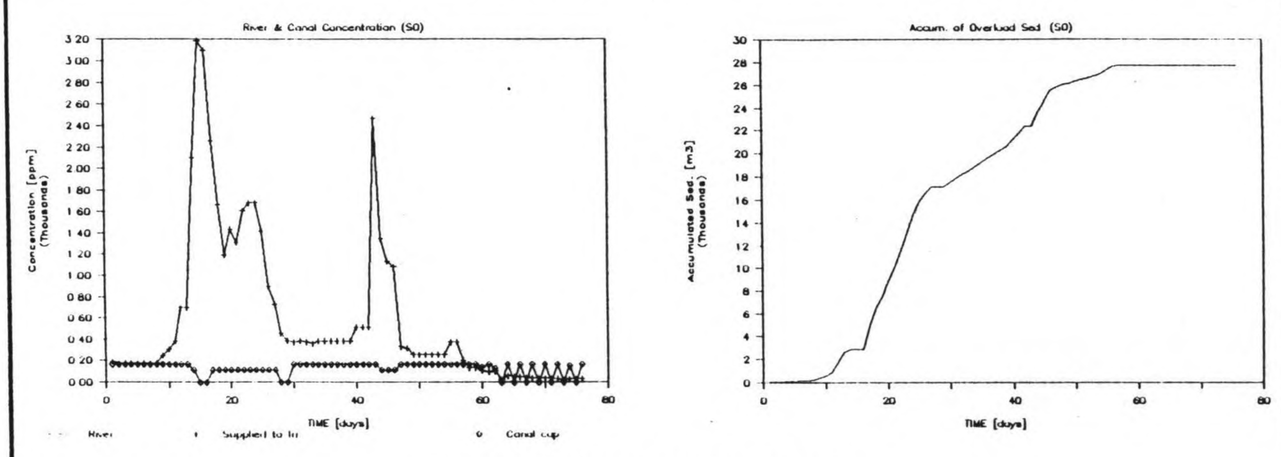


Figure 4.6 Strategy 0 test results.

Figure 4.6.b. shows that with this measure only, during the simulated time-period the system is still facing a problem of about 27,756 cubic meters of accumulation of the sediment that is entering the irrigation system and cannot be transported by the canal due to overload condition. The problem cannot only be solved by the intake gate regulation because it will come to a condition that the intake gate should be closed or lowered during most of the simulated time period.

The strategy 1 test results are presented in figures 4.7. In the strategy 1 tests, the intake gate is closed during the period of river discharges higher than 900 m^3/s and no other intake gate regulation is applied for minimising the entering sediment.

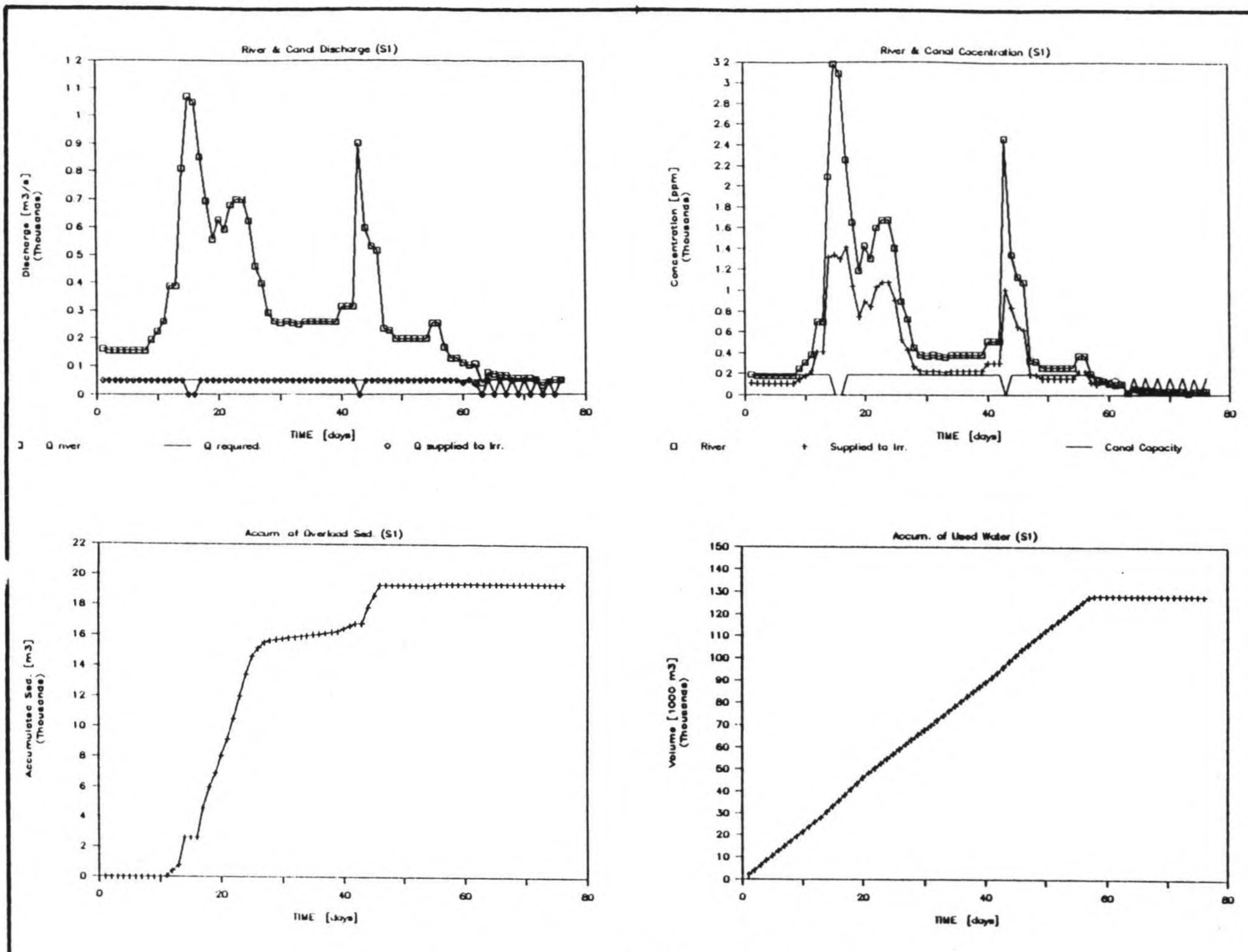


Figure 4.7 Strategy 1 test results.

Figure 4.7.a. shows the way how the intake gate is regulated. In Figure 4.7.b. the sediment concentration in the river are shown, the concentration after passing the tunnel excluder and the sediment transport capacity of the canal under that condition. The tunnel excluder can decrease the amount of accummulation overload sediment entering the canal up to 19,500 cubic meters (see figure 4.7.c.) and more days that the required irrigation discharge can be diverted. For this sediment exclusion, about 128 million cubic meters of water are used without disturbing the irrigation supply. It is shown in figure 4.7.d.

The strategy 2 test results are presented in figures 4.8. The intake gate is regulated in the same way as in the strategy 1 test. The sandtrap is only effective during the first 15 days. After that period, the sandtrap is piled up with sediment and cannot be flushed due to the high river discharges. The sandtrap can be flushed on the 49th day because on that day the river discharge is within the flushing window. In the same simulated time period, this causes about 21,500 cubic meters of accumulation of overload sediment entering the canal. For the exclusion of sediment, only 8,640 cubic meters of water are used for flushing the sandtrap, but the irrigation will be disturbed during the flushing operation.

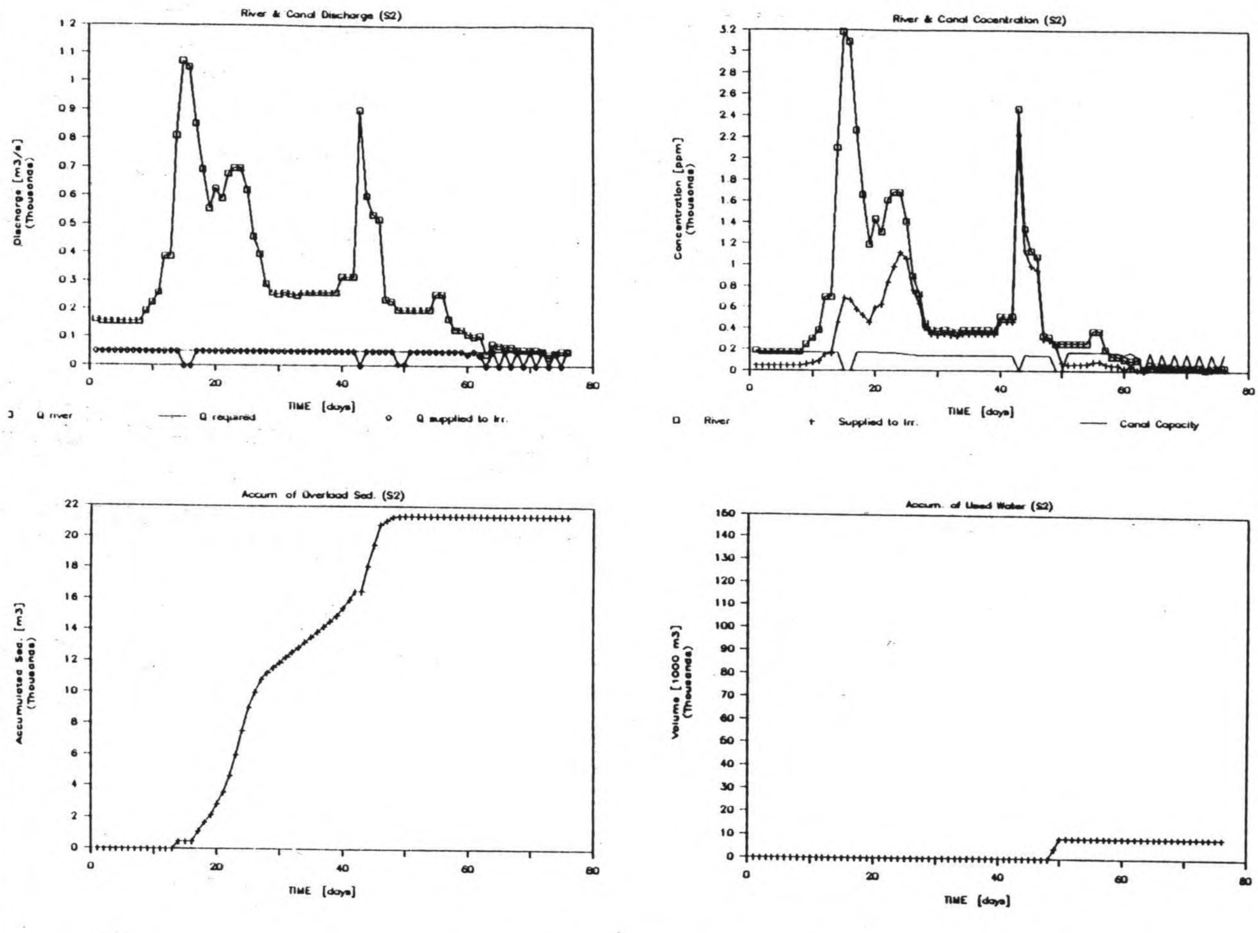


Figure 4.8 Strategy 2 test results.

The strategy 3 test result are presented in figures 4.9. The intake gate is regulated in the same way as mentioned in the strategy 1 test. The application of a tunnel excluder and a sandtrap and by operating them in series can decrease the accumulation of overload sediment entering the canal to about 7,250 cubic meters (see figure 4.9.b and 4.9.c.). For the exclusion of sediment, 140 million cubic meters of water are used (see figure 4.9.d.). The irrigation will only be disturbed when the sandtrap being flushed.

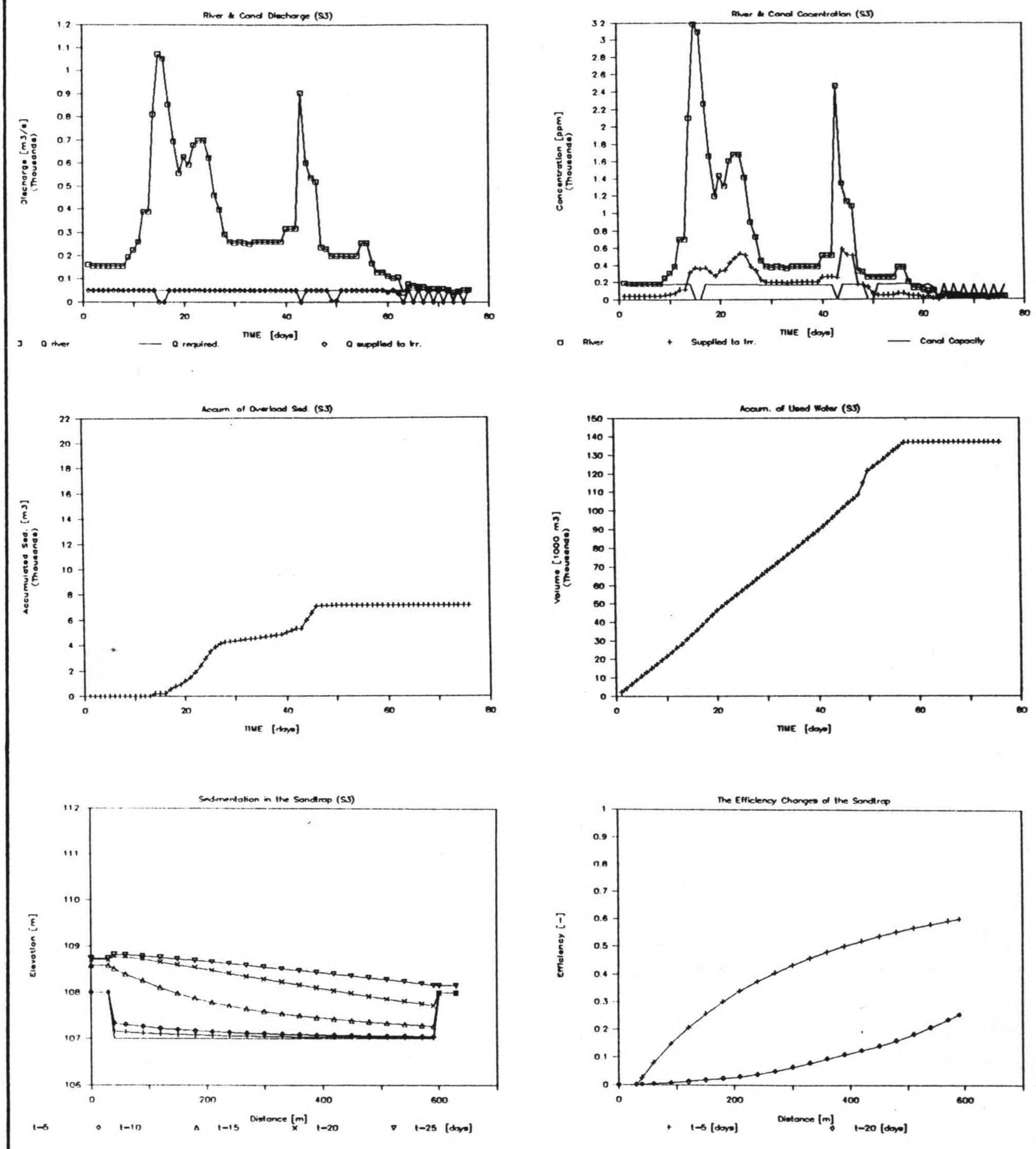


Figure 4.9 Strategy 3 test results.

The flushing window of the sandtrap is too small. Considering the hydraulic design, the sandtrap should be flushed in a two weeks' cyclus. During time interval September 1, 1987 - October 18, 1987 the sandtrap should be flushed three times. In fact, it cannot be done due to the high downstream water level. This condition decreases the sandtrap efficiency. The accumulation of settled sediment in the sandtrap and the change of sandtrap efficiency during this time interval are shown in figure 4.9.e. and figure 4.9.f. respectively.

In order to minimise the overload of sediment to the system that still occurs in the strategy 3 test, an attempt will be made to regulate the intake gate in the following two ways:

(i) In the period of river discharges 500 $m^3/s \leq Q$ river $\leq 900\ m^3/s$, the diverted water to the intake will be lowered from 50 m^3/s to 25 m^3/s. Later, this test will be called the strategy 4.1. test.

(ii) In the period of river discharges 500 $m^3/s \leq Q$ river $\leq 900\ m^3/s$, the diverted water to the intake will be lowered from 50 m^3/s to 30 m^3/s. Later, this test will be called the strategy 4.2. test.

In both strategies, the intake gate will be closed during the period of river discharges higher than 900 m^3/s. The test result are presented in figures 4.10 and 4.11. The intake gate regulation of both tests can be seen in figure 4.10.a. and figure 4.11.a. During the simulated time period, the accumulation of overload sediment entering the canal is 175 m^3 for the strategy 4.1. (see figure 4.10.b. and 4.10.c.) and 650 m^3 for the strategy 4.2. (see figure 4.11.b. and 4.11.c). For the exclusion of sediment, 140 million cubic meters of water are used in both strategy.

The strategy 5 test results are presented in figure 4.12. In the strategy 5, the intake gate is regulated in the same way as in the strategy 3 test and is shown in figure 4.12.a. The sandtrap can be flushed on the 27th day, which is the advantage of a heightening of the weir crest. During the simulated time period, the accumulation of overload sediment entering the canal is 4200 cubic meters (see figure 4.12.b. and 4.12.c.). For the exclusion of sediment, 141 million cubic meters of water are used (see figure 4.12.d.). During the simulated time period, the irrigation will be disturbed for three days in total for flushing the sandtrap.

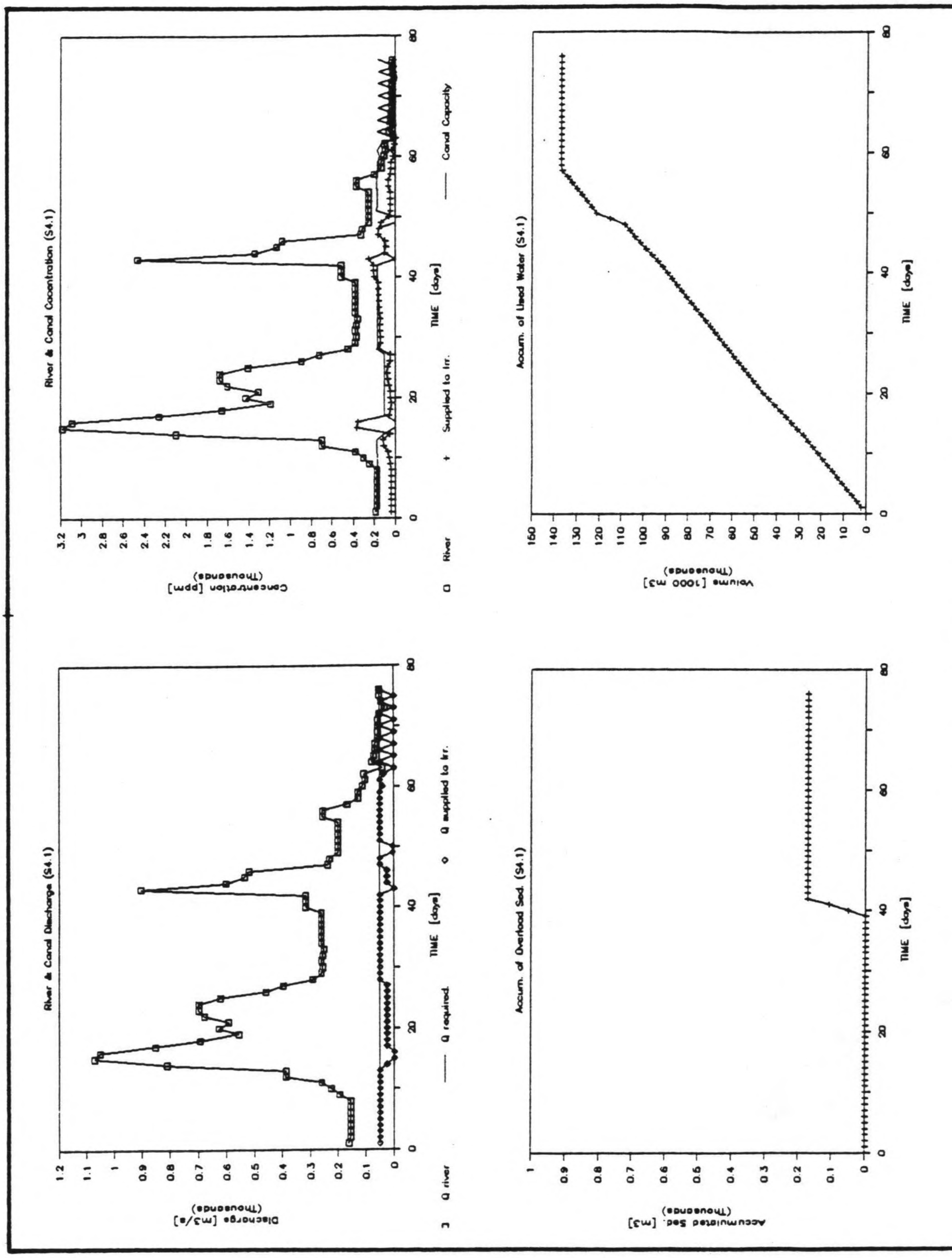


Figure 4.10 Strategy 4.1 test results.

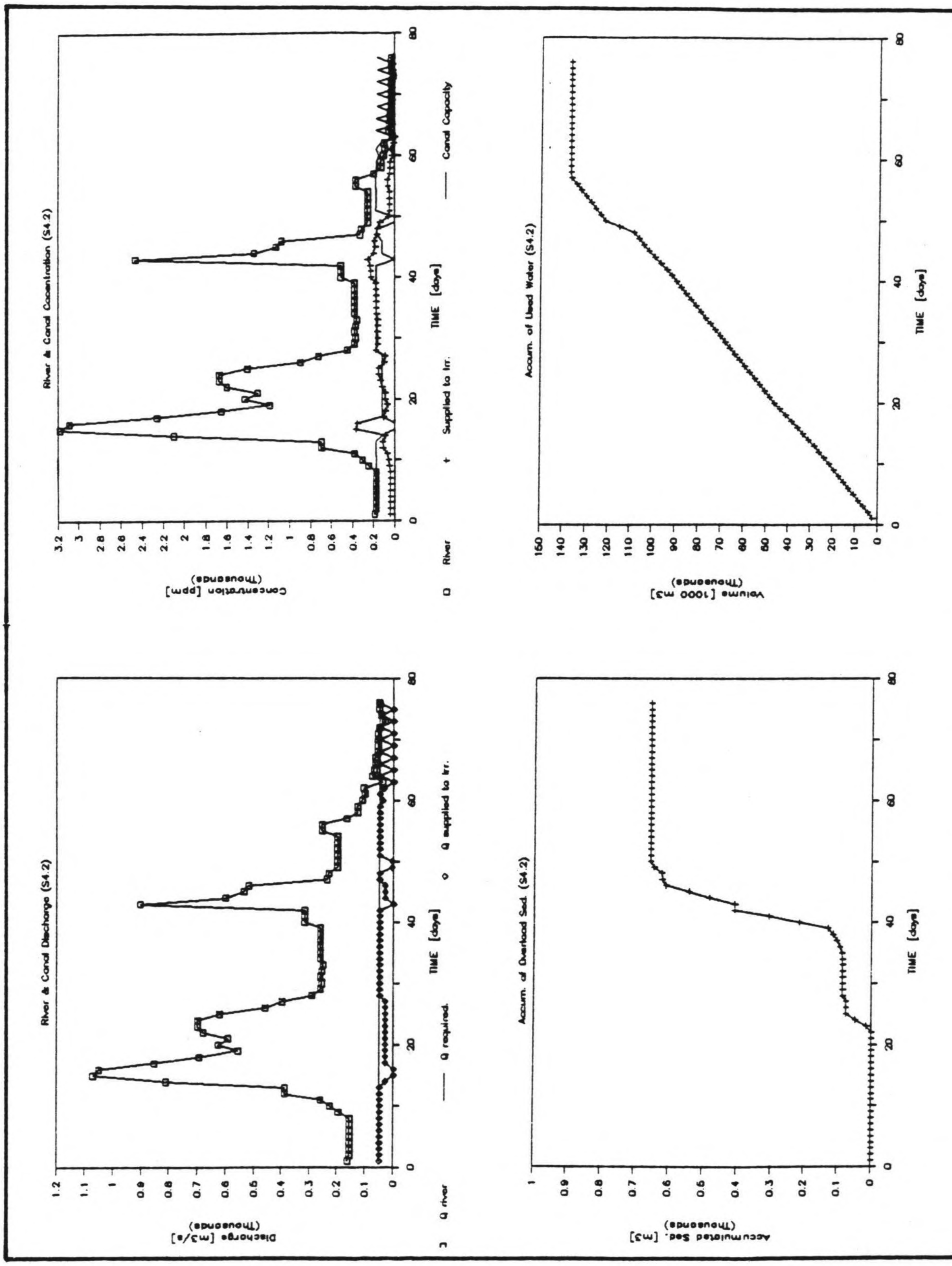


Figure 4.11 Strategy 4.2 test results.

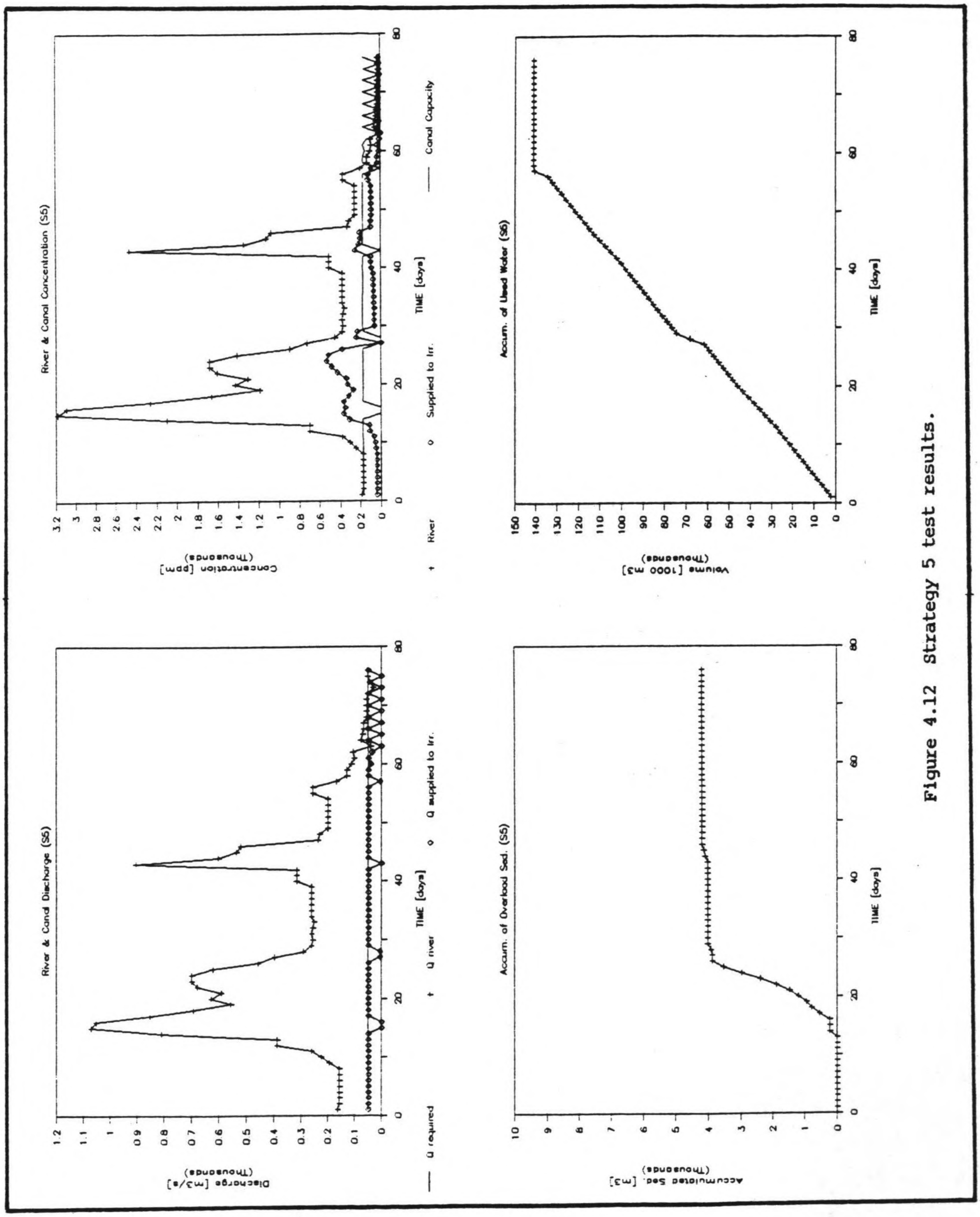


Figure 4.12 Strategy 5 test results.

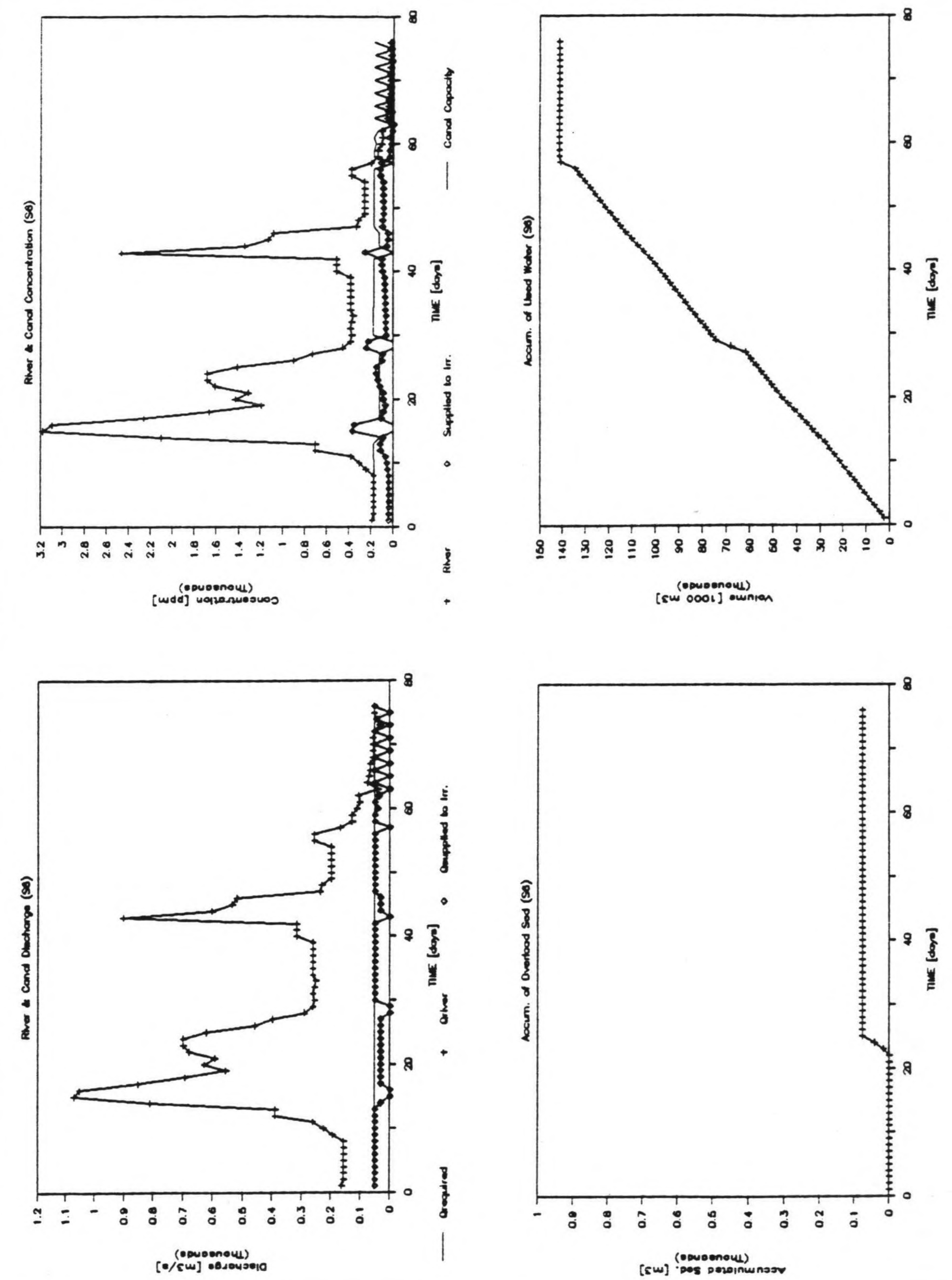


Figure 4.13 Strategy 6 test results.

The strategy 6 test results are presented in figure 4.13. The intake gate is regulated in the same way as mentioned in the strategy 4.2. test and is illustrated in figure 4.13.a. In the same simulated time period, the accumulation of overload sediment entering the canal is only 75 cubic meters (see figure 4.13.b. and 4.13.c.). The same amount of water as in the strategy 5 test are used for excluding the sediment.

In the strategy 7 test, the sandtrap takes a great part in the sediment exclusion. The tunnel excluder is operated with a smaller flushing discharge, the maximum flushing discharge is 20 cubic meters per second. It can be seen in figure 4.14.d. The intake gate is regulated in the same way as in the strategy 4.2.

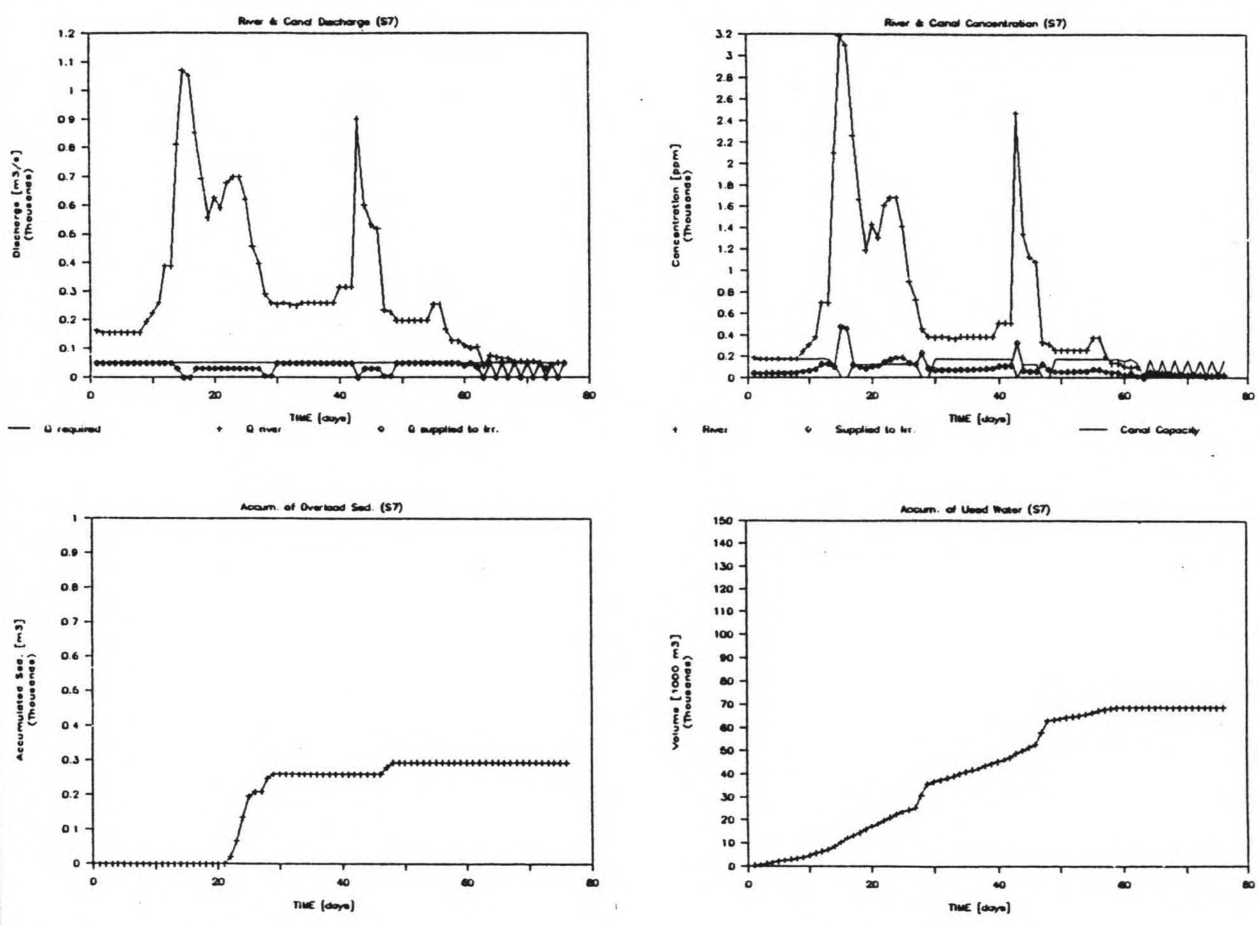


Figure 4.14 Strategy 7 test results.

Some remarks can be made from the test results :

(i) As long as the sandtrap is not yet filled up by the sediment, it means that the trapping efficiency of the sandtrap is still high enough, the sandtrap can solve the entering sediment problem. It can be concluded from figure 4.14.b.

(ii) It should be realised that if the rate of sedimentation in the sandtrap becomes higher, the needed frequency of the flushing operation will also be higher.

(iii) To fulfil that requirement, the heightening of the crest elevation of the Kabo weir is very important.

From figure 4.14.c. can be seen that in the same period, the accumulation of overload sediment entering the canal is only 300 cubic meters.

The mentioned test results can be summarised in a matrix form and presented in table 4.3.

Strategy	Irrigation Cond.[1000 m^3]		Accumulated Overload Sediment[m^3]	Used Water for excluding [1000 m^3]
	Required	Supplied		
0	328 320	256 608	27 750	-
1	328 320	282 528	19 500	128 000
2	328 320	274 752	21 500	8.640
3	328 320	274 752	7 500	140 000
4.1	328 320	242 352	175	140 000
4.2	328 320	248 832	650	140 000
5	328 320	269 568	4 193	141 000
6	328 320	243 648	75	141 000
7	328 320	243 648	300	68 670

Table 4.3. Summary of test results.

Note : Considering the available water in the river, the maximum volume of water that can be supplied in the simulated time period is only 295.488 million cubic meters.

From the simulation, applying seven specific strategies, can be concluded that the facing sediment problems in the Ye U irrigation scheme can be solved by providing a tunnel excluder upstream of the Ye U intake and a sandtrap downstream of the Ye U intake structure. The devices should be supported by the intake gate regulation. The intake gate should be closed during a river discharge higher than 900 m³/s. During the river discharge $500\ m^3/s \leq Q_{river} \leq 900\ m^3/s$, the diverted discharge to the intake should be lowered from 50 m³/s to 30 m³/s. If that is not acceptable, a higher discharge can be diverted with a consequency some over-capacity of sediment will enter the irrigation system.

The difference of the accumulation of overload sediment entering the canal under the condition with and without an intake gate regulation is 6600 cubic meters (consider the strategy 3 and strategy 4.2. test results). If the crest of the Kabo weir is heightened by 0.30 meter, the difference will be 4,118 cubic meters (consider the strategy 5 and strategy 6 test results).

It should be realised that the intake gate regulation gives a decrease of about 25.52 million of the supplied water to the irrigation scheme. This decreasing of supplied irrigation water may cause crop damages. The approach for analysing the crop damages due to the mentioned intake gate regulation will be discussed in general in section 5.1.

The heightening of the crest of the Kabo weir by 0.30 meter gives a higher possibility of sandtrap flushing operation. It means the sandtrap efficiency can be kept higher. The result of this effort can be seen by comparing the amount of the accumulation of overload sediment in the strategy 3 to the strategy 5 or the strategy 4.2 to the strategy 6.

More details of the output concerning the grain size of transported sediment after passing a sediment exclusion device are presented in the Appendices. This data are important when dealing with a hydro-power plant.

If the requirement of a stable canal design is fulfilled, the unbalance between the amount of sediment entering the irrigation canal and the transport capacity of the canal will not generate an errosion problem. Otherwise, a further analysis concerning the canal stability should be carried out.

5. **Water and Sediment Management**

5.1 General

The present chapter discusses in general some important aspects with respect to irrigation practices, which are of direct relevance for predicting the impact of the intake regulation to the crop damages and other aspects that should be considered for choosing a measure or measures of sediment exclusion. In the last part the cost and benefit of sediment exclusion will be summarised.

5.2 Water Management at Intakes

5.2.1 General

It is necessary to decide which effort will give the highest benefit. By considering the following possibilities:

(i) Regulating the intake gate to prevent an overload sediment entering the system resulting in possible crop damages.

(ii) Fulfilling the requirement of irrigation discharge with a consequence of a higher cost of canal maintenance that is necessary.

To carry out the analysis, the data concerning the system of water distribution in the irrigation scheme and the impact of a shortage of water to the crop are needed.

5.2.2 Background of the Group and Staggering System

In practice one tries to promote a better distribution of water demand in time by introducing the group system. A group generally refers to a number of geographycal units of irrigated areas, well-distributed over a project area. All farmers in one group are supposed to start their field work within a certain period of time.

A refiner of this method is a so called staggering period. A specific starting date for land preparation is assigned to each group. Farmers in a specific group are supposed to start gradually with their field work within a certain period of time.

An example of a planned group system and the way it can be schematised is shown in figure 5.1. The planned system consists of 6 groups with staggering periods of 6 weeks and intervals of two weeks between their starting dates.

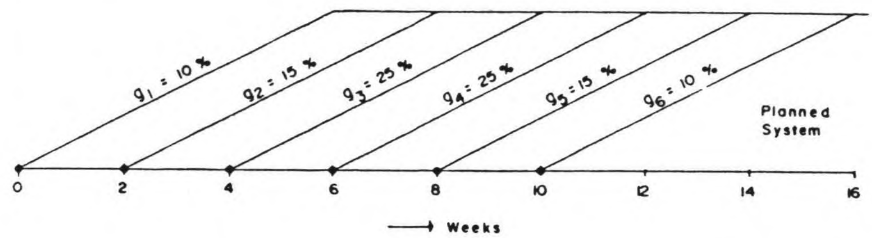


Figure 5.1 Schematisation of a planned group system (After Grijsen, 1988)

This condition of irrigation practices also gives a good impact from a point of view to minimise the entering sediment to the irrigation scheme. The group system will result a different in growth stage of the crop.

5.2.3 <u>Yield Response to the Water Stress</u>

Water shortage in the soil leads to a water stress in plant. Factors which determine the effect of water stress on a yield are among other things the duration of the stress, the intensity of the stress and the growth stage of the crop.

The duration of the stress period can be measured by the number of stress days. For rice the stress period begins on the fourth consecutive day without water. The water table and capillary movement of water should also be considered in this case.

Due to the complexity of the multiple interactions influencing the soil - plant - water relationships at various stages of plant growth, it is much easier to quantify the effect of water shortage on the yield than to quantify it (Grijsen, 1988). In order to quantify the effect of water stress on yield, a general relationship between the decreasing of relative yields and the relative evapotranspiration deficit, which is used in the

FAO approach, of the following form is adopted:

$$1 - \frac{Ya}{Yp} = ky \cdot \left(1 - \frac{Ea}{Ep} \right) \qquad (5\text{-}1)$$

where:

- Ya = actual harvested yield.
- Yp = potential harvested yield.
- ky = yield response factor as shown in table 5.1.
- Ea = actual evapotranspiration.
- Ep = potential evapotranspiration.

From equation (5-1) and table 5.1. can be seen that for rice plantation, the decreasing in yield due to water deficits during particular growth periods is relatively small for the vegetative and ripening periods, but relatively large for the flowering and yield formation periods. In terms of optimal water management, this should imply that in a case of water shortages, maximum total production is obtained by directing the limited water supply towards meeting the full water requirement in a limited acreage during such sensitive growth periods.

Stage	Establishment		Vegetative growth		Flowering period		Yield Formation		Ripening	
Crop	days	ky	days	ky	days	ky	days	ky	days	ky
Rice	10	1.1	40-60	1.2	10-15	2.5	25-35	0.4	10-20	0
Maize	15	0.4	30	0.4	20	1.5	30	0.5	10	0.2
Groundnut	10	0.2	25	0.2	30	0.8	30	0.6	10	0.2
Soybean	10	0.2	30	0.2	25	0.8	30	1.0	10	0.2
Green bean	10	0.2	25	0.2	20	1.1	30	0.75	20	0.2
Potato	15	0.4	15	0.45	20	0.8	45	0.7	10	0.2
Sugar cane			150-350			0.75	70-200	0.5	60	0.1

Table 5.1. Yield response factors for various crops.

Source: Grijsen, 1988

5.2.4 The Management of Intake Regulation

Based on a short description of the application of group and staggering system in irrigation practices which gives a different growth stage of crop, the starting day of a stress period due to a water shortage and to gain a maximum total crop production, the following items which are of direct relevance for intake regulation should be considered:

(i) Closing the intake gate to prevent a high concentration of sediment entering the system should not be longer than the period during the water stress begins. In this case lowering the intake discharge or an intermittent opening is preferable.

(ii) In case the requirement of irrigation discharge cannot be fulfilled, the supplied water should be distributed by considering the requirement of maximum total crop production.

(iii) If point (i) and (ii) are carried out, the intake gate regulation will not damage the crop.

5.3 Comprehensive Consideration of Sediment Management

5.3.1 General

To optimise the sediment exclusion at intakes, there are still many aspects that have not been discussed explicitly in the previous chapters. In this section more detail consideration about choosing the type of devices, needed hydraulic head and water used for excluding, and also construction and maintenance cost of devices will be discussed in a qualitative way.

5.3.2 Type of device

In chapter 2 has been described the basic consideration for choosing the type of devices from the standpoint of the mechanism of sediment transport.

The field experiences show that sandtrap has the highest effectivity. On the other hand, a sandtrap also needs more space, higher hydraulic head for flushing and higher construction cost. If dredging is chosen for removing the sediment from the settling basin, it will need a higher maintenance cost and a longer dredging time. These difficulties will decrease the average trapping efficiency of the sandtrap. These are the reasons why a sandtrap is not always preferred.

The tunnel excluder is preferable due to the simplicity of the structure. The tunnel excluder can flush a sedimentation upstream of the intake to maintain a skimming wall effect. In this way the tunnel excluder can reduce the entering bed load transport even if it is not being operated. These advantages lead to a posibility of intermittent operation.

The structure of the tunnel extractor and vortex tube are simple and can easily be constructed more than once in an irrigation system. However, they need an additional part of water from the river, particularly when they are constructed downstream of the intake. It also means that a wider intake structure is needed. The vortex tube is preferable to exclude the additional entering bed load from the area downstream of the intake.

5.3.3 Needed Hydraulic Head

The needed hydraulic head for a sediment exclusion device often becomes a constrain and cannot be provided due to the following reasons:

- (i) A higher construction cost for head works structure and upstream dikes are needed, also a wider impounding area.
- (ii) The constrains of river morphology, water level and ground water-level change in upstream and/or downstream.
- (iii) The back water can influence the other water resources system in the river basin.

The mentioned construction cost should be optimised with the resulted trapping efficiency of devices and the annual maintenance cost which is needed to reach a same effectivity.

5.3.4 Used Water for Sediment Exclusion

In chapter 4 has been described that the volume of used water for sediment exclusion is not the only parameter. It should be realised when, and how long the water is used and how the water can be used again by the other system. These are also important parameters.

In the Ye U case for example, the tunnel excluder can be operated without disturbing the irrigation water, although the volume is huge. The used water can be used by the downstream system without any losses, because it is directly flown back to the river. Not always this happens with a sandtrap or vortex tubes, because the used water for sediment exclusion can also be flown back to the drainage system.

5.4 **Benefit and Cost of Sediment Exclusion**

This section discusses an approach to the costs and benefit of the sediment exclusion and further needed development of the model.

The measures taken for minimising the amount of entering sediment can be supported by a certain water distribution policy to reduce crop damage. The policy mentioned has been described in section 5.1. In section 4.3 has also been shown that the water used for sediment exclusion can be managed in a such way that losses of water are avoided and it will not disturb the irrigation. Briefly, the adverse effect of sediment exclusion can be minimised.

An overload of sediment entering the system will cause sedimentation in the irrigation canal. Eventually resulting in a partial blocking of the canal and causing a decrease in flow capacity. Three major problems will be faced:

(i) Costly canal in maintenance operation is necessary.
(ii) The irrigation area cannot be cultivated for a certain period during overall canal maintenance.
(iii) A water shortage will occur in the irrigated area. The decreasing of irrigated area means a decreasing of the total production.

The designed model cannot simulate this process, in the present study.

An additional sub model could be added for simulating the morphological changes of at least the main canal. The additional model can be developed on the basis of the analysis of Ribberink and Van der Sande (1984) of aggradation in a river due to overloading. Another possibility use the same approach that has been used for the sandtrap. With this addition, the decreasing of canal flow capacity, a definite time and total volume of sediment that must be excavated can be determined. This data are an important input for a river basin model. The benefit of sediment exclusion can be analysed more clearly and can be compared to the needed cost for constructing and operating the sediment exclusion devices.

6. Conclusions and Proposed Further Study

6.1 Conclusions

(i) In the present study a model has developed for optimising a system of sediment exclusion devices. The designed model can be used for choosing the most appropriate sediment exclusion device or devices for a particular system. The model can also be used for optimising the operation of a system of sediment exclusion devices by considering several aspects.

(ii) The sub-models of the developed model were tested by a more advanced model and/or with field data. Some important conclusions of this testing are listed below:

- The sandtrap sub model gives a good result if compared to the results of SUTRENCH 2D model when analysing the trapping efficiency. The tests were carried out by considering a range of parameters which are representative for the conditions met in most of the irrigation schemes and hydropower plants.
- For this range, the analysis can be done by using relatively large space-steps and time steps without losing accuracy. It makes the computation faster, and it is a desireable requirement for a model which will be used in a preliminary design stage.
- The sandtrap sub model is sensitive to the applied sediment transport formula. In the model it is possible to choose between four sediment transport formulas giving more possibilities for calibrating the model.
- The sub model for predicting the trapping efficiency of a tunnel excluder, tunnel extractors and vortex tubes, yields predictions that are over-estimating the efficiency. This has been concluded from a test carried out by the Hydraulics Research Ltd, Wallingford. Unfortunately, the parameters used in the tests were not described in detail.

(iii) The test results show that the developed model is sensitive to the characteristics of transported sediment, particularly the chosen representative grain size of the transported sediment. To prevent an over-or under-estimated prediction, extensive field data of the characteristics of transported sediment are required.

(iv) The assumption adopted in the developed model is that a sandtrap will completely empty after a flushing operation. In practice, it is common to partly flush a sandtrap when the required hydraulic head is not available. Because up to now there is no sub-model for simulating a flushing of a sandtrap, the developed model is not yet capable for analysing the results of this operation.

6.2 Proposed Further Study

(i) It is very important to know the required hydraulic head for a system of sediment exclusion device. The availability of quantitative design methods for calculating the required head of some exclusion devices allow for completing the present design model with a sub model for estimating it. For a tunnel excluder type, Memed (1983) introduced a graph for predicting the required head as a function of upstream and downstream water level, gate opening, the length and height of a tunnel excluder. Atkinson (1984) described a design method for calculating the required head of a tunnel extractor. This method can also be used for a vortex tube type of sediment exclusion device.

(ii) As mentioned in section 6.1., a sub-model for simulating a flushing of a sandtrap should be included. For a flushing with moderate Froude numbers, it is not too difficult. In case of a flushing with higher Froude numbers, where super critical flow is occurring, an extensive study should be carried out.

(iii) In the present study, the trapping efficiency theory of tunnel excluder, tunnel extractor and vortex tube are only set up by Sanmuganathan (1976) based on the Einstein formulation of sediment transport. Using the same basic principle, but a different formulation of the sediment transport, a different approach to estimate the trapping efficiency can be set up. The different approach is necessary for comparison.

(iv) An additional sub model could be added for simulating the morphological changes of at least a main canal. With this addition the decrease of canal-flow capacity, the definite time and total volume of sediment that must be excavated can be determined. In this stage, the benefit of sediment exclusion can be analysed more clearly and can be compared to the needed cost for providing the sediment exclusion devices.

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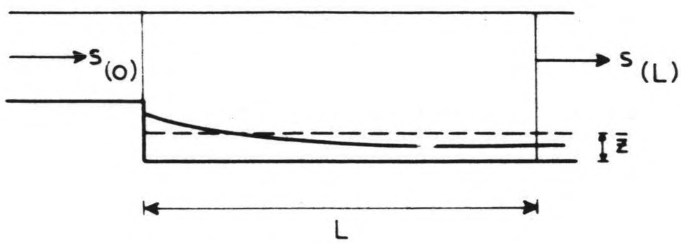
TRUNCATION ERROR

The aim of this analysis is to know the magnitude of an error or to know the influence of some chosen parameters with respect to the error due to the used solution algorithm of the bed-level changes.

The mathematical formulation of the bed level changes:

$$\frac{\partial \bar{z}}{\partial t} = \frac{s_o - s_L}{L}$$



Finete difference solution

$$\frac{\bar{z}^{\,n+½} - \bar{z}^{\,n-½}}{\Delta t} = \frac{s_o - s_L \,\frac{(\bar{z}^{\,n+½} + \bar{z}^{\,n-½})}{2}}{L}$$

1) The generated error of left hand part:

$$z^{n+½} = z + \frac{½\ \Delta t}{1!}\left(\frac{\partial z}{\partial t}\right) + \frac{(½\Delta t)^2}{2!}\left(\frac{\partial^2 z}{\partial t^2}\right) + \frac{(½\Delta t)^3}{3!}\left(\frac{\partial^3 z}{\partial t^3}\right) + \frac{(½\Delta t)^4}{4!}\left(\frac{\partial^4 z}{\partial t^4}\right)\ \frac{(½\Delta t)^5}{5!}\left(\frac{\partial^5 z}{\partial t^5}\right) + \text{h.o.t}$$

$$z^{n-½} = z - \frac{½\ \Delta t}{1!}\left(\frac{\partial z}{\partial t}\right) - \frac{(½\Delta t)^2}{2!}\left(\frac{\partial^2 z}{\partial t^2}\right)\ \frac{(½\Delta t)^3}{3!}\left(\frac{\partial^3 z}{\partial t^3}\right) - \frac{(½\Delta t)^4}{4!}\left(\frac{\partial^4 z}{\partial t^4}\right) - \frac{(½\Delta t)^5}{5!}\left(\frac{\partial^5 z}{\partial t^5}\right) + \text{h.o.t}$$

$$\frac{\bar{z}^{n+½} - \bar{z}^{n-½}}{\Delta t} = \frac{\partial \bar{z}}{\partial t}\ \frac{(½\Delta t)^2}{3!}\left(\frac{\partial^3 z}{\partial t^3}\right) + \frac{(½\Delta t)^4}{5!}\left(\frac{\partial^5 z}{\partial t^5}\right) + \text{h.o.t}$$

The generated truncation error:

$$T_1 = \frac{(½\ \Delta t)^2}{3!}\left(\frac{\partial^3 z}{\partial t^3}\right) + \frac{(½\ \Delta t)^4}{5!}\left(\frac{\partial^5 z}{\partial t^5}\right) + \text{h.o.t.}$$

2) The generated error of righthand part:

$$\text{Say } \tilde{z} = \frac{\bar{z}^{n+½} - \bar{z}^{n-½}}{2} - z$$

The righthand part can be written

$$\frac{s_o - s_L\left(z + \dfrac{\bar{z}^{n+½} + \bar{z}^{n-½}}{2}\right)}{L} =$$

$$\frac{s_o - s_L\,(z + \bar{z})}{L} = \frac{1}{L}\left[s_o - \{s_{L(z)} + \frac{\partial s_L}{\partial z}\,.\,\tilde{z} + ½\,\frac{\partial^2 s_L}{\partial z^2}\,\tilde{z}^2 + h.o.t\}\right]$$

$$= \frac{s_o - s_L\,(z)}{L} - \frac{1}{L}\,\{\frac{\partial s_L}{\partial z}\,.\,\tilde{z} + ½\,\frac{\partial^2 s_L}{\partial z^2}\,\tilde{z}^2 + h.o.t\}$$

The generated truncation error:

$$T_2 = -\frac{1}{L}\,\{\frac{\partial s_L}{\partial z}\,.\,\tilde{z} + ½\,\frac{\partial^2 s_L}{\partial z^2}\,\tilde{z}^2 + h.o.t\}$$

where for the term of $\tilde{z}$ can be derived as follow:

$$z = z_o + L\,z_{ox} + \frac{L^2}{2!}\,z_{oxx} + \frac{L^3}{3!}\,z_{oxxx} + h.o.t.$$

$$\bar{z} = \frac{1}{L}\int_o^L z\,dx$$

$$= z_o + ½\,.\,L\,.\,z_{ox} + \frac{1}{6}\,L^2\,.\,z_{oxx} + \frac{1}{24}\,L^3\,.\,z_{oxxx} + h.o.t.$$

$$\bar{z}_{(t\,+\,½\Delta t)} = \{z_{(o,t)} + ½\Delta t\,.\,z_{ot} + \frac{(½\Delta t)^2}{2!}\,z_{ott} + h.o.t.\} +$$

$$½\,L\,\{z_{x(o,t)} + ½\Delta t\,z_{oxt} + \frac{(½\Delta t)^2}{2!}\,z_{oxtt}\} + h.o.t.$$

$$\bar{z}_{(t\,-\,½\Delta t)} = \{z_{(o,t)} - ½\Delta t\,.\,x_{ot} + \frac{(½\Delta t)^2}{2!}\,z_{ott} + h.o.t.\} +$$

$$½\,L\,\{z_{x(o,t)} + ½\Delta t\,z_{xot} + \frac{(½\Delta t)^2}{2!}\,z_{oxtt} + h.o.t.\} + h.o.t.$$

$$\frac{\bar{z}^{n+½} - \bar{z}^{n-½}}{2} = \{½\,.\,\Delta t\,z_{ot} + \frac{(½\Delta t)^3}{3!}\,z_{ottt} + h.o.t.\} +$$

$$½\,.\,L\,.\,\{½\,.\,\Delta t\,.\,z_{oxt} + \frac{(½\Delta t)^3}{3!}\,z_{oxttt} + h.o.t.\} + h.o.t.$$

$$\tilde{z} = \frac{\bar{z}^{n+½} - \bar{z}^{n-½}}{2} - z$$

$$= \{½\,.\,\Delta t\,.\,z_{ot} + \frac{(½\Delta t)^3}{3!}\,z_{ottt} + h.o.t.\} + ½\,.\,L\,\{½\,.\,\Delta t\,.\,z_{oxt} +$$

$$\frac{(½\Delta t)^3}{3!}\,z_{oxttt} + h.o.t.\} - z_o - L\,.\,z_{ox} - \frac{(L)^2}{2!}\,z_{oxx}\,\frac{(L)^3}{3!}\,z_{oxxx}$$

The term of $\partial s_L/\partial z$ can be derived as follow:

For the condition $B_0 = B_1$ and $s_o >> s_L$, the Eysink and Vermaas equation can be formed:

$$s_L = s_o * \text{Exp}\ (\frac{-\alpha\ .\ L}{h_L})$$

$$\text{where } \alpha = 0.24\ \{\frac{h_L}{k_n}\}^{1/6}\ \frac{w}{u}\ [1 + 8\ \{\frac{h_L}{k_n}\}^{1/6}\ \frac{w}{u}]\ [1 + 4.1\ \{\frac{h_L}{k_n}\}^{-0.25}]$$

$$h_L = H - z$$

$$u = \frac{q}{H - z}$$

For the time under consideration of bed level changes, q and H can be assumed constant.

$$\frac{\partial s_L}{\partial z} = s_0 * \text{Exp}\ (\frac{-\alpha_z L}{h_{L(z)}}) * (\frac{\alpha'_{(z)}\ .\ h_{L(z)} - h'_{L(z)}\ .\ \alpha_{(z)}\ .\ L}{\alpha^2_{l(z)}})$$

$$\alpha'_{(z)} = \frac{\partial \alpha}{\partial z} = f'_{(h_L)} * \frac{\partial h_{(z)}}{\partial z} + f'_{(u)} * \frac{\partial u_{(3)}}{\partial z}$$

$$f'_{(h_L)} = \frac{0.24}{6}\ .\ (\frac{h_L}{k_n})^{-5/6}\ .\ \frac{1}{k_n}\ .\ \frac{w}{u}\ .\ [1 + 8\ \{\frac{h_L}{k_n}\}^{1/6}\ \frac{w}{u}][1 + 4.1\ \{\frac{h_L}{k_n}\}^{-1/4}] +$$

$$0.24\ (\frac{h_L}{k_n})^{1/6}\ .\ \frac{w}{u}\ .\ [1 + 8\ \{\frac{h_L}{k_n}\}^{-5/6}\ \frac{w}{u}]\ [1 + 4.1\ \{\frac{h_L}{k_n}\}^{-1/4}] +$$

$$0.24\ (\frac{h_L}{k_n})^{1/6}\ .\ \frac{w}{u}\ .\ [1 + 8\ \{\frac{h_L}{k_n}\}^{1/6}\ \frac{w}{u}]\ [1 + 4.1\ \{\frac{h_L}{k_n}\}^{-5/4}\ \frac{1}{k_n}]$$

$$h'_{L(z)} = \frac{\partial h_L}{\partial Z} = -\ 1$$

$$f'_{(u)} = -\ 0.24\ .\ (\frac{h_L}{k_n})^{1/6}\ .\ \frac{w}{u^2}\ .\ [1 + 8\ \{\frac{h_L}{k_n}\}^{1/6}\ \frac{w}{u}][1 + 4.1\ \{\frac{h_L}{k_n}\}^{-1/4}]$$

$$-\ 0.24\ .\ (\frac{h_L}{k_n})^{1/6}\ .\ \frac{w}{u}\ .\ [1 + 8\ \{\frac{h_L}{k_n}\}^{1/6}\ \frac{w}{u^2}][1 + 4.1\ \{\frac{h_L}{k_n}\}^{-1/4}]$$

$$\frac{\partial u}{\partial z} = \frac{q}{(H - z)^2} = \frac{q}{h_L^2}$$

One method which can be used in an attempt to quantify the value of other derivative terms is transforming the derivative term to a finite-difference form (Perkins, 1970 and Ralston, 1965).

For example:

$$\frac{\partial^3 z}{\partial t^3} \approx \frac{\nabla^3 z}{(\Delta t)^3}$$

∇ denotes a backward difference

$$\frac{\partial^3 z}{\partial t^3} \approx \frac{1}{\Delta t^3} (z^{n+1} - 3\ z^{n} + 3\ z^{n-1} - z^{n-2})$$

INTERGRAL FORM OF THE TRAPPING EFFICIENCY OF AN SEDIMENT EXTRACTOR

Refering to equation (2-13), when it is integrated yields:

$$p_j = \frac{p_{\alpha j}}{p_{1_j}}$$

(i) when a is small:

$$p_{\alpha_j} = \lambda \,.\, a \,.\, \frac{u^{*'}}{u^{*}}$$

$$+ e^{15.\, s_j .\, a} \{[1 + \frac{\lambda}{\kappa} - \frac{\lambda}{\kappa} \ln 15\, s_j]\, [\frac{1 - e^{-15 s_j \,.\, \alpha}}{15\, s_j}]$$

$$+ \frac{\lambda}{\kappa} \frac{1}{15 s_j} [y^{*} - e^{-15 s_j \alpha} \ln 15 s_j \alpha \;+ E_i\, (-15\, s_j \alpha]\}$$

$$- [a + \frac{\lambda}{\kappa} a \ln a] \qquad (1)$$

$$p_{1_j} = p_{\alpha_j} \Big|_{\alpha = 1.0} \qquad (2)$$

where: $- E_{i(-x)}$ = exponential integral

$$= \int_t^{\alpha} \frac{e^{-t}}{t}\, dt$$

y^{*} = Euler constant

= - 0.57722

$$s = \frac{w_s}{u^{*}}$$

(ii) when a is relatively large ($a \geq 0.01$)

$$p_{\alpha_j} = \lambda \,.\, a \frac{u^{*'}}{u^{*}}$$

$$+ \frac{e^{15.\, s_j .\, a}}{15\, s_j} \{[1 + \frac{\lambda}{\kappa}]\, [e^{-15.\, s_j .\, a} - e^{-15\, s_j \,.\alpha}]$$

$$+ \frac{\lambda}{\kappa} [e^{-15 s_j a} \ln a - e^{-15\, s_j \alpha} \ln a - E_i\, (-15\, s_j a]$$

$$- E_i\ [-15\ s_j\ \alpha]\} \tag{3}$$

$$p_{1_j} = p_{\alpha_j} \Big|_{\alpha = 1.0} \tag{4}$$

The first term in equation (1) and (3) stems from the movement of sediment in the bed layer. The remaining terms represent the suspended sediment.

TOTAL SEDIMENT TRANSPORT FORMULAS

In the designed model, four sediment transport formula are implemented:

1 Engelund and Hansen
2 Ackers and White
3 Einstein and Brown
4 van Rijn

Define ϕ as a transport parameter and ψ as a Shields-like flow parameter:

$$\phi = \frac{s}{(\Delta \cdot g \cdot D^3)^{1/2}}$$

$$\psi = \frac{hi}{\Delta D}$$

where:

s	=	sediment transport in volume of grain	[m³/s]
Δ	=	relative density under water	[-]
D	=	representative grain diameter	[m]
h	=	water depth	[m]
i	=	hydraulic gradient or slope	[-]

(i) The Engelund and Hansen formula can be written as:

$$\phi = 0.1\ f^{-1}\ \psi^{2.5} \tag{1}$$

where $f = \frac{2g}{C^2}$

$D = D_{50}$

(ii) Ackers and White formula can be written as:

$$\phi = c \cdot \psi^{1/2} \left(\frac{u}{u_*}\right)^{1+n} \left(\frac{F_{gr}}{\delta} - 1\right]^m \tag{2}$$

where:

$$F_{gr} = (\psi)^{1/2} \left(\frac{u'_r}{u_*}\right)^{1-n} \tag{3}$$

$$u'_* = \frac{u}{5.64 \log \left(\frac{10h}{D}\right)}$$

$$D_{gr} = D \left(\frac{\Delta g}{\nu^2}\right)^{1/3}$$

$$D = D_{35}$$

and n, m, c and A are a function of D_{gr}.

For $1 < D_g < 60$

$$n = 1.00 - 0.56 \log D_{gr}$$

$$m = 9.66\, D_{gr}^{-1} + 1.34$$

$$c = 10^{[2.86 \log D_{gr} - (\log D_{gr})^2 - 3.53]}$$

$$A = 0.23\, D_{gr}^{-1/2} + 0.14$$

For $D_{gr} > 60$

$n = 0$

$m = 1.5$

$c = 0.025$

$a = 0.17$

(iii) Einstein and Brown formula can be written as:

$$\phi = 40 \left\{\left(\frac{2}{3} + 36 \frac{\nu^2}{g\Delta D^3}\right)^{0.5} - \left(\frac{\nu^2}{g\Delta D^3}\right)^{0.5}\right\} . \psi^3 \tag{4}$$

(iv) The van Rijn formula is divided in a part for suspended bed-material transport and a part for bed load. The suspended sediment equation reads:

$$s_s = 0.012\, u\, D_{50} \left\{\frac{u - u_c}{[\Delta g D_{50}]^{0.5}}\right\}^{2.4} [D_g]^{-0.6} \tag{5}$$

The bed load equation reads:

$$s_b = 0.005\, u\, .\, h \left\{\frac{u - u_c}{[\Delta g D_{50}]^{0.5}}\right\}^{2.4} \left[\frac{D_{50}}{h}\right]^{1.2} \tag{6}$$

where u_c = critical flow velocity for initiation of bed particle motion.

The equations (5) and (6) are an approximation of the complicated van Rijn method develop by DELFT HYDRAULICS. The approximation is valid for:

$0.1 \text{ m} < h < 20.0 \text{ m}$

$0.5 \text{ m/s} < u < 2.5 \text{ m/s}$

$0.1 \text{ mm} < D_{50} < 2 \text{ mm}.$

STRUCTURED DIAGRAM OF THE EXTRACTOR SUB MODEL

DECLARATION

INPUT DATA : 1. Discharge entering the system
2. Flow condition upstream the device
3. Extraction ratio

Assume $h' = h$

5 Calculate Fr, $u*'$ and Y'

Find the value of $k_u = u*' / u*$ from figure 2.6

Compute $h'_{new} = h \cdot k_u^2$

If $|h'_{new} - h'| > 0.001\, h$

Then:

$h' = h'_{new}$
Go to 5

Else:

$h' = h'_{new}$
$= u / u*' \cdot k_u$
$u* = u \cdot \lambda$

Calculate α_u (dimensionless height where $u = u*$)

$$\alpha_u = \mathrm{Exp}\left(-0.6 - 0.4 \frac{1}{\lambda} \right)$$

Find height of dividing stream line (α)

If $R \leq \alpha_u \cdot \lambda$

Then:

$$\alpha = \frac{R}{\lambda}$$

Else:

Find from $R = \alpha + \frac{\lambda}{\kappa} \alpha \ln \alpha$

by trials and errors.

Calculate $\alpha_D = 2 \; (D_j / h)$

Compute dimensionless bed layer thickness

If $\alpha_D \leq \alpha_u$

Then:

$a = \alpha_u$

Else:

$a = \alpha_D$

COMPUTE TRAPPING EFFICIENCY

$$p_j = \frac{p_{\alpha j}}{p_{1j}}$$, see Appendix A-2

SUMMARY OF THE TRAPPING EFFICIENCY

ENDING PART

STRUCTURED DIAGRAM OF SANDTRAP SUB MODEL

DECLARATION

INPUT DATA : 1. Hydraulic design of the sandtrap
2. Concentration entering the sandtrap
3. Discharge

CALCULATE DOWNSTREAM WATER LEVEL

Consider an overflow spillway in end part of sandtrap

CALCULATE THE FLOW CONDITION ON THE OTHER GRIDS

Do for J = JJ, 1, -1

Flow field = Rigid lid

Then	Else
$H_{J-1} = H_J$ $h_{J-1} = H_{J-1} - z_{j-1}$	Considering backwater condition. Modified Euler method is used.

CALCULATE THE TRAPPING EFFICIENCY AND SEDIMENTATION PROCESS

Do for Iter = 1, Iteration

CALL EYSINK (analyse trapping efficiency)

Do for J = 0, JJ

Predict bed level changes due to sedimentation

Calculate new flow field condition

CALL EYSINK (analyse final trapping efficiency)

Do for J = 0, JJ

Calculate final bed level changes

SUMMARIES TRAPPING EFFICIENCY

Do for I = 1, Fraction

$$p_i = [\, S_{(i,0)} - S_{(i,jj)} \,] \,/\, S_{(i,jj)}$$

$$p = \frac{1}{Frac.} \sum_{i=1}^{Frac.} p_i$$

SET FLOW AND BED LEVEL CONDITION FOR THE NEXT TIME STEPS

END PART

STRUCTURED DIAGRAM OF SUBROUTINE EYSINK

DECLARATION

CALCULATE TRAPPING EFFICIENCY SECTION BY SECTION

Do for J = 0, (JJ-1)

> Calculate trapping efficiency of each fraction
>
> Do for I = 1, Fraction
>
> > Calculate sediment transport in equilibrium condition
> >
> > Calculate $S_{(j+1),i}$ using Eysink and Vermaas method
>
> $$S_{(j+1)} = \sum_{i=1}^{Frac} S_{(j+1),i}$$

Input Data 1.
Operation of Sediment Exclusion Device.
Series / Strategies : 7.

Irrigation Canal Data :

Width of canal [m]	Side slope (1 for vert)	Longitudinal slope	Chezy roughness [m/s+]	Sed. Transport equation [see note]
30. ,	1.5 ,	.000143 ,	40.0 ,	4

Note for sediment transport equation :
- Engelund and Hansen : 1
- Ackers and White : 2
- Einstein and Brown : 3
- Van Rijn : 4

Provided Sed. Exclusion Device :

T. Excluder	T. Extractor	Vortex Tube	Sandtrap
'YES' ,	'NO' ,	'NO' ,	'YES'

Technical data of Tunnel Excluder :
(if no tunnel excluder is provided, write 'C' on the first column of the line data)

Width of T Excluder	Width of gate	Elevation of tunnel floor	slab
20.42 ,	18.28 ,	107.00 ,	108.15

Number of provided Tunnel Extractor Max 3 (if no tunnel extractor is provided, write 'C' on the first column of the line data) :

Number of T. Extrac.	Width of Tunnel Extractor(i) (respectively)	El. of Tunnel Extractor (i) (respectively)
,	,	

Number of provided Vortex Tube Max 3 (if no vortex tube is provided, write 'C' on the first column of the line data) :

Number of V. Tube	Width of Vortex Tube(i) (respectively)	El. of Vortex Tube(i) (respectively)
,	,	

Numbers of Total Time Steps :

Total Time Steps [-]
76

Operation Data of devices :

No.	Set elv. of T Exc. Crest	Flushing Discharge [m3/s] T Exclud	T Extrac1	T Extrac2	T Extrac3	V Tube1	V tube2	V Tube3	Sandtrap Flushing (YES/NO)	Dicharge [m3/s]	Duration of operation[days]
1	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
2	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
3	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
4	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
5	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
6	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
7	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
8	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
9	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
10	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
11	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
12	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
13	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
14	110.6	15.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
15	110.6	20.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
16	110.6	20.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
17	110.6	15.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
18	110.6	15.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
19	110.6	15.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
20	110.6	15.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
21	110.6	15.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
22	110.6	15.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
23	110.6	15.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
24	110.6	15.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
25	110.6	15.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
26	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
27	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
28	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'YES'	50.0	1.

29	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'YES'	50.0	1.
30	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
31	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
32	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
33	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
34	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
35	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
36	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
37	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
38	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
39	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
40	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
41	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
42	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
43	110.6	20.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
44	110.6	15.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
45	110.6	15.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
46	110.6	15.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
47	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'YES'	50.0	1.
48	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'YES'	50.0	1.
49	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
50	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
51	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
52	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
53	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
54	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
55	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
56	110.6	10.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
57	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
58	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
59	110.6	5.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
60	110.6	0.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
61	110.6	0.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
62	110.6	0.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
63	110.6	0.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
64	110.6	0.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
65	110.6	0.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
66	110.6	0.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
67	110.6	0.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
68	110.6	0.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
69	110.6	0.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
70	110.6	0.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
71	110.6	0.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
72	110.6	0.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
73	110.6	0.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
74	110.6	0.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
75	110.6	0.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.
76	110.6	0.0	0.0	.0	.0	0.0	.0	.0	'NO'	0.0	1.

Input Data 2
River dicharges and transported sediment data
Series / Strategies : 7

General Data for Sediment Transport

Density [kg/m3] Sediment	Density [kg/m3] Water	Acc. of gravity [m/s2.]	Porosity of sediment [-]	Kinematic viscosity
2650.	1000.	9.81	.4	1.01E-6

Representation Sediment Data :

Number of fraction	1.	2.	3.	4.	5.	6.	7.	8.	9.	10
5	.06	.12	.19	.32	.65					

(Representative Diameter [mm])

Water and Sediment Balance Data:

No.	Elv. Riv. Surface	Discharge [m3/s] River	Irr. req.	Supplied	Diameter [mm] d35	d50	d65	Concentration by Fraction [ppm] 1.	2.	3.	4.	5.	6.	7.	8.
1	110.6	163.	50.0	50.0	.064	.083	.107	114.8	40.6	21.0	10.3	4.1			
2	110.6	156.	50.0	50.0	.064	.083	.107	107.5	38.0	19.7	9.6	3.8			
3	110.6	156.	50.0	50.0	.064	.083	.107	107.5	38.0	19.7	9.6	3.8			
4	110.6	156.	50.0	50.0	.064	.083	.107	107.5	38.0	19.7	9.6	3.8			
5	110.6	156.	50.0	50.0	.064	.083	.107	107.5	38.0	19.7	9.6	3.8			
6	110.6	156.	50.0	50.0	.064	.083	.107	107.5	38.0	19.7	9.6	3.8			
7	110.6	156.	50.0	50.0	.064	.083	.107	107.5	38.0	19.7	9.6	3.8			
8	110.6	156.	50.0	50.0	.064	.083	.107	107.5	38.0	19.7	9.6	3.8			
9	110.6	194.	50.0	50.0	.064	.083	.107	148.9	52.6	27.3	13.3	5.3			
10	110.6	224.	50.0	50.0	.064	.083	.107	184.6	65.3	33.8	16.5	6.6			
11	110.6	260.	50.0	50.0	.064	.083	.107	230.7	81.6	42.3	20.7	8.2			
12	110.6	388.	50.0	50.0	.064	.083	.107	419.5	148.4	77.0	37.7	14.9			
13	110.6	388.	50.0	50.0	.064	.083	.107	419.5	148.4	77.0	37.7	14.9			
14	111.4	811.	50.0	30.0	.064	.083	.107	1262.4	446.5	231.6	113.4	45.1			
15	111.4	1071.	50.0	0.0	.064	.083	.107	1912.6	676.4	350.9	171.8	68.3			
16	111.4	1051.	50.0	0.0	.064	.083	.107	1859.4	657.7	341.2	167.1	66.4			
17	111.4	852.	50.0	30.0	.064	.083	.107	1358.9	480.6	249.3	122.1	48.5			
18	111.4	694.	50.0	30.0	.064	.083	.107	1000.2	353.7	183.5	89.8	35.7			
19	111.4	556.	50.0	30.0	.064	.083	.107	718.2	254.0	131.8	64.5	25.6			
20	111.4	628.	50.0	30.0	.064	.083	.107	861.5	304.7	158.0	77.4	30.7			
21	111.4	592.	50.0	30.0	.064	.083	.107	788.8	279.0	144.7	70.8	28.1			
22	111.4	679.	50.0	30.0	.064	.083	.107	968.1	342.4	177.6	87.0	34.6			
23	111.4	699.	50.0	30.0	.064	.083	.107	1011.0	357.6	185.5	90.8	36.1			
24	111.4	699.	50.0	30.0	.064	.083	.107	1011.0	357.6	185.5	90.8	36.1			

25	111.4	622.	50.0	30.0	.064	.083	.107	849.2	300.3	155.8	76.3	30.3
26	110.6	459.	50.0	30.0	.064	.083	.107	539.3	190.7	98.9	48.4	19.2
27	110.6	398.	50.0	30.0	.064	.083	.107	435.8	154.1	79.9	39.1	15.5
28	110.6	291.	50.0	5.0	.064	.083	.107	273.0	96.5	50.0	24.5	9.7
29	110.6	260.	50.0	5.0	.064	.083	.107	230.7	81.6	42.3	20.7	8.2
30	110.6	255.	50.0	50.0	.064	.083	.107	224.1	79.2	41.1	20.1	8.0
31	110.6	260.	50.0	50.0	.064	.083	.107	230.7	81.6	42.3	20.7	8.2
32	110.6	255.	50.0	50.0	.064	.083	.107	224.1	79.2	41.1	20.1	8.0
33	110.6	250.	50.0	50.0	.064	.083	.107	217.5	76.9	39.9	19.5	7.7
34	110.6	260.	50.0	50.0	.064	.083	.107	230.7	81.6	42.3	20.7	8.2
35	110.6	260.	50.0	50.0	.064	.083	.107	230.7	81.6	42.3	20.7	8.2
36	110.6	260.	50.0	50.0	.064	.083	.107	230.7	81.6	42.3	20.7	8.2
37	110.6	260.	50.0	50.0	.064	.083	.107	230.7	81.6	42.3	20.7	8.2
38	110.6	260.	50.0	50.0	.064	.083	.107	230.7	81.6	42.3	20.7	8.2
39	110.6	260.	50.0	50.0	.064	.083	.107	230.7	81.6	42.3	20.7	8.2
40	110.6	316.	50.0	50.0	.064	.083	.107	308.7	109.2	56.6	27.7	11.0
41	110.6	316.	50.0	50.0	.064	.083	.107	308.7	109.2	56.6	27.7	11.0
42	110.6	316.	50.0	50.0	.064	.083	.107	308.7	109.2	56.6	27.7	11.0
43	111.4	903.	50.0	0.0	.064	.083	.107	1482.2	524.2	272.0	133.2	52.9
44	111.4	602.	50.0	30.0	.064	.083	.107	808.7	286.0	148.4	72.6	28.9
45	110.6	536.	50.0	30.0	.064	.083	.107	679.9	240.5	124.7	61.1	24.3
46	110.6	520.	50.0	30.0	.064	.083	.107	649.8	229.8	119.2	58.4	23.2
47	110.6	235.	50.0	5.0	.064	.083	.107	198.3	70.1	36.4	17.8	7.0
48	110.6	229.	50.0	5.0	.064	.083	.107	190.8	67.5	35.0	17.1	6.8
49	110.6	199.	50.0	50.0	.064	.083	.107	154.7	54.7	28.3	13.9	5.5
50	110.6	199.	50.0	50.0	.064	.083	.107	154.7	54.7	28.3	13.9	5.5
51	110.6	199.	50.0	50.0	.064	.083	.107	154.7	54.7	28.3	13.9	5.5
52	110.6	199.	50.0	50.0	.064	.083	.107	154.7	54.7	28.3	13.9	5.5
53	110.6	199.	50.0	50.0	.064	.083	.107	154.7	54.7	28.3	13.9	5.5
54	110.6	199.	50.0	50.0	.064	.083	.107	154.7	54.7	28.3	13.9	5.5
55	110.6	255.	50.0	50.0	.064	.083	.107	224.1	79.2	41.1	20.1	8.0
56	110.6	255.	50.0	50.0	.064	.083	.107	224.1	79.2	41.1	20.1	8.0
57	110.6	168.	50.0	50.0	.064	.083	.107	120.1	42.4	22.0	10.7	4.2
58	110.5	128.	50.0	50.0	.064	.083	.107	80.0	28.3	14.6	7.1	2.8
59	110.5	128.	50.0	50.0	.064	.083	.107	80.0	28.3	14.6	7.1	2.8
60	110.5	112.	50.0	40.0	.064	.083	.107	65.5	23.1	12.0	5.8	2.3
61	110.5	102.	50.0	50.0	.064	.083	.107	57.0	20.1	10.4	5.1	2.0
62	110.5	107.	50.0	35.0	.064	.083	.107	61.2	21.6	11.2	5.5	2.1
63	110.5	41.	50.0	0.0	.064	.083	.107	14.6	5.1	2.6	1.3	.5
64	110.5	77.	50.0	50.0	.064	.083	.107	37.4	13.2	6.8	3.3	1.3
65	110.5	71.	50.0	0.0	.064	.083	.107	33.1	11.7	6.0	2.9	1.1
66	110.5	66.	50.0	50.0	.064	.083	.107	29.7	10.5	5.4	2.6	1.0
67	110.5	66.	50.0	0.0	.064	.083	.107	29.7	10.5	5.4	2.6	1.0
68	110.5	56.	50.0	50.0	.064	.083	.107	23.2	8.2	4.2	2.0	.8
69	110.5	56.	50.0	0.0	.064	.083	.107	23.2	8.2	4.2	2.0	.8
70	110.5	56.	50.0	50.0	.064	.083	.107	23.2	8.2	4.2	2.0	.8
71	110.5	56.	50.0	0.0	.064	.083	.107	23.2	8.2	4.2	2.0	.8
72	110.5	51.	50.0	50.0	.064	.083	.107	20.2	7.1	3.7	1.8	.7
73	110.5	31.	50.0	0.0	.064	.083	.107	9.6	3.4	1.7	.8	.3
74	110.5	46.	50.0	45.0	.064	.083	.107	17.3	6.1	3.1	1.5	.6
75	110.5	51.	50.0	0.0	.064	.083	.107	20.2	7.1	3.7	1.8	.7
76	110.5	51.	50.0	50.0	.064	.083	.107	20.2	7.1	3.7	1.8	.7

TECHNICAL DATA OF SANDTRAP DESIGN

1. Data of downstream control section for defining water level in the sandtrap.

Width of the Control section [m]	Height of the Crest from the upstream floor [m]	Elevation of the Crest [m]
20. ,	.875 ,	108.875

2. Data of schematized sandtrap.

Number of compartment	Number of division along the sandtrap	side slope (1 for Vert.)	Cond. of W level along the sandtrap
1 ,	25 ,	1.5 ,	'NO'

3. Data for the accuracy of sedimentation :

Numbers of needed predictor-corrector	Number of refiner time steps for simulating piling up of sediment in the sandtrap
3 ,	4

4. Additional request for detail output (for computation checking).

Numbers of set of output	Specified time steps when the output are needed
4 ,	20, 21 , 22, 23

5. Detail dimension of sandtrap.
Give (Number of division + 1) set of data

Distance in axis direction [m]		Bottom Width [m]		Bottom Elevation [m]	
0.0	,	27.5	,	108.0	,
30.0	,	27.5	,	108.0	,
40.0	,	27.5	,	107.0	,
60.0	,	27.5	,	107.0	,
90.0	,	27.5	,	107.0	,
120.0	,	27.5	,	107.0	,
150.0	,	27.5	,	107.0	,
180.0	,	27.5	,	107.0	,
210.0	,	27.5	,	107.0	,
240.0	,	27.5	,	107.0	,
270.0	,	27.5	,	107.0	,
300.0	,	27.5	,	107.0	,
330.0	,	27.5	,	107.0	,
360.0	,	27.5	,	107.0	,
390.0	,	27.5	,	107.0	,
420.0	,	27.5	,	107.0	,
450.0	,	27.5	,	107.0	,
480.0	,	27.5	,	107.0	,
510.0	,	27.5	,	107.0	,
540.0	,	27.5	,	107.0	,
570.0	,	27.5	,	107.0	,
590.0	,	27.5	,	107.0	,
600.0	,	27.5	,	108.0	,
630.0	,	27.5	,	108.0	,
640.0	,	27.5	,	108.0	,

STRATEGIES 7

EFFICIENCY OF THE EXCLUSION DEVICES :

TIME [Days]	Fraction Number / Rep. Diameter [mm]	1 / .060	2 / .120	3 / .190	4 / .320	5 / .650	T. Concen.[ppm]	Efficiency [%]
		Concentration [ppm]						
1.00								
	River	114.80	40.60	21.00	10.30	4.10	190.80	.00
	Tunnel Excluder	106.49	28.37	8.68	1.39	.03	144.96	60.32
	Sandtrap	44.81	.38	.00	.00	.00	45.19	91.31
2.00								
	River	107.50	38.00	19.70	9.60	3.80	178.60	.00
	Tunnel Excluder	99.72	26.56	8.14	1.29	.03	135.74	60.32
	Sandtrap	43.57	.40	.00	.00	.00	43.97	90.96
3.00								
	River	107.50	38.00	19.70	9.60	3.80	178.60	.00
	Tunnel Excluder	99.72	26.56	8.14	1.29	.03	135.74	60.32
	Sandtrap	44.10	.41	.00	.00	.00	44.52	90.84
4.00								
	River	107.50	38.00	19.70	9.60	3.80	178.60	.00
	Tunnel Excluder	99.72	26.56	8.14	1.29	.03	135.74	60.32
	Sandtrap	44.66	.43	.00	.00	.00	45.09	90.72
5.00								
	River	107.50	38.00	19.70	9.60	3.80	178.60	.00
	Tunnel Excluder	99.72	26.56	8.14	1.29	.03	135.74	60.32
	Sandtrap	45.24	.44	.00	.00	.00	45.68	90.59
6.00								
	River	107.50	38.00	19.70	9.60	3.80	178.60	.00
	Tunnel Excluder	99.72	26.56	8.14	1.29	.03	135.74	60.32
	Sandtrap	45.85	.46	.00	.00	.00	46.31	90.46
7.00								
	River	107.50	38.00	19.70	9.60	3.80	178.60	.00
	Tunnel Excluder	99.72	26.56	8.14	1.29	.03	135.74	60.32
	Sandtrap	46.48	.48	.00	.00	.00	46.96	90.32
8.00								
	River	107.50	38.00	19.70	9.60	3.80	178.60	.00
	Tunnel Excluder	99.72	26.56	8.14	1.29	.03	135.74	60.32
	Sandtrap	47.14	.50	.00	.00	.00	47.63	90.17
9.00								
	River	148.90	52.60	27.30	13.30	5.30	247.40	.00
	Tunnel Excluder	138.12	36.76	11.28	1.79	.04	187.99	60.32
	Sandtrap	58.00	.51	.00	.00	.00	58.51	91.32
10.00								
	River	184.60	65.30	33.80	16.50	6.60	306.80	.00
	Tunnel Excluder	162.82	36.32	7.95	.62	.00	207.71	71.48
	Sandtrap	65.83	.46	.00	.00	.00	66.29	91.66

11.00							
River	230.70	81.60	42.30	20.70	8.20	383.50	.00
Tunnel Excluder	203.48	45.38	9.94	.78	.00	259.59	71.48
Sandtrap	77.90	.50	.00	.00	.00	78.40	92.12
12.00							
River	419.50	148.40	77.00	37.70	14.90	697.50	.00
Tunnel Excluder	370.01	82.53	18.10	1.42	.01	472.07	71.48
Sandtrap	124.11	.60	.00	.00	.00	124.71	93.15
13.00							
River	419.50	148.40	77.00	37.70	14.90	697.50	.00
Tunnel Excluder	370.01	82.53	18.10	1.42	.01	472.07	71.48
Sandtrap	128.77	.67	.00	.00	.00	129.45	92.88
14.00							
River	1262.40	446.50	231.60	113.40	45.10	2099.00	.00
Tunnel Excluder	1060.22	197.45	31.41	1.27	.00	1290.35	77.60
Sandtrap	101.93	.01	.00	.00	.00	101.93	98.08
15.00							
River	1912.60	676.40	350.90	171.80	68.30	3180.00	.00
Tunnel Excluder	1278.65	104.20	3.61	.01	.00	1386.47	90.89
Sandtrap	467.39	8.78	.02	.00	.00	476.19	98.08
16.00							
River	1859.40	657.70	341.20	167.10	66.40	3091.80	.00
Tunnel Excluder	1239.10	99.80	3.38	.01	.00	1342.29	90.98
Sandtrap	449.65	8.25	.02	.00	.00	457.91	98.08
17.00							
River	1358.90	480.60	249.30	122.10	48.50	2259.40	.00
Tunnel Excluder	1141.45	212.78	33.82	1.36	.00	1389.41	77.65
Sandtrap	122.88	.01	.00	.00	.00	122.89	97.85
18.00							
River	1000.20	353.70	183.50	89.80	35.70	1662.90	.00
Tunnel Excluder	840.19	156.66	24.89	1.00	.00	1022.75	77.67
Sandtrap	104.04	.01	.00	.00	.00	104.05	97.52
19.00							
River	718.20	254.00	131.80	64.50	25.60	1194.10	.00
Tunnel Excluder	603.34	112.55	17.88	.72	.00	734.49	77.68
Sandtrap	84.97	.02	.00	.00	.00	84.99	97.18
20.00							
River	861.50	304.70	158.00	77.40	30.70	1432.30	.00
Tunnel Excluder	723.76	135.07	21.44	.86	.00	881.13	77.70
Sandtrap	108.52	.02	.00	.00	.00	108.55	97.00
21.00							
River	788.80	279.00	144.70	70.80	28.10	1311.40	.00
Tunnel Excluder	662.74	123.75	19.65	.79	.00	806.93	77.71
Sandtrap	110.95	.03	.00	.00	.00	110.99	96.65
22.00							
River	968.10	342.40	177.60	87.00	34.60	1609.70	.00
Tunnel Excluder	813.43	151.94	24.12	.96	.00	990.45	77.73
Sandtrap	145.53	.05	.00	.00	.00	145.58	96.42

23.00								
	River	1011.00	357.60	185.50	90.80	36.10	1681.00	.00
	Tunnel Excluder	849.52	158.75	25.19	1.00	.00	1034.47	77.75
	Sandtrap	169.51	.08	.00	.00	.00	169.59	96.00
24.00								
	River	1011.00	357.60	185.50	90.80	36.10	1681.00	.00
	Tunnel Excluder	849.57	158.82	25.20	1.00	.00	1034.59	77.76
	Sandtrap	191.18	.13	.00	.00	.00	191.31	95.48
25.00								
	River	849.20	300.30	155.80	76.30	30.30	1411.90	.00
	Tunnel Excluder	713.64	133.42	21.16	.84	.00	869.08	77.78
	Sandtrap	186.11	.22	.00	.00	.00	186.34	94.75
26.00								
	River	539.30	190.70	98.90	48.40	19.20	896.50	.00
	Tunnel Excluder	433.97	68.24	7.93	.16	.00	510.30	81.31
	Sandtrap	137.24	.31	.00	.00	.00	137.55	93.58
27.00								
	River	435.80	154.10	79.90	39.10	15.50	724.40	.00
	Tunnel Excluder	350.68	55.14	6.41	.13	.00	412.37	81.31
	Sandtrap	123.67	.40	.00	.00	.00	124.06	92.80
28.00								
	River	273.00	96.50	50.00	24.50	9.70	453.70	.00
	Tunnel Excluder	245.80	59.32	14.90	1.54	.01	321.58	68.23
	Sandtrap	207.17	22.03	.75	.00	.00	229.94	74.71
29.00								
	River	230.70	81.60	42.30	20.70	8.20	383.50	.00
	Tunnel Excluder	207.71	50.16	12.61	1.31	.01	271.80	68.23
	Sandtrap	87.08	.84	.00	.00	.00	87.92	91.28
30.00								
	River	224.10	79.20	41.10	20.10	8.00	372.50	.00
	Tunnel Excluder	197.66	44.05	9.66	.76	.00	252.13	71.48
	Sandtrap	68.87	.37	.00	.00	.00	69.23	92.87
31.00								
	River	230.70	81.60	42.30	20.70	8.20	383.50	.00
	Tunnel Excluder	203.48	45.38	9.94	.78	.00	259.59	71.48
	Sandtrap	71.51	.39	.00	.00	.00	71.90	92.80
32.00								
	River	224.10	79.20	41.10	20.10	8.00	372.50	.00
	Tunnel Excluder	197.66	44.05	9.66	.76	.00	252.13	71.48
	Sandtrap	71.24	.41	.00	.00	.00	71.65	92.61
33.00								
	River	217.50	76.90	39.90	19.50	7.70	361.50	.00
	Tunnel Excluder	191.84	42.77	9.38	.74	.00	244.73	71.48
	Sandtrap	71.04	.43	.00	.00	.00	71.47	92.39
34.00								
	River	230.70	81.60	42.30	20.70	8.20	383.50	.00
	Tunnel Excluder	203.48	45.38	9.94	.78	.00	259.59	71.48
	Sandtrap	75.50	.46	.00	.00	.00	75.96	92.38

STRATEGIES 7

THE CHANGES OF GRADATION OF THE TRANSPORTED SEDIMENT :

TIME [Days]	Exclusion Device	Diameter [mm] D15	D35	D50	D65	D90	T. Concentration [ppm]	Efficiency [%]	Used Water [m3/s]
1.00									
	Tunnel Excluder	.038	.055	.070	.088	.150	144.96	60.32	5.00
	Sandtrap	.016	.024	.031	.040	.072	45.19	91.31	.00
2.00									
	Tunnel Excluder	.038	.055	.070	.088	.150	135.74	60.32	5.00
	Sandtrap	.016	.024	.031	.040	.072	43.97	90.96	.00
3.00									
	Tunnel Excluder	.038	.055	.070	.088	.150	135.74	60.32	5.00
	Sandtrap	.016	.024	.031	.040	.073	44.52	90.84	.00
4.00									
	Tunnel Excluder	.038	.055	.070	.088	.150	135.74	60.32	5.00
	Sandtrap	.016	.024	.031	.040	.073	45.09	90.72	.00
5.00									
	Tunnel Excluder	.038	.055	.070	.088	.150	135.74	60.32	5.00
	Sandtrap	.016	.025	.032	.041	.073	45.68	90.59	.00
6.00									
	Tunnel Excluder	.038	.055	.070	.088	.150	135.74	60.32	5.00
	Sandtrap	.016	.025	.032	.041	.073	46.31	90.46	.00
7.00									
	Tunnel Excluder	.038	.055	.070	.088	.150	135.74	60.32	5.00
	Sandtrap	.016	.025	.032	.041	.073	46.96	90.32	.00
8.00									
	Tunnel Excluder	.038	.055	.070	.088	.150	135.74	60.32	5.00
	Sandtrap	.016	.025	.032	.041	.073	47.63	90.17	.00
9.00									
	Tunnel Excluder	.038	.055	.070	.088	.150	187.99	60.32	5.00
	Sandtrap	.016	.024	.031	.040	.072	58.51	91.32	.00
10.00									
	Tunnel Excluder	.035	.051	.064	.080	.135	207.71	71.48	10.00
	Sandtrap	.015	.024	.030	.039	.071	66.29	91.66	.00
11.00									
	Tunnel Excluder	.035	.051	.064	.080	.135	259.59	71.48	10.00
	Sandtrap	.015	.023	.030	.039	.070	78.40	92.12	.00
12.00									
	Tunnel Excluder	.035	.051	.064	.080	.135	472.07	71.48	10.00
	Sandtrap	.015	.023	.029	.038	.069	124.71	93.15	.00
13.00									
	Tunnel Excluder	.035	.051	.064	.080	.135	472.07	71.48	10.00
	Sandtrap	.015	.023	.029	.038	.069	129.45	92.88	.00

14.00								
Tunnel Excluder	.033	.048	.059	.074	.124	1290.35	77.60	15.00
Sandtrap	.010	.016	.021	.028	.054	101.93	98.08	.00
15.00								
Tunnel Excluder	.025	.037	.046	.058	.098	1386.47	90.89	20.00
Sandtrap	.018	.027	.034	.044	.077	476.19	98.08	.00
16.00								
Tunnel Excluder	.025	.036	.046	.058	.098	1342.29	90.98	20.00
Sandtrap	.018	.027	.034	.043	.077	457.91	98.08	.00
17.00								
Tunnel Excluder	.033	.048	.059	.074	.124	1389.41	77.65	15.00
Sandtrap	.010	.016	.021	.028	.054	122.89	97.85	.00
18.00								
Tunnel Excluder	.033	.048	.059	.074	.124	1022.75	77.67	15.00
Sandtrap	.010	.016	.021	.028	.054	104.05	97.52	.00
19.00								
Tunnel Excluder	.033	.048	.059	.074	.124	734.49	77.68	15.00
Sandtrap	.010	.016	.021	.028	.055	84.99	97.18	.00
20.00								
Tunnel Excluder	.033	.048	.059	.074	.124	881.13	77.70	15.00
Sandtrap	.010	.016	.022	.029	.055	108.55	97.00	.00
21.00								
Tunnel Excluder	.033	.048	.059	.074	.124	806.93	77.71	15.00
Sandtrap	.011	.017	.022	.030	.056	110.99	96.65	.00
22.00								
Tunnel Excluder	.033	.048	.059	.074	.124	990.45	77.73	15.00
Sandtrap	.011	.017	.023	.030	.057	145.58	96.42	.00
23.00								
Tunnel Excluder	.033	.048	.059	.074	.124	1034.47	77.75	15.00
Sandtrap	.011	.018	.023	.031	.058	169.59	96.00	.00
24.00								
Tunnel Excluder	.033	.048	.059	.074	.124	1034.59	77.76	15.00
Sandtrap	.012	.018	.024	.032	.060	191.31	95.48	.00
25.00								
Tunnel Excluder	.033	.048	.059	.074	.124	869.08	77.78	15.00
Sandtrap	.012	.019	.025	.033	.062	186.34	94.75	.00
26.00								
Tunnel Excluder	.031	.045	.056	.069	.116	510.30	81.31	10.00
Sandtrap	.013	.021	.027	.035	.065	137.55	93.58	.00
27.00								
Tunnel Excluder	.031	.045	.056	.069	.116	412.37	81.31	10.00
Sandtrap	.014	.022	.028	.036	.067	124.06	92.80	.00
28.00								
Tunnel Excluder	.036	.053	.067	.083	.140	321.58	68.23	10.00
Sandtrap	.026	.038	.048	.060	.101	229.94	74.71	50.00
29.00								
Tunnel Excluder	.036	.053	.067	.083	.140	271.80	68.23	10.00
Sandtrap	.016	.025	.032	.041	.073	87.92	91.28	50.00
30.00								
Tunnel Excluder	.035	.051	.064	.080	.135	252.13	71.48	10.00
Sandtrap	.015	.023	.029	.038	.069	69.23	92.87	.00

31.00									
Tunnel Excluder	.035	.051	.064	.080	.135	259.59	71.48	10.00	
Sandtrap	.015	.023	.030	.038	.069	71.90	92.80	.00	
32.00									
Tunnel Excluder	.035	.051	.064	.080	.135	252.13	71.48	10.00	
Sandtrap	.015	.023	.030	.038	.070	71.65	92.61	.00	
33.00									
Tunnel Excluder	.035	.051	.064	.080	.135	244.73	71.48	10.00	
Sandtrap	.015	.023	.030	.039	.070	71.47	92.39	.00	
34.00									
Tunnel Excluder	.035	.051	.064	.080	.135	259.59	71.48	10.00	
Sandtrap	.015	.023	.030	.039	.070	75.96	92.38	.00	
35.00									
Tunnel Excluder	.035	.051	.064	.080	.135	259.59	71.48	10.00	
Sandtrap	.015	.023	.030	.039	.070	77.62	92.20	.00	
36.00									
Tunnel Excluder	.035	.051	.064	.080	.135	259.59	71.48	10.00	
Sandtrap	.015	.023	.030	.039	.071	79.42	92.02	.00	
37.00									
Tunnel Excluder	.035	.051	.064	.080	.135	259.59	71.48	10.00	
Sandtrap	.015	.024	.030	.039	.071	81.37	91.81	.00	
38.00									
Tunnel Excluder	.035	.051	.064	.080	.135	259.59	71.48	10.00	
Sandtrap	.015	.024	.031	.039	.071	83.47	91.59	.00	
39.00									
Tunnel Excluder	.035	.051	.064	.080	.135	259.59	71.48	10.00	
Sandtrap	.016	.024	.031	.040	.071	85.72	91.35	.00	
40.00									
Tunnel Excluder	.035	.051	.064	.080	.135	347.37	71.48	10.00	
Sandtrap	.015	.023	.030	.039	.071	106.63	91.99	.00	
41.00									
Tunnel Excluder	.035	.051	.064	.080	.135	347.37	71.48	10.00	
Sandtrap	.015	.024	.030	.039	.071	110.46	91.69	.00	
42.00									
Tunnel Excluder	.035	.051	.064	.080	.135	347.37	71.48	10.00	
Sandtrap	.016	.024	.031	.040	.071	114.60	91.36	.00	
43.00									
Tunnel Excluder	.024	.035	.044	.056	.095	1023.60	91.88	20.00	
Sandtrap	.017	.026	.033	.042	.075	324.62	91.36	.00	
44.00									
Tunnel Excluder	.033	.048	.059	.074	.124	827.83	77.81	15.00	
Sandtrap	.010	.016	.021	.028	.054	66.69	98.04	.00	
45.00									
Tunnel Excluder	.029	.042	.053	.066	.110	599.00	85.18	15.00	
Sandtrap	.010	.016	.021	.028	.054	56.87	97.82	.00	
46.00									
Tunnel Excluder	.029	.042	.053	.066	.110	572.47	85.18	15.00	
Sandtrap	.010	.016	.021	.028	.054	58.66	97.65	.00	
47.00									
Tunnel Excluder	.036	.053	.067	.083	.140	233.61	68.23	10.00	
Sandtrap	.019	.029	.036	.046	.081	126.70	84.32	50.00	

48.00								
Tunnel Excluder	.036	.053	.067	.083	.140	224.80	68.23	10.00
Sandtrap	.016	.025	.032	.041	.073	77.18	90.72	50.00
49.00								
Tunnel Excluder	.038	.055	.070	.088	.150	195.33	60.32	5.00
Sandtrap	.015	.024	.031	.039	.071	54.94	92.19	.00
50.00								
Tunnel Excluder	.038	.055	.070	.088	.150	195.33	60.32	5.00
Sandtrap	.015	.024	.031	.039	.071	55.76	92.07	.00
51.00								
Tunnel Excluder	.038	.055	.070	.088	.150	195.33	60.32	5.00
Sandtrap	.016	.024	.031	.040	.071	56.64	91.94	.00
52.00								
Tunnel Excluder	.038	.055	.070	.088	.150	195.33	60.32	5.00
Sandtrap	.016	.024	.031	.040	.072	57.59	91.80	.00
53.00								
Tunnel Excluder	.038	.055	.070	.088	.150	195.33	60.32	5.00
Sandtrap	.016	[illegible]	.031	.040	.072	58.60	91.65	.00
54.00								
Tunnel Excluder	.038	.055	.070	.088	.150	195.33	60.32	5.00
Sandtrap	.016	.024	.031	.040	.072	59.68	91.49	.00
55.00								
Tunnel Excluder	.035	.051	.064	.080	.135	252.13	71.48	10.00
Sandtrap	.015	.023	.030	.039	.070	75.28	92.22	.00
56.00								
Tunnel Excluder	.035	.051	.064	.080	.135	252.13	71.48	10.00
Sandtrap	.015	.023	.030	.039	.070	76.98	92.03	.00
57.00								
Tunnel Excluder	.038	.055	.070	.088	.150	151.60	60.32	5.00
Sandtrap	.016	.025	.032	.041	.074	55.68	89.69	.00
58.00								
Tunnel Excluder	.038	.055	.070	.087	.149	100.42	60.79	5.00
Sandtrap	.017	.026	.034	.043	.076	46.19	86.95	.00
59.00								
Tunnel Excluder	.038	.055	.070	.087	.149	100.42	60.79	5.00
Sandtrap	.017	.026	.034	.043	.076	46.66	86.80	.00
60.00								
Tunnel Excluder	.017	.064	.083	.107	.076	108.70	.00	.00
Sandtrap	.014	.022	.029	.037	.068	21.13	93.50	.00
61.00								
Tunnel Excluder	.014	.064	.083	.107	.068	94.60	.00	.00
Sandtrap	.019	.028	.036	.046	.081	41.83	84.50	.00
62.00								
Tunnel Excluder	.019	.064	.083	.107	.081	101.60	.00	.00
Sandtrap	.012	.019	.025	.033	.062	12.23	95.99	.00
63.00								
Tunnel Excluder	.012	.064	.083	.107	.062	2.92	.00	.00
Sandtrap	.010	.015	.021	.027	.053	.58	95.99	.00
64.00								
Tunnel Excluder	.010	.064	.083	.107	.053	62.00	.00	.00
Sandtrap	.022	.032	.041	.051	.088	34.95	79.36	.00

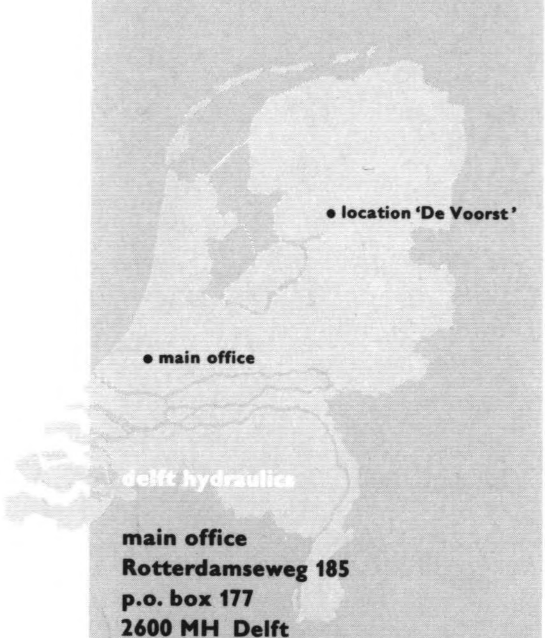
65.00									
	Tunnel Excluder	.022	.064	.083	.107	.088	30.93	.00	.00
	Sandtrap	.016	.025	.032	.041	.073	25.99	79.36	.00
66.00									
	Tunnel Excluder	.016	.064	.083	.107	.073	49.20	.00	.00
	Sandtrap	.025	.036	.045	.057	.096	29.97	76.44	.00
67.00									
	Tunnel Excluder	.025	.064	.083	.107	.096	29.97	.00	.00
	Sandtrap	.019	.028	.036	.045	.079	25.72	76.44	.00
68.00									
	Tunnel Excluder	.019	.064	.083	.107	.079	38.40	.00	.00
	Sandtrap	.029	.042	.052	.064	.107	25.05	72.28	.00
69.00									
	Tunnel Excluder	.029	.064	.083	.107	.107	25.05	.00	.00
	Sandtrap	.022	.033	.042	.052	.089	21.19	72.28	.00
70.00									
	Tunnel Excluder	.022	.064	.083	.107	.089	38.40	.00	.00
	Sandtrap	.029	.042	.052	.065	.107	25.09	72.18	.00
71.00									
	Tunnel Excluder	.029	.064	.083	.107	.107	25.09	.00	.00
	Sandtrap	.023	.033	.042	.052	.089	21.22	72.18	.00
72.00									
	Tunnel Excluder	.023	.064	.083	.107	.089	33.50	.00	.00
	Sandtrap	.032	.045	.056	.069	.114	22.98	68.72	.00
73.00									
	Tunnel Excluder	.032	.064	.083	.107	.114	10.93	.00	.00
	Sandtrap	.026	.038	.047	.058	.097	9.32	68.72	.00
74.00									
	Tunnel Excluder	.026	.064	.083	.107	.097	28.60	.00	.00
	Sandtrap	.024	.036	.045	.056	.095	17.41	76.57	.00
75.00									
	Tunnel Excluder	.024	.064	.083	.107	.095	20.32	.00	.00
	Sandtrap	.018	.028	.035	.045	.079	17.45	76.57	.00
76.00									
	Tunnel Excluder	.018	.064	.083	.107	.079	33.50	.00	.00
	Sandtrap	.032	.046	.056	.070	.114	23.03	68.55	.00

STRATEGIES 7

SEDIMENT AND WATER BALANCE

TIME [Days]	RIVER Discharge [m3/s]	RIVER Concentration [ppm]	Irrigation Requirement [m3/s]	SUPPLIED Discharge [m3/s]	SUPPLIED Concentration [ppm]	Used Water for Flushing [1000 m3]	Sed. Transport Cap. of Canal [ppm]	Sediment Balance [m3]
1.00	163.00	190.80	50.00	50.00	45.19	432.00	175.05	-211.69
2.00	156.00	178.60	50.00	50.00	43.97	432.00	175.01	-213.62
3.00	156.00	178.60	50.00	50.00	44.52	432.00	175.00	-212.71
4.00	156.00	178.60	50.00	50.00	45.09	432.00	174.98	-211.75
5.00	156.00	178.60	50.00	50.00	45.68	432.00	174.96	-210.75
6.00	156.00	178.60	50.00	50.00	46.31	432.00	174.95	-209.71
7.00	156.00	178.60	50.00	50.00	46.96	432.00	174.93	-208.62
8.00	156.00	178.60	50.00	50.00	47.63	432.00	174.91	-207.48
9.00	194.00	247.40	50.00	50.00	58.51	432.00	175.03	-189.94
10.00	224.00	306.80	50.00	50.00	66.29	864.00	175.16	-177.47
11.00	260.00	383.50	50.00	50.00	78.40	864.00	175.19	-157.79
12.00	388.00	697.50	50.00	50.00	124.71	864.00	175.31	-82.49
13.00	388.00	697.50	50.00	50.00	129.45	864.00	175.28	-74.72
14.00	811.00	2099.00	50.00	30.00	101.93	1296.00	123.37	-20.96
15.00	1071.00	3180.00	50.00	.00	476.19	1728.00	.00	.00
16.00	1051.00	3091.80	50.00	.00	457.91	1728.00	.00	.00
17.00	852.00	2259.40	50.00	30.00	122.89	1296.00	123.36	-.46
18.00	694.00	1662.90	50.00	30.00	104.05	1296.00	123.36	-18.89
19.00	556.00	1194.10	50.00	30.00	84.99	1296.00	123.36	-37.53
20.00	628.00	1432.30	50.00	30.00	108.55	1296.00	123.36	-14.49
21.00	592.00	1311.40	50.00	30.00	110.99	1296.00	123.35	-12.09
22.00	679.00	1609.70	50.00	30.00	145.58	1296.00	123.35	21.74
23.00	699.00	1681.00	50.00	30.00	169.59	1296.00	123.35	45.23
24.00	699.00	1681.00	50.00	30.00	191.31	1296.00	123.34	66.49
25.00	622.00	1411.90	50.00	30.00	186.34	1296.00	123.31	61.65
26.00	459.00	896.50	50.00	30.00	137.55	864.00	123.26	13.99
27.00	398.00	724.40	50.00	30.00	124.06	864.00	123.21	.83
28.00	291.00	453.70	50.00	5.00	229.94	5184.00	2.47	37.08
29.00	260.00	383.50	50.00	5.00	87.92	5184.00	2.54	13.92
30.00	255.00	372.50	50.00	50.00	69.23	864.00	175.28	-172.87
31.00	260.00	383.50	50.00	50.00	71.90	864.00	175.27	-168.51
32.00	255.00	372.50	50.00	50.00	71.65	864.00	175.25	-168.88
33.00	250.00	361.50	50.00	50.00	71.47	864.00	175.23	-169.15
34.00	260.00	383.50	50.00	50.00	75.96	864.00	175.22	-161.81
35.00	260.00	383.50	50.00	50.00	77.62	864.00	175.20	-159.08
36.00	260.00	383.50	50.00	50.00	79.42	864.00	175.19	-156.11
37.00	260.00	383.50	50.00	50.00	81.37	864.00	175.16	-152.90
38.00	260.00	383.50	50.00	50.00	83.47	864.00	175.14	-149.44
39.00	260.00	383.50	50.00	50.00	85.72	864.00	175.12	-145.73
40.00	316.00	513.20	50.00	50.00	106.63	864.00	175.19	-111.75

41.00	316.00	513.20	50.00	50.00	110.46	864.00	175.15	-105.47	
42.00	316.00	513.20	50.00	50.00	114.60	864.00	175.12	-98.66	
43.00	903.00	2464.50	50.00	.00	324.62	1728.00	.00	.00	
44.00	602.00	1344.60	50.00	30.00	66.69	1296.00	123.36	-55.43	
45.00	536.00	1130.50	50.00	30.00	56.87	1296.00	123.37	-65.04	
46.00	520.00	1080.40	50.00	30.00	58.66	1296.00	123.36	-63.29	
47.00	235.00	329.60	50.00	5.00	126.70	5184.00	2.52	20.24	
48.00	229.00	317.20	50.00	5.00	77.18	5184.00	2.54	12.17	
49.00	199.00	257.10	50.00	50.00	54.94	432.00	175.14	-195.94	
50.00	199.00	257.10	50.00	50.00	55.76	432.00	175.12	-194.58	
51.00	199.00	257.10	50.00	50.00	56.64	432.00	175.11	-193.12	
52.00	199.00	257.10	50.00	50.00	57.59	432.00	175.09	-191.55	
53.00	199.00	257.10	50.00	50.00	58.60	432.00	175.07	-189.87	
54.00	199.00	257.10	50.00	50.00	59.68	432.00	175.05	-188.08	
55.00	255.00	372.50	50.00	50.00	75.28	864.00	175.21	-162.91	
56.00	255.00	372.50	50.00	50.00	76.98	864.00	175.19	-160.10	
57.00	168.00	199.40	50.00	50.00	55.68	432.00	174.84	-194.25	
58.00	128.00	132.80	50.00	50.00	46.19	432.00	174.48	-209.15	
59.00	128.00	132.80	50.00	50.00	46.66	432.00	174.45	-208.33	
60.00	112.00	108.70	50.00	40.00	21.13	.00	152.61	-171.47	
61.00	102.00	94.60	50.00	50.00	41.83	.00	173.48	-214.62	
62.00	107.00	101.60	50.00	35.00	12.23	.00	139.06	-144.73	
63.00	41.00	24.10	50.00	.00	.58	.00	.00	.00	
64.00	77.00	62.00	50.00	50.00	34.95	.00	171.60	-222.77	
65.00	71.00	54.80	50.00	.00	25.99	.00	.00	.00	
66.00	66.00	49.20	50.00	50.00	29.97	.00	169.18	-226.94	
67.00	66.00	49.20	50.00	.00	25.72	.00	.00	.00	
68.00	56.00	38.40	50.00	50.00	25.05	.00	164.84	-227.88	
69.00	56.00	38.40	50.00	.00	21.19	.00	.00	.00	
70.00	56.00	38.40	50.00	50.00	25.09	.00	164.74	-227.66	
71.00	56.00	38.40	50.00	.00	21.22	.00	.00	.00	
72.00	51.00	33.50	50.00	50.00	22.98	.00	161.64	-226.05	
73.00	31.00	15.80	50.00	.00	9.32	.00	.00	.00	
74.00	46.00	28.60	50.00	45.00	17.41	.00	159.02	-207.76	
75.00	51.00	33.50	50.00	.00	17.45	.00	.00	.00	
76.00	51.00	33.50	50.00	50.00	23.03	.00	161.49	-225.71	



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