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Publication date:
1999

Document Version
Accepted author manuscript, peer reviewed version

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August 1999

Published 1999 by
Hydraulics & Coastal Engineering Group
Aalborg University

Printed in Denmark by
Centertrykkeriet, Aalborg University

ISSN 0909-4296 SERIES PAPER NO. 19

PREFACE

The present thesis is submitted as one of the requirements for the Danish Ph.D. degree according to the notice (no. 989) of December, 1992 from the Danish Ministry of Education. The study was carried out at Aalborg University, Department of Civil Engineering with Associate Prof. Kjeld Schaarup-Jensen as supervisor. The work was financially supported by the Danish Technical Research Council within the framework program on "Solids in Sewage Systems".

Besides the Ph.D. thesis following publications were prepared:

> Flemming Schlütter (1999), A Conceptual Model for Sediment Transport in Combined Sewer Systems., *Water Science and Technology*, Vol. 39 (9), (UDM98)
>
> Flemming Schlütter and Michael R. Rasmussen (1998), Image Processing as a Method for Analysis of Wastewater Quality., 3rd Int. Conference on Innovative technologies in urban storm drainage (NOVATECH), Lyon, France.
>
> Flemming Schlütter and Kjeld Schaarup-Jensen (1998), Sediment Transport Under Dry Weather Conditions in a Small Sewer System., *Water Science and Technology*, Vol. 37 (1), pp.155-162.

I would like to thank the researchers at the CERGRENE-LABAM laboratory from "L'Ecole Nationale de Ponts et Chausées - Ecole Nationale du Génie Rural des Eaux et des Fôrets - Université Paris Val de Marne" for their kind collaboration during my stay at the laboratory and for letting me exploit some of the data collected from the Le Marais catchment in Paris. I would like to pay special thanks to M.Sc. M. Ahyerre, Prof. G. Chebbo and M.Sc. M.C. Gromaire-Mertz for their constructive collaboration.

Finally, I would like to thank my colleague Michael R. Rasmussen for many fruitful discussions and always inspiring attitude.

Aalborg, August, 1999 Flemming Schlütter

CONTENTS

List of symbols

Abreviations

AD	Advection dispersion.
ANN	Artificial neural network.
ARMA	Auto regression moving average.
BMP	Best management practices.
CM	Conceptual model.
CSO	Combined sewer overflow.
dis	Domestically and industrially derived sewer sediments.
DWF	Dry weather flow.
LCM	Lumped conceptual model.
PE	Person equivalent.
PM	Physically-based model.
RTC	Real time control.
SM	Statistically-based model.
STNeuro	Sediment transport neural network model.
STSim	Sediment transport simulation.
SWMM	Storm water management model.
WWTP	Wastewater treatment plant.

Pollutant parameters

BOD_5	Biological oxygen demand
N_{Kj}	Kjeldahl nitrogen
P_{tot}	Total phosphorous
Cd	Cadmium
COD	Chemical oxygen demand
Cr	Chromium
Cu	Copper
DEPH	di(2-ethyle) phtalate
DO	Dissolved oxygen
Fe	Iron
H2S	Hydrogen sulphide
Hg	Mercury
LAS	Linear alcylbenzenesulphonates
Mn	Manganese
NH3	Ammonia
Ni	Nickel
NPE	Nonylephenolethoxylates
PAC	Polocyclic aromatic hydro carbons
PAH	Polyaromatic hydrocarbons
Pb	Lead
SS	Suspended solids
TOC	Total organic carbon
TSS	Total suspended solids
VSS	Volatile suspended solids
Zn	Zinc

Parameters

α_{acc}		Numerical coeficient.
α_{acc}	[kg/ha/day]	Accumulation rate on the surface.
α_{dep}	[1/day]	Deposition rate in sewer pipes.
α_{ero}	[$1/m^2$]	Erosion rate in sewer pipes.
α_{rem}	[1/day]	Removal rate.
α_{sc}	[%]	Pipe storage capacity in percent of pipe volume.
$\alpha_1,\ \alpha_2,\ \alpha_3,\ \alpha_4$		Numerical coefficients.
α_w	[mm^{-1}]	Washoff coeficient.
β		Momentum value.
Δ		Specific density $\rho_s/\rho_w - 1$
ϵ_{RMS}		Root mean square error.
η		Learning rate.
$\overline{d}$	[m]	Mean particle diameter.
ρ_s	[kg/m^3]	Density of particles.
ρ_w	[kg/m^3]	Density of water.
τ_{cd}	[N/m^2]	Critical bed shear stress for deposition.
τ_{ce}	[N/m^2]	Critical bed shear stress for erosion.
τ_0	[N/m^2]	Mean bed shear stress.
A	[m^2]	Area.
C	[mg/l]	Mean concentration at water level y.
c	[mg/l]	Local concentration.
$c_1,\ c_2,\ c3$		Numerical coefficients.
C_V		Volumetric concentration.
D		Longitudinal dispersion coeficient.
d_{50}	[m]	Particle diameter corresponding to the 50 % fractile.
E		Error function.
g	[m/s^2]	Gravity.
h	[m]	Total water depth.
I		Gradient of the energy line
$i(t)$	[$\mu m/s$]	Rainfall intensity at time t.
I_0		Slope of the bottom of the sewer pipe.
$I10_{max}$	[$\mu m/s$]	Maximum 10 minutes average rain intensity.
$I5_{max}$	[$\mu m/s$]	Maximum 5 minutes average rain intensity.

k_s		Nikuradse equivalent sand roughness.
M_{acc}		Normalized accumulated load.
$M_{c,initial}$	[kg/ha]	Initial load on the catchment surface at the beginning of the rainfall event.
$m_c(t)$	[kg/ha]	Mass deposited on the surface at time t.
m_{max}	[kg/ha]	Maximum load on the surface.
$M_{p,initial}$	[%]	Initial mass in pipes given as a percentage of the pipe volume.
$m_p(t)$	[kg/m]	Mass deposited in sewer pipe at time t.
$m_a(t)$	[kg/ha]	Mass deposited on the surface at time t.
$me(t)$	[kg/ha]	Washed off mass at time t.
Mt_{accu}	[kg]	Accumulated mass transport for a complete rainfall event.
Mt_{max}	[kg/s]	Maximum mass transport rate.
P	[m]	Wetted perimeter.
$p(t)$	$[m^3/s]$	Pollutant flux at time t.
$pe(t)$	$[m^3/s]$	Pollutant flux entering into suspension at time t.
Q	$[m^3/s]$	Discharge.
Q_{DWF}	$[m^3/s]$	Discharge during dry weather period.
q_b	[mm/h]	Specific surface runoff.
$r(t)$	$[m^3/s]$	Runoff rate from impervious surface at time t.
R^2		Correlation coeficient.
R_h	[m]	Hydraulic radius.
S_m	[mm]	Soil depression storage.
t	[s]	Time.
T_{rain}	[min]	Duration of rainfall event.
T_{sss}	[min]	Time since start of rain event.
tff	[s]	First flush duration.
TSS_{DWF}	$[kg/s]$	Flux of suspended solids during dry weather period.
TSS_{STORM}	$[kg/s]$	Flux of suspended solids during a rainfall event.
u	[m/s]	Local velocity.
U_*	[m/s]	Shear velocity.
U_f	[m/s]	Friction velocity.
V_{acc}		Normalized accumulated discharge volume.
W_s	[mm/s]	Settling velocity.

Summary

The present thesis considers modelling of sediment transport in sewer systems. Emphasis is put on numerical modelling of suspended solids in combined sewer systems. Different modelling approaches are applied and evaluated in the light of results from the other applied models.

First part of the thesis presents a description of pollutant sources and characteristics of sediments. It also renders a description of the physical processes involved in sediment transport and in particular sediment transport in sewer systems.

Second part of the thesis considers how to approach numerical modelling dependent on the aims for the modelling. Subsequently, three significantly different sediment transport models are presented. Each model is applied on a case study catchment named Le Marais, which is located in the centre of Paris.

The thesis, finally, discusses the advantages and drawbacks of the different model types.

Summary in Danish

Nærværende afhandling omhandler modelering af sediment transport i kloaksystemer. Der er lagt vægt på numerisk modelering af af suspenderet materiale i fælleskloakerede systemer. Der anvendes forskellige fremgangsmåder for modeleringen, som evalueres på baggrund af resultaterne ved modelering ved hjælp af andre modeller.

Første del af afhandlingen præsenterer en beskrivelse af forureningskilder og karakteristika af sedimenter. Der bliver yderligere fremlagt en beskrivelse af de involverede fysiske processer relateret til sediment transport og nærmere betegnet til transport i kloaksystemer.

Anden del af afhandlingen omhandler mulige indfaldsvinkler til numerisk modelering anfhængende af hvilke målsætninger der kræves. Tre forskellige sediment transport modeller præsenteres. Hver model anvendes på et specifikt opland, som hedder "Le Marais". Dette opland er en del af Paris centrum.

Afhandlingen diskuterer slutteligt fordele og ulemper ved de forskellige modeltyper.

CHAPTER 1

Introduction

Depending on the point of view a combined sewer system needs to perform a number of functions. During dry weather (DWF) it should be able to transport domestically and industrially derived wastewater with its content of solids and solutes to the wastewater treatment plant (WWTP). During wet weather it must be able to drain the catchment area of the rain water. In the course of the rain event it is important that the large volumes of water are transported downstream without causing flooding in buildings or on the surface. During severe rain events not all the wastewater can be treated at the WWTP, some often needs to be discharged via overflow structures to local recipients such as channels, rivers, lakes, etc.

Historically, the sole concern has been to design sewer systems in such a way that stormwater is routed as quickly as possible from urban areas to the local recipient in order to avoid flooding. This approach has been a great success improving the health and life expectancy for people in the western world significantly. Only during app. the last three decades, there has been a concern about the detrimental effect on the receiving waters. In this connection quality modelling has been developed for sewer systems and receiving waters. The most crucial part of quality modelling is the ability to model the physical, biological, and chemical processes leading to the transport of solids.

At different locations and points in time, it is thus desirable to know not only the hydraulic conditions but also the quality of the wastewater. The existence and transport of sediments plays a significant role in connection with both the hydraulic conditions and the wastewater quality. Sediment deposits may reduce the hydraulic capacity of the system both locally and in general. During dry weather these deposits can result in septicity possibly accompanied by gas and corrosive acidity production (Ellis and Marsalek 1996).

Combined sewer overflow (CSO) should have only a small influence on recipients both regarding the hydraulic effects and the effects of pollutant load discharged. According to (IDA 1998) the following list (translated) of effects on recipients should be considered:

1. Hydraulic overloading resulting in erosion and flooding.
2. Deposition of sludge and other sediments from the sewer system.
3. Esthetic pollution by gross solids.
4. Hygienic pollution with potential health risk to humans and animals.
5. Discharge of organic material which can cause oxygen depletion and/or penetrate into spawning grounds.
6. Discharge of nutrients which can result in eutrofication in lakes and fjords.
7. Discharge of substances with toxic effects including anthropogenic substances (heavy metals, pesticides, oil, ammonia, hydrogen sulphide, etc.) which can entail a deterioration of biological conditions in the recipient indicated by the fauna index or saprobie index or similar.

The effects on the recipient may have very different timescales. The effects may either be termed acute or cumulative and the effects may subsequently entail different recovery duration for the recipient (Ellis and Hvitved-Jacobsen 1996).

1.1 Motivation

Being able to predict deposition and transport of sediments in sewer systems would be beneficial in many respects. Modelling of sediment transport will facilitate modelling of the transport of pollutants and modelling of processes going on in the sewer and ultimately help in prediction of the environmental impacts.

As already indicated it is of interest to be able to predict the location of sediment deposits in combined sewer systems and thus the self-cleansing ability of the system. This is the case both when evaluating existing systems and predicting the performance of new or renovated systems.

In combination with the introduction of real time control (RTC) in a sewer system it is important to ensure the transport capacity for sediments under the controlled conditions.

In order to be able to predict the discharge of sediments and attaining pollutants during CSO events modelling of the mass transport of sediment arriving at the CSO structure is essential. Furthermore, prediction of the mass transport arriving during rain events at the WWTP is interesting. These issues becomes even more relevant when dealing with the renewal or extension of a sewer system. Modelling of the sediment transport will help in decision making and design of retention basins in connection with rain induced discharges from CSO structures. As a dilemma overdesign of the sewer and retention may endanger the WWTP and the self-cleansing ability of the sewer system (Ellis 1986).

Another incentive to sediment transport modelling is that it may turn out to be important to be able to evaluate the influence of introducing retention basins, RTC, and BMP's (e.g. stormwater infiltration) locally in the system. Modelling allows for "what-if" scenarios.

1.2 Aspects in modelling

There may be different objectives in developing sediment transport models. Some models can be seen as "research models" rather than models developed for "easy application". When dealing with models of the latter type the predominant belief is that the models must be tailored to suit the application in hand. The "research" model is often developed to get increased insight into the phenomenae it describes.

It is of course relevant to be able to model a number of different sewer systems such as traditional combined sewer systems, separate systems, open channel systems (developing countries), systems with stormwater infiltration, etc. This study concentrates on combined sewer systems as a number of problems can be associated with these systems. These problems may be build-up of sediment deposits and with that the attaining problem of maintenance. Flush out of sediments and attaining pollutants cause problems to receiving waters and may cause overloading of the WWTP. Furthermore, the combined sewer system is so far the most predominant in major urban areas and it is the structural maintenance and extension of these systems which could benefit from transport modelling of both water and solids.

When concerned about the impact of urban runoff on receiving waters what is ultimately desirable is to model the ecosystem in order to evaluate the sustainability. The discharge from CSO's is one of the inputs. This means that prediction of the mass transport during rain events for wastewater arriving at the CSO and the WWTP is of paramount importance. The liable location for build-up of sediments during dry weather is also important when the sewer systems are physically changed or extended or when the input of wastewater changes. These entities are thus the main aim of the modelling efforts in this study.

Development of quality models must be seen in context with expected demands for documentation of sewer systems performance in the future. Up till present time all emphasis has been put on avoiding flooding. This has been achieved by using relatively rough design methods and subsequently choosing slightly larger pipe diameters than estimated by calculation. This practice has some inherent safety factors. Development of more precise design methods using the dynamic wave model has necessitated new standards. One of these standards is the European standard EN 752-4 (CEN 1997). This standard allows for reduced return periods for design events when calculations are performed using models enabling a check of flooding conditions. This development towards relying on numerical models may be expected within quality standards as well. The European standard stipulates that "design shall be such that the receiving water will be protected against overloading of its self-purifying capacity". The necessary emission limits can be set "by the relevant authority using, where appropriate, water quality simulation models". The standard also recognizes that sediment deposits may increase the risk of flooding and pollution, but only basic self-cleansing criteria are suggested.

This thesis presents developments of new sewer sediment transport modelling approaches and evaluation of these compared with existing models. The aim is to determine the robustness and reliability of different model types given realistically obtainable input data. This will facilitate the choice of model type to develop and apply for numerical modelling of sediment transport in combined sewer systems.

In this thesis chapter 2 describes pollutant sources and sediment characteristics. Prior to the description of numerical modelling, chapter 3 renders a short description of the physics involved in transport of solids in sewer systems. This chapter also presents a selected review of sediment transport theory for pipe hydraulics. Chapter 4 presents a case study catchment named "Le Marais". Chapter 5 till 9 deals with different approaches in numerical sediment transport modelling. These are physically based modelling, conceptual modelling, and finally, black-box modelling. The different models are evaluated and compared in chapter 10.

CHAPTER 2

Pollutant sources and characteristics

Even though this dissertations main topic is numerical modelling of sewer sediment transport it is imperative to describe pollutant sources and characteristics. The objective is to depict the variability in the sources and characteristics. This assists in putting the reliability of numerical modelling of these pollutants into perspective. This chapter also puts forward arguments on why it can be reasonable to concentrate on modelling total suspended solids in this study rather than trying to encompass the modelling of all pollutant constituents.

2.1 Pollutant sources

Pollutant sources are numerous and can be subdivided into sources with different characteristics. A list of possible sources are given in Ashley et al. (1999):

1. Atmosphere
2. Surfaces on the catchment
3. Domestic sewage
4. Industrial and commercial effluents
5. The environment and processes in sewer pipes
6. Construction sites

Some of the pollutants interacts before they reach the sewer system and interacts with pollutants already existing in the system.

Pollutants may encompass a number of substances, whereas sediments are always particles which has the ability to settle, in this case in water. A number of the pollutants are connected closely with the sediments, as will be described in paragraph 2.5. Some pollutants and pollutant level indicators can be listed as seen in table 2.1.

Group	Pollutant / pollutant indicator examples
Gross solids	Faecal solids, sanitary refuse, etc.
Suspended solids	TSS
Biodegradable organic matter	BOD, COD, TOC
Toxic pollutants	Heavy metals - Cu, Zn, Cd, Pb, Ni, Cr, Hg Organic micropollutants - pesticides, herbicides, clorinated hydrocarbons, phenols, polyaromatic hydrocarbons (PAH), linear alcylbenzenesulphonates (LAS), nonylephenolethoxylates (NPE), di(2-ethyle) phthalate (DEPH) Other types - ammonia (NH3), hydrogen sulphide (H2S), hydrocarbons (oils and greases)
Nutrients	Nitrogen, Phosphorous
Bacteria and viruses	Indicator organisms: E.Coli, faecal coli, faecal streptococci

Table 2.1: *Pollutants connected with combined sewer systems.*

2.2 Surface pollutants

The sources of pollutants ending up on the catchment surface may be atmospheric fall out of small particles, soil erosion transported to the catchment area, leaves, roadside spraying, etc.

Some of the fine particles may not fallout onto the surface before the beginning of the rain event. These particles suspended in the atmosphere originates from aerosols, heating, auto traffic, waste incineration, industry, construction sites, erosion of soils, etc. (Ashley et al. 1999). Concentration levels in the precipitation are seen in table 2.2.

The ranges shown in table 2.2 are very broad. This is due to high variability between sites. Washoff from roof surfaces can often be contained within the shown ranges even though additional pollutants comes from degradation of the

Parameter	Range	
COD	20 – 160	mg/l
Cd	0.15 – 17	μg/l
Cu	0.5 – 78	μg/l
Fe	0.1 – 300	μg/l
Mn	1 – 15	μg/l
Ni	1.5 – 50	μg/l
Pb	1.6 – 110	μg/l
Zn	5 – 800	μg/l
Hydrocarbons	0.02 – 0.07	μg/l

Table 2.2: *Concentration levels in atmospheric precipitation*
(from Ashley et al. 1999).

roof itself, bird dropping and vegetation.

Surfaces which attributes to the rainfall runoff are in general roofs, parkings, roads, highways, and pavements. The street runoff is, compared with the complete catchment surface, introducing the largest part of sediments into the sewer. A study in Karlsruhe showed that app. 90 % of the sediments originates from this source (Xanthopoulos and Augustin 1992). Most street runoff sediments had a particle size between 50–80 μm. These fine particles were found to be contaminated with heavy metals, polycyclic aromatic hydrocarbons (PAH) and mineral oils, with a degree dependent on the traffic density.

At the level of streets and parking areas the solids occur due to (Ashley et al. 1999):

- The erosion of road material
- The wear of vehicles tires
- The dry deposition of fine solids from the atmosphere
- Solid wastes and litter ("gross solids")
- Animal droppings and vegetation
- De-icing materials
- Construction sites

A number of these sources are illustrated on figure 2.1.

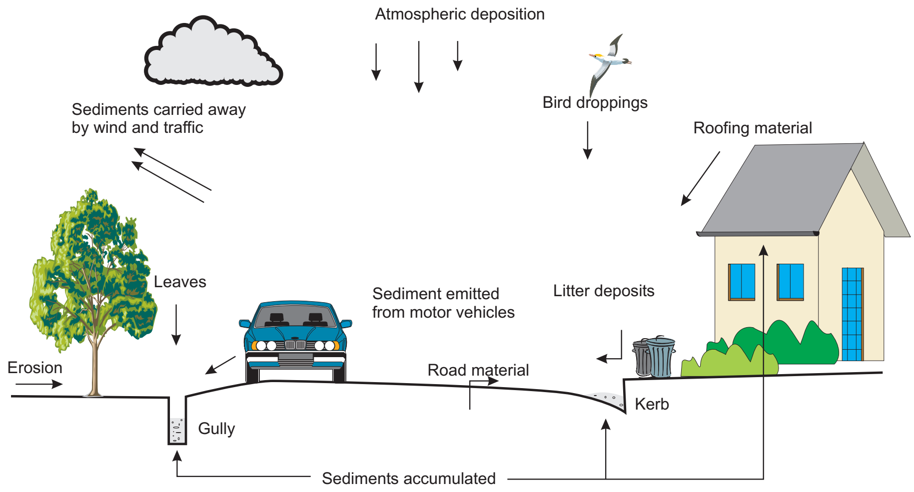


Figure 2.1: *Sources of pollutants on the catchment surface.*

2.2.1 Surface sediment characteristics

The sediments deposited on the surface are mainly (90 %) rather small particles with a grain size below 80 μm. Up till 40 % of the sediments have been found to be organic (Ashley and Crabtree 1992). Variations between different locations are large, both considering particle sizes and organic content.

As a special issue, the difference between winter and summer conditions is considerable in areas where winter grittings and de-icing is used. Winter grittings can in some places be the single most significant contributor to sediment loading of the sewer system (Ashley and Crabtree 1992). Rock salt used for de-icing contains 5-10 % of insoluble solids resulting in an increased sediment loading where used. Other influences during winter increases the load as well, e.g. lowered combustion efficiency of motor vehicles. Sansalone and Buchberger (1997) investigated heavy metal runoff from snowmelt compared with rainfall runoff and found that the heavy metals are connected to the smallest particle fractions, but there were no clear difference between concentration levels for snowmelt and rainfall runoff respectively. Comparing lead measurements from earlier investigations a decrease in concentration was found, which is attributed to the removal of Pb from gasoline.

The average accumulation rate of sediments on the surface vary dependent on the catchment characteristics. Ashley et al. (1999) presents a comparison of result from different researchers. This comparison reports a range of the loads, which for residential areas is 95–3200 kg/ha/year, for commercial areas 50–1220 kg/ha/year, and for industrial areas is 400–1700 kg/ha/day. The deposited sediments on the surface at the initiation of a rain event cannot all be assumed to be washed off during the event and therefore it is difficult to exploit the reported build-up rates. Taking a longer period of time containing a number of events the build-up and removal of materials from the surface will of course be equal. Additional build-up rates have been reported by Butler et al. (1992) and Mance and Harman (1978).

2.3 Wastewater pollutant contributions

The domestic wastewater is in general very predictable. This is valid for both the diurnal cycle of wastewater production and the production of solids (e.g. see Schlütter and Schaarup-Jensen 1997). The input from commercial areas and industry is more difficult to assess. For domestic wastewater the solids can be categorized as

- Fine faecal and other organic matter
- Large faecal and organic matter (gross solids)
- Paper, rags, and misc. sewage litter (sanitary litter)

Of course the input of solids with the dry weather flow becomes more important the larger and more densely populated the catchment is (Ashley and Crabtree 1992). The BOD characteristic varies considerably in the wastewater during the diurnal cycle (Butler et al. 1995). The change of organic content in the wastewater has also been observed in connection with this Ph.D. study. Investigations at Frejlev (Schaarup-Jensen et al. 1998) has shown considerable higher contents of organic material in the wastewater during the morning hours.

The toilet generates 30-40 % of the domestic wastewater. It is therefore interesting to determine the characteristics of the wastewater originating from the toilet. Most of the flushes are only urine related, but it is estimated that Europeans produces 100-130 g_{wet}/capita/day of faecal matter (Friedler et al. 1996). Besides faecal matter a number of items are typically flushed through the toilet. These items are among others: toilet paper, tissues, condoms, cotton buds, nappies, plastic wrapping, razor blades, and sanitary protection items (Friedler et al. 1996).

A number of the items above are transported in the sewer system as gross solids. Also faecal matter can be transported in this mode, even though it most commonly disintegrates into fine faecal matter.

The most common approach is, however, to consider the concentration levels of wastewater in a dry weather flow downstream in the sewer system where the fluctuations from single dwellings does not influence the wastewater characteristics significantly. One person typically generates about 151 l/day (Danish conditions, see Henze et al. 1995), with a range between 80-380 l/day. Dependent on the wastewater type different concentration levels can be estimated as seen in table 2.3.

Parameter	Symbol	Unit	Wastewater type Concentrated	Moderate	Diluted
Suspended solids	SS	mg/l	450	300	190
Volatile suspended solids	VSS	mg/l	320	210	140
Biological oxygen demand	BOD_5	mg/l	350	250	150
Chemical oxygen demand	COD	mg/l	740	530	320
Total organic carbon	TOC	mg/l	250	180	110
Fats, oil, and grease	-	mg/l	100	70	40
Kjeldahl Nitrogen	N_{Kj}	mg/l	80	50	30
Total phosphorous	P_{tot}	mg/l	23	16	10
Lead	Pb	μg/l	80	65	30
Zinc	Zn	μg/l	300	200	130
Copper	Cu	μg/l	100	70	40
Coliform bacteria		no./l	10^9	10^9	10^9

Table 2.3: *Concentration levels for typical wastewater (from Henze et al. 1995).*

Phosphorus levels will typically be lower in catchments where detergents without phosphate is used.

2.4 In-sewer sediment characteristics

The complexity of the composition of the sediments and pollutants in the sewer system is high and varies dependent on the characteristics of the catchment and the sewer system. The characteristics of sewer sediments have been studied comprehensively (Ashley et al. 1999; Ashley and Crabtree 1992; Butler et al. 1992; Xanthopoulos and Augustin 1992; Verbanck 1990; Ristenpart 1995; Michelbach 1995; Chebbo and Bahoc 1992; Chebbo 1992).

Typical sewer sediments categories divided into four primary classes, not including gross solids, are given in table 2.4.

Sediment type	Description /where found	Wet density $10^3 kg/m^3$	% by granular particle size minimum–mean–maximum			Organic content %
			< 0.063	0.063–2.0	2.0–50	
A	coarse granular bed material widespread	1.72	1–6–30	3–61–87	3–33–90	7
C	mobile, fine grained found in slack zones, in isolation of overlaying type A	1.17	29–45–73	5–55–71	0	50
D	organic pipe wall slimes and zoogleal biofilms around mean flow level	1.21	17–32–52	1–62–83	1–6–20	61
E	fine grained mineral and organic material found in CSO storage tanks	1.46	1–22–80	1–69–85	4–9–80	22

Table 2.4: *Sewer sediment classes.*
(from Ashley et al. 1999).

The sediments composed of frictional noncohesive material may furthermore be characterized by their grain size distribution curve. This may, however, be misleading, as most sediments deposited in the sewers are more or less agglutinated or cemented and/or mixed with gross solids and cohesive sediments. Finally the sediment can be characterized by the settling velocity, which however is quite dependent on the method by which this settling velocity is determined (Lucas-Aiguier et al. 1997).

The sediment characteristics are often dependent on where in the sewer system they are located. E.g. in large flat interceptor sewers coarse sediment are found to accumulate (Ahyerre 1997). Figure 2.2 shows areas within which grain size curves are located.

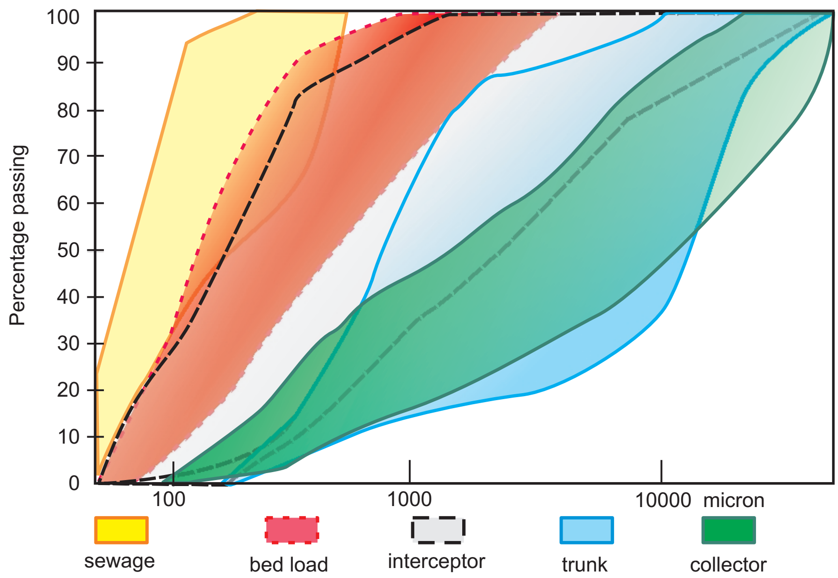


Figure 2.2: *Range for particle size distributions for different sediment sources.*
(reproduced from Ashley and Crabtree 1992).

Figure 2.2 gives rather broad bands for possible particle sizes. Chebbo (1992) investigated suspended solids for four different storm sewers and five combined sewers during rain events. Table 2.5 presents results for three different particle size fractions.

Storm sewer	$D_{10}(\mu m)$	$D_{50}(\mu m)$	$D_{90}(\mu m)$	$\% < 100(\mu m)$
Average	7.4	32.1	617	81
St. deviation	1.1	3.5	442	3.3
Combined sewer				
Average	6.78	34.1	331	75
St. deviation	3.25	6.4	112	5.5

Table 2.5: *Particle sizes for suspended solids during rain events.*
(from Chebbo 1992).

Coarse sediments (frictional materials) will typically have a density close to 2650 kg/m^3, which can be useful in determinating approximate settling velocities. Another useful characteristic is the bulk density of the sediments. Table 2.6 shows values for different sediment classes measure in Dundee (Ashley and Crabtree 1992). These findings are consistent with other studies.

Class	Average (10^3kg/m^3)	Max. (10^3kg/m^3)	Min. (10^3kg/m^3)	No. of -	St. deviation (10^3kg/m^3)
A	1.60	1.97	1.00	10	0.316
A/C	1.58	2.15	1.00	40	0.243
C/A	1.51	1.78	0.97	25	0.176
C	1.070	1.448	0.972	63	0.079

Table 2.6: *Bulk densities for different sediment classes.*
(from Ashley and Crabtree 1992).

Due to decomposition processes, agglutination, consolidation, etc. the characteristics of the sediments changes. The rheology of the sediments have been investigated e.g by Ristenpart (1995) showing increasing density and decreasing volatile solids content with time. This entails possible changes in shear strength of the deposits.

2.5 Correlation between pollutant parameters

The pollutants are closely connected with the particles for surface sediments (Gromaire-Metz et al. 1998) . High percentages (80 % on the average) of BOD, P_{tot}, and SS have been found to be attached to sediments (Hogland et al. 1984).

Heavy metals such as cadmium, copper, lead, and zinc are associated with the fine runoff particles. Computations of mass balances shows that between 50–80 % of the heavy metals discharged originates from washoff from roofs and streets (Boller 1997). The proportion of the heavy metals, which are particle bound lies between 72 % and 97 % percent for street runoff (Gromaire-Metz et al. 1998).

In the sewer system pollutant parameters are still closely related to the sediment particles. Chebbo (1992) states that 83–92 % of COD, 77–91 % of BOD_5, 78–82 % of N_{Kj}, 82–99 % of hydro carbons, and 79–99.98 % of Pb pollution is particle bound.

Besides pollutants connection with particles the pollutant parameters has been shown to be correlated (Onderdelinden and Timmer 1986).

Parameter 1	Parameter 2	Correlation coefficient
BOD	N_{Kj}	0.82
COD	N_{Kj}	0.87
COD	P_{tot}	0.95
COD	Pb	0.82
TSS	P_{tot}	0.95
TSS	COD	0.97

Table 2.7: *Examples of reached correlation between pollutants.*
(from Onderdelinden and Timmer 1986).

As seen in table 2.7 there is a rather high correlation between shown parameters. This supports the concept of using one or a few parameters such as COD or TSS as overall pollutant indicators.

As presented in this chapter the materials transported in the sewer system vary from everything between nappies and sand. The different materials does not behave in the same manner when transported with the flow. There is also a degree of interaction between the different classes of sediments. The complicated structure of the sewer sediments must be kept in mind when developing numerical models for sediment transport.

Figure 2.3: *Yet another exiting "thing" found in most sewer systems.*

CHAPTER 3

Sediment transport in sewers

The processes of sediment transport in a sewer system is complex. Numerous physical, chemical, and biological processes are going on at the same time and are interacting. The processes vary with the highly varying hydraulic conditions. This chapter aims at giving an overview of the main transport modes and possible empirical or semi-empirical models.

The understanding of the sediment transport phenomena has advanced considerably during the last decades. A considerable amount of experience has been transferred from sediment transport in rivers and estuaries (alluvial streams). The ultimate success to apply the theories from this field has not been obtained, mainly due to the differences between the two systems. In sewers the hydraulic conditions may change rapidly both in time and space which of course will change the conditions for sediment transport as well. Another important difference is the influence of the cross-sectional shape and cross-section changes in the sewer system. This induces turbulence and local possibilities for sedimentation. Together with the influence of the cross-section comes the limited availability of sediments for erosion. Finally, particle types, sizes, and distributions are relatively well definable in rivers, but in sewers a number of different types of particles occur, such as minerals, organic particles, etc. (see chapter 2). These particles does not necessarily behave in the same manner when eroded, transported, or deposited. They may interact and change due to compacting of deposits and chemical and biological processes.

The timescales of the different processes vary imposing subsequent problems when trying to describe the system. Atmospheric precipitation of particles during the rain event, washoff of sediments from the surface, build-up, rheology and erosion and transport of sediment through the gullies, transport of sediments during dry weather, build-up of deposits in the sewer during the dry weather period, transport and erosion of sediments during rain events in suspension or near the bed, including first flush, deposition of sediments at the decline of the rain event, etc. does not have simply definable timescales as the processes depends on both the hydraulic conditions, time, and the composition of the sediments themselves.

3.1 The physical system

As already indicated a number of different phenomena goes on in the course of transporting sediments in a sewer system. Some of the phenomena and terms are illustrated on figure 3.1.

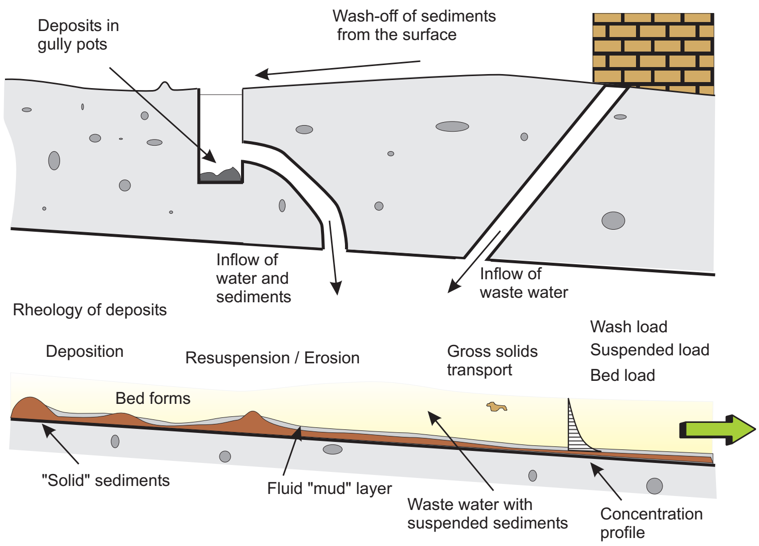


Figure 3.1: *Sediment transport phenomena.*

In storm sewers the only input of sediments into the system should be surface sediments resulting in significantly lower concentrations than in combined sewers. Often there seem to be a certain percentage of illicit sanitary connections to the storm sewers. A Canadian study showed between 5–13 % of illicit connections. The influence on the sewer system has been illustrated by Saget et al. (1998) and the comparison of storm sewers, storm sewers with illicit connections, and combined sewers during wet weather is shown in table 3.1. Comparison of storm

Constituent	Storm sewer	Polluted storm sewer	Combined sewer
	Concentration [mg/l]		
TSS	160–460	240–400	240–670
COD	80–320	180–470	350–570
BOD	13–130	35–120	90–270

Table 3.1: *Comparison of storm sewers and combined sewers.*
(from Saget et al. (1998)).

sewers and combined sewers can give an indication of how much the contribution from the surface is. Table 3.1 indicates that washoff from the catchment surface gives a relatively large contribution to suspended solids but not to the same degree organic material. The contribution to suspended solids is of the order 10–40 % depending on the characteristics of the sewer system.

The sediments on the surface, deposits during dry weather periods on roofs and streets. A continuous process of export and import of material to the specific surface area goes on. Sediments are transported to and from the area by wind and traffic depositing in sheltered locations with low wind velocities and less traffic. Approximately 50 % of the solids ends up in the gutter.

The deposited sediments on the catchment surface undergoes changes due to abrasion and biological and chemical processes. Materials are disintegrating into smaller particles and organic material decomposes. As the deposits are wetted, e.g. by falling dew, a cementation may take place. Other processes can bind particles together such as agglutination by oils.

At the initiation of a rain event the sediments deposited on the surface are wetted along with the surface itself. Already, before rainwater actually starts flowing towards the sewer inlets sediment particles on the surface are moved by the impact of individual rain drops. When runoff is established sediment particles may be transported with the flow either saltating along the bottom of the flow or fully suspended in the flow. Potential sediment movement depends on rain intensity and particle characteristics. It has been shown that all sediment deposits on the surface cannot be expected to be transported during rainfall, between 3 and 10 times the amount of solids are available compared with the amounts which are being washed off. Fine particles ($d_{50} < 100 \mu m$) are easily washed off. Street cleaning practices will typically have a significant influence on the washoff loads during rain events. Street cleaning can reduce the stormwater load of TSS from the surface with 30–50 % (Ellis 1986).

Heavy metals are associated with the fine particles on the surface (see 2.5) and it is these particles which are flushed off the surface at the beginning of the rain. This expectable flush effect of heavy metals is of course not critical as environmental impacts from heavy metals are of accumulative nature. It is though not irrelevant in combination with estimation of the load of heavy metals discharged from CSO. Computations of mass balances shows that between 50–80 % of the heavy metals discharged from the sewer system originates from roofs and streets (Boller 1997).

Litter from the streets are also moved towards the sewer inlets. At this point these gross solids are retained on the surface by the drain gratings, given there is one. In most countries roadside catchpits are connected to the sewer pipe. There are two types of these catchpits or gullies; with retention and without. The gully pot has two functions. It serves as an odour seal and as a trap for coarse sediments. Following a storm the gully pot is filled with water and sediments. The hydraulic conditions quickly becomes quiecent and sediment particles starts to settle due to gravity. The settling process can take days depending on particle size and specific gravity of the particle. During dry weather periods biological and chemical processes may lead to increased concentration levels of COD, BOD, ammonium, and nitrate in the liquid (Butler et al. 1995). Measurements of bulk densities of the deposited sediments in the gully pot has shown a range from 100–1700 kg/m^3. Build-up of sediments in the gully pot also takes place during dry weather periods due to transport of sediments to the inlet grating by wind and traffic (Morrison et al. 1995).

During wet weather the inflowing water impacts vertically into the gully pot generating a lot of turbulence. Most of the surface sediments are transported through the gully pot and some sediments are re-suspended from the gully pot and exported into the sewer pipe. Some of the coarser particles may be trapped in the gully pot. The trapping efficiency depends on the flow through the gully pot and the particle size. As most of the sediments deposited in the gully pot is relatively coarse an armouring of the deposits surface can take place. This happens when the finer fractions of the sediments in the upper part of the deposits are washed out of the gully pot and the remaining deposits are trapped by a layer of larger particles. The gullies can also have a reduced capacity for transporting runoff water due to clogging by litter, branches etc.

Contrary to the transport process on the surface the domestic input of pollutants to the sewer system is continuous. In dry weather periods the flow in combined sewers has a distinct diurnal cycle, at least in predominantly residential areas. The size of the dry weather flow is mainly based on how many people are living upstream for the location where the flow is monitored. Often this amount of water, app. 150 l/PE/day on the average in Denmark, cannot explain the total flow. Additional flow originate from infiltration due to a groundwater level higher than the pipes and/or street cleaning water, etc. There will be a change in the flow pattern on working days compared with weekends and holidays. Figure 3.2 shows an example of a diurnal cycle of wastewater. The changing levels of the

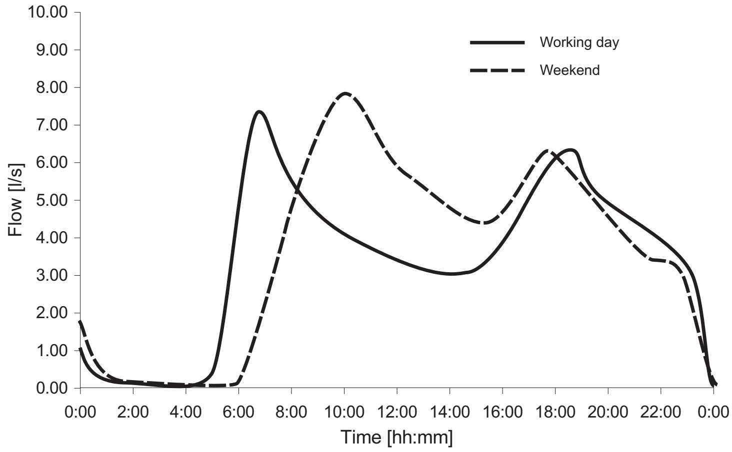


Figure 3.2: *Diurnal flow pattern for the small village of Frejlev, Denmark. The village holds app. 2000 inhabitants and there is no infiltration.*

flow also results in changes in sediment movements. Some particles transported in the flow are not sediments. They are either neutrally buoyant or so small that they will not deposit during their transport from the source to the WWTP as their settling time in quiecent water is longer than the transport time in the sewer. The fine particle fraction considered as wash-load has a particle size less than app. 1μm. They are susceptible to flocculation and may thus change to sediments and possibly settle out.

Sediments suspended in the flow are kept in suspension by the eddies of fluid turbulence and are moved downstream with the flow by drag and shear induced lift forces. The particle can deposit if its settling velocity is high enough. Turbulence and shear stress are introduced by changing cross-section, flow direction and roughness conditions.

During dry weather the cross-sectional flow will seldomly be fully mixed. The concentrations of SS will be less in the upper part of the flow than closer to the pipe bottom or sediment bed. This results in a concentration profile with a considerably higher concentration in the lower part of the flow.

Some of the particles moving close to the bottom are interacting with the sediment bed by saltation, sliding or rolling. Particles which impacts with the sediment bed from time to time are transported in the fraction of the mass transport termed bed-load. The proportion in mass of the bed-load compared with the total load may vary, but can e.g. amount to 12 % (Ashley et al. 1992). Even though the concentration level is much higher in this near bed zone flow velocities are smaller than the mean flow velocity and the extent of the near bed zone is limited. Some evidence show that the particle sizes found in the near bed zone should be able to deposit. When this is not happening it may be due to hindered settling, where the exact reasons are not yet fully understood. The sediments transported close to the bottom are also named near bed solids and the near bed zone is also termed a *fluid mud layer* or *dense under current*. The different terms are considered to cover the same basic physical processes with some deviations due to the specific flow conditions, sediment characteristics and sewer design.

The transport processes in the near bed zone in sewers thus differs from the bed-load as perceived in alluvieal stream hydraulics, where particles only moves by saltation, sliding or rolling(Graf 1984). Dependent on flow conditions and particle sizes ripples, dunes, plane bed, or antidunes may form. Ripples and dunes are formed when particles are traveling up a slope reaching the summit and falls down into a sheltered zone from the fluid action. It has been shown that movement of frictional materials in sewers also results in different bed forms.(Nalluri and El-Zaemey 1993; Perrusquía 1992; Kleijwegt 1992; Mark 1992). It is not completely clear if the same conditions are valid for smaller combined sewers with a mixture of sediments which are considerably different from fine sands. Bed forms in combined sewers are also observed at cross-section changes, locations with local disturbances of the flow due to partly clogging by e.g. gross solids, etc.

During dry weather there is often a general tendency to build-up or accumulation of sediments in the combined sewer system. This means that the system is not completely self-cleansing. This may be due to changed conditions from those assumed during design. Flow conditions may have changed and certainly sewer pipes are not placed on a straight slope as indicated on the design plans and may subsequently move when the surrounding soil is consolidating creating local depressions. Areas with low flow velocities and thus low shear stress are potential locations for build-up of sediment deposits. There is a limit to how large deposits can build-up. As the size of the cross-section is getting smaller flow velocities will increase and an equilibrium will be reached. Investigations of sediment deposits shows that a layer of fine sediments will build-up on top of the more coarse sediments which are moved only during storm events (Ahyerre 1997).

The sediment deposits undergoes changes due to compacting, cementation, agglutination, and biological processes. The bulk density and total solids content increases with the age of the deposit, whereas the amount of volatile solids, COD, and BOD decreases (Ristenpart 1995).

A continuous build-up of biofilm also takes place especially growing in the area around the mean water level in the sewer during dry weather. Biofilm may also form on the surface of sediment deposits potentially altering the conditions for resuspension (Vollertsen 1998).

The processes taking place during a rainfall event are quite complex. Sediments from the surface and the gullies enters the sewer pipe together with the rainfall runoff. Here they are mixed with the dry weather flow. As the flow increases the dense undercurrent is overtaken by the wave of stormwater which quickly mixes the near bed zone into the whole flow section creating a first foul flush. This is illustrated on figure 3.3. In the course of the rain event the top layer

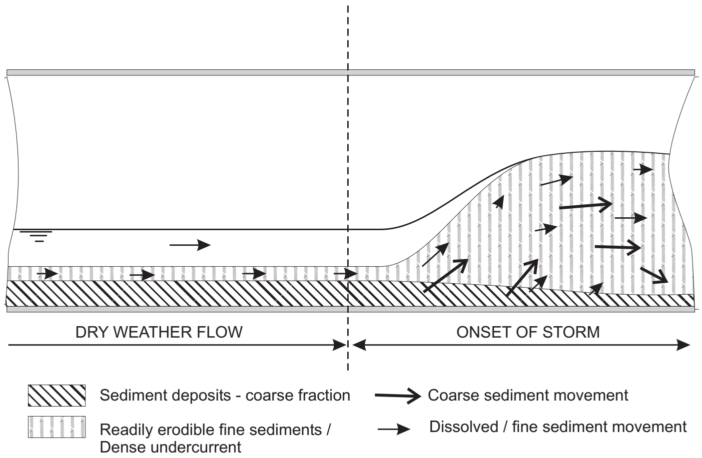


Figure 3.3: *Illustration of resuspension of the near bed zone.*
(from Ashley et al. 1992).

of sediments deposited during the dry weather period is eroded. Dependent on the discharges occurring during the event coarser sediments deposited during preceeding rain events can be eroded and transported downstream. In this way erosion and deposition takes place simultaneously at different locations. It is even possible to imagine deposition of coarser material taking place in the same location and at the same time as finer material is eroded from the sediment bed.

During the rain event tearing off of biofilm occur. This may contribute significantly to the COD load. Michelbach and Wöhrle (1992) showed that the biofilm growth reaches a maximum thickness after app. 15 days and during a storm event up till 30 g/m^2 of old biofilm was eroded. The maximum thickness was recorded as $90 - 100$ g/m^2. It has subsequently been shown that the eroded biofilm erodes into settable solids (Michelbach 1995).

At the beginning of a new dry weather period new sediment deposits may have formed and existing has been remoulded. The aging of the sediment deposits reduces the pollution content regarding BOD_5 and COD to less than one fifth of the fresh deposits (Ristenpart 1995). At the same time the pollution level of solutes in the interstitial water rises. These changes are caused by bio-degradation processes.

In the following paragraphs the physical phenomena described above are quantified by empirical or semi-empirical models. Classical deterministic sediment transport models are not dealt with in this chapter, but will be treated in chapter 6.

3.1.1 First flush in sewer systems

The resulting pollutograph at the intercepting sewer from a catchment often shows distinct characteristics. If the maximum concentration is reached before the maximum discharge in a rain event it can be termed as a flush effect. If the main part of the load is transported during the first part of the storm event the event is sometimes termed "a first flush event". The definition of first flush is ambiguous. This paragraph aims at reviewing different first flush definitions and reported extent of the phenomenon. Typically, the procedure to evaluate whether a rain event has resulted in a first flush of pollutants is to plot the normalized accumulated load M_{acc} as a function of the normalized accumulated discharged volume V_{acc} according equation 3.1 (Bertrand-Krajewski et al. 1998).

$$M_{acc}(V_{acc}) = \frac{\sum_{i=1}^{j} C_i Q_i \Delta t_i}{\sum_{i=1}^{N} C_i Q_i \Delta t_i} = f\left(\frac{\sum_{i=1}^{j} Q_i \Delta t_i}{\sum_{i=1}^{N} Q_i \Delta t_i}\right) \tag{3.1}$$

where

N		:	Total number of measurements.
C_i	[mg/l]	:	Concentration.
Q_i	[l/s]	:	Discharge.
Δt_i		:	Time interval.

This results in a curve of the type shown on figure 3.4. The storm event can be said to exhibit a significant first flush effect if the curve lies above the bisector line. The adequate distance above the bisector depends on which definition for first flush that is applied. Some measured curves can also happen to be located below the bisector. This is the case when the constituent is diluted during the event, which e.g. often can be the case for N_4^+. Based on a review of earlier

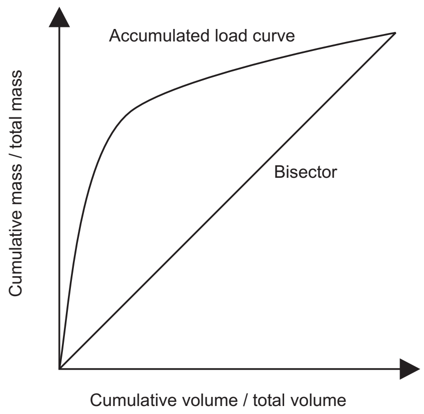


Figure 3.4: *Illustration of accumulated load curve.*

suggested first flush criteria a new criterion has been suggested (Saget et al. 1996; Bertrand-Krajewski et al. 1998). The $M(V)$ curve is numerically fitted to a simple power function of the type shown in equation 3.2.

$$f(x) = x^b \tag{3.2}$$

If the numerical coefficient b lies within the range $0 < b < 0.185$ then the event is termed as a first flush event. This definition corresponds to a first flush where at least 80% of the pollutant mass is transported in the first 30% of the volume. Several other pairs of values have been suggested and there is no specific argument for the definition above. A quite compliant definition is given by Stotz and Krauth (1984) expressed as: "The maximum pollution load in kg/min appears before the maximum water flow in m^3/min and the pollution load decreases at a more rapid rate than the water flow".

Another definition has proposed that the curve needs a deflection larger than 20 % above the bisector at any point in order to be a first flush event (Geiger 1984). This is a more restrictive requirement than the one proposed by Bertrand-Krajewski. A third definition has been put forward by Gupta and Saul (1996). In this case two curves are plotted; the cumulative load curve and the cumulative flow curve plotted as a function of the cumulative time of the event. The first flush is defined as the mass at the location where these two curves show the largest divergence. This definition is illustrated in figure 3.5. The advantage of the definition put forward by Gupta and Saul is that design of a retention basin able to catch a certain amount of the first flush load becomes easy and a first flush duration tff comes with the definition, which can be convenient in

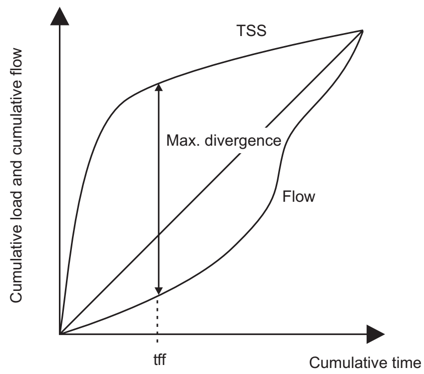


Figure 3.5: *Illustration of alternative first flush definition.*

connection with control.

A regression model was established for the determination of the first flush load for a number of monitored events from England (Bertrand-Krajewski et al. 1998). The model is based on the storm duration, maximum rainfall intensity, and ADWP yielded correlation coefficients (R^2) between 0.52 and 0.71. The model indicated strong site dependence.

Application of the first flush definition of Saget defined by a b coefficient below 0.185 on 12 French catchment resulted in only one first flush event out of 197 concluding that first flush is a very rare phenomenon (Bertrand-Krajewski et al. 1998). Correlation between first flush and different parameters such as impervious area, time of concentration, average slope, ADWP, rain depth, and 5min max. rain intensity were investigated without finding any significant correlation (Saget et al. 1996).

3.2 Build-up on the catchment surface

In order to model washoff of sediments from catchment surfaces an estimate of the accumulated mass of sediments on the surface at the initiation of the rain event is needed. For continuous modelling it becomes necessary to model the build-up process as a function of time.

The build-up process depends on the ADWP, the residual mass deposited on the surface after the last event, street cleaning practices, traffic intensity and traffic speed, vegetation, urbanization and surface characteristics. Two types of empirical models are used for build-up, either a linear or exponential function. The linear build-up is stopped at a maximum value m_{max}, see equation 3.3.

$$\begin{aligned} m_a &= \alpha_{acc}\, t \quad &&\text{, when } m_a < m_{max} \\ m_a &= m_{max} \quad &&\text{, when } m_a \geq m_{max} \end{aligned} \tag{3.3}$$

where

α_{acc}	[kg/ha/day]	:	Rate of accumulation.
t	[day]	:	Period after last rain event.
m_a	[kg/ha]	:	Mass deposited on the surface after time t.

A power function for the build-up model has also been suggested. This model can be seen in equation 3.4, where α_{acc2} is a numerical coefficient.

$$\begin{aligned} m_a &= \alpha_{acc}\, t^{\alpha_{acc2}} \quad &&\text{, when } m_a < m_{max} \\ m_a &= m_{max} \quad &&\text{, when } m_a \geq m_{max} \end{aligned} \tag{3.4}$$

It appears natural that there will be a certain degree of sediment decay and removal on the surface at the same time as the accumulation. This is the argument for reaching a maximum limit for accumulated mass on the surface. The removal rate α_{rem} depends on wind velocity, traffic intensity, etc. An exponential build-up function is formulated in equation 3.5.

$$\frac{dm_a(t)}{dt} = \alpha_{acc} - \alpha_{rem} m_a(t) \tag{3.5}$$

where

α_{acc}	[kg/ha/day]	:	Rate of accumulation.
α_{rem}	[day^{-1}]	:	Removal rate.
t	[day]	:	Period after last rain event.
$m_a(t)$	[kg/ha]	:	Mass deposited on the surface after time t.

Another exponential build-up function on nondifferential form can be written as equation 3.6.

$$m_a = m_{max}(1 - e^{-\alpha_{rem} t}) \tag{3.6}$$

Finally, an asymptotic model of the form shown in equation 3.7 has been suggested.

$$m_a = \frac{m_{max} t}{\alpha_{acc} + t} \tag{3.7}$$

Figure 3.6 shows the course of the various build-up functions, but of course the curves can change according to chosen accumulation rates etc. In order to apply

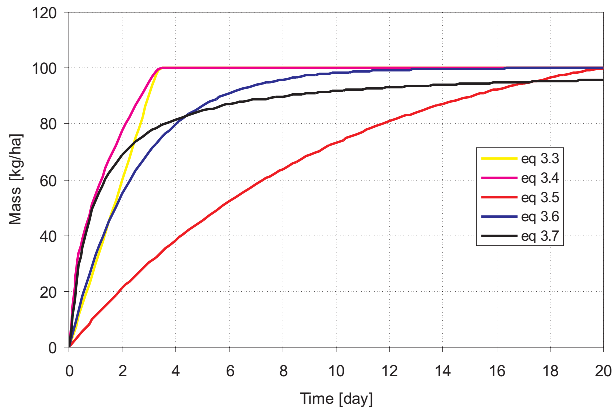


Figure 3.6: *Illustration of empirical build-up models.*

the models it is in principle necessary to calibrate them with local data. As this is seldomly feasible the functions are normally fitted together with the washoff model with which they are used. The model shown in equation 3.5 is the one most frequently used and it is e.g. implemented in the SWMM model. There is no especially preferred model and the precision of the model mainly depends on the quality of the calibration to local conditions.

The period it takes before an equilibrium between accumulation and removal of sediments are reached may range between 5 to 20 days, e.g. 5 days reported for measurements carried out in London (Ellis 1986). Street cleaning has a definite influence on the average build-up rate, where the build-up may be reduced with up till 30 % for frequent street cleaning (3-5 days/week). A number of accumulation rates are reported. Mance and Harman (1978) estimates a value of 380 kg/ha/year, but the range of values are within 95–3200 kg/ha/year according to the comparison in Ashley et al. (1999).

3.3 Washoff from the catchment surface

The washoff process depends on rainfall intensity, duration, depth, sediment characteristics, surface conditions, area, slope of the surface, etc. As the detailed process is very complicated, as explained in paragraph 3.1, washoff is not attempted modelled by a detailed physically based model. This is due to both the complexity and the stochastic nature of the process. Therefore, statistical/empirical models are applied.

One of the most commonly used models, e.g. implemented in the SWMM model, is formulated as

$$\frac{dm_a(t)}{dt} = -\alpha_w \; i(t) \; m_a(t) \tag{3.8}$$

where

α_w	[mm^{-1}]	:	Washoff coefficient.
$i(t)$	[mm/h]	:	Rainfall intensity at time t.
m_a	[kg/ha]	:	Mass left on the surface at time t.

The washoff coefficient used in SWMM is $\alpha_w = 0.18\text{mm}^{-1}$, but of course this coefficient should be calibrated with measurements. Equation 3.8 can also be supplemented with an exponent on the rainfall intensity.

Another model still related to the rainfall intensity, but also dependent on shear stress levels for erosion and deposition, is

$$\frac{dm_a(t)}{dt} = a_i i(t)^{1.5} + a_e(\tau_0 - \tau_{ce}) + a_d(\tau_{cd} - \tau_0) - \frac{m_a(t) q_b}{K q_b + S_m} \tag{3.9}$$

where

a_i, a_e, a_d		:	Price-Mance coefficients.
$i(t)$	[mm/h]	:	Rainfall intensity at time t.
τ_0	$[N/m^2]$	:	Mean shear stress.
τ_{ce}	$[N/m^2]$	:	Critical shear stress for erosion.
τ_{cd}	$[N/m^2]$	:	Critical shear stress for deposition.
q_b	[mm/h]	:	Specific surface runoff.
S_m	[mm]	:	Soil depression storage.
K		:	Linear reservoir parameter.

The model in equation 3.9 poses some difficulties when procuring suitable data for calibration. If entities such as τ_0, τ_{cd} and τ_{ce} are considered as simple numerical coefficients it is difficult to justify using this equation instead of another. If on the other hand measurements determining τ_0, τ_{cd} and τ_{ce} is required then the applicability of this model decreases. It has though also been suggested to leave out the second and third term of equation 3.9.

A model derived based on field data has been proposed by Servat (cited in Bertrand-Krajewski et al. 1993)

$$m_e = \alpha_w m_a^{c1} \; I_{max5}^{c2} \; V^{c3} \tag{3.10}$$

where

α_w	[mm^{-1}]	:	Washoff coefficient.
m_e	[kg]	:	Washed off mass.
m_a	[kg]	:	Accumulated mass at the initiation of the rain.
$I5_{max}$	[mm/h]	:	Maximum rainfall intensity during a time step of five minutes.
V	[m^3]	:	Runoff volume.
$c1, c2, c3$	-	:	Numerical coefficients.

As seen, this model in (3.10) is not time dependent and thus yields the total mass washed off during the rain event.

Some models which avoid the direct dependency on rainfall intensity also exists utilizing the surface runoff instead. One of these models is the NPS (Non Point Source) model

$$\begin{array}{ll} m_e(t) = \alpha_w\ Q_s^{c1} & ,\text{when}\ \ m_e(t) \geq m_a(t) \\ m_e(t) = m_a(t) & ,\text{when}\ \ m_e(t) > m_a(t) \end{array} \tag{3.11}$$

where

$m_e(t)$	[kg/ha]	:	Washed off mass at time t.
$m_a(t)$	[kg/ha]	:	Accumulated mass at time t.
Q_s	[mm]	:	Surface runoff on impervious area.
$c1$	-	:	Numerical coefficient.

In general, the rainfall intensity and subsequently the runoff rate mainly control the washoff process. Common for all the models is that they require calibration with measurements and if done carefully the quality of the models are all in the same range. The accumulation models and washoff models are often used as a part of a bigger model, as it will be presented in chapter 7 through 6. If the surface washoff is of relatively small importance to the total sediment transport then it becomes less important which washoff model is chosen.

3.4 Sediment transport in sewer pipes

The sediment transport through a given cross-section of a sewer at a specific point in time can generally be formulated as an integral over the cross-section of the flow, see equation 3.12.

$$S = \int_A u(y,x) \cdot c(y,x)\ dA \tag{3.12}$$

where u is the longitudinal flow velocity and c the concentration and x the horizontal coordinate perpendicular to the flow direction. Presuming that two-dimensional variations can be neglected, entails that the sediment transport (S) through a cross-section without a deposited bed can be calculated in one dimension as

$$S = \int_{y=0}^{y=Y} U(y) \cdot C(y) \cdot s_y(y)\ dy \simeq Q\ \bar{C} \tag{3.13}$$

where U is the mean velocity at height y, C the mean concentration at height y, s_y is the inside width of the pipe y above the bottom of the cross-section and $\bar{C}$ is the mean concentration in the cross-section.

3.4.1 Concentration profiles

However, as described in paragraph 3.1, it is not generally possible to assume a fully mixed cross-sectional flow. There will under most circumstances exists a concentration profile with indication of high concentration levels in the near bed zone.

Concentration profiles has been measured frequently in different sewers, e.g. (Ristenpart 1995). Most measurements are carried out in large diameter sewers. To illustrate the concentration profile measurements from Schlütter and Schaarup-Jensen (1998) are seen in figure 3.7 As seen on figure 3.7, a Rouse

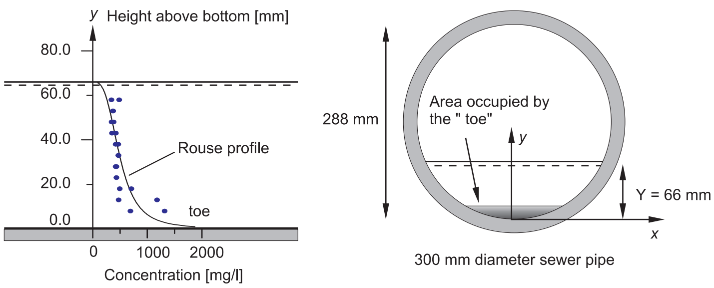


Figure 3.7: *Measured concentration profile during DWF in a 300mm combined sewer.*

profile have been fitted to the data even though it can be questioned whether that profile is valid for sewage flow in pipes (Ashley and Verbanck 1996). At the right hand side of figure 3.7 the extent of the area with higher concentrations (toe) is illustrated. The Rouse profile can be formulated as:

$$C(y) = C_b \left(\frac{Y-y}{y} \, \frac{b}{Y-b} \right)^Z \tag{3.14}$$

where

$C(y)$	[mg/l]	:	Concentration at level y.
Y	[m]	:	Total water depth ($Y = 0.066$m).
C_b, b	[mg/l], [m]	:	Constants related to the concentration at the bottom.
Z		:	Rouse number, $Z = W_s/(U_f \cdot \kappa)$, W_s being the particle settling velocity and U_f the friction velocity $U_f = \sqrt{gYI}$ and $\kappa = 0.4$ the Karman constant.

The fit of the Rouse profile to the measured data yields the following constants: $C_b = 590$mg/l and $b = 0.0238$m. The Rouse profile does not fit especially well ($R^2 = 0.53$), which has also been the case for other measurements in sewers. Another model of the concentration profile (reported by Ashley and Verbanck 1996) is divided into an outer region of the flow and an inner region close to the bed

$$\begin{aligned} \frac{C(y)}{C_a^*} &= e^{\eta\left(1-\frac{y}{a^*}\right)} \quad &&, \text{for} \quad y > \tfrac{1}{4}Y \\ \frac{C(y)}{C_a^*} &= \left(\tfrac{y}{a^*}\right)^{-\eta} \quad &&, \text{for} \quad y \leq \tfrac{1}{4}Y \end{aligned} \tag{3.15}$$

where

C_a^*		:	Volumetric solid concentration at $y = a^*$.
a^*	[m]	:	Depth of inner suspension region.
η		:	Sedimentation parameter.

3.4.2 Deposition and erosion criteria

The most frequently used criteria for deposition and erosion is to consider the level of the bed shear stress and compare this value with a threshold value for deposition (τ_{cd}) and for erosion (τ_{ce}). Figure 3.8 illustrates the relation between the transport modes and grain size with attaining settling velocity and the shear velocity u_* which is found from the bed shear stress by

$$\tau_0 = \rho_w \cdot u_*^2 \tag{3.16}$$

where ρ_w is the density of water and τ_0 the bed shear stress. The shear velocity is a measure of the intensity of the turbulent fluctuations. A number of criteria

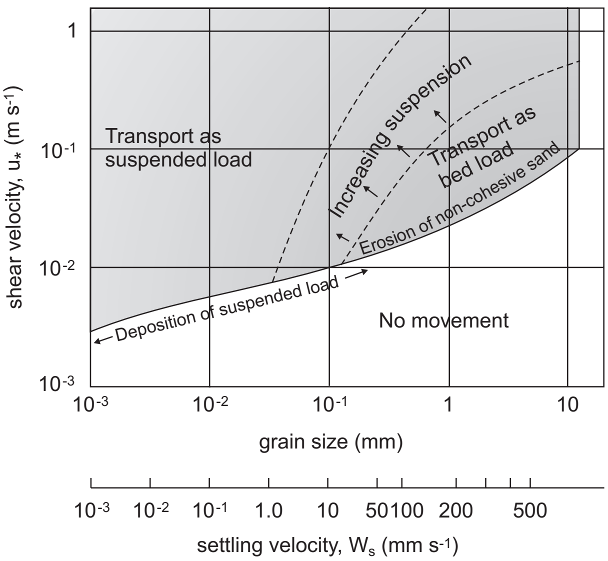


Figure 3.8: *Relation between grain size, flow velocity and transport mode.*

exists for incipient motion of the sediment particles or critical bed shear stress for erosion τ_{ce}. The most well-known is Shields criteria (Graf 1984), which is developed for a plane sediment bed with uniform sediment particles. Adapted for a distributed grain size curve the criteria can for fully turbulent flow ($Re \geq 400$) be formulated as (Andersen et al. 1984)

$$\tau_{ce} = 0.06 \cdot \rho_w g \Delta d_{50} \tag{3.17}$$

where $\Delta = \rho_s/\rho_w - 1$. Both more complex as well as simpler criteria has been suggested, a simple criteria (Graf 1984) which does not depend on the flow but only on the mean grain size ($\bar{d}$) reads

$$\tau_{ce} = 166 \cdot \bar{d} \tag{3.18}$$

Equation 3.17 and 3.18 determines a critical level where sediments starts to be eroded. The induced average shear stress by the flow can be calculated by

$$\tau_0 = \rho_w \ g \ R_h \ I \tag{3.19}$$

where

R_h	[m]	:	Hydraulic radius.
I		:	The gradient of the energy line.

Equation 3.19 is valid for pipes with a uniform roughness distribution along the cross-section of the inner pipe wall. If a sediment bed is present it is necessary to adjust for the different roughness on the wall and the bed. An representative value for the overall average shear stress can be found by (Perrusquia et al. 1995)

$$\tau_0 = \frac{\tau_{wall} P_{wall} + \tau_{bed} P_{bed}}{P} \tag{3.20}$$

where P denotes the wetted perimeter.

The applicability of the Shields criteria to real sewer sediment can be questioned. Kleijwegt et al. (1990) concludes that it is not applicable as the particles tends to move at a significantly lower level of the bed shear stress.

The actual value for τ_{ce} and τ_{cd} depends on the present hydraulic conditions and sediment characteristics. A comparative study of two sewers with a diameter of 150 cm has been presented by Ristenpart and Uhl (1993). During dry weather conditions τ_{ce} was estimated to fall within the range $0.44 - 1.02$ N/m^2 and the critical bed shear stress for deposition τ_{cd} within the range $0.04 - 0.67$ N/m^2. An average value for rain events resulted in $\tau_{ce} = 1.67$ N/m^2, having a range between $0.38 - 2.9$ N/m^2. This study also concluded that the bed shear stress is the most appropriate entity for estimation of incipient motion compared with a critical flow velocity.

The near bed fluid sediment layer reported by Ashley and Verbanck (1996) has been observed to be mixed with the suspended load at bed shear stress levels of app. 1 N/m^2.

3.4.3 Wash load

The wash load mainly consists of fine particle which are transported with the flow. Some simple models for the total load may include this fraction more or less explicitly. However, dissolved substances and wash-load can successfully (see e.g. Garsdal et al. 1995) be modelled by the advection-dispersion transport equation (Vestergaard 1989)

$$\frac{\partial(AC)}{\partial t} + \frac{\partial(QC)}{\partial x} - \frac{\partial}{\partial x}\left(D \cdot A \, \frac{\partial C}{\partial x}\right) = m \tag{3.21}$$

where

A	[m^2]	:	Cross section area.
D	[m^2/s]	:	Longitudinal dispersion coefficient.
m	[g/m/s]	:	Source/sink term.
x	[m]	:	Space coordinate.
t	[s]	:	Time.

The application of this model can be seen in chapter 6.

3.4.4 Suspended load and bed load

A number of sediment transport model have been proposed based on deterministic models developed for alluvial hydraulics. Furthermore, models which are empirical or semi-empirical has been developed for both suspended load and bed-load transport. These are all based on laboratory studies using artificial sediments (mostly sands) and steady hydraulic conditions. This type of model has so far proven limited applicability for modelling of combined sewers, whereas storm sewers are more liable to be modelled successfully. Most of these models study the sediment transport at the limiting condition for erosion/deposition, see (May 1995; Nalluri and Alvarez 1992; Nalluri and El-Zaemey 1993; Perrusquía 1993, etc.).

A model which has been applied on measurements from Brussels main trunk sewer is (Verbanck 1996)

$$C_V = \frac{1}{5.16} \frac{u_{*,b}^3}{g \Delta W_S R_h} \tag{3.22}$$

where

C_V		:	Volumetric concentration $C = \rho_s \cdot C_V$.
$u_{*,b}$		:	Shear velocity at the bed (see equation 3.23).
W_S	[m/s]	:	Effective settling velocity $W_S = \sum_i (p_i W_i)$.
R_h	[m]	:	Hydraulic radius.

The average shear velocity at the bed is determined as

$$u_{*,b} = \frac{U_m}{\frac{1}{\kappa} \ln \left(\frac{12\ h}{k_s} \right)} \tag{3.23}$$

where

κ		:	The von Karman constant ($\kappa \cong 0.4$).
h		:	The total flow depth.
U_m	[m/s]	:	Mean flow velocity.
k_s		:	Nikuradse equivalent sand roughness.

A different approach is to model the total suspended transport by a linear regression model. This has been attempted by Coghlan et al. (1996) applying one model for dry weather conditions and one for storm events, calculating the total load from one event. A dry weather model also used with good results at other sites (e.g. Schlütter and Schaarup-Jensen 1998) is

$$m = \alpha_1 \cdot Q^{\alpha_2} \tag{3.24}$$

where α_1 and α_2 are numerical coefficients. A model applied for storm events is dependent on the velocity (U), the time since the start of the event (T_{sss}), and the antecedent dry weather period $ADWP$

$$m = \alpha_1 \cdot U + \alpha_2 \cdot T_{sss} + \alpha_3 \cdot ADWP + \alpha_4 \tag{3.25}$$

Modelling by these regression models avoids the need for knowledge about the sediment particle sizes, which are difficult to measure for each sample and thus frequently is used as "numerical coefficient" in other models.

When transport models are implemented numerically it gives some additional possibilities. An often used concept is to state two critical conditions for deposition and erosion respectively. This is done in the Velikanov model, which is based on turbulent energy analysis. The two limit sediment concentrations C_{min} and C_{max} are

$$\begin{aligned} C_{min} &= \eta_{min} \cdot \tfrac{\rho_s \cdot \rho_m}{\rho_s - \rho} \cdot \tfrac{U}{w_s} \cdot I_0 \\ C_{max} &= \eta_{max} \cdot \tfrac{\rho_s \cdot \rho_m}{\rho_s - \rho} \cdot \tfrac{U}{w_s} \cdot I_0 \end{aligned} \tag{3.26}$$

where

η_{min}, η_{max}		:	Efficiency coefficients.
ρ_m	[kg/m^3]	:	Density of sand + water mixture.
U	[m/s]	:	Mean flow velocity.
I_0		:	Slope of the bottom of the sewer pipe.

If the concentration is lower than C_{min} erosion takes place. Within the two limits neither erosion or deposition goes on and if the concentration is larger than C_{max} then deposition takes place.

Instead of limit concentrations limit velocities or shear stress may be applied. During the deposition mode a possible rate of deposition can be determined using the model (Bertrand-Krajewski 1992)

$$\begin{array}{ll} \frac{dm(t)}{dt} = \alpha_{dep} \cdot (\alpha_{sc} - m(t)) & , \quad m(t) < \alpha_{sc} \\ \frac{dm(t)}{dt} = 0 & , \quad m(t) \geq \alpha_{sc} \end{array} \tag{3.27}$$

where α_{dep} is the deposition rate and α_{sc} the storage capacity of the sewer system.

The erosion develops mainly as a function of the flow rate ($Q(t)$), leading to the differential equation 3.28.

$$\frac{dm(t)}{dt} = -\alpha_{ero} \cdot m(t) \cdot Q(t) \tag{3.28}$$

Some of the models stated in the paragraph 3.2 to 3.4.4 forms part of numerical models presented later in this thesis, where more complete descriptions may be given.

CHAPTER 4

Data basis for the case study "Le Marais"

All numerical models except for *entirely* physically based models, needs measured data for calibration and verification. This is also the case for all the models presented in this thesis. Even though some models are termed physically based they often include formulations which are empirically or statistically based and thus needs to be calibrated anyhow. This is the case for *all* numerical water quality models for combined sewer systems.

This chapter renders general thoughts on measuring data, calibration of models and presents the case study site used for evaluating the models in this thesis.

Performing measuring campaigns in sewer systems usually proves to be difficult. It is though necessary to carry out measurements in order to depict the characteristics of the specific catchment. The level of detail and the density of measurements must be able to resolve the phenomena which are going to be modelled by the water quality model. Furthermore, sufficient parameters needs to be monitored to make the model calibration feasible. Given that calibration parameters are site and event specific and independent it is necessary to measure at least one more entity than the number of calibration parameters in the model. It is evident that the quality of the data influences the quality of the modelling. If calibration data is of poor quality the precision of the model will be of poor quality as well.

The case study site is presented in the last part of this chapter. First of all the catchment itself is described and subsequently the data used for calibration of the models are scrutinized. The measuring campaigns has been carried out very thoroughly by the CERGRENE-LABAM laboratory of L'Ecole Nationale des Ponts et Chausees.

4.1 Measuring techniques and precision

The aim of modelling sediment transport in combined sewer systems is ultimately to be able to assess the impact on receiving waters after discharges of pollutants during storm events and dertermine the load on the WWTP. Geiger (1984) states that "it was realized in the early 70ties that realistic conclusions concerning receiving water pollution cannot be derived from rainfall-runoff measurements or simulations of singular events". Numerous measurements are thus imperative if the complete range of extreme events must be covered by measuring. By numerical modelling it is possible to have indications of the transport during all types of rain events, although conclusions based on extrapolation should be cautious.

A very important part of the variability in results from measurement campaigns stems from the uncertainty arising from how measurements are carried out and subsequent analysis of water samples are performed. Before setting up a field study a number of considerations are needed. The purpose of the study can be divided into data leading to a better understanding of the physical processes involved and sources and loads of the pollutants e.g. for the development of design guidelines or secondly data for use in local planning and documentation for acquired standards (Geiger 1986). The procurement of data has a number of aims:

- Identification of pollutant sources.
- Explanation of underlying mechanisms.
- Time dependency and relation with rainfall, runoff characteristics.
- Establishment of pollutant loads and concentration ranges.
- Determination of mass balances.
- Assessment of pollutant impacts.

In order to be able to make sound conclusions regarding the issues above it is of paramount importance to choose field study sites carefully. The phenomenon which is the main target of interest needs to occur in the catchment with a sufficient frequency. To identify hydrographs and pollutographs sampling resolution has to be high enough to avoid aliasing. As the variability in the studied processes are high the result is often too many samples making it impossible to analyse all samples in the laboratory. Thus a compromise is often unavoidable. Furthermore, the correct time synchronization between measurements of different entities is necessary. The accuracy of the different measurements should also be comparable. E.G. there is no reason to do very accurate pollutant transport measurements if hydraulic data accuracy is very poor. Another concern is the number of events that should be monitored. This of course depends on what is being investigated. If for instance statistical properties of extreme events (e.g. CSO events) are sought a high number of events are needed. Sometimes less than extreme events are in fact being monitored but conclusions are non-the less made on the characteristics of the extreme events. This is an extrapolation, which should not be done. Last but not least come the inaccuracies originating from the uncertainties in the measuring and analysis methods. A typical investigation may consist of the following:

- Measurement of precipitation.
- Measurements of discharges.
- Acquisition and analysis of water samples.

The importance on how and where to carry out measurements of rainfall is stressed by numerous researchers. The uncertainties introduced by correlation rainfall data from one gauge, maybe not even located in the catchment, can be large especially if the rain events are not dominated by consistent weather fronts. In all cases it is important not to introduce time lag between rainfall measurements and flow measurements on an average basis taking into account the time of concentration.

The quality of flow measurements depends on the method applied. Traditionally, measurements of water depth over weir structures have been used. Presently, the combination of a pressure sensor and a Doppler velocimeter is often used. The newest development is electromagnetic flowmeters, which introduces no obstacles into the flow. Beside these principles a lot of sensors exists based on potentiometers, ultra sound, etc. Common for all the measuring devises is that they are commercial and the producers usually quite optimistically estimate the obtainable precision. Dependent on the type of sensor system uncertainties varies from a few percent of the flow rate up till 50 % (Maksimovic and Radojkovic 1986).

Besides rainfall and runoff measurements the intricate issues of taking water samples are important. Some investigations are based on grab samples, others on automatic aqua samplers using either vacuum pumps or peristaltic pumps. How large discrepancies this introduces is difficult to assess, but the result of using the different approaches is not the same. Next question is where in the cross-sectional flow to take the sample. Many agree that the only feasible way to sample is to use a flexible tube pointing downstream placed in the flow well above any deposits (Balmforth et al. 1995). Other investigations have tried to find an influence of suction velocity, probe size, etc., where variations has been shown to be less than those introduced by e.g. the subsequent laboratory analysis (Schlütter and Schaarup-Jensen 1998). Others have though expressed the importance of the intake velocity (Kleijwegt 1992). It is important to realise that using an automatic aqua sampler will never depict the mass transport of near bed solids or bed load in general. Geiger (1984) recommends homogenising a part of the flow before sampling takes place. When water samples are analyzed the analysis procedure introduces additional uncertainty. If for instance one large sample is subdivided and analyzed for total suspended solids (TSS) a coefficient of variation as high as $CV = 20$ % may be expected (Schlütter and Schaarup-Jensen 1998). Field studies at multiple sites also introduce in-between site discrepancies due to the fact that a number of different people are carrying out the measuring campaigns and analysis of samples and data.

4.1.1 Calibration data versus model

A model can be illustrated by the transfer of a number of input variables $(x_1, x_2, x_3, ...)$ to one or more outputs $(y_1, y_2, ...)$. The model itself contains a number of calibration constants $(\alpha_1, \alpha_2, \alpha_3, ...)$. The model definition is seen in figure 4.1. By tuning the model parameters either by hand or by an optimiza-

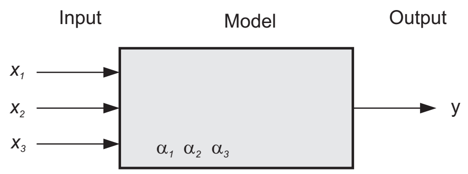


Figure 4.1: *Illustration of a model.*

tion procedure the model is calibrated to a set of calibration data. When already possessing calibration data it is of paramount importance to choose a model for the description of the process which has a suitable complexity. If a too complex model is applied there will be a risk of over-parameterization, which again may entail that the optimum calibration is hard to determine due to local minima on the objective function. The objective function is the mathematical formulation of the difference between the model and the desired output.

Before carrying out the actual calibration, a subject which is treated further in chapter 5, it is important to evaluate the quality of the input data. The results from the model will be influenced by any errors in the calibration data, i.e. low quality of input data will result in low quality of output data (Gaume et al. 1998). Different analyses are recommendable when scrutinizing the input data (Jewell 1981):

1. Plot data to pinpoint outliers.
2. Determine correlation matrix.
3. Plot data sets and fit simple regression models.
4. Examine the sensitivity of the regression models to the input parameters.

The four items listed above serves different purposes. First of all the basic time series plots are important in order to get an impression of how the monitored entity varies in time and as indicated pinpoint outliers and discover any other errors in the data which are possible to determine visually. To see how measured input variables are related, the correlation coefficient is calculated by

$$R = \frac{covar(x,y)}{s(x)s(y)} = \frac{\frac{1}{n}\sum(y_i - \bar{y})(x_i - \bar{x})}{\sqrt{\frac{1}{n}\sum(x_i - \bar{x})^2}\sqrt{\frac{1}{n}\sum(y_i - \bar{y})^2}} \tag{4.1}$$

where $\bar{x}$ and $\bar{y}$ are the mean values of the input parameters. Next step is to determine how important the different input parameters are to the output. This can e.g. be done by fitting a multiple linear regression model to the output data using all the input parameters. Then one input parameter is excluded at the time and another regression model is fitted. The drop in the degree of explanation of the variance indicated, the importance of the excluded parameter. It can be relevant to perform a logtransform of the data in order to shift the weight towards the more extreme events. Finally a simple sensitivity analysis can be performed on the full regression model.

4.2 Presentation of the case study catchment "Le Marais"

This paragraph contains a short description of the "Le Marais" catchment and its sewer system. It is mainly based on information derived from two reports of CERGRENE: "Caracterisation et equipment du basin versant experimental "Le Marais"" (Mertz et al. 1996) and "Characteristiques et origines de la pollution des eaux urbains de temps de sec et de temps de pluie" (CERGRENE 1997).

The investigated catchment "Le Marais" forms part of a bigger urban area in Paris called Le Marais. This is in general a very old quarter with predominantly narrow one way streets. Houses are 2-6 stories high with shops, bars and restaurants at street level. The area was residence area of the royal family during the 17th century but was abandoned during the revolution. Since the 1960'ties the area has slowly become fashionable and restoration of the old houses are going on.

4.3 Catchment description

The catchment comprises a total area of approximately 42 ha. The landuse in the area has been carefully recorded by CERGRENE. The shape of the catchment is more or less rectangular with the long side ($\approx 800m$) pointing east - west and the short side ($\approx 600m$) pointing north - south. The intercepting sewer extends from the outlet from the south west corner of the catchment. A photo from a typical street corner in the catchment is seen in figure 4.2 and a street map of the area is shown in figure 4.3.

Figure 4.2: *Street corner in the "Le Marais" catchment.*

As seen on the map main streets are constituted by Rue Saint Antoine, Rue de Rivoli, Rue Vielle du Temple, Rue Archives and Rue des Francs Bourgeois. The slope of the streets vary from almost levelled to more than 7 %. In table 4.1 is seen some characteristics of the slopes of streets and gutters.

There are 186 road sections with a total length of 10.5 km. The land use is distributed according to the following values seen in table 4.2. For a more elaborate

Figure 4.3: *Street map of the "Le Marais" catchment (encircled).*

Slopes [%]	Street	Gutter
Minimum	0.06	0.0
Maximum	7.29	8.14
Median	0.76	0.94
Weighted average	0.91	1.08
Average	0.84	-

Table 4.1: *Characteristics of street and gutter slopes.*

table see Mertz et al. (1996).

Category	Surface [ha]	Percentage of total surface
Roofs	22.8	54.5 %
Streets (tarmac)	9.4	22.4 %
Streets (Stone pavement)	0.3	0.7 %
Yards and gardens	9.4	22.4 %
	41.9	

Table 4.2: *Distribution of land use.*

The resident population was as of 1990, 12.372 persons resulting in a population density as high as 295 inhabitants/ha.

The area's main activities lies within the service industry of different kinds as well as commerce. Table 4.3 shows a list of different activities carried out in Le Marais. The activities susceptible to discharge of the main part of the industrial sewage are listed separately. Traffic in the area is dense but of course with a

Activity	Number of locations
Restaurants and bars	156
Hotels, schools and kindergartens	9
Bakers, butchers, florists and the like	38
Hair dressers, laundries, pharmacists, photo lab., etc.	33
Others (banks, offices, clothes stores, etc.)	557

Table 4.3: *Activities in Le marais.*

much smaller circulation of cars on the small one way streets compared to the main passageways. Rue de Rivoli and Rue Saint Antoine carries about 24.000 cars a day and Rue Archives about 11.000 a day.

4.4 Characteristics of the sewer system

The construction of the Parisian sewer system with its gravity system was commenced in 1850 by Baron Haussmann (prefect for the Seine) and the engineer Eugene Belgrand. Until then, almost all sewage had been diverted by open sewers of which the last one was closed in 1920. The System of open sewers were taken in use from as early as around 1200. In 1878, 600 km of sewers had been build and this system is now extended to 2100 km.

The subcatchment considered in Le Marais is only a small fraction of the whole system in Paris. The total length of its main sewers are 5.8 km but if all sewers are included the total length amounts to more than 8.7 km or 0.4 % of the Parisian sewer system. The weighted average slope of the 8.7 km pipes is 0.9 %. 90 % of the main sewers has a slope less than 1.5 %.

All sewers are man entry sewers. Figure 4.5 shows examples of the most frequently used cross-sections and figure 4.4 shows some photos from the Parisian sewer.

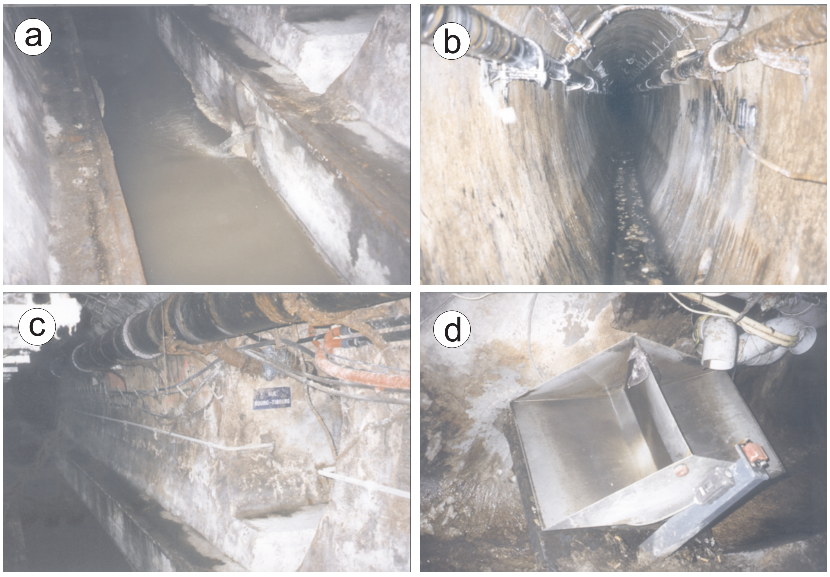


Figure 4.4: *Photos from the Parisian sewer in Le Marais. Photo a and c from a trunk sewer and b from a cunette shaped sewer. Photo d shows a tipping buket for measurements of runoff from the surface.*

The two cross-sections at the top of figure 4.5 are used for the main sewers and also contains pipelines for the supply of drinking water and water used for street cleaning (also visible on photos 4.4.b and 4.4.c).

4.5 Measurements in "Le Marais"

A quite rigorous measurement project has been carried out in the catchment. At the outlet dry weather campaigns has been carried out determining the waste-

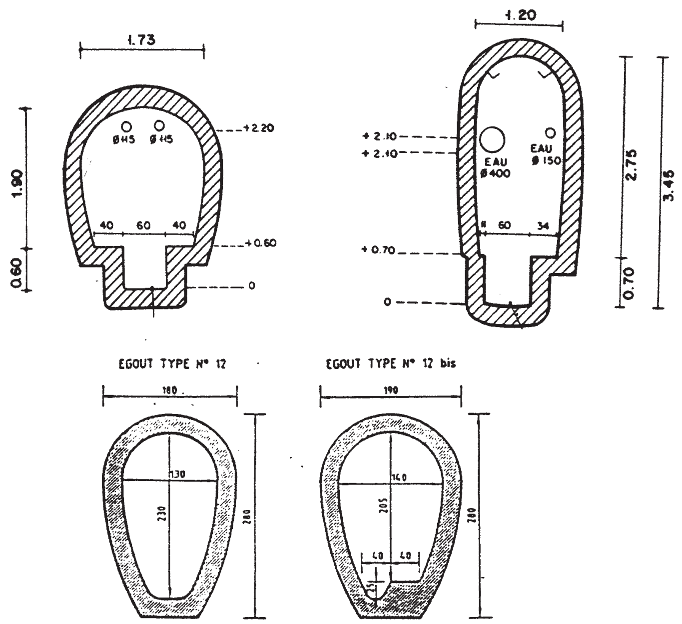


Figure 4.5: *Cross-sections for sewer pipes.*

water quality by TSS, COD, BOD_5, VS, heavy metals and hydrocarbons. These parameters has also been determined for a number of rain events. Furthermore, the water quality from surface runoff during rain events from several locations of roofs and streets has been investigated.

The types of data used for calibration of the numerical model of sediment transport and hydraulic conditions are listed below:

- Rain intensity measurements
- Surface runoff from streets
- Suspended solids in surface runoff (washoff)
- Discharge measurements at the outlet
- Velocity measurements at the outlet
- Water level measurements at the outlet
- Suspended solids measurements at the outlet, converted into mass transport rates

4.5.1 Measurement equipment

Precipitation is measured with a classical rain gauge with a tipping bucket. The rain gauge has a resolution of 0.2 mm. Street runoff is measured in a similar fashion using a tipping bucket with a volume of 20 l but also a combination of a pressure sensor and a V-notch shaped weir is used.

The water level at the outlet is measured with both a pressure sensor and an ultrasonic sensor. The ultrasonic sensor (Endress+Hauser) has a shooting length from 0.5 m up till 9 meters with a concentrated beamwidth less than one meter. The ultrasonic sensor is able to filter out echo noise from fixed structures. Water velocity is measured with three sets of ultra sonic sensors at different levels. Samples are taken with automatic water samplers extending a loose suction tube into the sewage flow.

4.6 Description of data used for modelling

For the purpose of modelling, 13 rainfall events and dry weather conditions has been selected. For most of the 13 events complete attaining pollutographs exists computed from TSS in water samples taken during the rain events at a flow proportional rate.

4.7 Dry weather conditions

Figure 4.6 shows the measured discharge and mass transport at the outlet during dry weather. The curves are composed by averaging of several diurnal cycles obtained during one month. The DWF undergoes both seasonal changes as well as changing from working days and days off. The chosen dry weather measurements are considered to be applicable for its purpose in the modelling. If the complete volume of water discharged during one diurnal cycle is divided by the number of inhabitants ($5690m^3/12372$) the result is 460 l/PE/day. This is a high rate of water consumption and can be explained by a sizable use of water for street cleaning etc., more than 20 % (Mertz et al. 1996).

The measured transport of suspended solids during dry weather follows the flow rate closely. If measured concentrations of TSS are plotted against the discharge a regression curve can be fitted. An example of such a fit of a regression model can be seen in figure 4.7. Likewise a regression model can be fitted to the mass transport as seen in figure 4.8. The exponent of the regression curve in figure 4.8

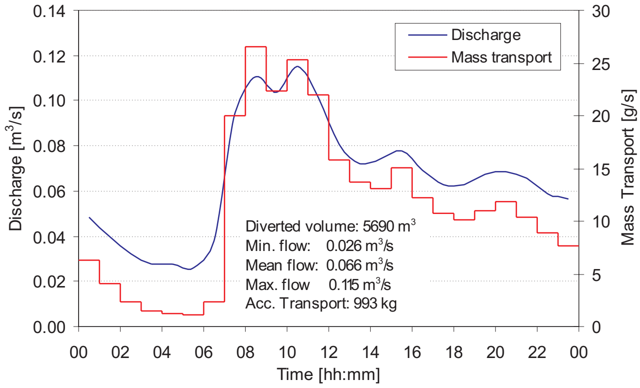


Figure 4.6: *Dry weather discharge and mass transport rate.*

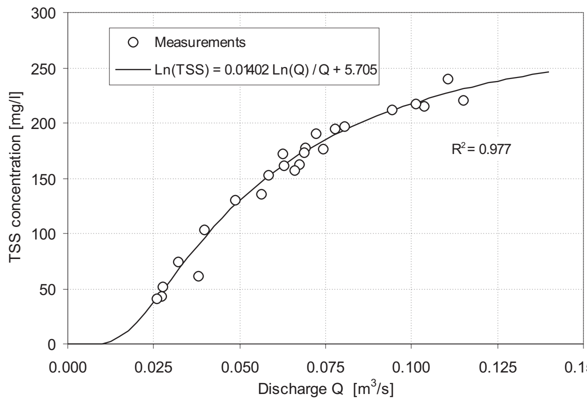


Figure 4.7: *Regression model for TSS concentrations as a function of discharge.*

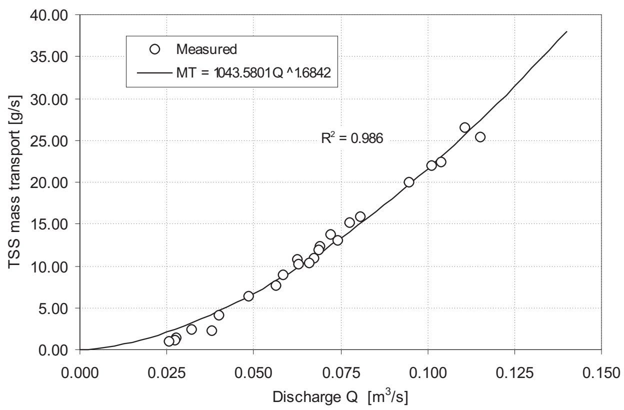


Figure 4.8: *Regression model for mass transport rate as a function of discharge.*

of 1.68 corresponds to a similar regression model from a small urban catchment in Denmark (Schlütter and Schaarup-Jensen 1998) where the exponent equaled 1.69.

4.8 Hydrological and hydraulic data from rain events

The hydrological data consists of rain intensity measurements and measured street runoff at different locations. An example of a set of rainfall and runoff curves can be seen in figure 4.9. The rainfall and subsequently runoff results in an increased discharge at the outlet. A plot of the discharge at the outlet can be seen in figure 4.10. As seen the abrupt changes in the rainfall intensity and runoff is dampened by the sewer system yielding slower variations in the discharge.

The set of rain events comprises a range of different intensities, rain durations etc. Table 4.4 shows some of the main characteristics of the rain events. Figure 4.10 and 4.11 shows plots of concentration curves and mass transport respectively for one rainfall event as an example.

In general, the diverted mass transport rates of suspended solids shows that the system yields some flushing effects, i.e. a relatively larger mass transport takes place during the initial part of the rain event compared with the rest of the event. As sediments collected during non-extreme rain events shows a high

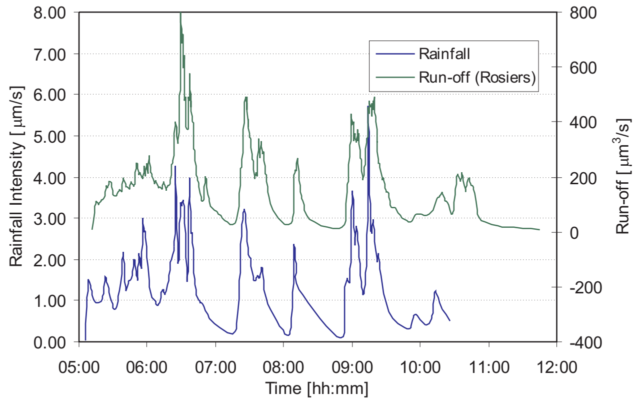


Figure 4.9: *Curves for rainfall intensity and surface runoff.*

Model -	Rain [yymmddhh]	T_{rain} [min]	ADWP [day]	I_{max} [μm/s]	$I10_{max}$ [μm/s]
M17	96070504	321	5.97	5.72	3.13
M16	9608061x	97	29.91	40.00	24.79
M15	96081017	177	0.12	22.22	10.32
M14	96082200	169	1.26	3.17	2.56
M13	96100103	490	3.47	5.72	2.90
M12	961127	326	2.48	0.92	0.71
M05	97050604	417	0.71	18.17	8.35
M06	97051600	67	1.73	28.58	13.95
M07	97062516	142	1.89	2.40	1.63
M08	97063000	170	0.73	22.22	13.48
M09	97070413	122	0.07	2.78	2.31
M10	97080518	231	10.84	66.67	17.13
M11	97080604	123	0.25	40.00	19.42

Table 4.4: *General rain event data. (ADWP: Antecedent dry weather period.)*

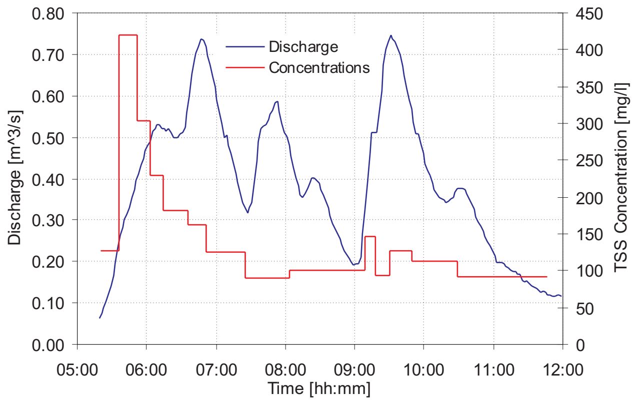


Figure 4.10: *Suspended solids concentration during M17.*

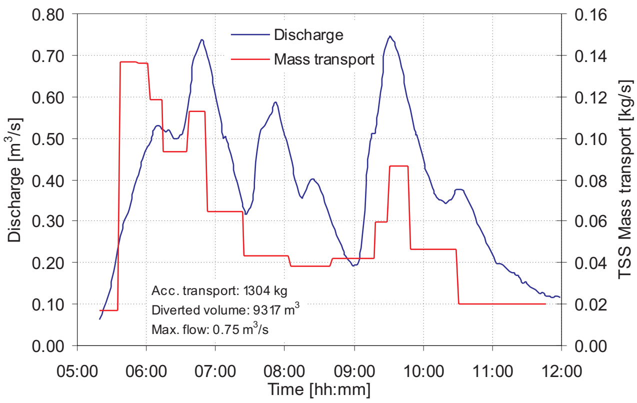


Figure 4.11: *Mass transport rate during M17.*

content of organic matter it is reasonable to assume that the recorded mass transport mainly originate from fine sediments build-up on the surface and in the sewer system in particular, during the antecedent dry weather period. The readily erodable amount of sediments for non-extreme events are thus limited.

Some hydraulic and mass transport related information is given in table 4.5. Different entities such as diverted volume, ADWP, $I10_{max}$, etc. has been cor-

Model	Rain	Div. Vol.	Max. Dis.	Acc. trans.	Max. trans
-	[yymmddhh]	[m^3]	[m^3/s]	[kg]	[kg/s]
M17	96070504	9317.28	0.747	1304	0.135
M16	9608061x	3891.12	1.205	2086	0.660
M15	96081017	4731.84	1.098	887	0.330
M14	96082200	3733.20	0.796	425	0.126
M13	96100103	3234.36	0.510	478	0.067
M12	961127	3951.96	0.250	573	0.062
M05	97050604	8458.56	0.865	1368	0.163
M06	97051600	2630.88	0.801	797	0.403
M07	97062516	2452.08	0.377	368	0.065
M08	97063000	5510.40	1.204	1029	0.223
M09	97070413	2548.10	0.540	298	0.066
M10	97080518	5089.92	1.338	1808	0.696
M11	97080604	–	–	–	–

Table 4.5: *General hydraulic and mass transport related data.*

related with the accumulated transport. There is a tendency that the heavier the rainfall intensity the more mass is transported. Correlations are though not very strong with correlation coefficients (R^2) ranging from 0.0006 to 0.67. in some cases the lack of correlation can be explained, e.g. correlation with ADWP is expectedly weak due to regular street cleaning. A nonlinear function with the input of rain duration (T_{rain}) and $I10_{max}$ has also been fitted to the accumulated transport (M_{accu}) ($R^2 = 0.92$). This function reads

$$\mathrm{M_{accu}} = 23.10\ (\mathrm{i10_{max}})^{1.38} + 5734.95\ (\mathrm{T_{rain}})^{0.057} - 7334.58 \tag{4.2}$$

A plot of the regression model of equation 4.2 is shown in figure 4.12.

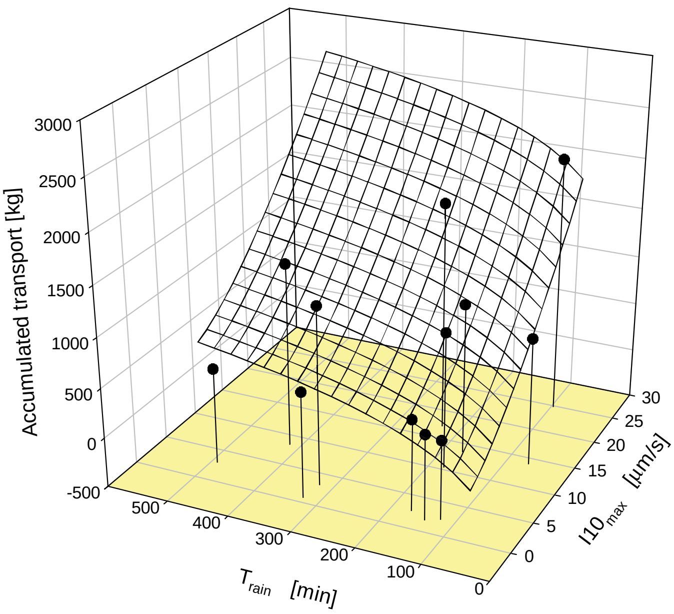


Figure 4.12: *Regression model for accumulated transport.*

CHAPTER 5

Numerical modelling approaches

A model is, in the framework of this study, a mathematical description of a natural process, either being a physical, chemical or biological process (Lei and Schilling 1994).

5.1 Model definitions

Prior to discussing different prospects within modelling of sediment transport in sewer systems it is imperative to present a number of definitions of model types. Hemain (1986) defines two model groups:

- **Deterministic models** which comprise causal relationships between various variables controlling the processes modelled.
- **Stochastic models** which comprise relationships between probability levels of the variables included in the processes.

If a deterministic model is feed with identical input it will yield the same output each time. This may also be the case for a stochastic model and the relationship between probability levels for input and output is established by repeated modelling with what is basically a deterministic model. The values of the input variables can in this case be generated by e.g Monte Carlo simulations. If the model uncertainty is modelled stochastically as well the model will no longer give the same output for repeated identical input. The advantage of stochastic models is the possibility to track the propagation of the uncertainty on input to the resulting uncertainty on the output. In order to use a stochastic model it is, however, necessary to have measured data showing the input variables and model parameters probability density distributions or at least have a clear idea about these distributions. Such a coherent data basis has not been available for this study.

Therefore, in this thesis only deterministic models are considered.

The deterministic models can be subdivided into different classes:

- Physically-based models (PM), which are derived from theoretical approaches, within the fields of e.g. fluid mechanics, forces exerted on singular particles, etc.
- Conceptual models (CM), which borrow features from both PM and SM. They have a structure which mimic the main properties suggested by e.g. a PM and can be validated by field data.
- Statically-based models (SM), developed from statistical analysis of experimental data or field data. Regression models and neural network models belongs to this class, but can furthermore be named black-box models as physical parameters are not recognized in the model descriptions.

If a number of conceptual models are combined forming a model complex it can be termed a lumped conceptual model (LCM) (Gaume et al. 1998). The LCM typically consists of a number of empirical relationships which models processes like sediment build-up on the catchment surface, etc.

The advantage of a statistically-based approach to modelling is that only input parameters which contribute significantly to explaining the output is included in the model. In a PM some of the parameters may not be excluded due to the theoretical considerations even though measured data does not substantiate the significance of the parameter.

When deciding how to approach numerical modelling of sediment transport in sewer systems it is important to realize what the objectives are. First of all it is deemed unfeasible to develop a fully deterministic model able to account for the high spatial and temporal variability of the processes involved. The involved complexity has been described in chapter 2 and 3.

A well known principle of parsimony should be observed in modelling or as Albert Einstein (1879-1955) has put it: "Make everything as simple as possible, but not simpler". This means that the model complexity should only be increased as long as increased precision of desired output variables are obtained without requiring non-accessible input. Non-accessible input data means data which cannot be measured with known technology or data which are unrealistically expensive to procure. In this context model complexity should also be defined. In this case the model complexity is defined as the number of model constants $\alpha_1, \alpha_2, \alpha_3, ...$ to be found by calibration (Larsen 1997).

One of the drawbacks of purely statistically based models (black-box) is their inability to reliably predict output which is out of the range of the outputs encompassed by the calibration data. Physically based models which only includes models constants which can be determined by experiments, e.g. the viscosity of water, may be used for predictions leading to a larger range of outputs. This is of course given that the processes described does not change. CM having some physical basis, may also be used for extrapolations, but this has to be done with great caution.

5.2 *Calibration/Validation*

SM models needs calibration of model constants. Three steps is recommended to test a given model. The process is illustrated by figure 5.1. The example shown

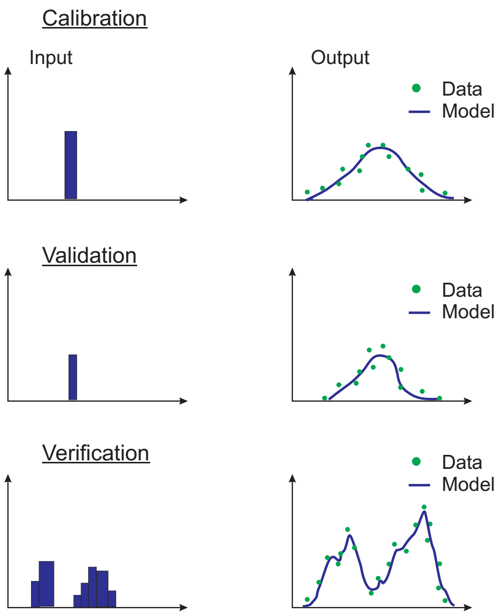


Figure 5.1: *Illustration of test procedure for models.*

in figure 5.1 could be modelling of a rainfall-runoff process. First, the model is calibrated applying one set of input data represented by the rainfall intensity. The model is then validated by applying a new set of input data keeping the model constants obtained during the calibration. The quality of the models fit to the measured data can be determined by the correlation coefficient (see equation 4.1) and/or by the mean residual sum of the error between the model estimates and the measured data also called the root mean square error given by

$$\varepsilon_{RMS} = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(y_i - \hat{y}_i)^2} \tag{5.1}$$

where y_i is the measured values, $\hat{y}_i$ is the values estimated by the model and n is the number of data.

In order to verify that the model is generally applicable it should be transposed to another catchment, repeating the process of calibration and validation exploiting the data obtained from the new site.

The choice of model complexity is limited by the available measured input data for calibration. If a too complex model is applied to a limited set of data, there is risk of overparametrization (Gaume et al. 1998). This can be illustrated by figure 5.2.

The model example given above is very simple. It is in simple cases easy to evaluate whether the obtained calibration constitutes the best possible result. When dealing with lumped models this issue of optimization quickly becomes intricate with the increase in model complexity.

Each combination of chosen model constants will result in a specific residual error in the estimation, this results in the objective function. If the lumped model is poorly formulated or the model constants are interdependent local minima may arise on the objective function (Gaume et al. 1998; Beven and Binley 1992). Some of the local minima can represent a model fit which is almost as good as the global minimum. This combination of model constants can though turn out to be located at a completely different location on the objective function and at the same time represent a more realistic set of model constants when related to the physics that are modeled.

Another problem is to find the optimal calibration. The approach for obtaining the global minimum, must depend on the model structure, as it may not always be practicable to apply sophisticated optimization routines. This will be further discussed in chapter 8.

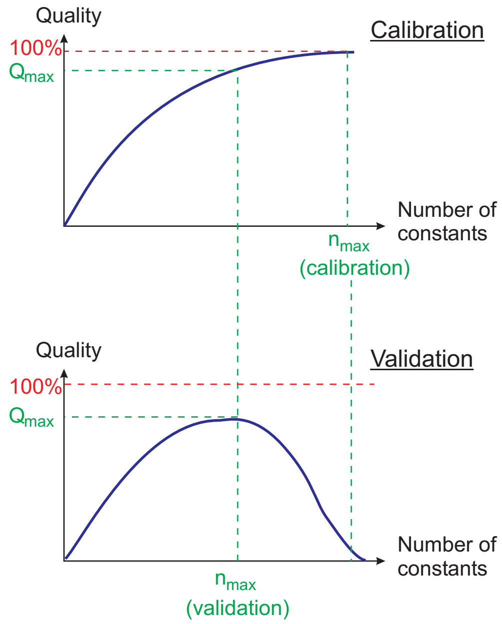


Figure 5.2: *Illustration of the influence of model complexity on the calibration and validation. Q_{max} pinpoints the optimal quality of the model, i.e. where the discrepancy between modelled and measured calibration and validation data is smallest.*

A good fit between measured and simulated data cannot always be taken as a success. If the uncertainty on model constants remains large the model will turn out to be more or less useless. The uncertainty of the model constants is indicated by the models sensitivity to the choice of constants, low sensitivity indicating a large uncertainty.

5.3 Model developments

The following paragraphs (5.4 - 5.6) renders a short overview of some of the different models which have been presented within this field of research. The review comprises lumped conceptual models, physically based models and black-box models.

The need for reliable and robust modelling of sediment and pollutant transport in sewer systems is apparent. The European Union are making regulations where design criteria depends on the sophistication of the model (Gent et al. 1996). This is one of the incentives for the model development. So far, the main effort has been aimed at modelling pollutant spills from CSO structure. Initial focus was, in addition, put on development of physically based models, trying to exploit the theories developed for sediment transport within riverine sediment transport. As these models have not proven undisputably successful, developments within LCM models are becoming more the center of focus.

One of the concerns when dealing with complex models, either PM or SM, is the model uncertainty and applicability. In this context a number of German models have been compared in a project presented by Russ (1998).

Data were collected during the period 1986-1991 from two catchments. Ten models were calibrated and compared, where it was the contenders who performed the calibration based on detailed information about the catchments and the calibration data.

From the results it was concluded that the persons running the models are the most important. Differences, where the user influences results, arises due to different approaches for the calibration procedure and due to lack of knowledge about the model. I.e. the user qualifications are decisive for getting good results. This entails that the user interface, model documentation and guided use of the model is very important factors for success for models, when it is the sewer system managers who needs to apply the models.

5.4 Review of lumped conceptual models

A number of lumped conceptual models have been presented during the last decade. They are all deterministic in nature and to a certain extent physically based. The models attempts to model the solids production and build-up during dry weather and the washoff and transport of pollutant/sediments during wet weather.

The models which are briefly described in the following paragraphs does not constitute a comprehensive list of all the models build, but a selection of the most well-known attempts to apply LCM's to transport of sewer sediments. The models differ in how extensively they have been reported in literature and thus the descriptions below cannot be used as a guide to which model is the best.

5.4.1 Flupol

Flupol is a French model developed by "Agence de l'Eau Seine-Normandie, Syndicat des Eaux d'Ile-de-France" and "Compagnie Générale des Eaux" (Bujon et al. 1992). The model calculates the flow in the sewer and the pollution transport downstream from an urban catchment. The pollution parameters are TSS, BOD_5, COD and N_{Kj}.

It is regarded necessary that the model includes:

- Production and accumulation of polluting matter on surfaces and in the system.
- Runoff and washoff from surfaces.
- Transport of pollution in the sewer system.
- Build-up and erosion of deposits.

For the build-up of sediments on the surface the model uses, as many other models, the formulation used in SWMM, see equation 3.5. The runoff is modelled by a linear reservoir technique and the flow in the sewer system is either modelled by kinematic wave theory, the Cunge-Muskingum method or diffusive wave theory. The differential equation governing the transport of pollutants in the sewer system is

$$K\,\frac{dp}{dt} + p(t) = pe(t) \tag{5.2}$$

where

K	[-]	:	Proportionality factor dependent on the physical characteristics of the catchment (catchment area, average slope, imperviousness, rainfall duration, length of catchment and rain depth).
$p(t)$	$[m^3/s]$	:	Pollutant flux at time t.
$pe(t)$	$[m^3/s]$	:	Pollutant flux entering into suspension at time t.

Deposition and resuspension is governed by the Velikanov theory as seen in equation 3.26. If the concentration $C > C_{max}$ then deposition takes place and contrary if $C < C_{min}$ erosion occurs. In between the limits the turbulent energy level is just adequate enough to keep the solids in suspension.

Bujon et al. (1992) presents a calibration of the model to two catchments and a validation on a third catchment. The model shows promising results for the rain events presented.

5.4.2 SEWSIM

The SEWSIM model has been put forward by Ruan and Wiggers (1998). The model is build with the aim of repeated event modelling and continuous modelling, e.g. predicting CSO emissions. It is therefore developed with a very simple structure.

The runoff process on the catchment surface is modeled by a double linear reservoir and the outflow from the sewer system by a cascade of linear reservoirs.

Build-up of sediments on the surface is again done by the empirical relation in equation 3.5. The washoff of sediments from the surface is modelled by an adaptation of equation 3.8

$$\frac{dp(t)}{dt} = -\alpha_w \; r(t)^{\alpha_{w2}} \; p(t) \tag{5.3}$$

where

$p(t)$	[kg/s]	:	Pollutant flux at time t.
α_w, α_{w2}		:	Numerical coefficients.
$r(t)$	$[m^3/s]$	:	Runoff rate from the impervious surface.

One of the original aspects of this model is that it reuses the formulations for build-up and washoff of sediments on the surface in the sewer system. This makes the model very simple.

Results from the model are presented predicting the total load from the system for a number of rain events (Ruan and Wiggers 1998). The average discrepancy between measured and modelled results is 35 %.

5.4.3 HYPOCRAS

The objective of the HYPOCRAS model is also to model the production and transport of solids in sewer systems. The model is developed in France (Bertrand-Krajewski 1992) and makes use of simplified modelling of the hydraulics. The hydrological and hydraulic processes are modeled by two linear reservoirs.

First part of the model concerns the build-up of solids during dry weather periods. The second part deals with the transport of solids during rain events. A illustration of the model structure can be seen in figure 5.3.

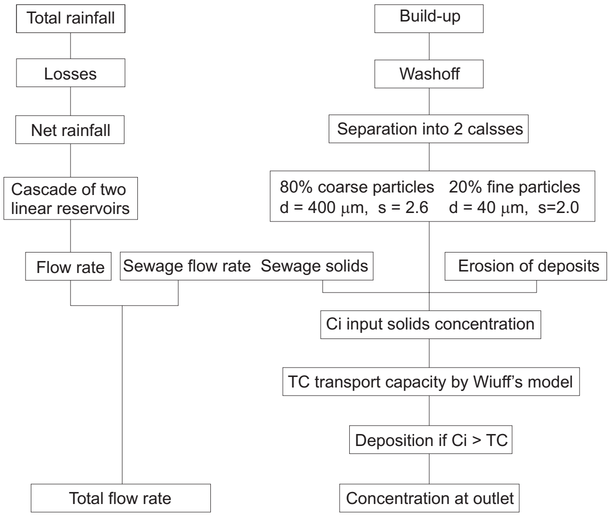


Figure 5.3: *Illustration of the structure of the HYPOCRAS model (from Bertrand-Krajewski 1992).*

The production of solids during dry weather is modelled by

$$C(t) = \alpha \cdot Q_{DWF}(t)^{\alpha_2} \tag{5.4}$$

where

$C(t)$	$[kg/m^3]$	:	Concentration at time t.
α, α_2	[-]	:	Numerical coefficients.
$Q_{DWF}(t)$	$[m^3/s]$	:	Flow during dry weather.

The transport of solids are modelled by Wiuff's model (Wiuff 1985) adapted for dealing with two fractions of solids - a coarse and a fine fraction. The process of deposition and resuspension is modelled by equation 3.27 and 3.28 respectively.

In the model the real actual sewer system is replaced by one pipe with the diameter equal to the pipe diameter at the outlet and a slope equal to the mean slope of the sewer system. The pipe has no length, but a time lag between flow and solids transport is introduced instead.

The model computes suspended solids pollutographs, total bed load and the variation of deposits for each class of particles at the outlet. HYPOCRAS has been verified on e.g. Mantes-la-ville, Entzheim and Dundee sewer systems (Bertrand-Krajewski et al. 1993). From application of the model it was concluded that the initial conditions are the most important for the results, next comes the sensitivity to catchment data and least important is the geometry of the sewer system (Bertrand-Krajewski et al. 1993).

5.4.4 SIMBA-SEWER

The SIMBA-SEWER model is not extensively reported in literature yet. It is part of a model complex build in MATLAB/SIMULINK, which entails that it is not a stand alone application. It has been used in Norway in combination with two projects (Risholt et al. 1998).

One of the incentives for establishing the model is the change of strategy from the Norwegian authorities. There is a shift from giving end-of-pipe permits towards issuing permits for total pollutant discharge, i.e. only limited discharge of pollutants are permited.

The flow in the sewer system is modelled by the diffusive wave approximation of the St. Venant equations. The transport of pollutants and solids are modelled using a advection-dispersion model. Finally, the sedimentation is governed by Shield equation and re-suspension is modelled as a function of the critical bed shear stress.

As indicated above this model is to a large extent a physically based model. Risholt et al. (1998) does not present results in the reference.

5.4.5 HORUS

The HORUS model is developed in France and is a quite extensive model. It is a conceptual total load model based on the Velikanov theory (Zug et al. 1998a). An illustration of the model structure is seen in figure 5.4. As indicated on this figure, this model has an optimization algorithm included for the calibration process. This is different from the other LCM's. The optimization minimizes an objective function which includes the differences between measured and modelled values of mass transport, total load and peak mass flux.

The Runoff from surfaces is modeled by a linear reservoir model, the hydraulics in the sewers are modeled by the Muskingum model.

Build-up on the catchment surface is modelled like the other models by the SWMM expression (equation 3.5). The pollutographs are propagated by convection and deposition and re-suspension is governed by the Velikanov expressions.

The model has shown promising results, where the overall discrepancy is smaller

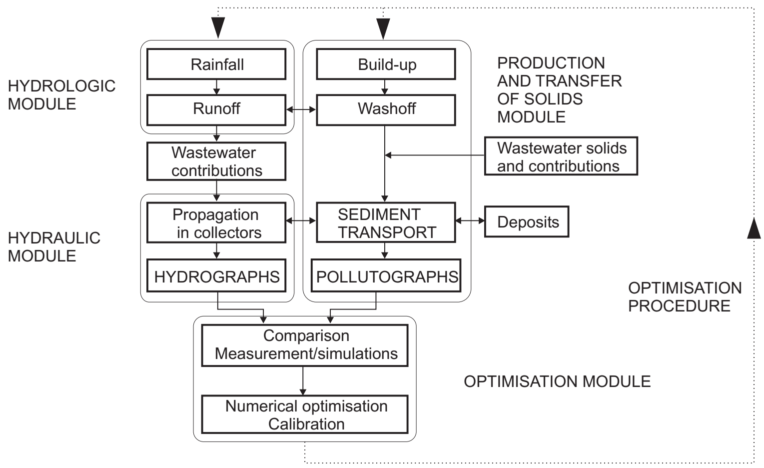


Figure 5.4: *Illustration of the structure of the HORUS model (from Zug et al. 1998a).*

than 20 %. The largest modelling errors occurs when predicting low concentrations (Zug et al. 1998b). Also COD has been modelled successfully through the use of potency factors (Zug et al. 1998).

5.5 Physically based models

When labeling the following models as physically based it does not indicate that the models listed in paragraph 5.4 are not physically based and furthermore models listed in this paragraph may be lumped as well. It is not possible to give a proper review of the PM models available here. These models are SWMM, Thalia, Mosquito, Hydroworks-Qm, MOUSE TRAP among others.

The Storm Water Management Model (SWMM) is one of the oldest models, whereas Hydroworks-QM and MOUSE TRAP are newer developments. Hydroworks-QM from Wallingford software and MOUSE TRAP from the Danish Hydraulic Institute are commercial software packages.

The physically based models have not yet proven entirely successful eventhough they have been on the market for a number of years. It may ofcourse be explained by lack of knowledge by the endusers and lack of high resolution calibration data. If this should be the reason it will be natural to question whether these models have the right layout compared with current level of knowledge and data.

The MOUSE TRAP model is described in chapter 6. This model has been presented in a number of articles. Mark et al. (1995) states that sediment transport in sewers are similar to transport in rivers except for the fact that there is a limited supply of sediments in the sewer, the sediments in the sewer can have a wider variability in their characteristics, the spatial and temporal variability of the hydraulic conditions are higher in sewers and the influence of the fixed boundaries of the sewer on the transport is not fully known. The MOUSE TRAP model has been compared with laboratory data with good results and it is concluded that the sediment transport formulation of Engelund-Fredsøe (Engelund and Hansen 1972) adapted from river hydraulics is the most suitable. (Mark et al. 1995). A 120 km sewer tunnel in Rya catchment, Sweden, was modelled. The tunnel routes the sewage from 793000 people equaling 321000 m^3/day. Results were encouraging, but peak transport rates of sediments had a time lag compared with measured ones. It was concluded that the quality of the modelling would improve if more data were available on sediment grain sizes and densities and sediment deposits location and depths in the tunnel.

Another study aimed at predicting locations of deposits in the Ljubljana catchment (Jugoslavia), which holds 240000 people in a 4000 ha sized catchment. The deposits was build-up during dry weather and moved and/or eroded during subsequent wet weather. Locations of sediment deposits were found with satisfactory results (Mark et al. 1996). Also, the influence of sediments on the volumes of CSO spills were investigated.

The MOUSE TRAP model has also been applied for the modelling of dissolved substances in the sewer (Garsdal et al. 1995). In this case the advection-dispersion model included in the MOUSE TRAP package was used. It is concluded that the model works well with dissolved substances ($DO, COD_{dissolved}$).

Due to the success of PM within river hydraulics it has been a natural step to try to apply the same techniques within modelling of sediment transport in sewer systems. Unfortunately, the results from these efforts have not so far been convincing (see e.g. Jack and Ashley 1996). Sparse, elaborate verifications of the PM's have been presented and the user requirements for these models are still unpractical to the sewer managers.

5.6 Black-box models

Black-box modelling within urban storm drainage have shown many successful applications. A black-box model correlates input with output and the form of this response function is normally not decided beforehand. The black-box model can be subdivided into groups: Regression models, time series models, artificial neural network models, and genetic algorithm models. The black-box models are in general easy to adapt to the problem in hand and they give fast answers. Calibration is done automatically as an inherent feature of the models. The disadvantages of the black-box model is that it needs relative large amounts of measured data to yields good results. The black-box model will never be better than the data that feeds it. Another drawback is that it seldomly gives information on the intermediate processes which are modelled as a PM may do. Finally, a black-box model will need a new calibration if, e.g. the modelled sewer system is changed.

The following paragraphs outlines the structure of the different types of black-box models and refers to some application examples.

5.6.1 *Regression models*

The most simple regression model possible is to relate input x with output y by multiplying the input with a factor α. This equals a straight line through origo. A model formulated as a straight line may read

$$y = \alpha \cdot x + \beta \tag{5.5}$$

where β is the shift from origo. This is of course a very simple regression model, but it often turns out to be successful. The next step is to add linear term to equation 5.5, which results in a multiple linear regression model. This model can be made nonlinear and will then take the following form

$$y = \sum_{i=1}^{n} \left(\alpha_{1,i} \cdot x_i^{\alpha_{2,i}} \right) + \beta \tag{5.6}$$

where

x_i	:	A number of independent input variables.
$\alpha_{1,i}, \alpha_{2,i}$	:	Numerical coefficients.

As already stated, the addition of extra fitting coefficients will inevitably lead to better correlation between measures and modelled data. But at the same time the model looses its ability to generalize. It is thus a continuous balancing act to choose the right model. This is in fact a general dilemma of black-box models.

Fitting the model to the measured data is quite often done by minimizing the sum of least squares, which is the sum of the vertical distance between measured and predicted output values put to the second. By applying this approach straight forward the data contributes equally to the sum of squares. This may not always be desirable as one may wish to put more weight on e.g. a range of small input values. In this case transformation of x and y, e.g. a linear, logarithmic or normal transformation may result in a more robust regression model. A way to check the behavior of the model is to examine the residuals between measured and predicted values (Draper and Smith 1998).

Regression modelling has been widely used in urban storm drainage. Coghlan et al. (1996) have attempted to apply regression modelling to load models for suspended solids for dry weather conditions and wet weather, respectively. Site specific models were build for the catchment of Dundee. This catchment is 340 ha and inhabits 14590 persons. According to Coglan et al. (1996) the fitted dry weather model is

$$TSS_{DWF} = 955 \cdot Q^{0.8} \tag{5.7}$$

where

TSS_{DWF}	[kg/s]	:	The flux of suspended solids.
Q	[m^3/s]	:	Flow rate.

The multiple regression model for the mass flux of suspended solids during rain events for the Dundee catchment is seen in equation 5.8.

$$TSS_{STORM} = 104.4 + 416.41U - 0.79674TSSS - 3.1238ADWP \tag{5.8}$$

where

TSS_{STORM}	[kg/s]	:	The flux of suspended solids during wet weather.
U	[m/s]	:	Mean flow velocity.
$TSSS$	[hours]	:	Time since the start of the rain event.
$ADWP$	[hours]	:	Antecedent dry weather period.

The choice of these simple models were based on a wish to avoid inclusion of data on sediment characteristics. The models were subsequently compared with the physically based model of Ackers-White and the regression models proved to perform better than this model (Coghlan et al. 1996).

It is clear that the force of this type of model is its simplicity and the models in this example are not able to yield a very wide range of results. The applicability of the models to other sites may not be straightforward.

5.6.2 Time series models

Time series models comprises ARMA, ARIMA, etc. models, which are Auto Regression Moving Average models (Box et al. 1994). These models becomes relevant when input data is interdependent. In that case the data shows a certain amount of autocorrelation. The auto regressive part of the time series model includes the influence of prior output and the moving average part smoothen out input data. This type of model have not been applied within this thesis and will not be dealt with in details.

Ruan and Wiggers (1997) presents an application of time series models to the modelling of CSO overflows. The model is applied to the catchment of Loenen (Holland), with a catchment area of 56.5 ha, 28 % imperviousness, and 5.2 mm storage capacity. Time series from 63 rain events are modelled, predicting hydrographs, water levels and pollutographs at the CSO. The quality of the models predicting hydrographs and water levels were very promising.

5.6.3 *Artificial neural network models*

With the development of fast computers the Artificial Neural Network (ANN) became an option within modelling in a number of fields. ANN's have been successful within stock market prediction, the medicine industry, etc. In the last years ANN has been shown to be very useful within urban storm drainage as well.

The force of ANN is their adaptability to specific problems and their ease of use. In general when feed input the ANN produces output. In order to produce relevant output the ANN needs to be learned by training examples.

The basic element of the ANN is the neuron. An illustration of such a neuron is seen in figure 5.5. The neuron is composed by its inputs $x_{i,j-1}$ from either other

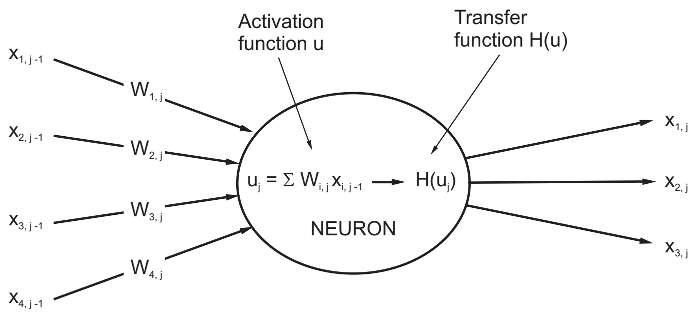


Figure 5.5: *Illustration of a neuron.*

neurons or measured input data. The indata is associated with synaptic weights $W_{i,j-1}$. In the neuron an activation function calculates u_j and these values is processed by a transfer function $H(u_j)$, producing outputs equal to inputs to other neurons $x_{i,j}$ or final output y.

The neurons are combined in a network. The structure of the neurons and the choice of transfer function results in different kinds of networks (see e.g. Loke et al. 1997). The most commonly used type of network is the backpropagation network. This network type is described in detail in chapter 9.

A number of applications of ANN's have been reported in literature (Loke et al. 1997; Djebbar and Kadota 1998; Gong et al. 1996). The applications shows the variability of the application possibilities of the ANN.

Loke et al. (1997) shows examples illustrating the ANN's capabilities in prediction, simulation, identification, classification and optimization. Computations are presented predicting runoff coefficients given catchment characteristics as input, and restoration of rainfall data for one gauge using data from other gauges. All the applications are successful, but of course the ANN's can only be successful if provided with sufficient input data of reliable accuracy. The ANN will never give better results than the input data allows. In modelling of sediment transport this may often turn out to be a limiting factor for the ANN as quality measured data in this field are often scarce and inhomogene.

An attempt to incorporate ANN techniques in combination with the HYPOCRAS model (see 5.4.3) has been carried out (Gong et al. 1996). The neurons were incorporated into the model in a network layout that mimics the structure of the HYPOCRAS model. Synaptic weights and neurons substitutes calibration parameters and variables in the HYPOCRAS model and assists in an easy automatic calibration of the model. The technique were applied for the hydraulic part of the HYPOCRAS model with good results. The greatest benefit were the ease of use when applying the ANN technique for automatic calibration (Gong et al. 1996).

Another type of problem was presented by Djebbar and Kadota (1998). An ANN was used for prediction of dry weather sewage production for a given catchment of a specific size and other catchment characteristics. The model was applied for the Greater Vancouver Sewage and Drainage District, which cover an area of 650 km^2 and is inhabited by 1.8 million people. Flow measurements from 40 monitoring sites was used for training the ANN. The input data consisted of catchment area, population, dwelling units, commercial area, industrial area, institutional area and other non residential area. The model turned out peak DWF and average DWF. All results from calibration and validation had an discrepancy less than 16 % between measured and modelled results.

This chapter has presented an introduction to different types of models which can be used to predict sediment transport in combined sewers. The following chapters presents specific models of different types and their application to the case study catchment "Le Marais". This is done to get an impression of the models suitability to sediment transport modelling. The literature study carried out in connection with this study does not reveal which model types are the most useful as both impressive and discouraging results have been presented in literature for most types of models.

CHAPTER 6

Physically based modelling

The aim of this chapter and chapter 9 is to get a better perspective of the quality of the lumped conceptual modelling compared with other modelling approaches. In this chapter modelling of the "Le Marais" sewer system is carried out using the MOUSE TRAP model from the Danish Hydraulic Institute (DHI 1993).

It is of course impossible to claim that the different comparisons of models are just. As earlier mentioned the quality of the modelling often also depends on the expertise of the modeller to use and apply a specific model to the case. As the author has not developed the MOUSE TRAP modelling package but has developed all the other models applied in this thesis it can be argued that if the MOUSE TRAP model does not perform well it may be due to this fact. The modelling has, however, been carried out to the best of the abilities of the modeller.

The MOUSE TRAP (Modelling Of Urban SEwers TRAnsport of Pollutants) modelling package contains several models. First of all it relies on the MOUSE model for calculation of the hydrodynamics. The TRAP package contains a lumped conceptual model for build-up and washoff of sediments on the catchment surface, an advection-dispersion model for transport of solutes and fine sediments in the sewer system, a sediment transport model for transport of non-cohesive sediments in the sewer and a water quality module. The model complex is illustrated in figure 6.1.

The following paragraphs gives a short introduction to the models. Additional information and application examples can be found in Mark (1995), Mark (1992), Mark et al. (1993), Mark et al. (1995), Mark et al. (1996), Crabtree et al. (1994), Crabtree et al. (1995), and Garsdal et al. (1995).

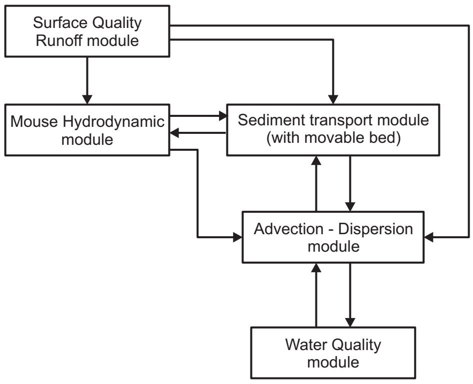


Figure 6.1: *Illustration of the MOUSE TRAP model complex. (from (DHI 1993)).*

6.1 Sediment surface model

The surface model is a lumped conceptual model as detailed deterministic modelling is too complex for the surface as explained in paragraph 3.

For the build-up of sediments on the catchment surface it is possible to chose either a linear or an exponential function, see equation 3.3 and 3.5. This model estimates the initial load on the surface before the succeding rainfall event based on the ADWP. If another initial load is desired than the value calculated by the buil-up function then it is possible to give a maximum value of the load and then just give a sufficiently high build-up rate to ensure that the model reaches this maximum value. It can be relevant to circumvent the build-up function in this way e.g. if measured data does not show a clear dependency on the ADWP.

The washoff of sediments is modelled by transport on the surface as bed load and transport in suspended mode. The bedload is calculated according to (DHI 1993)

$$q_b = K_b(\tau - \tau_c)^{\frac{3}{2}} \tag{6.1}$$

where

K_b : Factor dependent on the density of the sediments and the water.

The equation applied for calculation of the suspended fraction of solids in the flow on the surface is more complicated, taking into account the concentration profile in the cross-section of the flow

$$q_s = 11.6 u_* c_a a \left(2.303 log \left(\frac{30.2d}{k_s} \right) I_1 + I_2 \right) \tag{6.2}$$

where

u_*	:	shear velocity.
c_a	:	concentration at distance a from the bed.
d	:	water depth.
k_s	:	roughness height.
I_1, I_2	:	Integrals dependent on settling velocity, and flow characteristics.

The sediment modelled by the surface quality module can be divided into a fine and a coarse fraction of sediments. The input data requirements for the surface model is seen in table 6.1.

Model part	Required input
Build-up	Build-up rate (kg/ha/day) Maximum load (kg/ha) ADWP (days)
Coarse sediments	Mean grain size diameter (d_{50}) Sediment density Sediment porosity
Fine sediments	Mean grain size diameter (d_{50}) Sediment density Sediment porosity Detachment rate (m/hour)

Table 6.1: *Types of input required for the MOUSE TRAP surface module.*

The data stipulated in table 6.1 can be given in the MOUSE TRAP model as either global parameter values or associated with specific subcatchments.

Due to the variability of the sediment characteristics e.g. dependent on their location (land-use) the parameters should ideally be collected throughout the catchment. This is of course not feasible and therefore global values must be applied. In case that the sediment transport from the surface has been measured it is possible to calibrate the model by fine tuning some of the input parameters. The quality and coherence of measurements from real sewer catchments does, however, seldomly result in a basis for such a calibration. None of the references given in this chapter presents a verification of the surface model.

6.2 Sediment transport model

The sediment transport model contains four different transport formulations. One of the main aspects of the sediment transport module (ST) is to take into account the reduced hydraulic capacity of the sewer system due to the deposited sediments.

The flow resistance in the sewer system is a result of (DHI 1993):

- The resistance from the pipe material itself.
- The resistance from the sediment grains.
- The resistance from bed forms in the pipe.
- Other factors: aging effects, structural failure, biological growth.

The ST module calculates the transport of suspended load, bed load and how different bed form may appear dependent on the hydraulic conditions. Either ripples, dunes, plane bed or anti-dunes may appear. The four sediment transport formulae implemented in the ST module is:

- Ackers - White (Ackers and White 1973)
- Engelund - Hansen (Engelund and Hansen 1972)
- Engelund - Fredsøe - Deigaard (Engelund and Fredsøe 1976; Fredsøe 1984)
- Van Rijn (Rijn 1984a; Rijn 1984b)

The Ackers–White model and the Engelund–Hansen model calculates the sediment transport as a total load, whereas the remaining two models distinguishes between bed load and suspended load.

The models are based on physical interpretations of the flow resistance and the critical bed shear stress and different formulations of either total load or bed load and suspended load. As the description of the models in detail is quite comprehensive it will not be given here, but descriptions of the models and their implementation into MOUSE TRAP can be seen in DHI (1993) and Mark (1995) .

Another possibility for using the MOUSE TRAP model is to combine the surface module with transport of the sediments by the advection-dispersion module. Both modelling by use of the surface module combined with the sediment transport module and modelling combining the surface module with the advection–dispersion module will be presented in the following paragraphs.

6.3 Calculations using the ST module

The ST module is mainly aimed at modelling of coarse non-cohesive materials. The model allows for transport of graded sediment either defined by individual sediment fractions or by standard deviation of the mean particle size d_{50}. In this case the latter principle has been applied. All the four models have been applied, however, concentrating on the Engelund–Fredsøe–Deigaard model. None of the models yields satisfactory results and moreover, given the same results they do not yield results in the same range (Schlütter and Schaarup-Jensen 1998). The following paragraphs describes the input parameters used in the ST module. The input parameters, such as settling velocities etc., are as far as possible measured values from the case study cachment of "Le Marais".

The surface model requires input data regarding sediment characteristics and model parameters for the build-up model and the washoff model. The used sediment characteristics can be seen in table 6.2. As sediment detachment rate

Sediment fraction	d_{50} [mm]	Density [kg/m^3]	Porosity
Coarse sediments	1.0	2600	0.35
Fine sediments	0.1	2600	0.35

Table 6.2: *Input values for the MOUSE TRAP surface module used for modelling of the "Le Marais" catchment.*

the default value (0.001) was used. During the use of the AD module it was concluded that the power coeficient of washoff model should be 1.9 in this case, so this value was used for the ST module as well.

The build-up model requires the ADWP, but as measurements from the Le Marais catchment does not show correlation between the ADWP and the sediment transport (see appendix A) another approach is used. From measurements by CERGRENE-LABAM is was found that 22.5 % of the sediments originated from the surface. This percentage has then been used as initial loads on the surface.

The pipe model requires a number of additional input parameters. The same porosity and density was used as for the surface model. The measurements of sediment characteristics suggests the mean gain size diameter should be $d_{50} = 0.4mm$ and the standard deviation defined as d_{84}/d_{16} is 4.5. Besides sediment characteristics the following input types are possible:

- Calibration factors
- Initial dune dimensions

- Parameters for graded sediments
- Sediment distribution in nodes
- Passive pipes
- Initial sediment depths
- Model parameters

For a number of the input types indicated above conditions can be given locally for selected pipes as well as globally. This results in an immense number of possibilities of changing the input for the ST model as only a limited amount of the information can be procured by measurements.

Different attempts were carried out to make the model perform as good as possible. Mean particle sizes and the attaining standard deviation and the critical bed shear stress was variated in order to find an optimum solution. Also detailed implementation of graded sediment was tried. Results from these applications did not show a decisively best set of input parameters. The mean grain size diameter was, however, lowered to 0.1 mm in order to make the sediments move more easily.

Another variation is how to execute the model. It is possible to execute the model explicitly without updating the bed shear stress and the bed level in the hydraulic module or to take these effects on the hydraulics into account. If both changed bed shear stress and bed level conditions are taken into account in the hydraulic model, then the sediment transport modelling turn out to be very poor. If only changed bed shear stress conditions is taken into account during the modeling, the results seems better.

The result of applying the Engelund–Fredsøe–Deigaard model with update of the bed shear stress can be seen in figure 6.2 for the calibration and in figure 6.3 for the validation. The calibration procedure was carried out by determining a common scaling factor for all the five events. Subsequently this factor was used for the validation events as well. As seen on figure 6.2 and 6.3 the results are not encouraging. This can also be seen by the obtained correlation coefficients, which can be seen in table 6.3.

6.4 Calculations using the AD module

As the sediments collected by the automatic aqua-sampler at the outlet of Le Marais is fine materials it is not inconceivable that the advection-dispersion

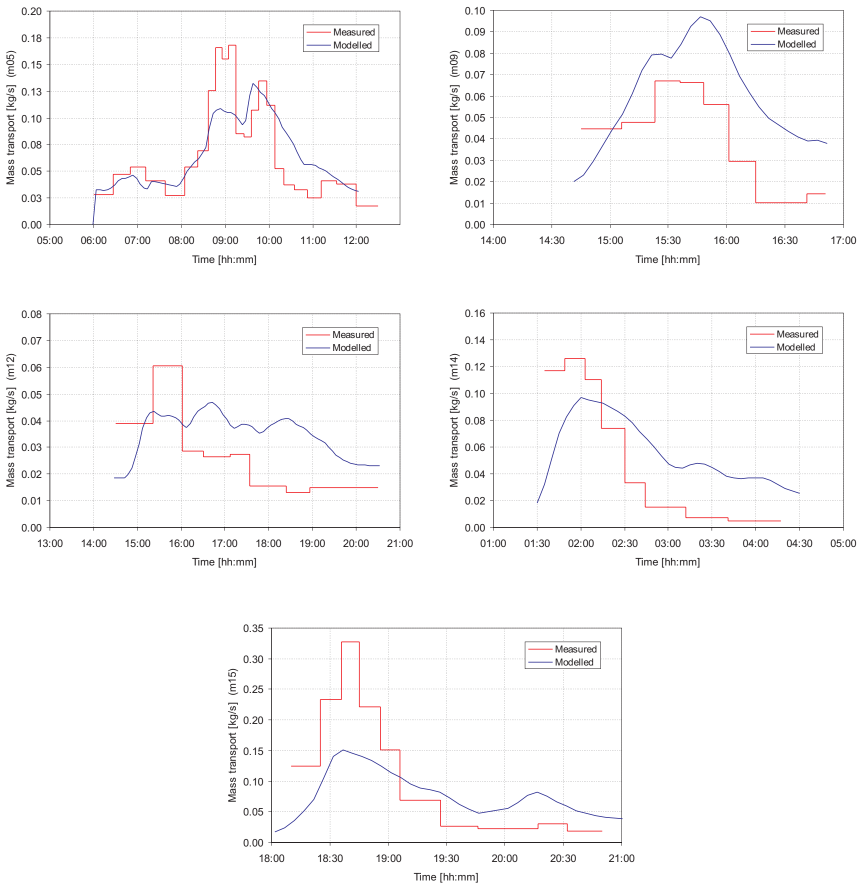


Figure 6.2: *Calibration results using the ST module of MOUSE TRAP. Upper-left: M05, Upper-right: M09, Middle-left: M12, Middle-right: M14, Lower: M15*

Event	R^2	Event	R^2
M05	0.67	M17	0.32
M09	0.47	M06	0.86
M12	0.07	M07	0.33
M14	0.51	M08	0.20
M15	0.72	M10	0.77
Average	0.49	Average	0.43

Table 6.3: *Obtained correlation coefficients.*

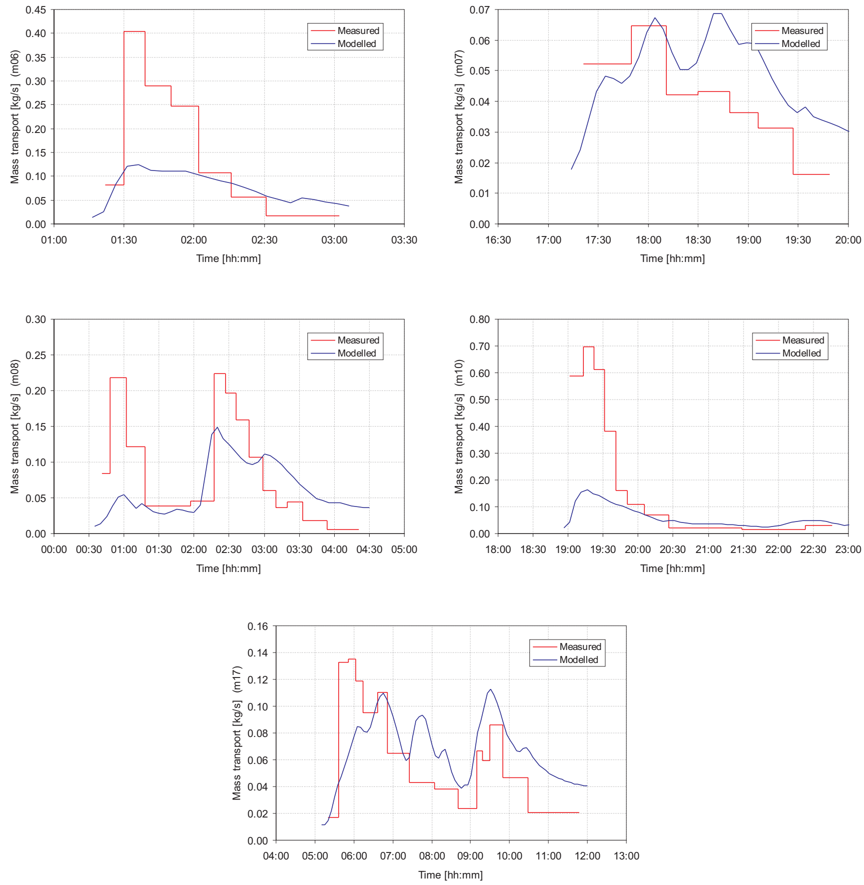


Figure 6.3: *Validation results using the ST module of MOUSE TRAP. Upper-left: M06, Upper-right: M07, Middle-left: M08, Middle-right: M10, Lower: M17*

module will be adequate for calculating the sediment transport. In the way the AD module is applied in this thesis there is no erosion and deposition of sediments taking place in the pipes. This means that a large amount of the sediments must be provided by the surface module. I.e. that the complete model used consists of a lumped conceptual model modelling the release of sediments into the system and the Advection-dispersion model transporting the sediments. This approach entails that the physical processes are not represented correctly by the model. The washoff of sediments from the surface and the resuspension of sediments in the sewer system is modelled by one and the same model, described in paragraph 6.1.

Like for the ST module the initial mass on the surfaces is not possible to access directly. In this case a procedure which is also used for the other lumped conceptual models presented in the thesis is applied. First all the five calibration events are modelled finding the optimal value of the initial mass on the surface for each event. Then a model, using the rain duration and $I10_{max}$ as input parameters is fitted to the obtained optimal values. This results in the regression model shown in figure 6.4 As seen from figure 6.4 the rain duration (T_{rain}) plays a smaller

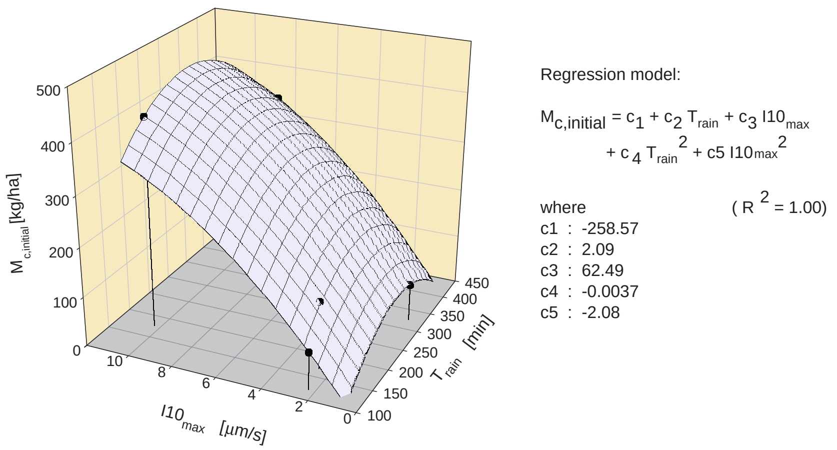


Figure 6.4: *Regression model for the estimation of the initial mass.*

role in the estimates. The optimal and estimated values of the initial mass can be seen in table 6.4.

The results of the modelling applying the AD-module can be seen in figure 6.5 and 6.6. Modelling using the AD-module yields correlation coefficients which are somewhat better than the ST-module. This can be seen in table 6.5. The correlation coefficient should only be taken as an indication of the quality of the modelling. There seem to be a good balance between the correlation coefficients

Event	Optimal $M_{c,initial}$	Estimated $M_{c,initial}$	Event	Estimated $M_{c,initial}$
M05	350	347.1	M17	206.7
M09	75	74.8	M06	332.7
M12	75	73.2	M07	60.1
M14	136	135.5	M08	455.2
M15	420	419.5	M10	488.4

Table 6.4: *Optimal and estimated values of the initial mass.*

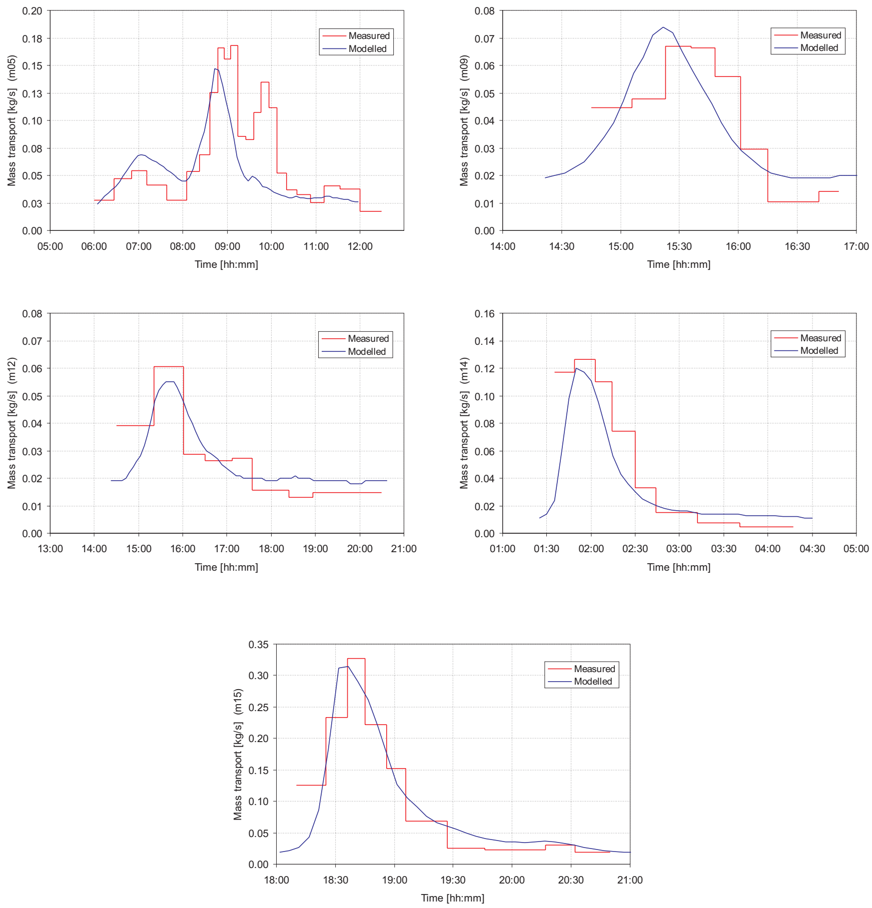


Figure 6.5: *Calibration results using the AD module of MOUSE TRAP. Upper-left: M05, Upper-right: M09, Middle-left: M12, Middle-right: M14, Lower: M15*

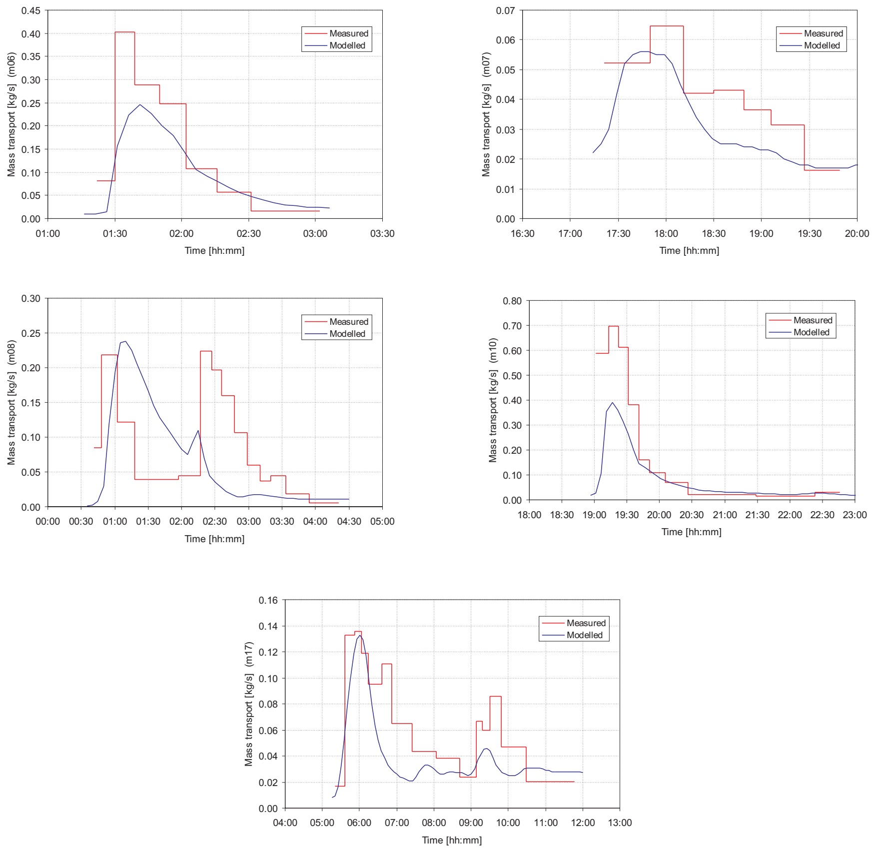


Figure 6.6: *Validation results using the AD module of MOUSE TRAP. Upper-left: M06, Upper-right: M07, Middle-left: M08, Middle-right: M10, Lower: M17*

Event	R^2	Event	R^2
M05	0.41	M17	0.60
M09	0.59	M06	0.82
M12	0.73	M07	0.73
M14	0.79	M08	0.18
M15	0.83	M10	0.76
Average	0.67	Average	0.62

Table 6.5: *Obtained correlation coefficients.*

of the calibration and the validation, indicating that little improvement is possible using the chosen model setups. The best way to judge the results, however, are to look at the graphs shown in figure 6.2, 6.3, 6.5 and 6.6.

Looking at the results from the AD-module it can be seen that multiple peaks are not modelled well. This composition of a lumped conceptual model for the surface and the AD model does clearly not represent the complex process of deposition and erosion in the sewer system. For other types of events the model performs well. The release of sediments from the surface module is based on an equally distributed load in kg/ha. The fact that this model setup works when applied on the sewer system suggests that the sediments in the sewer system itself is quite well distributed.

6.5 Summary

It can be concluded that the physically based model applied in this chapter has in general not performed well. This does not mean that the model is useless, it only shows that it is difficult to apply it with success on the chosen catchment of Le Marais.

The poor results may be due to the fact that the models are aimed at modelling other processes, i.e. modelling of solutes transport for the AD-module and sediment transport for coarse non-cohesive sediments for the ST-module. The fine sediments transported in the Le Marais catchment, with the geometrical layout of this specific catchment can maybe be considered atypical.

Another explanation for the relatively poor results of the ST module may result from not applying the optimal procedure for fine tuning the model. There is, however, not given any instructions on how to approach the modelling using this model. For the user it can be crucial, that the program documentation is adequate.

The best results were obtained combining the AD-model with a lumped conceptual model (the surface module). The approach of using conceptual models and black-box models will be investigated in the following chapters.

CHAPTER 7

Lumped conceptual modelling

When having obtained some overview of water quality modelling in sewer systems as it has been rendered in the previous chapters it is still a difficult issue to decide how to proceed with modelling of sediment transport in sewer systems. This chapter presents a new model (STSim) capable of simulating sediment transport in combined sewer systems, see also (Schlütter 1998).

7.1 Introduction to the STSim model

When choosing the approach for the STSim (Sediment Transport Simulation) model it is important to state which properties the model should be able to compute and with which precision. In this case the aim has been to be able to predict:

- The flux of sediments from a given sewer system during wet weather, depicting the temporal variation during the rain event. The pollutographs should depict the same shape as measured pollutographs without significant time lags between measured and simulated peaks and the accumulated volume of sediments during the event must be correctly reproduced.

- The build-up and erosion of sediment deposits in the sewer system making it possible to pinpoint pipe sections where sediment deposition constitutes a potential problem. It is thus "only" the aim to obtain qualitative information on sediment deposition.

In order to achieve the two objectives stated above it is unavoidable to realistically model the main physical processes going on in the sewer system at the scale of each sewer pipe. The approach of the conceptual STSim model is to copy the general concept of how the main physical processes interact and then use empirically derived statistical models to model each process, e.g. washoff, erosion and deposition. It is thus chosen to develop STSim as a lumped conceptual model.

The model is developed with emphasis put on applicability in solving practical problems concerning the discharge of sewer sediments and attaining pollutants. It should not be looked upon as a model which aims at increasing the insight into the complex processes involved. This is not the main aspect of the STSim model, as strongly indicated by the choice of LCM approach.

The usability of the model depends to a great degree on the requirement for input data. For the development of this model it is chosen not to include requirements for sediment characteristics such as particle sizes, density etc. The STSim model should be able to rely on data which are reasonably easy to procure when investigating a specific catchment and its sewer system. This entails that the following types of indata is accepted as required input for the model:

- Catchment characteristics (Area, imperviousness, surface inclination, population density etc.)
- Sewer system characteristics (location of manholes, pipes, pipe cross-section, special structures etc.)
- Rainfall data (thus also ADWP, max. intensity, rain depth, duration, etc.)
- Measured hydrographs of the discharge at the outlet of the catchment.
- Measured pollutographs of sediment transport through the cross-section at the outlet.

Most LCM's rely on simplified modelling of the hydraulics in the sewer system. This may very well be adequate when calculating total loads at the outlet. In this case spatially distributed results are desired and as a consequence of this the hydraulics (time varying discharges, velocities, waterlevels) needs to be modelled throughout the system. It is chosen to model the hydraulics using the full dynamic wave solution of the Saint Venant equations. In this case MOUSE (DHI 1997a) is chosen to act as the hydrodynamical basis for STSim. This approach of combining detailed physically based modelling of the hydrodynamics with a LCM of the sediment transport has not been investigated in depth prior to this study. The approach allows for spatial modelling of the sediment transport and will result in valuable information on the benefit of integrating detailed hydrodynamic modelling into sediment transport modelling.

One of the arguments for not choosing elaborate modelling of the hydrodynamics has been that it will decrease the applicability of the model for continuous simulation of historical rain data. Continuous modelling is important in connection with the estimation of annual loads from CSO structures. As the computer power increases and as the software for hydrodynamic modelling becomes faster and adapted for simulating historical records of rain events the stated concern becomes secondary.

7.2 Hydrodynamic modelling

This paragraph gives a brief introduction to the theory used for modelling of the hydrodynamics in the MOUSE model.

The MOUSE model is composed of different parts. One part models runoff from the catchment surfaces the other models the flow in the pipes of the sewer system. Figure 7.1 gives an illustration of the layout of the MOUSE model. It is

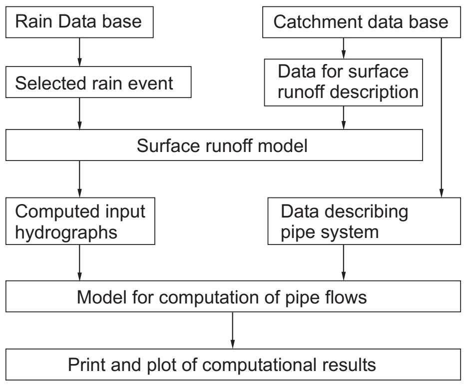


Figure 7.1: *Illustration of the structure of the MOUSE model* (from Lindberg and Jørgensen 1986).

possible to choose between two surface runoff models. The simple one relies on a time/area curve and a simple hydrological model incorporating initial loss and a catchment area reduction factor. The advanced approach uses kinematic wave theory and includes advanced hydrological modelling of evaporation, saturation of the surface, infiltration and surface magasination.

The pipe flow is routed by either a kinematic wave theory, diffusive wave theory or the complete dynamic wave theory. The three different models applies the same continuity equation seen in equation 7.1. For the impulse equation different approximations of the one dimensional St. Venant equation, where the dynamic wave applies the full equation.

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \tag{7.1}$$

$$\underbrace{\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\alpha' \frac{Q^2}{A}\right) + \underbrace{g\ A\ \frac{\partial h}{\partial x} + \underbrace{g\ A\ I_f = g\ A\ I_0}_{\text{Kinematic wave}}}_{\text{Diffusive wave}}}_{\text{Complete dynamic wave}} \tag{7.2}$$

where

Q	$[m^3/s]$	:	Discharge.
A	$[m^2]$	:	Cross-sectional flow area.
h	[m]	:	Water depth.
g	$[m/s^2]$	:	Gravity.
x	[m]	:	Distance in the flow direction.
t	[s]	:	Time.
α'		:	Velocity distribution coefficient.
I_0		:	Slope of the sewer.
I_f		:	Friction slope.

In pipe networks where loops and backwater does not occur it is possible to use the kinematic wave approximation. This entails increased calculation speed, but it is not really required. With present computer power there seem to be little reason not to use the complete dynamic wave model.

The pipe flow model is solved numerically using a six-point implicit finite difference scheme (DHI 1997b; Abbott and Basco 1989). Water levels and discharge is solved at alternating nodes in a number of nodes in each pipe branch.

The MOUSE model needs various input data in order to run. For the surface model the indata in question is:

- Subcatchment area.
- Imperviousness.
- Type of land-use.
- Initial loss.
- Hydraulic reduction factor.

- Type of time-area curve (rectangular, divergent, convergent).
- Time of concentration.
- Time series of rain intensity.

The pipe model relies on further indata, where the main indata is:

- Location of manholes and pipes.
- Location of special structures (basins, weirs, pumps, etc.) and the geometrical layout of these structures.
- Cross-section, roughness, and diameter of the pipes.
- Diameter, form of outlet, level of the top, and level of the bottom of manholes.
- Boundary conditions, e.g. Q-H relation at the outlet and surface runoff hydrographs.

Even though, it may seem difficult to use the MOUSE model at first, most of the indata has a static nature, that does not need changes from rain event to rain event. The model must be calibrated based on measured and simulated discharges from the sewer system.

The application of the model to the case study site is seen in paragraph 7.13.

7.3 Sediment build-up on the surface

It is generally agreed that the build-up of sediments on the surface goes on until a maximum limit is reached. At the same time a decay of sediments occurs due to wind and decomposition. A large number of models have been suggested for this process. The equation implemented in STSim describing an exponential asymptotic build-up, equal to equation 3.5, reads

$$\frac{dm_c}{dt} = \alpha_{acc} - \alpha_{rem}\ m_c(t) \tag{7.3}$$

where

α_{acc}	[kg/ha/day]	:	Rate of accumulation.
α_{rem}	[1/day]	:	Decay factor.
$m_c(t)$	[kg/ha]	:	Mass deposited on the surface at time t.

In principle, it is necessary to find the calibration factors α_{acc} and α_{rem} for each catchment through sampling programmes. Application of STSim aims at using global values for the whole catchment.

During the application process of STSim different build-up equations were evaluated, but equation 7.3 is used due to its logical division into a constant rate deposition process and a first order removal process. The build-up is, however, not important for the model. STSim is not aimed at continuous modelling between events, with the current model layout. The build-up process does, though, proceed during the rain event as well.

7.4 Sediment washoff from the surface

The washoff process is quite complicated. At initiation of rain the surface and the sediments are wetted and subsequently sediments are washed off the surface into gully pots and into the sewer. The washoff rate depends on the characteristics of the surface and the sediments and the discharge of water. A number of models relate the washoff with the rain intensity. In this case a model is applied which depends on the discharge from the surface

$$\frac{dm_c}{dt} = -\alpha_{w1}\ Q_c(t)^{\alpha_{w2}}\ m_c(t) \tag{7.4}$$

where

α_{w1},α_{w2}		:	Numerical coefficients.
$Q_c(t)$	[m^3/s]	:	Runoff discharge from the subcatchment.
$m_c(t)$	[kg/ha]	:	Mass deposited on the surface at time t.

7.5 Implementation of the surface model

Implementation of the surface model is simple. The model computes timeseries of deposition, washoff and mass on the surface for each subcatchment. Also, totals for the whole catchment of deposition, washoff and mass is saved.

Discretization of the differential equations (7.3) and (7.4) is done using the improved Euler method (or Heun's method), which technically probably is more accurate than absolutely necessary, but the method offers nice opportunity to exploit the intermediate results. For the surface model the following algorithm is thus used

for $i = 1$ to number of catchments do
begin
  for $n = 0$ to number of timesteps -1 do
  begin

$$\left.\begin{aligned} depk1 &= \Delta t\,(\alpha_{acc} - \alpha_{rem}\; m_{c\,i}^{\,n}) \\ depk2 &= \Delta t\,(\alpha_{acc} - \alpha_{rem}\; (m_{c\,i}^{\,n} + depk1)) \\ deposition_i^{n+1} &= \tfrac{1}{2}\,(depk1 + depk2) \end{aligned}\right\}\text{deposition process}$$

$$\left.\begin{aligned} washk1 &= \Delta t\,(-\alpha_{w1}\;(Q_{c\,i}^{\,n})^{\alpha_{w2}}\; m_{c\,i}^{\,n}) \\ washk2 &= \Delta t\,(-\alpha_{w1}\;(Q_{c\,i}^{\,n+1})^{\alpha_{w2}}\;(m_{c\,i}^{\,n} + washk1)) \\ washoff_i^{n+1} &= \tfrac{1}{2}\,(washk1 + washk2) \end{aligned}\right\}\text{washoff process}$$

$$m_{c\,i}^{\,n+1} = m_{c\,i}^{\,n} + deposition_i^{n+1} + washoff_i^{n+1}$$

  end
end.

7.6 The pipe model

In a sewer system consisting of pipes, manholes, various structures, etc. the sediment transport is an extremely complex physical process. Therefore, it is not considered feasible to build detailed numerical models able to compute the sediment transport at a given time and place in the sewer system. It is thus necessary to simplify the model. When every single physical subcomponent is not represented totally correct in the model it becomes necessary to calibrate the model. The more simple the model gets the fewer calibration factors are needed as each factor deals with the combined effect of a number of subprocesses. At the same time the level of detail in the (and abundance of) results decreases. How simple or conceptual to make the model is thus a balance of:

- The number of calibration constants which are feasible to determine on the basis of the measurements.
- Which resulting information are needed and with which precision.

The model considers one pipe at the time and all results, bed level, discharge, concentration, erosion, deposition, deposited mass, and suspended mass, are conceived as mean values for this pipe. At each time step water and sediments flows into the pipe from upstream and possibly from a surface through the upstream manhole. Additionally sediments enters the pipe from inhabitants on the subcatchment connected with the upstream manhole. Likewise, sediments are transported out of the pipe each timestep. In the pipe three sediment process modes may occur: Either sediments are deposited, or they are resuspended or neither deposition or resuspension occur.

Which of the three modes occur in a certain pipe at a certain timestep depends on the level of the bed shear stress (τ). The bed shear stress is calculated without any regard to the fact that there may be deposits on the bottom of the pipe increasing the bed shear stress. MOUSE returns one value of the discharge for each pipe and each timestep. This value may be located at the midway point in the pipe. In the implemented sediment transport model the discharge is considered to be representative for the discharge in the whole pipe.

Calculation of the bed shear stress is done according to

$$\tau = \rho_w g R_h \mid I_0 \mid \tag{7.5}$$

where

τ	[N/m^2]	:	Bed shear stress.
ρ_w	[kg/m^3]	:	Specific gravity of the water.
g	[m/s^2]	:	Gravitational acceleration.
R_h	[m]	:	Hydraulic radius.
I_0		:	Bottom slope of the pipe.

The hydraulic radius is calculated from the obtained results from MOUSE. An alternative possibility is to calculate R from the roughness of the pipe, the full flowing capacity and the discharge using Brettings formula in the case of circular pipes. This alternative entails that water level results from MOUSE is not needed for STSim, which can be an advantage with regard to computational memory and speed as handling of indata are time consuming. For non-circular pipes hydraulic radius calculations from MOUSE is used to interpolate the hydraulic radius.

Depending on the critical shear stress for erosion (τ_{ce}) and the critical shear stress for deposition (τ_{cd}) the three modes occur as following

$$\begin{array}{ccccccl} & & \tau & \leq & \tau_{cd} & \Longrightarrow & \text{Deposition.} \\ \tau_{cd} & < & \tau & \leq & \tau_{ce} & \Longrightarrow & \text{No deposition or resuspension.} \\ & & \tau & > & \tau_{ce} & \Longrightarrow & \text{Erosion.} \end{array}$$

In the succeeding paragraphs the deposition model and the erosion model is described.

7.7 Deposition model

Deposition in the pipe depends on the hydraulic conditions, the characteristics of the sediments (particle sizes and weight), the bed shear stress, the available space for deposition and the concentration of suspended solids. All these factors cannot be incorporated in the model used in this case. If the hydraulic conditions indicated by the bed shear stress allows for deposition and sediments in suspension are available the deposition process follows equation (7.6) or (7.7).

$$\frac{dm_p}{dt} = \alpha_{dep} \left(\alpha_{sc} - m_p(t) \right) \tag{7.6}$$

where

$m_p(t)$	[kg/m]	:	Mass per meter pipe.
α_{dep}	[1/day]	:	Deposition rate.
α_{sc}	[kg/m]	:	Storage capacity given as a fraction of the pipe volume per meter multiplied with a computational sediment bulk density (bulk density is defined as the ratio of the mass of dry solids to the bulk volume of sediments occupied by those dry solids).

It is remarkable that the above model does not directly depend on the actual level of the shear stress neither of the concentration in the suspension. Therefore, an alternative model has been implemented dependent on the bed shear stress relative to the critical bed shear stress for deposition.

$$\frac{dm_p}{dt} = \alpha_{dep} \left(\alpha_{sc} - m_p(t) \right) \frac{\tau_{cd} - \tau}{\tau_{cd}} \tag{7.7}$$

7.8 Erosion model

The first erosion or resuspension model used in STSim is very simple and only includes one calibration coefficient as seen in equation (7.8). This means that a number of physical phenomena are combined and that e.g. rheology effects are ignored.

$$\frac{dm_p}{dt} = -\alpha_{ero} \, m_p(t) \, Q_p(t) \tag{7.8}$$

where

α_{ero}	[1/m^2]	:	Pipe erosion rate.
$Q_p(t)$	[m^3/s]	:	Discharge in the pipe at time t.

The second implemented model is identical to the first but is multiplied with a factor dependent on the bed shear stress relative to the critical bed shear stress for erosion. In this way erosion rates are higher when the limit (τ_{ce}) is just exceeded than at higher levels of bed shear stress. This is aimed at modeling the effect of fine sediments being more readily erodable in the start of the event. With formula (7.9) this is of course also the case when bed shear stress decreases but at that time available mass is less and therefore the amplification smaller.

The second erosion rate formula is seen in equation 7.9.

$$\frac{dm_p}{dt} = -\alpha_{ero}\, m_p(t)\, Q_p(t) \left(0.0001 + (1 - 0.0001)\left(1 - \frac{\tau - \tau_{ce}}{\tau + \tau_{ce}}\right)^{\alpha_{ero2}}\right) \quad (7.9)$$

where

α_{ero2} [-] : Numerical value between one and zero.

The variation of the extra term on equation (7.9) compared with (7.8) can be seen in figure 7.2.

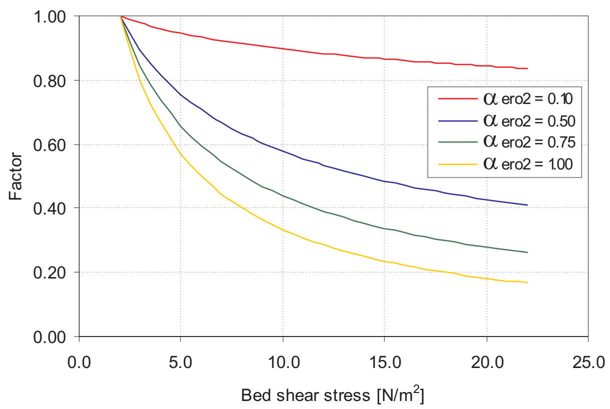


Figure 7.2: *Illustration of the adjustment factor in equation (7.9) ($\tau_{ce} = 2.0$).*

Finally a third erosion formula is available

$$\frac{dm_p}{dt} = -\alpha_{ero}\, m_p(t)\, Q_p(t)^{\alpha_{ero2}} \quad (7.10)$$

If no material is available for erosion no material is resuspended, of course. The consequence of using equation 7.9 or 7.10 is an additional calibration constant. This means that the two models must be significantly better in order to compensate for the inconvenience of complicating the model.

7.9 Computation of concentrations

Considering the level of detail in the conceptual model STSim the calculations of concentrations will evidently result in quite rough estimates. The principle in the calculation is to make a mass balance for each pipe where the inflow of sediments depends on the concentration of the wastewater in the upstream pipe, washoff of surface sediments into the upstream manhole and the additional sediments from domestic and industrial activity (*dis*). Transport out of the pipe depends on the concentration in the actual pipe. It is assumed that the sediments move with the same velocity as the transporting water. When the mass of sediments in suspension for the next timestep is calculated the concentration is derived by division with the water volume, thus assuming that the suspension is fully mixed. Figure 7.3 attempts to illustrate the mechanism behind the calculation and states the continuity equation for pipe stretch i. An effect of

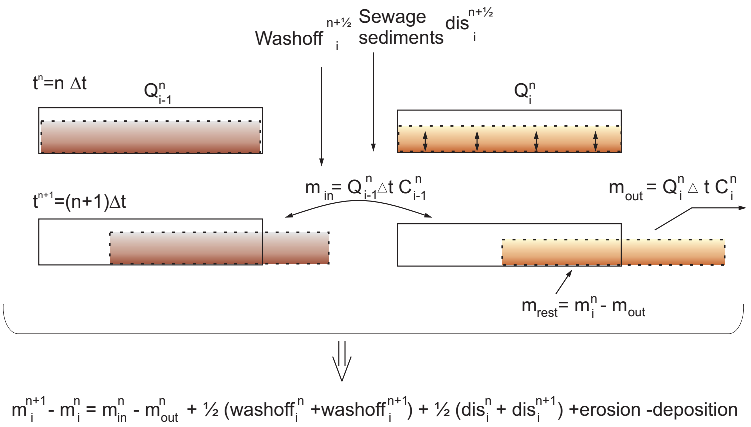


Figure 7.3: *Illustration of suspended mass balance calculation.*

calculating the concentration in this fashion is that an impulse of sediments will be dampened and distributed in several pipes due to the mixing condition even under steady state hydraulic conditions thus resulting in diffusion which is not directly included in the formulae.

As illustrated in figure 7.3 the following discretisized equations is applied when calculating the concentration (C) from the suspended mass in the pipe m_{ps}

$$\begin{aligned} & m_{ps\,i}^{n+1} - m_{ps\,i}^{n} = \Delta t\ Q_{p\,i-1}^{n} C_{i-1}^{n} - \Delta t\ Q_{p\,i}^{n} C_{i}^{n} \\ & + washoff_i^{n+\frac{1}{2}} + dis_i^{n+\frac{1}{2}} + erosion_i^{n+1} + deposition_i^{n+1} \end{aligned} \tag{7.11}$$

$$C_i^{n+1} = \frac{m_{ps\,i}^{n+1} \quad U_{p\,i}^{n+1}}{Q_{p\,i}^{n+1}} \cdot pipelength_i \tag{7.12}$$

The terms $washoff$, dis, $erosion$, and $deposition$ have been multiplied with Δt in the same fashion as shown for the discretization shown in paragraph 7.5. The biggest advantage of the approach in (7.11) is that upstream information at time $n+1$ is not required (explicit model) so that the complex task of composing the correct calculation sequence in the pipe network is avoided.

7.10 Implementation of the pipe model

Like for the surface model the improved Euler method is used for solving equation 7.6 and 7.8.

The domestically derived sediments (dis) is implemented as an equally distributed amount per day. It is evident that this contribution needs to be varied as a diurnal cycle and furthermore a variation around the mean value would be easy to implement. For simulations during wet weather it is though not deemed necessary, but in case of a online version of STSim this would become necessary.

Sediments are transported instantly through manholes. In case of a converging set of pipes to one manhole sediments are accumulated, but of course the rate of transport may not be the same in the different pipes meeting in the manhole. Where more than one pipe exerts downstream from the manhole, the amounts of sediments are distributed according to the distribution of the flow.

The STSim model is implemented in Delphi/Pascal and the use of the model is described in appendix B.1. When using the model the following set of indata files needs to be prepared, besides the *.CFL file for calculations with the surface model:

*.MIN : Text version of the catchment file from MOUSE called *.SWF.

*.MIF : File produced by user containing model constants, such as calibration factors.

*.PIN : File containing pipe slopes and lengths and full pipe flow capacities.

*.DFL : Time series of discharges in the pipes, subtracted as a text file from the *.PRF file produced by MOUSE.

*.VFL : Time series of velocities in the pipes, subtracted as a text file from the *.PRF file produced by MOUSE.

*.WFL : Time series of water levels in the pipes, subtracted as a text file from the *.PRF file produced by MOUSE.

CBFfile.txt : File containing data for special cross-sections.

Most of the model boundary conditions is given by the conditions originating from the setup and computations using MOUSE. The *.MIF file contains information about initial and boundary conditions related to the sediment transport. The file contains all the calibration constants and initial surface load, initial pipe load, initial wastewater concentration and daily PE discharge of solids. The initial pipe load can be given as a percentage of the storage capacity or this value multiplied with the relative steepness of each pipe.

7.11 Sensitivity analysis

The sensitivity analysis has in this case two objectives. First of all, the sensitivity analysis is performed in order to determine which effects changes made to input and calibration parameters has on different output characteristics of the model. Secondly, the analysis has been used to eradicate the last implementation errors, however, no significant errors were found. Selected results from the analysis are shown in this paragraph.

When performing a sensitivity analysis a number of model constants are varied. As indicated there are about ten model constants in STSim which could be included in the analysis. If for instance each of these parameters where changed between four levels and a complete set of computations were to be carried out, this would implicate 1.048.576 executions of the model. Therefore, it is necessary to limit the chosen combinations and in this case the analysis is not performed on the "Le Marais" catchment but on a smal catchment and therefore less complicated model.

This artificial sewer system includes one catchment (2 ha) and ten downstream circular pipes towards an outlet. Furthermore, a simple box-formed impulse of rain has been used for calculations. Figure 7.4 shows the different hydraulic conditions used for the simulations.

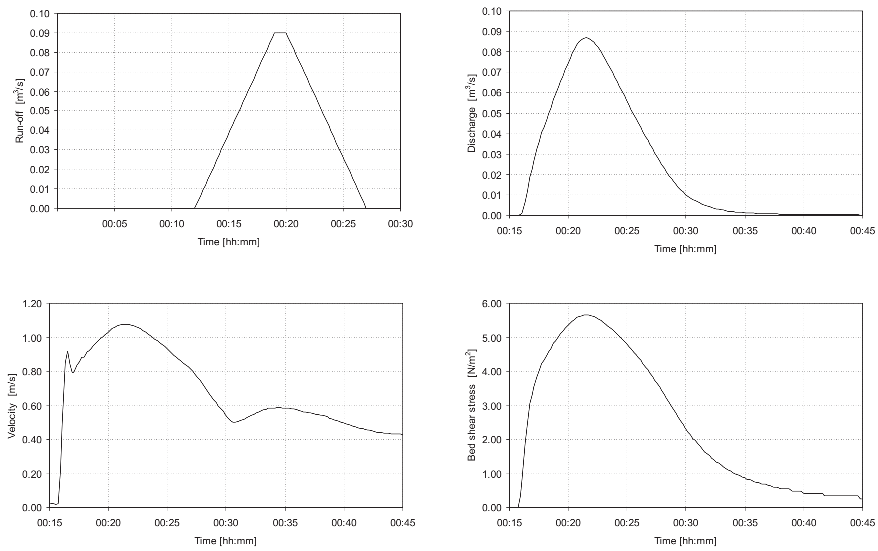


Figure 7.4: *Hydraulic conditions used for sensitivity analysis.*

First the surface washoff model has been considered. The washoff coefficient (α_{w1}) and the initial mass on the surface ($M_{c,initial}$) was alternated between two levels (100 kg/ha and 25 kg/ha) and the washoff exponent (α_{w2}) were changed in the range of 0.5 to 3.5. Peak values and accumulated washoff were monitored. Figure 7.5 shows the accumulated washoff dependency on the washoff exponent. The curves with $M_{c,initial} = 100kg/ha$ and $M_{c,initial} = 25kg/ha$ shows that the initial mass acts as a scaling factor, but does not change the nature of how the sediments are washed of the surface.

The erosion process in the sewer during rain events is the foremost important. Figure 7.6 and 7.7 shows the influence of the accumulated transport and the maximum transport rate respectively, when changing erosion rate (α_{ero}), see equation 7.8, and the initial mass parameter ($M_{p,initial}$).

Like for the surface the initial mass parameter acts as an almost linear scaling factor. The dependency on the erosion rate shows that eventually there is a pronounced effect of the limited availability of sediments for erosion.

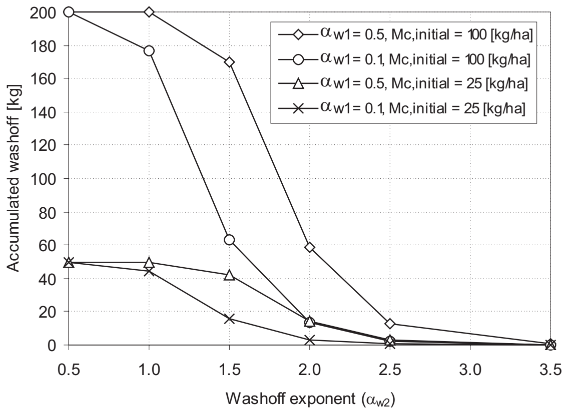


Figure 7.5: *Washoff as a function of washoff exponent.*

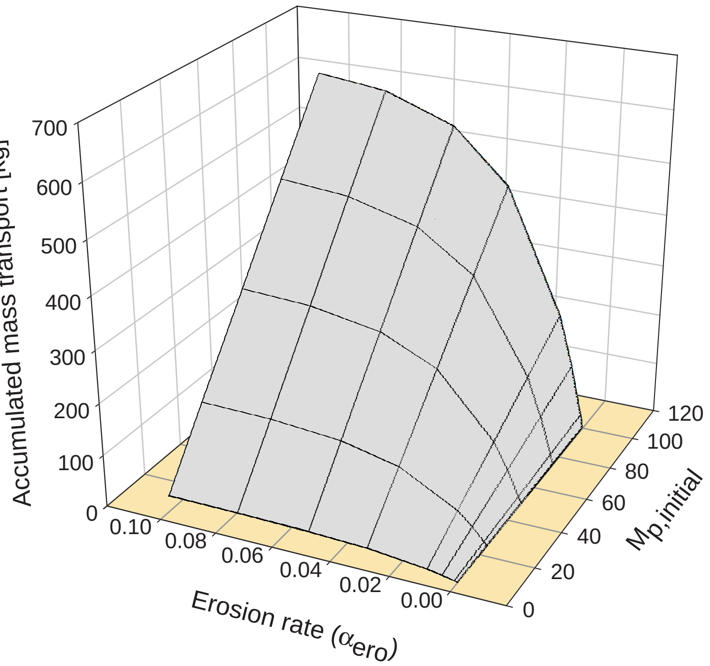


Figure 7.6: *The accumulated outlet mass transports dependency on* α_{ero} *and* $M_{p,initial}$.

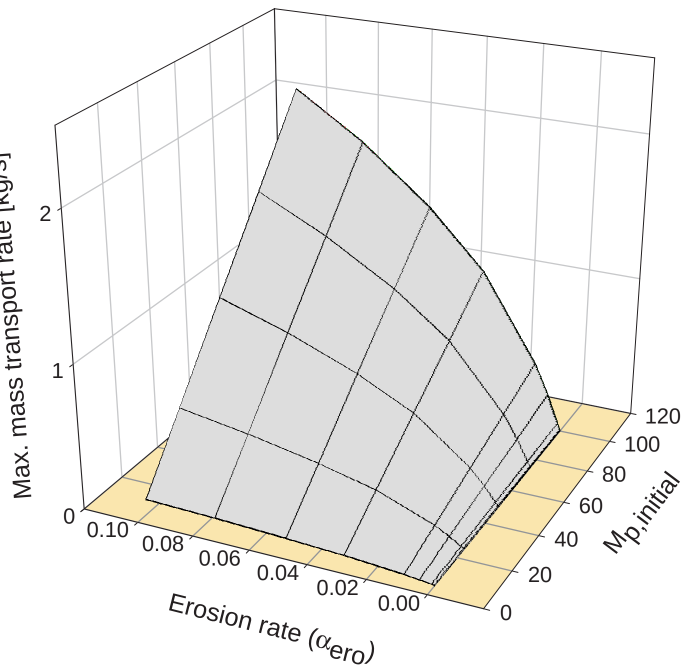


Figure 7.7: *The maximum outlet transport rates dependency on α_{ero} and $M_{p,initial}$.*

Variation of the critical bed shear stress for erosion results in the plot shown in figure 7.8. Figure 7.8 shows a good coherence with the calculated bed shear

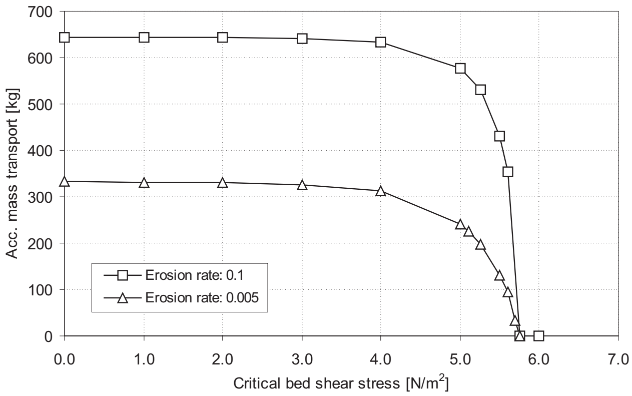


Figure 7.8: *Accumulated mass transports dependency on* τ_{ce}.

stresses shown in figure 7.4.

As the sensitivity analysis to some extent has been carried out unsystematically the relative importance of the individual parameters cannot be calculated. Table 7.1 therefore, presents the importance of the parameters based on experience from the sensitivity analysis and from general use of the model. As seen in table

Model part	Parameter	Estimated importance
Sewer system	$M_{p,initial}$	*****
	α_{ero}	****
	τ_{ce}	****
	α_{dep}	***
	τ_{cd}	***
	$M_{c,initial}$	*
Surface model	α_{w1}, α_{w2}	*
	α_{acc}, α_{rem}	-
dis	DWF sediments [kg/PE/day]	*
	Initial wastewater conc.	-

Table 7.1: *Parameter importance.*

7.1 the number of imperative parameters are five. Two of these parameters, namely τ_{ce} and τ_{cd}, are interdependent and physically based. This means that their values are more easy to estimate. The importance of the parameters entails that it may be possible to calibrate the model successfully exploiting a relatively small set of storm events.

7.12 Case study calculations and results

The following paragraphs (7.13-7.15) renders a case study using the STSim model. It is by using the model and presenting the results a reasonable grasp of the quality of the model can be achieved. This chapter only presents a manual calibration of the model. Chapter 8 will use a semi-automatic calibration procedure and explore the influence of modelling the system with a reduced level of detail compared with this chapter.

7.13 Modelling of the hydrodynamics

The hydraulic modelling of "Le Marais" is done using the MOUSE 3.41 model (DHI 1997b) from the Danish Hydraulic Institute. Thirteen rain events, most of them also used for sediment transport modelling (see table 4.4), and the DWF have been used a calibration. All events have been calibrated individually in order to obtain the best possible modelling of discharges and velocities at the outlet. The difference between the events, with respect to modelling, is a result of changing the hydraulic reduction parameter. For a common calibration of all rain events more events are necessary. Even though such hydraulic measurements are available they have not been used.

The hydraulics are modelled by first modelling the surface runoff and subsequently the hydraulic conditions in the sewer system.

The MOUSE model contains two surface models, model A which uses the time-area method and model B, which is a non-linear reservoir model (kinematic wave model) (DHI 1997b). Model A results more or less in runoff curves similar to the rain intensity, slightly dependent on the chosen time-area curve which indicates the shape of the catchment. For "Le Marais" a divergent catchment shape showed the best results comparing with measured runoff. Model B relies on calculating the effective precipitation as precipitation minus evaporation, surface saturation, infiltration and surface storage. Subsequently, the runoff is calculated based on a kinematic wave setup of the St. Venants equations.

The catchment surface can be described simply by the total area, and the percentage of imperviousness. Otherwise, a detailed description can be given furthermore requirering percentages of roof, semi-permeable and permeable area within the subcatchment. Model B and a detailed surface description was also applied for calibration of the surface model, but despite great effort the resulting hydrographs did not show as good a correlation with measured runoff as model A with a simple surface description.

The used version of MOUSE is able to predict DWF as "numerical water" has been minimised. In this case the model is run with a minimum water depth of 1 mm. DWF is added at each node with an inhabited subcatchment attached. The added flow depends on a diurnal cycle given as input. The model calculates water levels, discharges and velocities in a number of nodes on each pipe branch. The input of boundary conditions and initial conditions are extensive as indicated in the following paragraphs.

7.13.1 Geometrical input data for MOUSE 3.41

Geometrical data concerning the layout of roads and buildings, the sewer network described by pipes and their slopes, cross-sections etc. has been obtained from maps dating back to the 1940'ties as well as recently updated information. Below, in figure 7.9, is seen a small scaled image of the sewer system. This map were digitized and 153 nodes identified by changes in pipe direction or pipe intersections or cross-section changes or inlets of rain water. Subsequently, the area was divided into 153 subcatchments, one for each attaining node. The coordinates of the nodes and the area of each catchment was determined. Pipe connections between nodes results in 193 pipe sections each with a specific location, slope, cross-section, wall roughness etc. Eleven different cross-sections has been digitized for use with the MOUSE model. The most common cross-sections can be seen in figure 4.5. Table 7.2 shows the main geometrical parameters required to compile a MOUSE model.

Surface	Manholes	Pipes
Node name	Node name	Upstream node name
Area	x-coordinate	Downstream node name
Inclination	y-coordinate	Invert level upstream
Population density	Invert level	Invert level downstream
Imperviousness	Ground level	Cross-section
	Outlet shape (headloss)	Cross-section size
	Manhole diameter	Wall roughness

Table 7.2: *Main parameters for MOUSE model.*

As everybody must realize working with MOUSE modelling or using similar models requiring detailed catchment information it is decisive to check and recheck

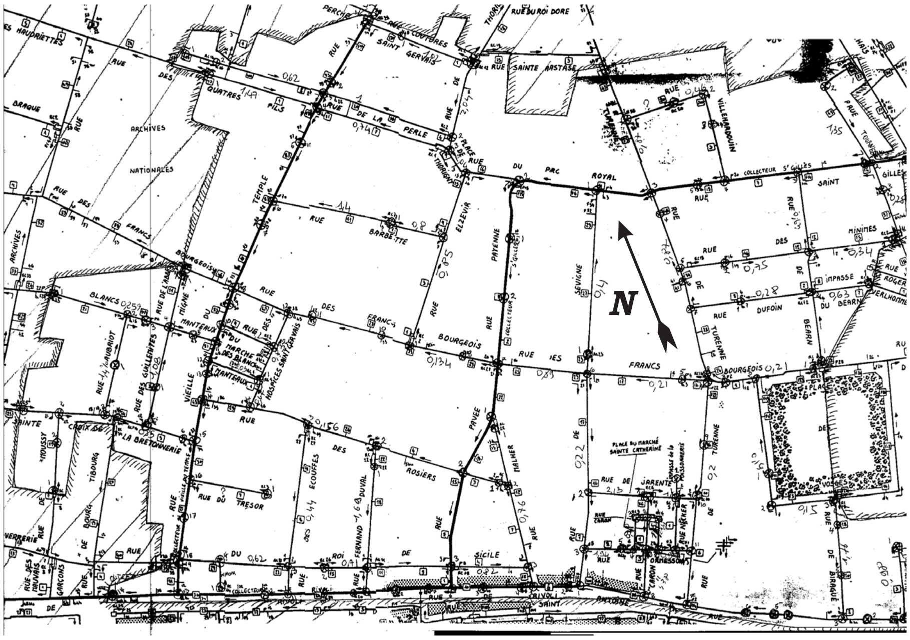


Figure 7.9: *Map of sewer system.*

that data has been entered correctly. This task has been taken on with great care in the case of the "Le Marais" model. The time used for input of data and validation has been estimated to at least 120 man hours.

7.13.2 Runtime boundary conditions

Three different time dependent boundary conditions have been used for the MOUSE model. First of all the rain intensity in μm/s as a function of time. Secondly, the earlier mentioned diurnal cycle. The discharge cycle seen in figure 4.6 was scaled and forwarded half an hour in time in order to take into account the mean transport time of the system. The last time dependent boundary condition is a prescribed discharge-water level (Q-H) relation at the outlet of the catchment. It was at an early stage of hydraulic modelling realised that the velocity of the wastewater / stormwater at the outlet is very dependent on the configuration of the downstream sewer system. In order to avoid modelling the rest of the Parisian sewer system the Q-H boundary was implemented. Based on all measured data a Q-H relation can be fitted. The data and the fitted relation is seen in figure 7.10. The figure also shows the relation actually implemented.

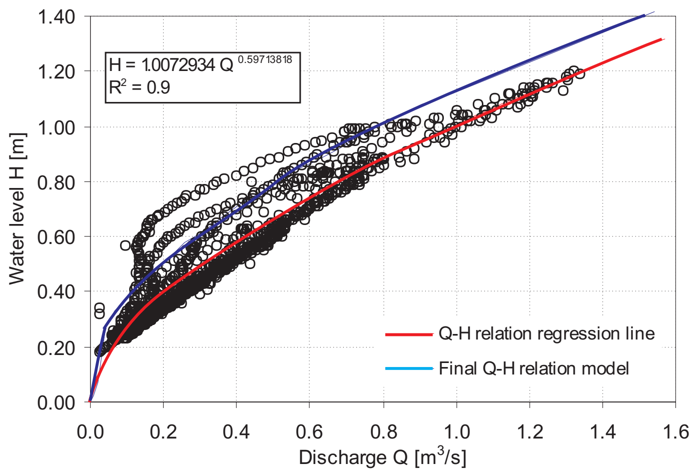


Figure 7.10: *General Q-H relation for the outlet node in "Le Marais".*

The reason why the implemented relation works better than the measured must be a consequence of the numerical modelling. As indicated on figure 7.10 the data shows in some cases a hysteresis effect, i.e. following another Q-H relation during the decrease of the rain event than during the increase. This is due to different pressure gradient conditions when the water level is rising compared to when it is decreasing.

7.13.3 Results from hydraulic modelling

For dry weather conditions measured and simulated curves of discharge is shown in figure 7.11. It is pleasing but not surprising that the model simulates the DWF

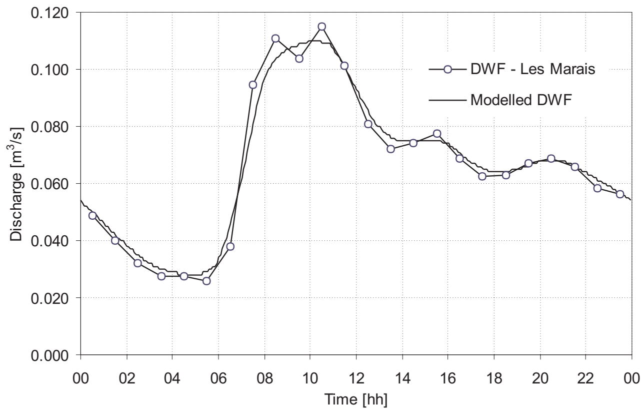


Figure 7.11: *Measured and simulated DWF.*

well as it is strongly guided by the input of the diurnal cycle. The level of the discharge should be noticed though. Comparing these levels with the discharge rates during the rain events shows the importance of including the DWF in the modelling of densely populated areas especially when the water consumption is as high as it is in Paris. If DWF is compared with a low intensity rain as "961127" it is seen that for the most part of the rain event the DWF constitutes more than 35 % of the total flow.

In order to make some general remarks about the hydraulic performance of the model the curves of discharge, water level and velocity are shown for rain event "97050604" in figure 7.12 till figure 7.14. Comparing with the other modelled events the event "97050604"is far from the best modelled event. As it is generally the case the discharge from the sewer system is modelled with high precision indeed. There is a tendency that the simulated data vary slightly less than the measured. Velocities are not only dependent on the flow and the local geometry of the sewer but also on the downstream conditions which may

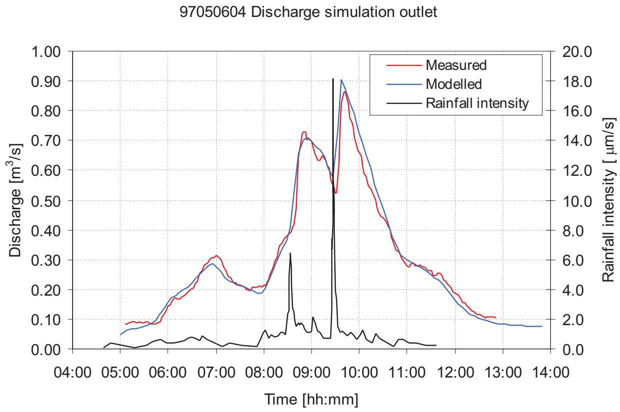


Figure 7.12: *Measured and simulated discharge at the outlet.*

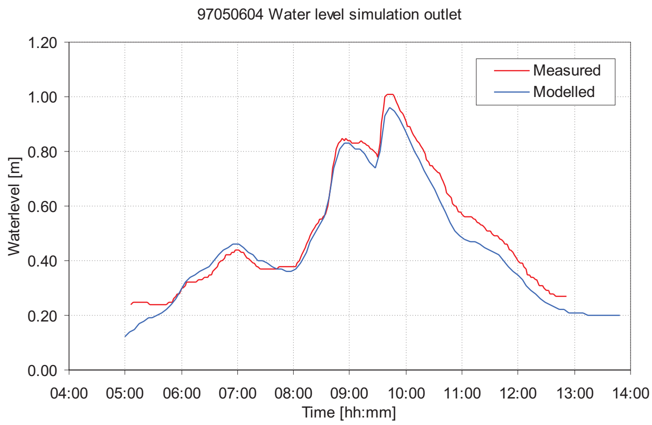


Figure 7.13: *Measured and simulated water level at the outlet.*

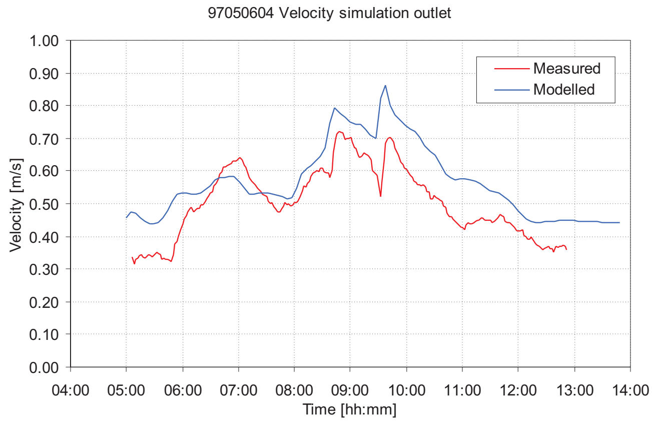


Figure 7.14: *Measured and simulated velocity at the outlet.*

encompass backwater. Therefore, the downstream conditions will not be the same for every rain event. This is the reason why velocities are modelled less precisely in some events than others as the same outlet boundary conditions is used.

7.13.4 The influence of permanent deposits on the hydraulics

The sewer system of "Le Marais" contains a big volume of quasi permanent coarse sediment deposits. These sediments are only moved during extreme rain events and are mainly located in the three main sewers (Vielle du Temple, St. Gilles and Rivoli). A plot showing the longitudinal profile of sediment deposits in the Vielle du Temple sewer, which holds the largest deposits, can be seen in figure 7.15. As seen most sediment deposits are located upstream. As the sewer is 2.4 m heigh in Vielle du Temple it is never full flowing eventhough there is a considerable deposition of sediments in the sewer. In order to investigate the influence of the presence of these sediments on the hydraulics, modelling has been carried out using the MOUSE ST module (DHI 1993). Deposits were given as initial conditions and fixed during calculations. The conclusion from these calculations is that there is no significant change of the flow conditions at the outlet except for a slight increase of velocities. Figure 7.16 shows the discharge at the outlet and in Vielle du Temple from calculations with and without sediment deposits. As seen the discrepancies however small is larger locally at Vielle du Temple where the sediments are located. Whether local changes of hydraulic conditions changes the overall characteristics of the modelling of the sediment

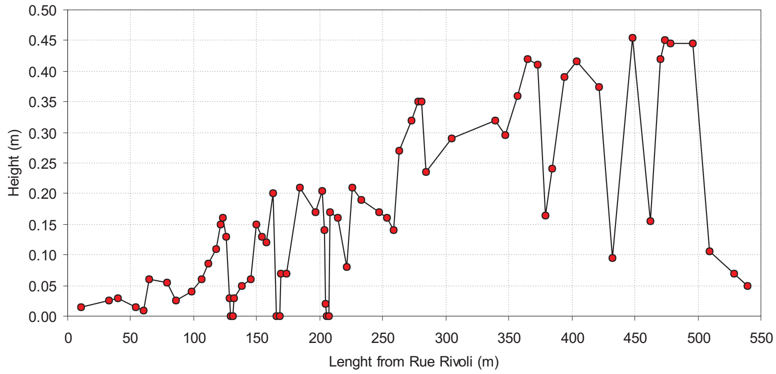


Figure 7.15: *Measured sediments deposits profile in Vielle du Temple.*

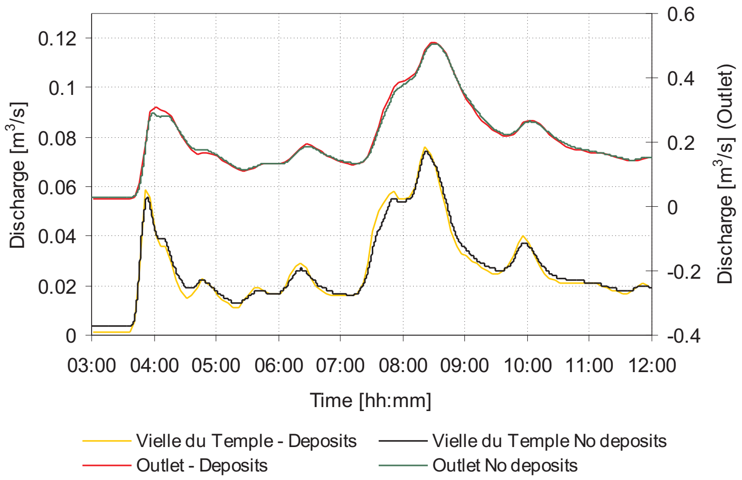


Figure 7.16: *Illustration of the influence of deposits on discharge.*

transport is not feasible to investigate. Though feasible to implement into the sediment transport model carrying out the hydraulic calculations is a very time consuming task due to low calculation speed of the MOUSE ST module.

7.14 Calibration and validation of STSim

If all processes leading to the required results concerning the sediment transport were independent and feasible to measure it would still be hard work to calibrate a model like STSim though comparably easy. As seen e.g. by the sensitivity analysis (see paragraph 7.11) the processes involved are not independent. Processes and thus calibration parameters are interdependent and as limited calibration data is available the calibration becomes intricate. In STSim there are about ten calibration parameters. In this case global initial conditions such as mass per ha. on the surface and the amount of readily erodable deposits in the sewer is included as calibration parameters. If all calibration events reacts in the same direction by changing an input parameter it should be possible to perform a reasonably good calibration manually if at all feasible. On the other hand an automated and optimised procedure for carrying out the calibration may yield better results.

7.14.1 Approach for manual calibration

Whether carrying out an automatic calibration or a manual one the first task is to choose calibration events. In this case 12 rain events were available with attaining measured pollutographs. Two events were subsequently disqualified for calibration. One due to lack of measurements during the middle of the event and one due to an unexplainable time lag between flow and mass transport. Five events were chosen by random for the calibration set. Most often, the calibration set is larger than the validation set, but in order to reduce calculations a relatively small calibration set was chosen. The five events comprises different types of rain events with variating rain intensity duration and different number of flow peaks.

The five events are labeled M05, M09, M12, M14, M15 (see table 4.4). It has been decided to calibrate against the mass transport rate at the outlet. Main entities for the calibration can be seen in table 7.3 where also the target values for the validation can be seen for later use. During calibration three entities have been monitored, namely the maximum transport rate, the accumulated transport for the event and the correlation coefficient R^2 between simulated and measured mass transport. Subsequently the deviation of these values from the target values in table 7.3 have been calculated as well as the following error

Event	$Mt_{ma,max}$ [kg/s]	$Mt_{ma,accu}$ [kg]	Event	$Mt_{ma,max}$ [kg/s]	$Mt_{ma,accu}$ [kg]
M05	0.163	1368	M17	0.135	1304
M09	0.066	298	M06	0.403	797
M12	0.062	573	M07	0.065	368
M14	0.126	425	M08	0.223	1029
M15	0.330	887	M10	0.696	1809

Table 7.3: *Target values for calibration events.*

function

$$E = \sum_{i=1}^{i=k} \left(\frac{|Mt_{mo,max} - Mt_{ma,max}|}{Mt_{ma,max}} + \frac{|Mt_{mo,accu} - Mt_{ma,accu}|}{Mt_{ma,accu}} + (1 - R^2) \right) \cdot 100\ \% \tag{7.13}$$

7.14.2 Calibration history

The reason for not starting the calibration using an automatic optimisation is to be able to monitor the progress thoroughly avoiding the risk of doing a lot of calculations in the wrong parameter ranges with the only possibility to make corrections after an optimisation session which may require substantial amounts of time. It is thus of paramount importance to know the proper ranges for the calibration parameters within which to seek the optimum. Initial manual calibration helps setting up the ranges and gives invaluable fingerspitzgefüll for using the model.

Below is seen a list of input parameters for STSim which must be taken into account when calibrating the model. All the parameters are assigned globally and even though they bear names normally assigned to parameters which is dependent on a complex set of conditions, e.g. critical bed shear stress for erosion is dependent on time, geometric conditions, sediment characteristics etc., their values should not be mistaken with the actual precise physical value.

All together including initial conditions 10 parameters are required to run STSim. These are:

$\underline{M_{c,initial}}$: Globally distributed initial mass on the surfaces at the beginning of the event in (kg/ha). This value can be estimated based on the ADWP, accumulation and decay rates for the build-up on the surface.

$\underline{\alpha_{acc}}$: Accumulation rate in (kg/ha/day) for the build-up process on the surface. This value has almost no influence on the washoff during a rain event as the rain event typically lasts for less than 12 hours and the build-up may reach its maximum after app. ten days.

$\underline{\alpha_{rem}}$: Decay factor in (1/day) for the build-up process on the surface. The same conditions apply for the decay factor as for the accumulation rate.

$\underline{\alpha_{w1}}$ and $\underline{\alpha_{w2}}$: Numerical coefficients for the washoff from the surface, α_{w1} being a factor and α_{w2} an exponent.

$\underline{M_{p,initial}}$: Initial conditions determining the amount of deposits in the sewer system. The mass can be distributed as a percentage of the pipe volume or dependent on both pipe volume and slope.

$\underline{\alpha_{dep}}$: Deposition rate for deposition process in the sewer system in (1/day).

$\underline{\tau_{cd}}$: Critical bed shear stress for deposition.

$\underline{\alpha_{ero}}$: Erosion rate for deposition process in the sewer system in $(1/m^2)$.

$\underline{\tau_{ce}}$: Critical bed shear stress for erosion.

For the first attempt to calibrate the model, the surface model was applied with the parameters $\alpha_{w1} = 0.08$ and $\alpha_{w2} = 2.0$. An exponent of 2.0 yields an amplified variation of the washoff compared with the discharge. The parameters results in a total washoff load of 25 % - 40 % of the diverted mass at the outlet $(Mt_{ma,accu})$. An initial surface load of 70 kg/ha was chosen in order to assure that sediments were abundant. Next, a set of pipe model parameters was chosen and initial conditions set with a value of $M_{p,initial}$ assuring abundant sediment deposits. This approach does not work as there is never a limited amount of erodable sediments. Modelled mass transport curves tends to follow the features of the discharge much more closely than observed.

The first approach of calibration, ignoring the build-up process during the rain event, leaves six parameters to be calibrated, namely:

$$\alpha_{w1},\ \alpha_{w2},\ \alpha_{dep},\ \tau_{cd},\ \alpha_{ero} \text{ and } \ \tau_{ce}$$

Realising the need for scarcity of sediments the prior value of the initial mass was lowered to an evenly distributed amount of sediments of 3600 kg equaling a thin layer of approximately 0.5 mm of material throughout the system. The 3600 kg ensures the availability of material as the "strongest“ rain event recorded diverts 2000 kg of material.

In order to obtain a first guess on critical bed shear stress values for deposition (τ_{cd}) and for erosion (τ_{ce}) the bed shear stresses were calculated from the hydraulic conditions during DWF for a number of pipes (11). The averages for maximum, mean and minimum levels is seen in table 7.4. On this background

Bed shear stress τ $[N/m^2]$	Average value
Maximum	1.01
Mean	0.74
Minimum	0.45

Table 7.4: *Average values for bed shear stress during DWF .*

τ_{cd} and τ_{ce} was chosen. Subsequently, each of the five calibration events were calibrated in order to obtain reasonable ranges for input parameters within which a common set of parameters can be searched. This procedure resulted in an average correlation coefficient of $R^2 = 0.75$. On this account three parameters τ_{cd}, α_{dep} and α_{ero} were varied systematically applying two levels for each parameter and carrying out computations of all combinations of these parameters. The parameter values used for this full factorial design (Box et al. 1978) is shown in table 7.5. For each run including all five calibration events the error function,

Level	τ_{cd}	α_{dep}	α_{ero}
1	0.15	0.05	0.00010
2	0.50	0.15	0.00017

Table 7.5: *Values for computations of main effects.*

equation 7.13, was calculated as well as its intermediate results. Completing the set of computations allows for the calculation of main effects. These calculations shows e.g. the average main effect of changing τ_{cd} from 0.5 to 0.15 on R^2. The results of the calculations are shown in table 7.6.

	Effect of changing $\tau_{cd} = 0.50 \rightarrow 0.15$ on			$\alpha_{dep} = 0.15 \rightarrow 0.05$ on			$\alpha_{ero} = 0.00017 \rightarrow 0.00010$ on		
	Mt_{max}	Mt_{accu}	R^2	Mt_{max}	Mt_{accu}	R^2	Mt_{max}	Mt_{accu}	R^2
M05	0.028	782	-0.017	0.028	422	0.165	-0.040	-472	-0.083
M09	0.035	410	0.231	0.037	212	0.003	-0.037	-68	-0.005
M12	0.033	765	-0.094	0.011	172	0.012	-0.015	-198	-0.190
M14	0.026	446	0.008	0.058	314	-0.046	-0.056	-274	0.021
M15	0.020	451	0.014	0.050	457	0.038	-0.068	-419	-0.024
Avg.	0.028	571	0.028	0.368	315	0.034	-0.043	-286	-0.056

Table 7.6: *Calculated main effects.*

It is seen that the greatest effect to improve the correlation coefficient is to raise the erosion rate. This means that further calculations should exploit this knowledge to increase α_{ero}. However, it is also seen that changing one parameter may have a positive effect on some events and a negative effect on others.

Throughout the described initial phase of calibration different models for deposition and erosion, described in paragraph 7.7 and 7.8, has been evaluated. No significant improvement of the modelling quality was found, using the different models as long as these were calibrated individually. All presented results are employing model 2 for deposition (see equation 7.7) and model 1 for erosion (see equation 7.8).

In general, the simulations tends to bee too closely related to the discharge compared with measured values of the mass transport rates. This can be illustrated by e.g. event M05 shown in figure 7.17. It is seen on figure 7.17 that the pro-

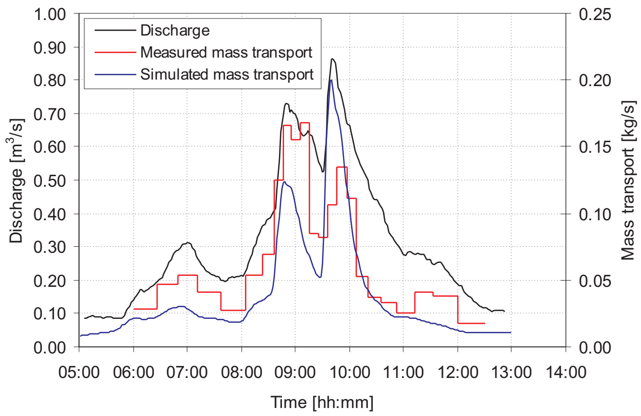


Figure 7.17: *Measured and simulated mass transport rates for M05.*

portions of the two peaks are simulated incorrectly. These initial results lacks some quality to be desired. As measurements and calculations of roof and street washoff has been carried out by CERGRENE it is of interest whether such information when applied to the conceptual model will increase the modelling quality. If so, it would be beneficial to carry out washoff measurements when utilising a conceptual model type like STSim.

The measured shape of the washoff curves suggests that a lower exponent (α_{w2}) between 0.7 and 1.6 should be applied and estimated values of the total washoff points at lowering the fraction of sediments originating from the surface to an average of 22.5 %. A review of literature also suggests a lower value than 2.0 of α_{w2}, this being for washoff models related to the rain intensity and not the runoff rate. But as the hydraulic calculations shows a great similitude between rain intensity and surface runoff a reasonable value for the exponent between 1.0 and 1.8 should be considered.

The surface model was calibrated against the measured total amount of washoff with an exponent of 1.0. With a lower input of sediments from the surface and a washoff which rate of change is lowered considerably it was imperative to introduce more erosion into the pipe model, e.g. erosion rate was changed from 0.00017 to 0.0025.

Calibrating the model with higher erosion levels yields an average correlation coefficient of $R^2 = 0.92$. This means that the shape of the mass transport curves are close to perfect, compared with measured values. But at the same time the quality of the estimates on peak values of mass transport rates and accumulated mass transport decreased to an average discrepancy of 47% and 44% respectively. As both surface model and pipe model parameters had been changed, the old value of the exponent of $\alpha_{w2} = 2.0$ was used again in order to determine whether the new results were an effect of changing the surface model parameters or the pipe model parameters. Returning to the prior surface model parameters but keeping higher erosion rates shows that the surface model is of little significance for the quality of the prediction of the mass transport rate at the outlet. Therefore, it seems sufficient to observe literature values for the surface model aiming at modelling the transport rate at the outlet. If of course the washoff from specific subcatchments are desirable to model, then the rough approach indicated above is not sufficient. In that case experience shows that measurements for calibration for each specific catchment is requisite and furthermore that physical surface washoff models are not obtainable due to the complexity and thus statistical relations must be applied (Bertrand-Krajewski et al. 1993), (Ruan and Wiggers 1998). The above conclusions implies that any thoughts of using the conceptual model to extrapolate "backwards" in the sewer system to the level of the surface model in order to find precise washoff quantities are completely impossible.

The reason for the ill predicted values of peak values for the mass transport rate and the accumulated transport may be found in the fact that the same initial mass condition is used for each event. If each calibration event is altered changing only the initial mass parameter $M_{p,initial}$ then the prediction of the maximum mass transport rate is 9.3% off on the average, the accumulated mass transport 5.5% off and the average correlation coefficient is 0.92. In this case the error function (equation 7.13) results in a value of $E = 112.5$. This may indicate that the amount of readily erodable material in the sewer system depends on the time of the event and the event itself. E.g. a higher mean water level during a rain event compared to another may expose sediments located higher upon the pipe walls to possible erosion.

7.14.3 Estimation of initial mass parameter

As the initial condition cannot easily be measured a relation between this value and known inputs concerning the rain event must be found in order to increase the quality of the model. Table 7.7 shows optimised $M_{p,initial}$ values for the five calibration events along with known input values. In order to establish

	$M_{p,ini.}$ [%]	Div. vol. $[m^3]$	Max. flow $[m^3/s]$	T_{rain} [minutes]	$I10_{max}$ $[\mu m/s]$	ADWP [days]	Max. int. $[\mu m/s]$
M05	0.38	8459	0.865	417	8.35	0.71	18.17
M09	0.12	2548	0.540	122	2.31	0.07	2.78
M12	0.40	3952	0.250	326	0.71	2.48	0.92
M14	0.09	3733	0.796	169	2.56	1.26	3.17
M15	0.23	4732	1.098	177	10.32	0.12	22.22

Table 7.7: *Data basis for initial mass parameter estimation.*

which parameters are the most important to include in the relation to calculate $M_{p,initial}$, a simple neural network technique using a single layer of input neurons and the adaptive back propagation algorithm was applied. First, the whole set of inputs seen in table 7.7 was used to train the neural network and the total error calculated by the sums of squares. Subsequently, six other neural network were trained each lacking one column of input. From this analysis it is possible to evaluate which parameters should be included in the relation. Results are shown in table 7.8. As seen information on rain intensity e.g. may be excluded.

Input	Sums of squares	Increase of error	Rank of importance
All	$0.397 \cdot 10^{-5}$	0.0	-
Div. vol.	$3.064 \cdot 10^{-5}$	$2.667 \cdot 10^{-5}$	1
Max. flow	$1.556 \cdot 10^{-5}$	$1.159 \cdot 10^{-5}$	4
T_{rain}	$1.614 \cdot 10^{-5}$	$1.217 \cdot 10^{-5}$	3
$I10_{max}$	$1.370 \cdot 10^{-5}$	$0.912 \cdot 10^{-5}$	5
ADWP	$1.780 \cdot 10^{-5}$	$1.383 \cdot 10^{-5}$	2
Max. Int.	$0.761 \cdot 10^{-5}$	$0.364 \cdot 10^{-5}$	6

Table 7.8: *Neural network investigation.*

Now multiple nonlinear regression analysis is performed using a regression model of the type shown in equation 7.14

$$y = ax^b + cz^d + y_0 \tag{7.14}$$

A good correlation between diverted volume, max. flow and the initial mass parameter was obtained with the characteristics shown in table 7.9. A more

	Value
a	$9.69 \cdot 10^{-5}$
b	0.9340
c	0.00561
d	-2.7600
y_0	-0.0767
R^2	0.97

Model	$M_{p,ini.}$	Predicted	Residual
M05	0.38	0.3831	-0.0031
M09	0.12	0.1010	0.0190
M12	0.40	0.4003	-0.0003
M14	0.09	0.1440	-0.0540
M15	0.23	0.1900	0.0400

Table 7.9: *Information on regression model.*

simple model depending only on the rain duration ($R^2 = 0.91$) reads

$$y = ax^b + y_0 \tag{7.15}$$

where $a = 0.1953$, $b = 0.2823$ and $y_0 = -0.6568$.

It can seem illogical that for instance the ADWP is not included in the estimation model for the initial mass. The fact that the diverted water volume and maximum flow is usable may be because higher flow rates entails higher water levels and therefore more deposits and wall slimes may be reached.

Computation with the completed set of input parameters using the estimates of $M_{p,initial}$ shown in table 7.9 results in the calibration results shown in table 7.10. Table 7.11 shows the results of the final validation.

	Mt_{max}	Mt_{accu}	R^2	ΔMt_{max} %	ΔMt_{accu} %	$(1 - R^2)$ %	E_i
M05	0.192	1340	0.862	17.8	-2.1	13.8	33.7
M09	0.065	295	0.962	-1.5	-1.0	3.8	6.3
M12	0.071	541	0.902	14.5	-5.6	9.8	29.9
M14	0.159	561	0.989	26.2	32.0	1.1	59.3
M15	0.324	800	0.902	-1.8	-9.8	9.8	2.4
Averages and $\sum E_i$			0.923	12.4	10.1	7.7	150.6

Table 7.10: *Results of final calibration.*

Plots of the results of the calibration and validation are shown in figure 7.18 and 7.19 respectively.

	Mt_{max}	Mt_{accu}	R^2	ΔMt_{max} %	ΔMt_{accu} %	$(1-R^2)$ %	E_i
M17	0.199	1481	0.855	47.4	13.6	14.5	75.5
M06	0.219	409	0.961	-45.7	-48.7	3.9	98.3
M07	0.055	329	0.935	-15.4	-10.6	6.5	32.5
M08	0.426	1094	0.573	91.0	6.3	42.7	140.0
M10	0.530	977	0.945	-23.9	-45.0	5.5	74.4
Averages and $\sum E_i$			0.854	44.7	24.8	14.6	420.7

Table 7.11: *Results of initial validation.*

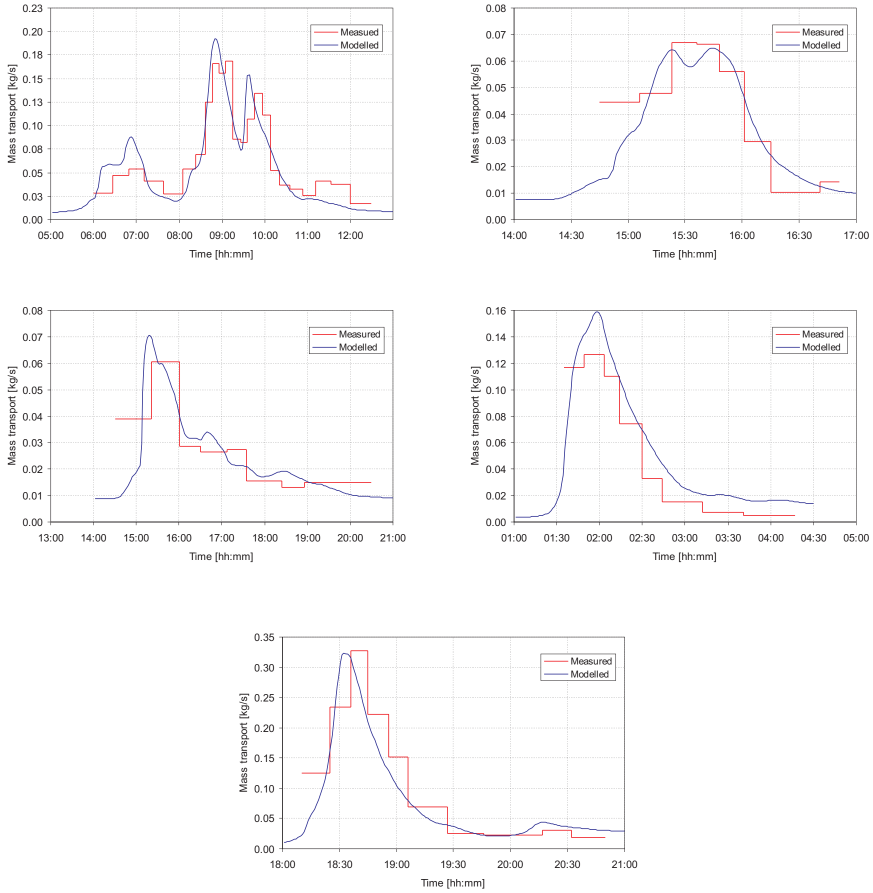


Figure 7.18: *Measured and simulated mass transport rates for the five calibration events. Upper-left: M05, Upper-right: M09, Middle-left: M12, Middle-right: M14, Lower: M15*

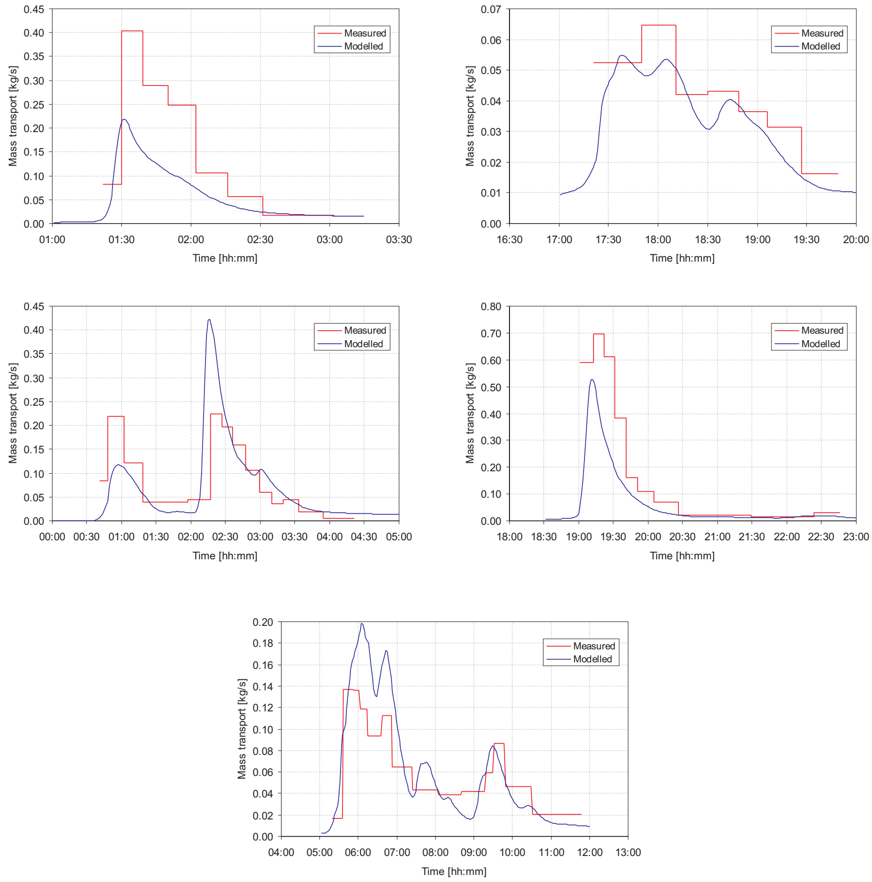


Figure 7.19: *Measured and simulated mass transport rates for the five validation events. Upper-left: M06, Upper-right: M07, Middle-left: M08, Middle-right: M10, Lower: M17*

As seen, the proportions between the peak values in calibration event M05 is now of the correct order. Compared with the initial calibration and validation results are much more promising. As listed the average correlation coefficient for the calibration is thus $R^2 = 0.923$ and for the validation the value is $R^2 = 0.854$.

Different conclusions were reached during the process of calibration and validation. First of all, calculations seems to indicate that the models sensitivity towards changes in the surface modelling of washoff is small when it comes to achieving coherent results at the outlet. Another conclusion reached, is that there is a need for an intelligent guess at the initial mass in the sewer system. If this is carried out as described in this report calibration and validation quality is improved. In this case the quality of the validation is significantly lower than that of the calibration. This may be because the calibration set is too small to establish a good relation for the initial mass coefficient - five data points are a quite small sample to fit a surface to. Furthermore, if the validation events are reviewed it is seen e.g. that event M10 is not encompassed by the calibration set as it is a much more extreme event. It is of course almost always desirable to have a larger calibration set, but this means more measurements which are costly, laborious and difficult.

The initial calibration have also given the experience that an automatic optimisation of the calibration should be tried.

7.15 Illustrations of spatial predictions

There are potential possibilities to exploit the spatial prediction capabilities of STSim. However, it should be remembered that there is little measured data available for verification of results. The results shown in this paragraph are therefore presented as relative and qualitative.

One possibility is to evaluate the accumulated mass transport at different locations in the sewer system. Figure 7.20 shows the results for rain event M14 (96082200) given as percentages compared with the total accumulated mass transport at the outlet. This kind of information can be composed for different rain events in order to see if the percentages changes.

Another possibility is to look at erosion and deposition during the rain event. A plot showing the conditions has been produced as seen in figure 7.21. In general, there is an indication that sedimentation would be expected upstream in the main sewers, but results are inconclusive compared with visual observations.

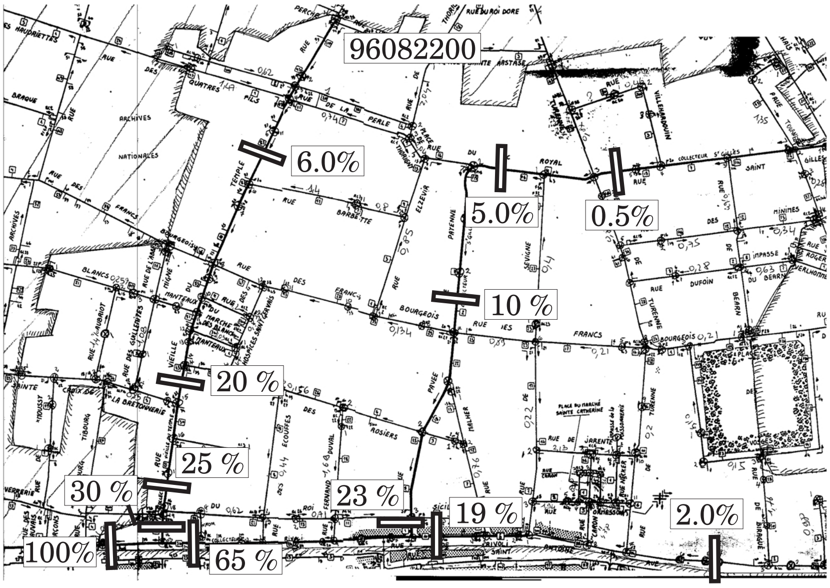


Figure 7.20: *Relative accumulated mass transport at different locations in the sewer system.*

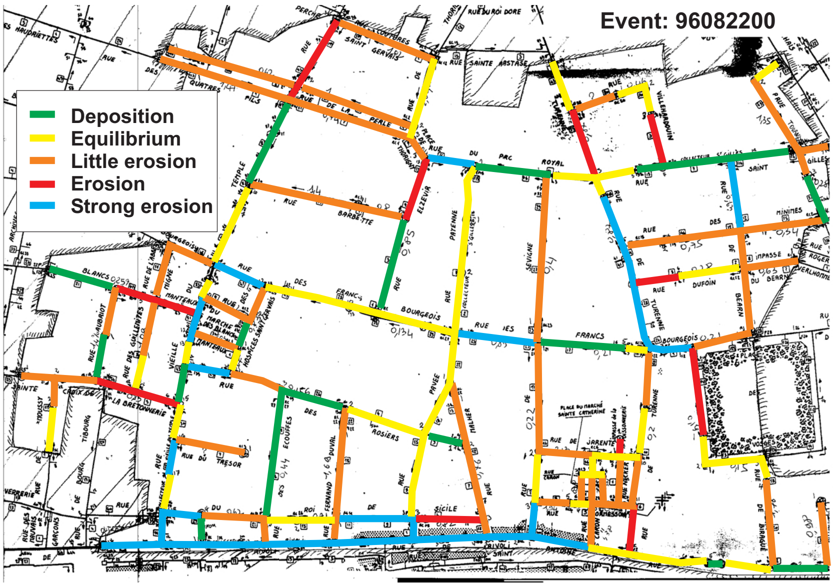


Figure 7.21: *Relative erosion conditions during event 96082200.*

7.16 Summary

The conceptual model STSim has been applied to measured data from the catchment "Le Marais" in Paris, exploiting data procured by CERGRENE.

The results of the calibration, which in this chapter were performed manually, and the validation are very promising. The model simulates the shape of mass transport curves at the outlet with great precision. In some cases the amounts of transported material through the cross-section at the outlet are less well estimated. The investigation has shown that this can be attributed to the sensitivity to the initial mass parameter in the model.

At the level of modelling the surface washoff it has proven sufficient to apply literature values for the calibration parameters as the sensitivity towards the surface model is little with regard to modelling the mass transport rates at the outlet.

CHAPTER 8

Simplified lumped conceptual modelling

The only critical drawback of modelling the hydrodynamics and subsequently the sediment transport at a very detailed level, as presented in chapter 7, is the computational time consumption. One thousand simulations with the complete sewer system of "Le Marais" and the five calibration events will take more than two weeks to perform even on a fast PC. This has for the detailed model ruled out the possibility to implement optimization routines for calibration of the STSim model.

In this chapter another approach is presented. Here thousands of computations with the STSim model is carried out, but instead a simplified representation of the sewer system of "Le Marais" is applied.

It is clear that the two different approaches using the STSim model also results in two different types of models. In the simplified model the possibility to obtain spatially distributed predictions as presented in paragraph 7.15 is lost. The simplified approach, on the other hand, should make conclusions regarding the importance of the modelling of the detailed system to the sediment transport modelling at the outlet feasible.

8.1 Model description and implementation

For this chapter the STSim model has been implemented again in a new version adapted for the purpose. A short description of the software layout is seen in appendix B.1.

The applied conceptual models are the same as the ones used in chapter 7, namely:

Surface build-up model	: Equation 7.3
Surface wash-off model	: Equation 7.4
Calculation of bed shear stress	: Equation 7.5
Pipe deposition model	: Equation 7.7
Pipe erosion model	: Equation 7.8

This model calculates the objective function (see equation 7.13) for a number of calibration events and the intermediate results of the objective function as well as the sum of squared errors between calibration data and modelled data. As none of the simulated results are saved it is neccesary subsequently to apply the original STSim model to obtain the modelled results for a given combination of input.

8.2 Calibration

The basic idea of the new implementation of the model is to have a calibration procedure which ascertains the combination of calibration parameters which entails the minimum value of the objective function.

As input values fourteen different inputs can be given by their lower limit, upper limit and the size of the step used in this range. The inputs encompass:

1. Surface accumulation rate
2. Surface decay rate
3. Surface washoff coeficient 1
4. Surface washoff coeficient 2
5. Pipe deposition rate
6. Pipe storage capacity
7. Pipe erosion rate
8. Critical bed shear stress deposition
9. Critical bed shear stress erosion

10. Initial surface load
11. Initial pipe load
12. Initial wastewater concentration
13. Computational sediment density
14. Daily PE discharge

The model combines all possible combinations of input values. This entails that the number of calculations develops explosively when refining steps sizes and including additional parameters to vary.

The structure of the model has been chosen for two purposes. First of all calculations with STSim (see chapter 7) suggested to some extent that the objective function may have local minima. This makes traditional procedures (steepest decent, etc.) for seeking the minimum of the objective function intricate. Secondly, the model can with its currently chosen structure be used for exhaustive sensitivity analysis.

Not all the parameters in the list above should be used as calibration parameters. As calibration to additional parameters quickly costs increased computational time, it is most rational only to fine tune the most important parameters according to the list seen in table 7.1.

8.3 Case study calculations and results

In this paragraph a simplified version of the "Le Marais" catchment and sewer system is presented. Modelling this simplified catchment makes it possible to evaluate the possible benefits of carrying out detailed modelling of the hydrodynamics as a basis for the sediment transport modelling.

8.3.1 Hydraulic modelling

The "Le Marais" catchment has been reduced to seven catchments and eight stretches of sewer pipes. The node points of the catchments are located on the interceptor sewers and the modelled sewer system follows the main sewer pipe of Le Marais. The new model layout can be seen in figure 8.1.

As seen the original sewer system is reduced to a very simple system. Despite the simplification the hydrographs are reproduced with a satisfactory precision.

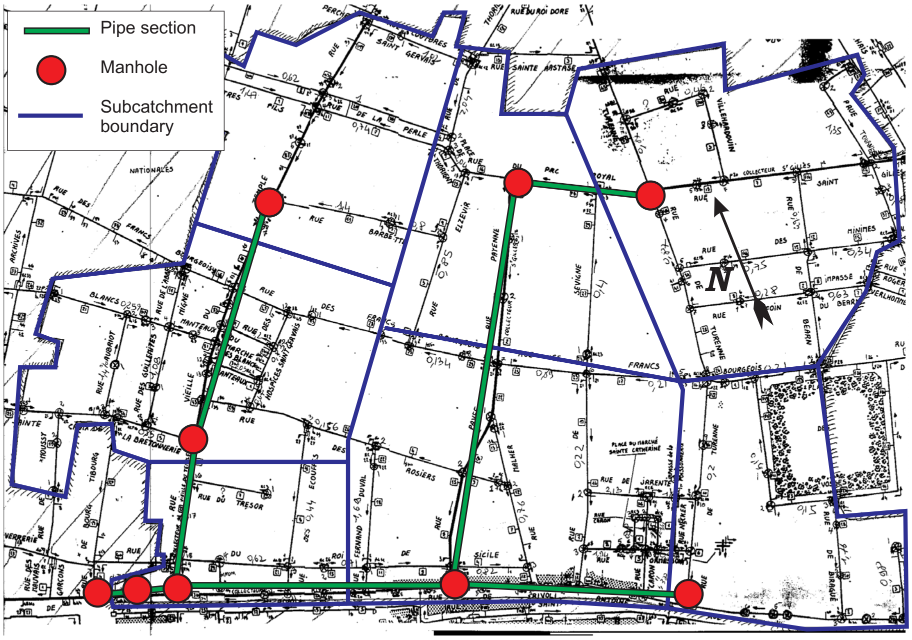


Figure 8.1: *Illustration of simplified sewer model for Le Marais.*

As seen on figure 8.2 the concentration time is a little to small in this case and the largest flow peak is enhanced compared with the measured values.

8.3.2 Sensitivity analysis

Where paragraph 7.11 presented a sensitivity analysis aimed at "getting a feeling" for the model, this paragraph presents a sensitivity analysis carried out for the reduced system of Le Marais and by that depicts the physical behaviour of this system in particular.

Before carrying out the sensitivity analysis the optimal combination of input parameters were found using the semi-automatic calibration technique. The results from this calibration is presented in the succeeding paragraphs. The optimal input combination is used while changing one or two parameters at the time. A number of plots is produced depicting the objective function (equation 7.13) as a function of each parameter.

Figure 8.3 shows the influence of the two coefficients used in the washoff equation.

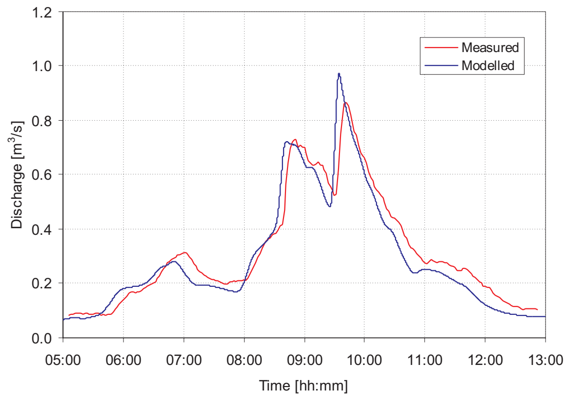


Figure 8.2: *Example of flow modelling (event M05) from the simplified sewer system.*

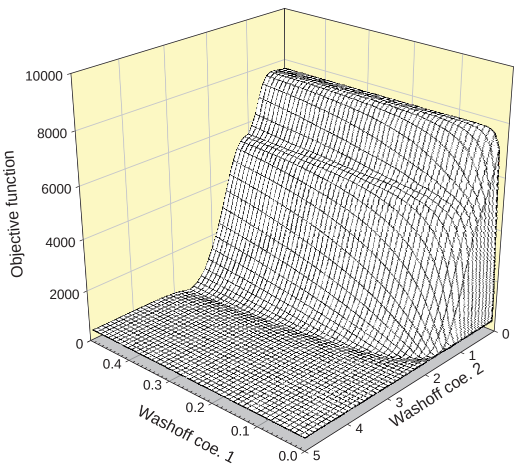


Figure 8.3: *Sensitivity to the washoff coefficients.*

As seen the value of the objective function rises to a quite high value. This is due to the possibility to erode a rather large amount of sediments from the surface. But measurements in the catchment has shown that only 22.5 % of the diverted mass at the outlet stems from the surface during the rain event. This sensitivity analysis yields results which entails much larger percentages of mass originating from the surface. The reason why this is possible is the relatively large value used for the initial surface load of 70 kg/ha. This is not an unrealistic value, but it may be the case that it is too high compared with the load which is readily erodable during the calibration events. As measured data shows only very weak correlation between the ADWP and the initial surface load, the build-up models cannot be used for the estimation of the initial surface load.

As indicated by table 7.1 the parameters connected to the pipe model is considered to be the most important. Figure 8.4 shows the results from the sensitivity analysis for four different parameters and figure 8.5 shows the influence of the critical levels for the bed shear stress. The plots in figure 8.4 shows that chang-

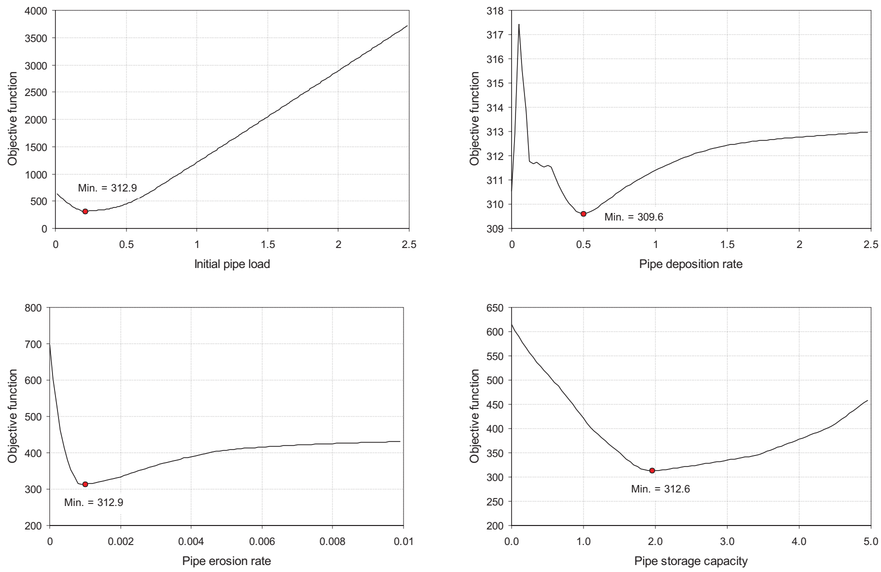


Figure 8.4: *Results of sensitivity analysis for pipe model parameters.*

ing one parameter at the time each parameter has a distinct optimum, but at the same time choosing a value a little different from the optimum does not decrease the quality of the modelling very much. This is true except for the initial pipe load and when choosing too small a value for the pipe erosion rate or too high a

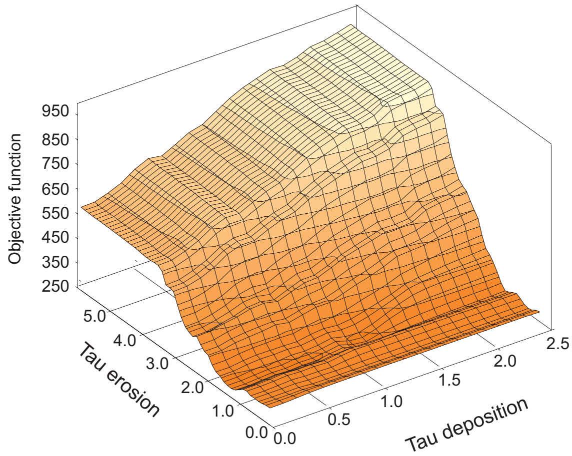


Figure 8.5: *Results of sensitivity analysis for τ_{cd} and τ_{ce} .*

value for the critical bed shear stress for erosion. This indicates that the erosion process modelled is the far most important.

The behaviour of the model is satisfactory as it is robust with regard to input parameters. It is, however, very sensible to the initial sediment load in the pipes. Improving the estimation of this initial condition may increase the quality of the model.

8.3.3 Optimised calibration

This paragraph explores the possibility to obtain a optimum of the input parameters and to which degree the simplified hydrodynamic modelling may deteriorate the quality of the sediment transport modelling at the outlet of Le Marais.

The same rain events used for calibration and validation, respectively, were used for modelling again, see paragraph 7.14.1. For the calibration, input parameters were changed according to table 8.1.

The calibration resulted in the values shown in table 8.2. These results are of course dependent on the chosen ranges and step sizes. The variation of the input parameters were a compromise between resolution and computational time. The calibration lasted about 18 hours on a 450 Mhz PII computer. This comprises 12600 combinations of the model as indicated in table 8.1. For comparison the 12600 combinations would have taken half a year (183 days) to complete if the detailed catchment description of "Le Marais" had been applied. Choosing one

Parameter		Lower limit	Upper limit	Step size
Surface accumulation rate	(kg/ha/day)	0.0	0.0	-
Surface decay rate	(1/day)	0.0	0.0	-
Surface washoff coe. 1	(-)	0.01	0.01	-
Surface washoff coe. 2	(-)	1.6	1.6	-
Pipe deposition rate	(1/day)	0.01	0.16	0.05
Pipe storage capacity	(% of vol.)	1.0	4.0	1.0
Pipe erosion rate	$(1/m^3)$	0.001	0.01	0.001
Critical bed shear stress deposition	(N/m^2)	0.1	0.6	0.2
Critical bed shear stress erosion	(N/m^2)	0.5	2.0	0.25
Initial surface load	(kg/ha)	70.0	70.0	-
Initial pipe load	(% of SC)	0.1	0.6	0.1
Initial wastewater concentration	(kg/m^3)	0.15	0.15	-
Computational sediment density	(kg/m^3)	1600.0	1600.0	-
Daily PE discharge	(kg/day/PE)	0.1	0.1	-

Table 8.1: *Variation of input parameters.*

more step for one of the parameters would have increased the time consumption considerably.

Parameter		Best value
Pipe deposition rate	(1/day)	0.11
Pipe storage capacity	(% of vol.)	2.0
Pipe erosion rate	$(1/m^3)$	0.001
Critical bed shear stress deposition	(N/m^2)	0.5
Critical bed shear stress erosion	(N/m^2)	1.0
Initial pipe load	(% of SC)	0.2

Table 8.2: *Obtained optimal values of varied input parameters.*

The model behaves very similar to the model of the complete sewer system. This means that it, like indicated by the sensitivity analysis, is still very sensitive to the initial load in the pipes. There seems not to be one single value which fits all the calibration events acceptable. For this reason the same procedure is carried out as with the detailed model (see paragraph 7.14.3). First the optimal value of the initial load is determined for each calibration event. Secondly a simple regression model is fitted to the obtained values. In this case a parbola is fitted based on the rain durations. Subsequently this regression model is used to estimate the initial mass parameter for all the events.

For the use of optimal individual values of the initial load, before any regression, the results are comparable with the complete model. The average correlation coeficient for the calibration is $R^2 = 0.87$ and the maximum transport rate is 8.7 % off and the accumulated transport is 5.7 % off and finally the objective function yields a value of $E = 138.1$. Comparative values can be seen in paragraph 7.14.2 were $R^2 = 0.92$, $\Delta Mt_{max} = 9.3\%$, $\Delta Mt_{accu} = 5.5\%$ and $E = 112.5$. As seen, the earlier obtained results using the complete model of Le Marais is only slightly better.

When the initial values from the regression is used as initial load the simulated polutographs can be seen in figure 8.6 together with the optimal choices of $M_{p,initial}$ as well as the measured values. The calibration using the estimated values of $M_{p,initial}$ results in the model accuracy depicted in table 8.3.

	Mt_{max}	Mt_{accu}	R^2	ΔMt_{max} %	ΔMt_{accu} %	$(1 - R^2)$ %	E_i
M05	0.157	1367	0.782	-3.5	-0.07	21.8	25.4
M09	0.069	334	0.948	4.1	12.1	5.2	21.4
M12	0.053	618	0.774	-15.2	7.9	22.6	45.7
M14	0.151	604	0.977	19.8	42.1	2.3	64.2
M15	0.248	699	0.848	-24.8	-21.2	15.2	61.2
Averages and $\sum E_i$			0.866	13.5	16.7	13.4	217.9

Table 8.3: *Results of calibration with the simplified Le Marais model.*

The results from the validation is seen in figure 8.7. The results on the accuracy of the model for the validation is seen in table 8.4.

	Mt_{max}	Mt_{accu}	R^2	ΔMt_{max} %	ΔMt_{accu} %	$(1 - R^2)$ %	E_i
M06	0.217	459	0.965	-46.1	-42.4	3.5	92.0
M07	0.050	315	0.840	-23.8	-14.4	16.0	54.2
M08	0.328	925	0.443	47.1	-10.1	55.7	112.9
M10	0.459	898	0.946	-34.1	-50.3	5.4	89.8
M17	0.136	1261	0.807	0.4	-3.3	19.3	23.0
Averages and $\sum E_i$			0.800	30.3	24.1	20.0	371.9

Table 8.4: *Results of validation with the simplified Le Marais model.*

Comparing the calibration and validation with the complete model (see chapter 7) the results from simplifying the system is relatively good. One of the reasons why this is the case may be the fact that the simplified model is assisted by a semi-automatic calibration. This is to some extent indicated by the fact that the objective function E is closer to each other for the calibration and validation compared with the complete model.

Even though the simplified model seems to use a more optimized set of input

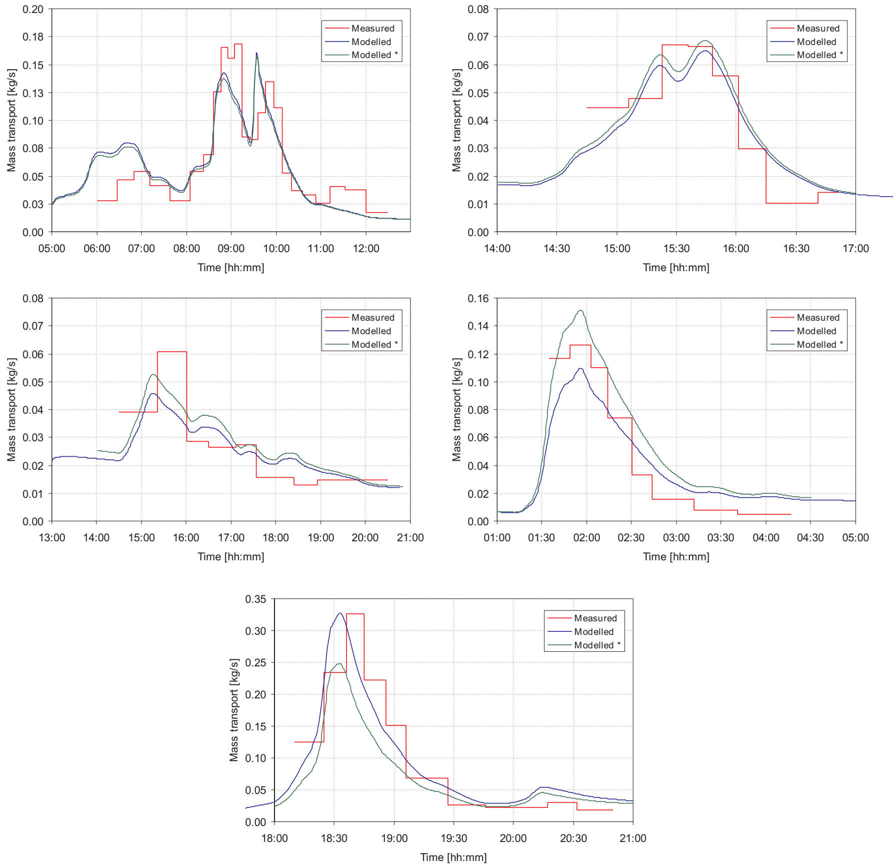


Figure 8.6: *Calibration results from the simplified Le Marais model. (* : $M_{p,initial}$ estimated from regression model) Upper-left: M05, Upper-right: M09, Middle-left: M12, Middle-right: M14, Lower: M15*

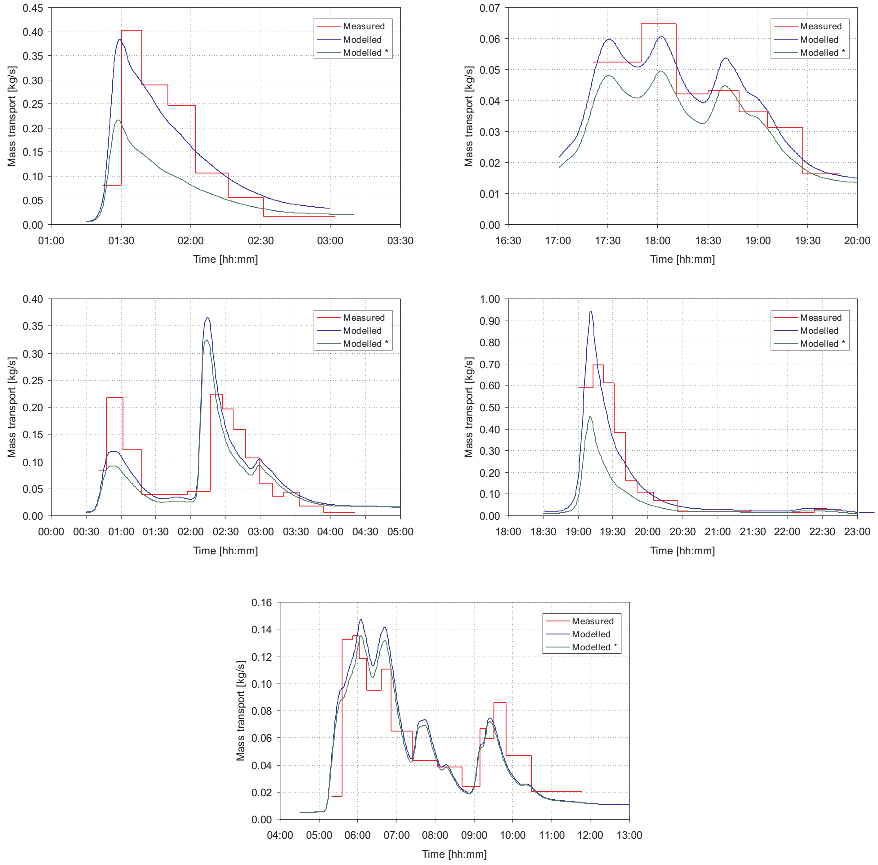


Figure 8.7: *Validation results from the simplified Le Marais model. (* : $M_{p,initial}$ estimated from regression model) Upper-left: M06, Upper-right: M07, Middle-left: M08, Middle-right: M10, Lower: M17*

parameters it is still performing slightly worse than the complete model. One conclusion can be to attribute this to the simplification, but as other factors influence the modelling as well this may be a too strong conclusion.

Another observation is that calibration parameters cannot be directly transferred between models. This means that it is not possible to start out with a simple model, calibrate it and then transfer the parameters to a more detailed model of the same system. This is because some of the parameters are dependent on the structure of the sewer system, e.g. the pipe storage capacity is given as a percentage of the pipe volume.

CHAPTER 9

Neural network modelling

The objective of this chapter is to explore the applicability of modelling sediment transport using a neural network model. The theory behind the neural network is elaborated a little compared to paragraph 5.6.3. The developed neural network model has only one output variable: the mass transport rate at the outlet based on input of rain characteristics. This model therefore constitutes another type of model than the previously described.

9.1 Description of STNeuro

STNeuro is the adaptation of a neural network to the modelling of sediment transport in sewer systems or other similar applications. A short description of the software is seen in appendix B.

The network used in this case is a feed-forward network. It consists of an input layer of neurons, one hidden layer of internal neurons and an output layer. The model can, however, be applied with more than one layer of hidden neurons. An illustration of a feed-forward network is seen in figure 9.1

When dealing with rain runoff and sediment transport is is imperative that the neural network is able to model the time, e.g. the time between a rain intensity peak and a flush of sediments at the outlet. This can be achieved in two ways. One possibility is to construct a network where output information is feed back into the neural network as an extra input. This approach has not shown promising results in this case. The other possibility is to construct a tab delay lined network. This means that the time varying input (rain intensity)

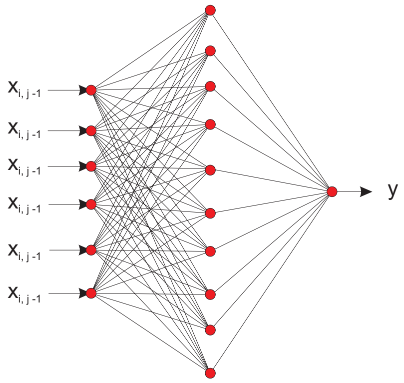


Figure 9.1: *Illustration of a feed-forward neural network with one output y, where $x_{i,j-1}$ is input from the i'th neuron in the $j-1$'th layer.*

is repeated as input to the network at a number of input neurons, but at each new input neuron the timeseries is shifted forward and zeroes are added. This results in indata in the manner shown in table 9.1, where time is progressing downwards.

Neuron 1	Neuron 2	Neuron 3	Neuron 4	Neuron 5
0.0	0.0	0.0	0.0	0.2
0.0	0.0	0.0	0.2	0.1
0.0	0.0	0.2	0.1	0.4
0.0	0.2	0.1	0.4	0.7
0.2	0.1	0.4	0.7	0.8

Table 9.1: *Start of a tab delayed input data for a neural network, here shown with five repetitions of the same input signal.*

The question is how many repetitions to include. The delay between the first column and the last one should be at least as long as the transport time in the sewer system. It may though still be beneficial to include a longer delay. The easiest way to examine what is the best approach is to experiment with different versions of the network.

In the STNeuro model it is possible to chose how many repetitions is wanted. Besides that other type of input data can be included. This could be the rain duration, the ADWP or other indata believed to be beneficial for the learning process of the network. It is subsequently possible to chose the number of hidden layers of neurons to be included in the network and the number of neurons in each layer. For each layer the type of transfer function for the neurons can be chosen as linear, sigmoid or tanh and also a learning rate η and a momentum value β is required.

For the learning of a neural network or calibration the same issues apply as described in paragraph 5.2. If too many calibration parameters (weights) are included by adding neurons a perfect correlation will be achievable but the the validation of the neural network will turn out to be poor. The network is then said to be grandmothered.

For STNeuro the back propagation algorithm is used for optimizing the network (Hertz et al. 1991). It is possible to stop after a certain number of iterations or when the sums of squared error between the model and the measured values becomes sufficiently small or decreases at a small rate or finally when a specified correlation coeficient is achieved.

9.2 Case study calculations and results

As primary input data for the neural network the rain intensities is used from the five calibration events M05, M09, M12, M14, and M15. The rain intensity time series of these events are combined to one long series adding a number of zero rain intensities between the events. The same procedure is of course applied to the output time serie. Figure 9.2 illustrates the way the time series are composed, however only showing one event.

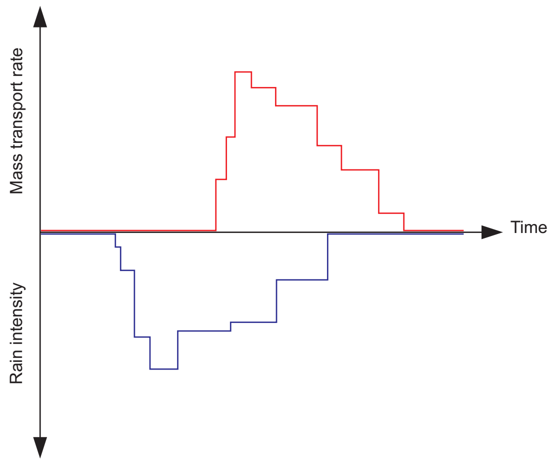


Figure 9.2: *Illustration of input and output time series.*

It quickly turned out that the network did not perform very well with the rain intensity as the sole input type. This may be because it has difficulties distinguishing between different rain events. By adding two extra input neurons the calibration improved considerably. One neuron was feed the rain duration and another the $I10_{max}$ rain intensity. It is on purpose that no hydraulic data like runoff volumes are used in the model. This would implicate hydraulic modelling and the ease of use of the black-box modelling would disappear.

There is virtually an infinite number of possible ways to build the network. The choice of the tab delay, the number of input neurons and the number and type of hidden neurons was done by trial and error. When adding input neurons and thereby increasing the tabbed delay the correlation coeficient of the calibration increases until an optimum, but then decreases again. This is shown in figure 9.3 and 9.4.

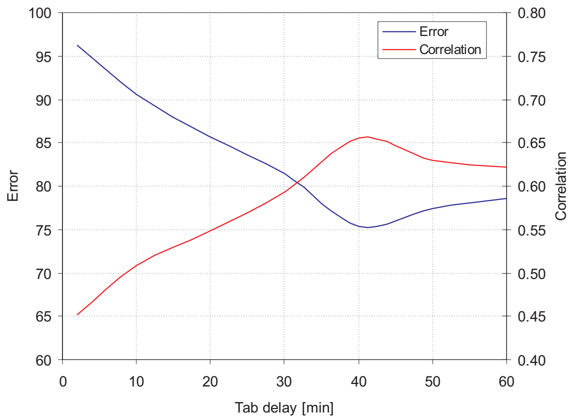


Figure 9.3: *Error and correlation coeficient R^2 as a function of tabbed delay. The network has two hidden layers, the first one having 10 neurons the second 6 neurons.*

As seen on figure 9.3 and 9.4 there is a optimum value of the tabbed delay. The delay is for some reason not the same for the two networks.

If instead of changing the tabbed delay the number of hidden neurons is changed the sum of squared errors decreases with an increasing number of hidden neurons. This is seen in figure 9.5. The curve for the correlation shown in figure 9.5 is increasing smoothly. In order to compute the curves seen in figure 9.5 it is neccesary to complete a sufficient number of iterations. Figure 9.6 shows the convergence of a neural network dependent on the number of iterations. The

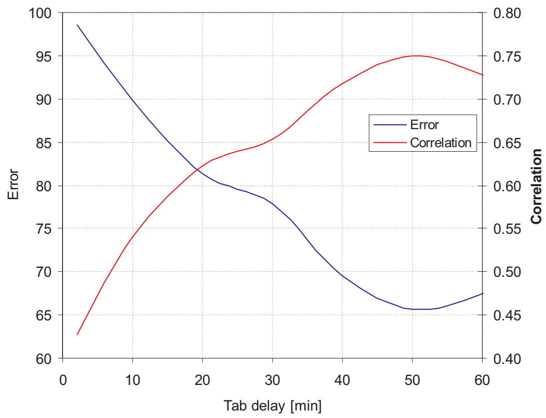


Figure 9.4: *Error and correlation coeficient* R^2 *as a function of tabbed delay. The network has one layer of 25 hidden neurons.*

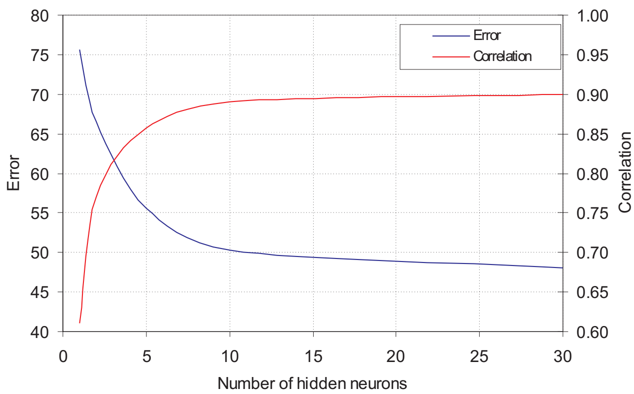


Figure 9.5: *Error and correlation coeficient* R^2 *as a function of the number of hidden neurons. The network has one layer of ten hidden neurons in this case.*

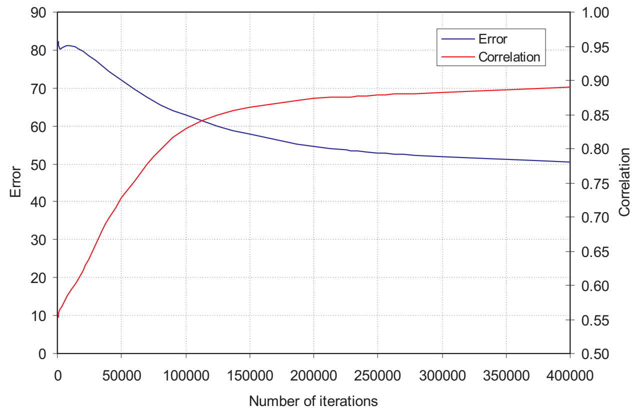


Figure 9.6: *Error and correlation coeficient* R^2 *as a function of the number of completed iterations. The network has one layer of ten hidden neurons in this case.*

convergence of the network may be a little different in the start from times to time even when it is the same network. The reason for this is that the training starts with a guess on all the weights. If this guess is very poor the the results may not be optimal in the start. This different starting point for each training session for the network may sometimes result in, that when the training is stopped the network has settled into a local minimum in the hyperspace of the objective function. The risk of this happening increases with the number of neurons.

The investigations regarding the layout of the network has resulted in a final choice of a network with one hidden layer with 10 neurons. The tabbed delay is 30 minutes equaling 15 input neurons. The input neurons has linear transfer functions, whereas the sigmoid function is used for the neurons in the hidden layer. Besides the rain intensity two extra input neurons is applied, one giving the rain duration and one the $I10_{max}$ rain intensity. A learning rate $\eta = 0.2$ is used and the momentum value was set to $\beta = 0.2$ as well.

The network was run for 400000 iterations in order to train the network. This results in a sum of squared errors of 50.4 and a correlation coeficient $R^2 = 0.88$. The validation with this network results in a correlation coeficient of $R^2 = 0.51$. The measured and modelled time series for the calibration and validation is seen in figure 9.7 and 9.8 respectively.

As seen from the results the neural network model STNeuro seems to estimate mass transport rates in the range between 0.05 kg/s to 0.25 kg/s, but outside this

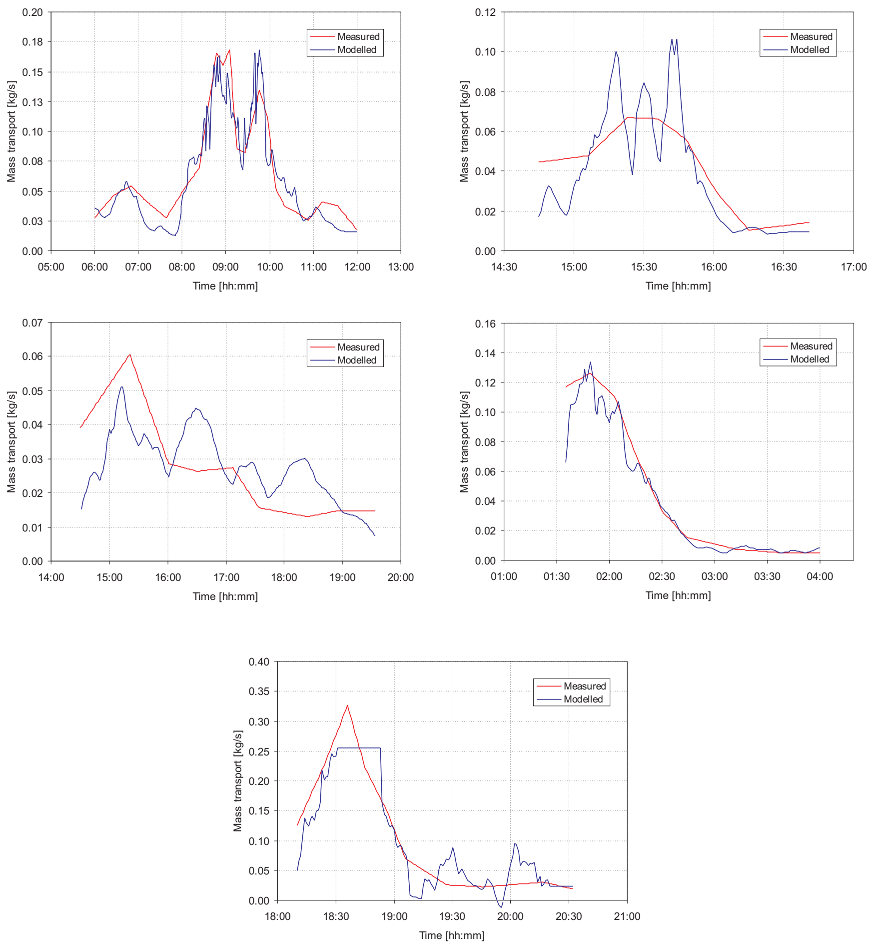


Figure 9.7: *Results of calibration using a neural network model. Upper-left: M05, Upper-right: M09, Middle-left: M12, Middle-right: M14, Lower: M15*

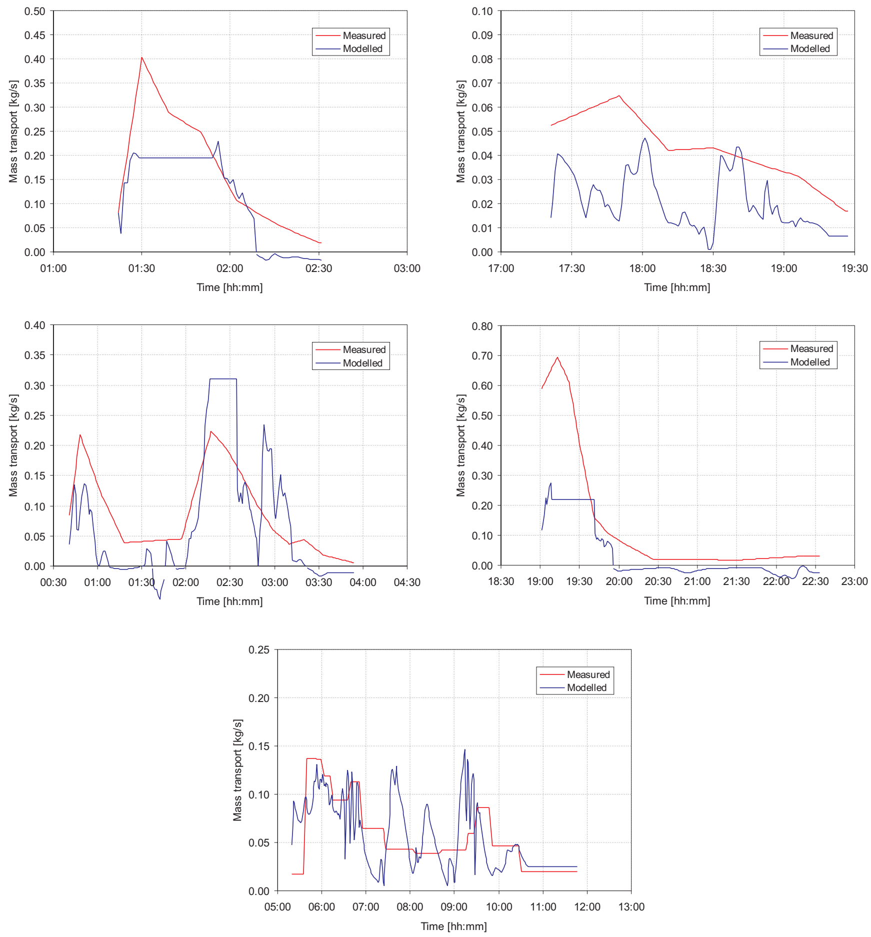


Figure 9.8: *Results of validation using a neural network model. Upper-left: M06, Upper-right: M07, Middle-left: M08, Middle-right: M10, Lower: M17*

range the predictions are quite poor. It is difficult to see which initiatives would improve the quality of the modelling. Inclusion of hydraulic measurements or modelling would probably help, but then one of the advantages of the black-box modelling disappears as the modelling get more laborious.

CHAPTER 10

Conclusions

This Ph.D. study has explored the possibilities of modelling sediment transport in combined sewer systems. The different modelling approaches have been evaluated by a case study, exploiting measured data from the "Le Marais" catchment in Paris procured by the CERGRENE–LABAM laboratory of L'Ecole Nationale des Ponts et Chausees.

A short conclusion is that it is possible but difficult to model the sediment transport in combined sewer systems. The system is very complex. The sewer system itself with it's constant change in pipe sizes, change in direction, special structures, structural condition, changing slopes of catchment and pipes results in local and temporal variations in the hydrodynamic conditions. As the sediment transport is closely connected to the hydrodynamic conditions it is of paramount importance to model the hydrodynamic conditions in the sewer system precisely during the rain events. The modelling carried out during this study has shown a clear benefit from modelling the hydrodynamic conditions using a detailed hydrodynamic model (MOUSE). This is supported by the fact that the STSim model using the detailed description of Le Marais yields the best results. Table 10.1 shows the obtained average correlation coefficients from the different models. Another key condition for the sediment transport is the characteristics of the sediments. The variability of the sediments further complicates the modelling of sediment transport. The sediments vary in gain size, cohesiveness, density, fall velocity, and type. Dependent on the sewer system and the behaviour of inhabitants in the catchment the amount of gross solids and coarse materials, from e.g. construction sites and winter grittings, vary. In the Parisian catchment most gross solids are disintegrated before reaching the outlet and coarse materials seem to build up in the main sewers staying there until manually removed.

Model	Calibration R^2	Validation R^2
MOUSE TRAP (ST-module)	0.49	0.43
MOUSE TRAP (AD-module)	0.67	0.62
STSim (detailed)	0.92	0.85
STSim (reduced)	0.87	0.80
STNeuro	0.88	0.51

Table 10.1: *Obtained correlation coefficients.*

The application of the most detailed physically based model, MOUSE TRAP ST-module, proved to be the least successful in the case study performed. The model has been shown to be able to model e.g. laboratory investigations of sediment transport (Mark 1995), but it does not apply well to "Le Marais". There may be a number of explanations to this. The model can simply be unsuitable for the modelling task in hand or it has been applied in an unadvantageous way. It may also be a result of having too little measured information to be able to calibrate a model with such detail as MOUSE ST.

As soon as a less detailed model with a more transparent model structure is chosen the modelling results improve. The combination of the MOUSE TRAP surface module and the AD module has proven to be applicable, actually to a higher degree than the obtained correlation coefficients suggests. This modelling approach is, however, very crude and a dedicated black-box model may be easier to use.

During this study a new model (STSim) was developed. The sediment transport model is a lumped conceptual model but this model is based on detailed modelling of the hydrodynamics. This is feasible as the model is event based and not continuous. If on-line modelling or modelling of many events for extreme statistics is desired the hydrodynamic modelling becomes laborious. With the current computer power and hydrodynamic models this problem is, however, manageable.

The STSim model performs well as seen in chapter 7 and in table 10.1. Different conceptual submodels were examined in connection with the case study of Le Marais. The investigation only showed small differences in the results when calibrating STSim applying different conceptual model for the subprocesses. In order to evaluate the possible submodels (e.g. wash-off, erosion, deposition) the semi-automatic version of STSim should be exploited.

The most crucial parameter in STSim is the initial mass in the sewer system ($M_{p,initial}$). The first test of the STSim model showed that individual values of the initial mass is needed for each event. A conceptual model less dependent on the initial mass could be a way to improve the model. It is difficult if not impossible to measure the initial mass in the sewer system. As the model is very dependent on a good estimate of the $M_{p,initial}$ parameter improvement of the model may be sought by improving the way to estimate $M_{p,initial}$. By making a continuous version of STSim, i.e. modelling both rain events as well as ADWP, the initial mass could be calculated. As the Le Marais catchment showed little correlation between ADWP and the sediment transport at the outlet (see appendix A) the ADWP cannot be modelled simply dependent on this parameter, equal to the build-up model for the surface. Which parameters to include during dry weather need further investigations.

Furthermore, it can be concluded that assisted calibration as performed by the semi-automatic version of STSim presented in chapter 8, relieves the user for a great work load. This version of the model successful helps in finding an optimal set of calibration parameters. This can be seen by the fact that the obtained correlation coeficient from the calibration and the validation respectively arrives closer to each other as seen in table 10.1.

Finally, a neural network model (STNeuro) has been developed and applied to the case study. This model does not perform quite as well as the STSim model. This indicates that there is a clear advantage of including the detailed modelling of the hydrodynamics in the modelling of sediment transport in combined sewer systems. Furthermore, the neural network lacks the possibility to make spatial predictions, e.g. regarding locations of sediment deposits.

A common condition for all the modelling carried out during this study is the choice of calibration and validation events. These were chosen by random, but a set of calibration events with higher rain intensities could yield better results. Of course, the best option would be to include more rain events all together. This does, however, often turn out to be problematic as measurement campaigns are expensive and difficult to carry out. This fact is one of the major problem involved in numerical modelling of sediment transport.

To summarize, the lumped conceptual model STSim has shown to be capable of modelling the sediment transport in the sewer system of "Le Marais". The success of this model type can partly be explained by the use of a detailed hydrodynamic model (MOUSE) as basis for the sediment transport modelling. Improved quality of the model can be investigated by testing different formulations of the different conceptual models used. Another logical step is to apply the models to other catchments.

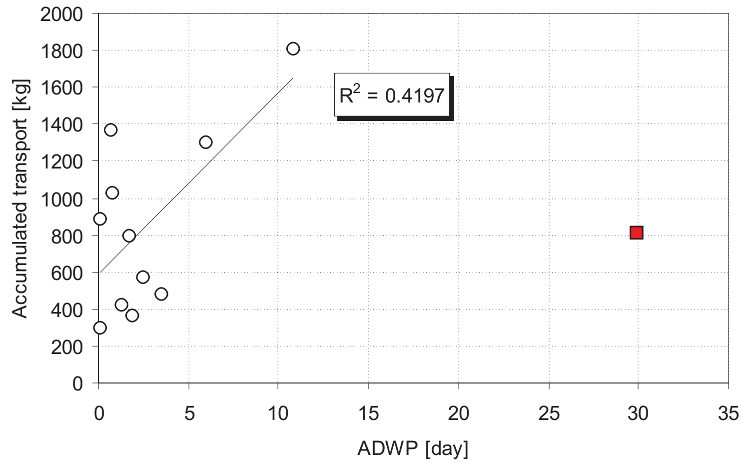
The perspective of the modelling is to be able to predict the mass transport of sediments at different locations in the sewer system with emphasis put on the outlet. This would facilitate dynamic modelling of CSO discharges to the environment. Prediction of sediment build-up in the system and self-cleansing efficiency is also relevant. Only the physically based model like MOUSE TRAP ST-module and the lumped conceptual model is aimed at modelling the spatial variability of the sediment transport. When improved modelling of the sediment transport is achieved as done in this study it becomes feasible to model other pollutant parameters such as COD and heavy metals through e.g. potency factors.

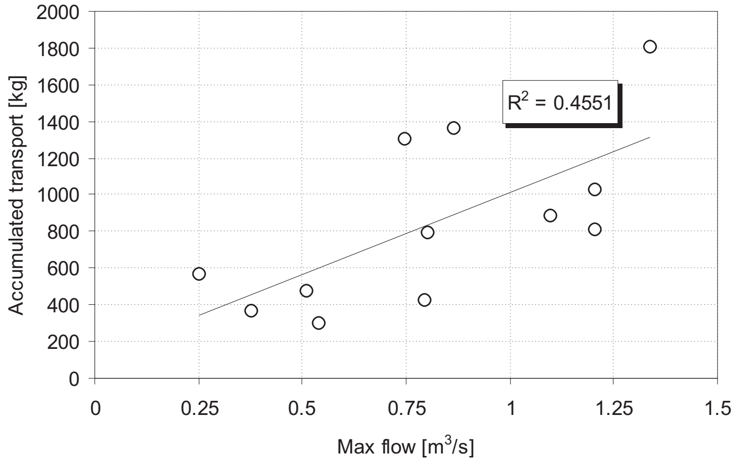
Different model types have been developed and tested during the study. The collocation of these models makes it possible to determine some of the strengths and weaknesses of each model type. Based on the results from this study the lumped conceptual model seems the most promising.

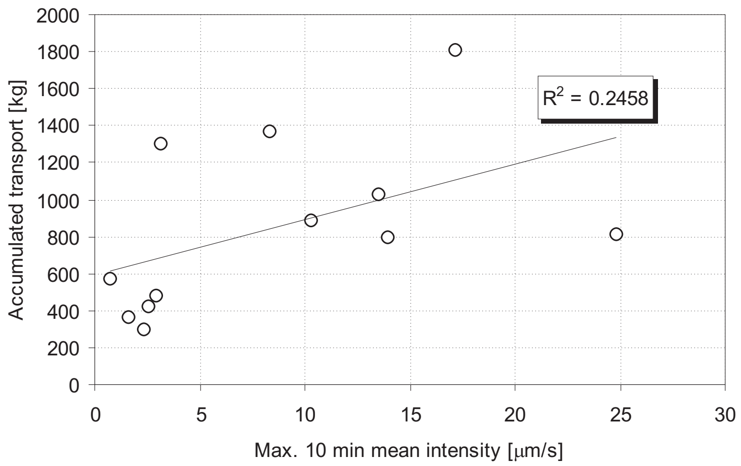
APPENDIX A

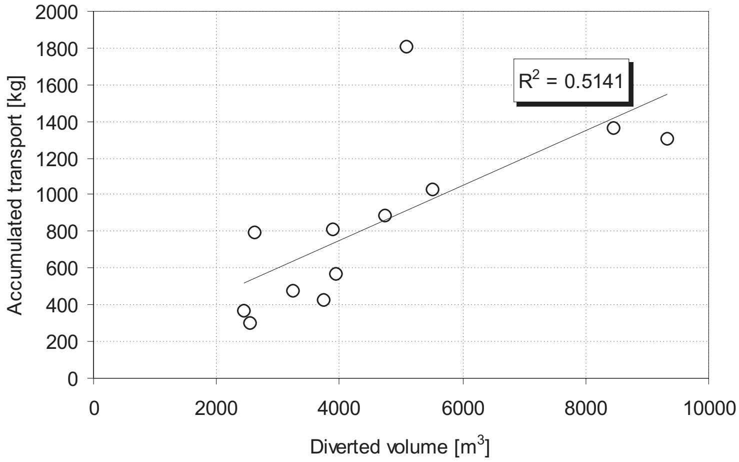
Les Marais data plots

This appendix shows various plots where the accumulated transport measured at the outlet of "Le Marais" is plotted against measured values of a number of parameters.

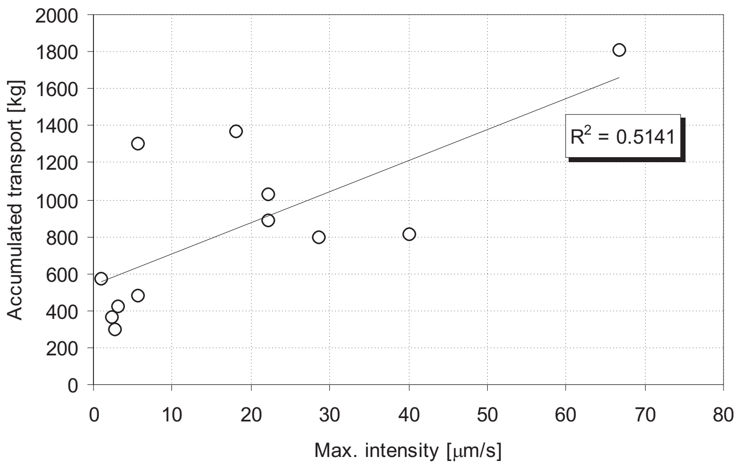



R² = 0.4551
Accumulated transport [kg]
Max flow [m³/s]
0
200
400
600
800
1000
1200
1400
1600
1800
2000
0
0.25
0.5
0.75
1
1.25
1.5

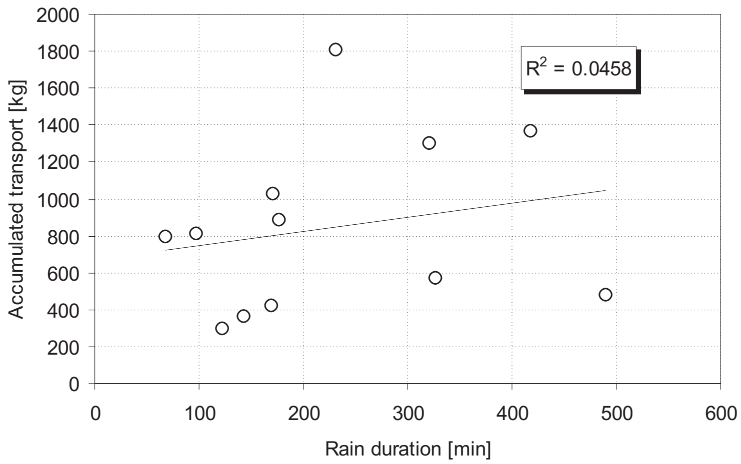

R² = 0.2458
Accumulated transport [kg]
Max. 10 min mean intensity [μm/s]
0
200
400
600
800
1000
1200
1400
1600
1800
2000
0
5
10
15
20
25
30

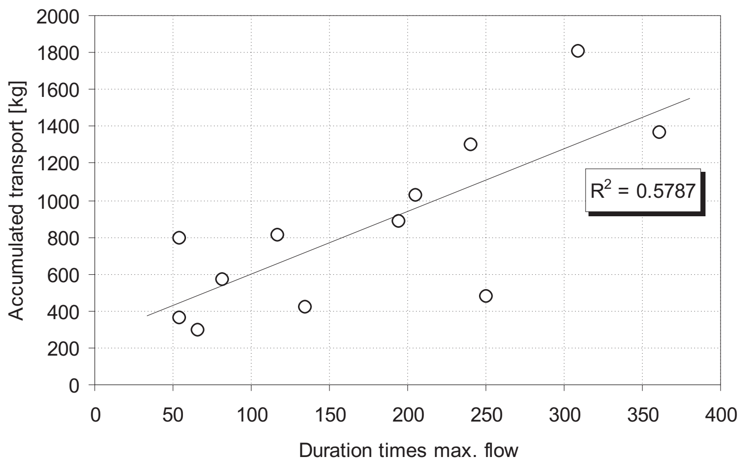


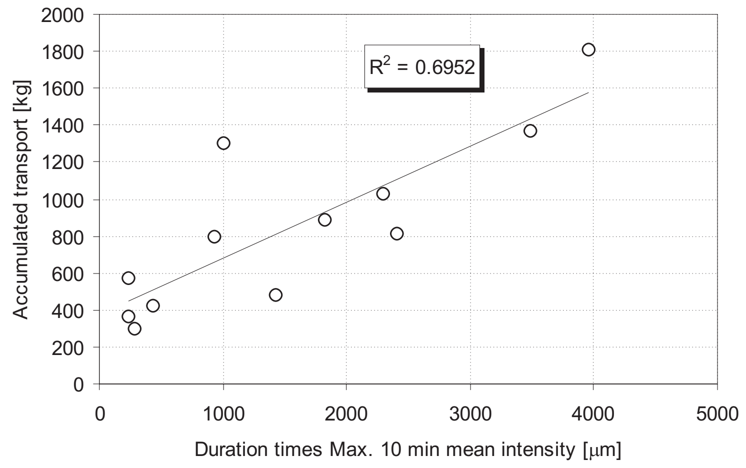
Accumulated transport [kg]
Diverted volume [m^3]
R^2 = 0.5141
0
200
400
600
800
1000
1200
1400
1600
1800
2000
0
2000
4000
6000
8000
10000



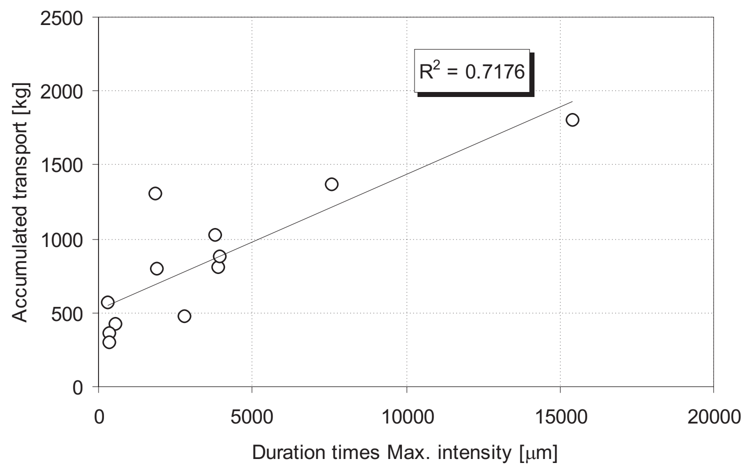
Accumulated transport [kg]
Max. intensity [μm/s]
R^2 = 0.5141
0
200
400
600
800
1000
1200
1400
1600
1800
2000
0
10
20
30
40
50
60
70
80


Accumulated transport [kg]
Rain duration [min]
R^2 = 0.0458
0
200
400
600
800
1000
1200
1400
1600
1800
2000
0
100
200
300
400
500
600


Accumulated transport [kg]
Duration times max. flow
R^2 = 0.5787
0
200
400
600
800
1000
1200
1400
1600
1800
2000
0
50
100
150
200
250
300
350
400



$R^2 = 0.6952$
Accumulated transport [kg]
Duration times Max. 10 min mean intensity [μm]
0
200
400
600
800
1000
1200
1400
1600
1800
2000
0
1000
2000
3000
4000
5000



$R^2 = 0.7176$
Accumulated transport [kg]
Duration times Max. intensity [μm]
0
500
1000
1500
2000
2500
0
5000
10000
15000
20000

APPENDIX B

Modelling software

This appendix serves as an introduction to three software programs developed for the modelling applied in this thesis. It gives an indication of how difficult the programs are to use. Furthermore, the manuals should help when the programs are used later on.

B.1 STSim Manual

When running the model it is possible that problems with the sizes of the windows and use of delimiters for time and decimal points may be experienced. The software is developed for a screen resolution of 800 by 600 pixels using small windows fonts. The time delimiter is ':', the date delimiter '-' and the decimal point '.'. It is recommended to apply these settings to Windows 95 before using the model. The software requires Windows 95, but may run on other platforms (Win 3.1 32s, Windows NT). In order to have a reasonable calculation speed a Pentium 133 Mhz or better should be used. Furthermore, plenty of free hard disk space is required for the windows swap file and for saving the results. At least 500 Mb are recommended, about 200 Mb will be taken up by the swap file in the case of "Le Marais".

Figure B.1 shows the main window in STSim which is opened when the program starts. In order to get as far as the window shows, namely running the model, one has to open the input files. This is done by the "open" button. The *.MIF file is chosen from the file dialog window. All other input files must be located in the same directory as the *.MIF file and bear the same name in front of the extension. After the input data has been read the model is ready to run. First

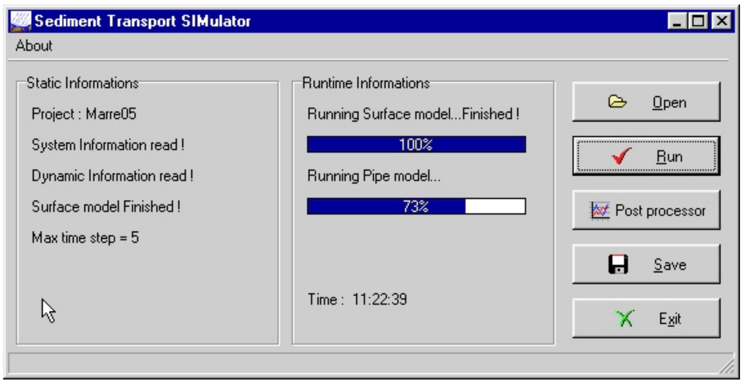


Figure B.1: *Main window in STSim.*

the surface model is executed and it is subsequently necessary to prompt for the execution of the pipe model. If the timestep in the prepared input data is considered to be too large there will be an possibility to stop the model. If the required timestep is close to the actual one, one may want to execute the model anyway. The computer gives at short beep when the surface model is finished and as well as when the pipe model is finished.

The model holds all input data and results in the computers memory and no results are saved on files unless this is specifically done by pressing the save button. This can be done prior or after using the post processor. Note that it is not possible to save data under new names, so if old results is desired to be kept this must be done by moving of renaming these files (*.CRF, *.MRF) before saving new data.

The post processor (see figure B.2) can be reached at any point in time.

Dependent on whether hydraulic input data or results are present in the memory or saved data is available on the disk different data types can be accessed. If input data has been opened from the main window and results from the pipe model are available and the "Hydraulic model" and the "Pipe model" checkboxes are checked the post processor will calculate mass transport rates and bed shear stresses in the system. The task of reading data from the disk and carrying out calculations may take a while dependent on the size of the job.

The use of the post processor is more or less self explanatory. As seen in figure B.2 it is possible to import measured data from a file in order to compare with calculated results. The file structure for the input file is as follows. First line must state a title. Data is give in two columns separated by space first column being the time and the second the values. Time is given as "hh:mm:ss". The

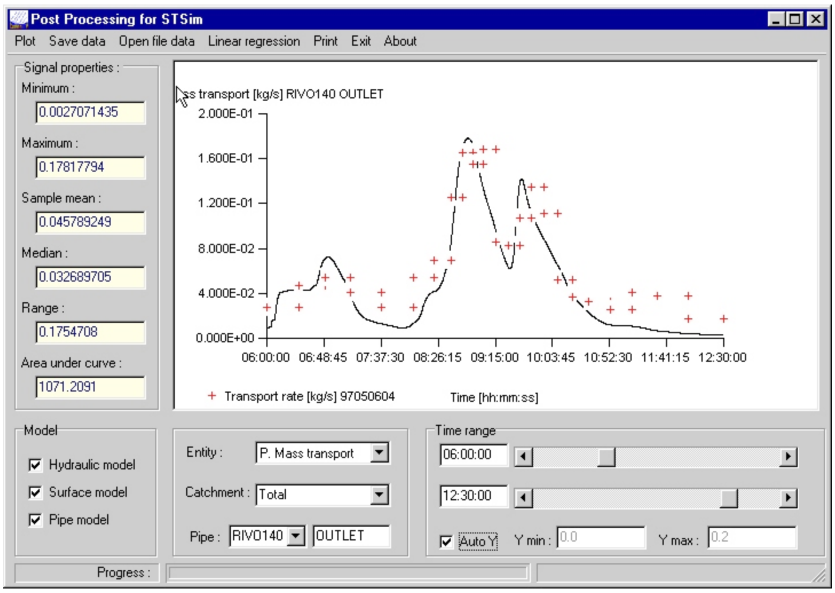


Figure B.2: *Post processor window in STSim.*

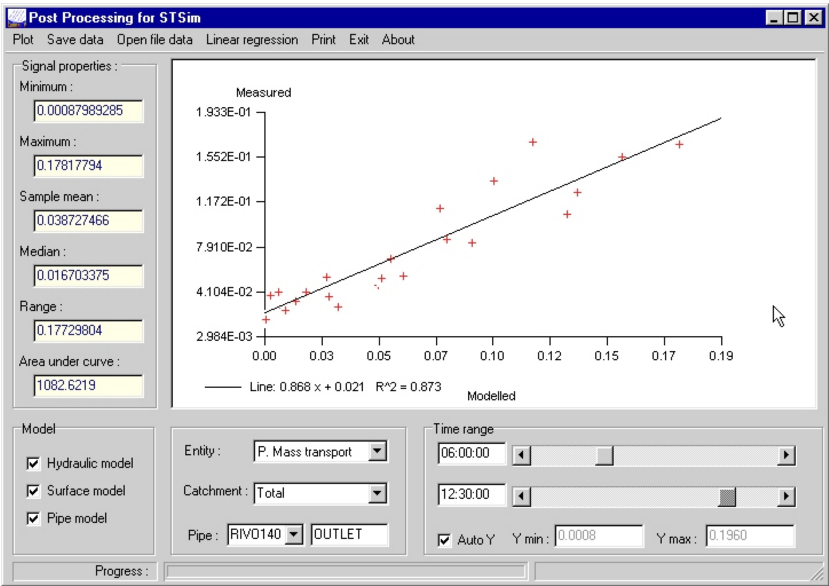


Figure B.3: *Post processor window with scatter plot in STSim.*

result of pressing the "Linear regression" option in the menu after data has been imported is seen in figure B.3. Data can also be saved on file with the option to subsample the calculated results or making a pooling average. All calculations shown in the post processor window as well as saving results are carried out on the data seen as the graph. Update of the plot is done by either pressing the "plot" option in the menu or simply by clicking somewhere in the graph area.

If when choosing a pipe the downstream pipe is not the desired one this can be entered manually. The only possibility to show more than one set of data in the graph area is first to save the data, plot the other graph and import the earlier data again.

B.2 Step by step preparation for STSim

The STSim model is not integrated with MOUSE, it merely uses the results from MOUSE when translated into a text format. Some consideration must be done when running the MOUSE model. Furthermore a set of text files must be extracted from MOUSE and afterwards prepared for STSim. The subtraction of data from the MOUSE result files may be done within MOUSE itself or by the use of the program called: "M11extra.exe" documented in M11extra.doc also distributed by DHI.

B.2.1 Using the Mouse model

When using the MOUSE model for later use with STSim it is important to notice:

1. The time between saving results from the run-off model must be the same as in the pipe model. This time must be equal to the timestep Δt applied in STSim.
2. The total length of the simulation time must be the same in the run-off model and the pipe model.
3. The number of timesteps in STSim equals that of MOUSE plus one.
4. The text browser page setup must not be changed, writing 66 lines of timesteps before a page shift. (only important if not using "M11Extra.exe").
5. STSim is developed based on MOUSE 3.40.

B.2.2 Preparing to use STSim

First of all a number of inputs is needed. Catchment size and location is acquired from a *.SWF file form MOUSE converted into text format and named *.MIN, where all "!" signs are removed. In this file information on the slope of the catchment and the number of inhabitants connected to each of the subcatchments is also located. In the model input file named *.MIF the size of the timestep, number of timesteps, number of catchments, calibration factors (α_{xxx}) and initial load on the surface must be given. Finally, the model needs the calculated run-off rates from MOUSE, which mouse saves in its *.rrf file. From this file results must be saved in text format in a file named *.CFL. As for all other files originating from MOUSE the "!" signs needs to be removed.

Above mentioned trouble of editing mouse-files can be avoided by using the M11Etra.exe program from DHI when it comes to *.cfl, *.dfl, *.vfl and *.wfl files. The program is used by first saving selections in MOUSE and then run M11Extra by e.g.:

```
m11extra Marre05.prf e:\marais\m05\Marre05.wfl wfl.rsf
```

In order to run STSim, 7 or 8 files need to be prepared which all have to be placed in the same directory. The files are prepared by:

- *.CFL : Catchment discharge file. In mouse choose runoff results, print and plot, select all data and print tables. By menus this is 4.G.5 1 3. Save the file and remove "!" signs by replacing with space. If the amount of data is too large for the DHI text browser save the selections and use the M11extra program to subtract the data.
- *.MIN : Convert the *.SWF file by use of catchment goodies menu A.8.2, SWF to TXT, MOUSE format text 2 and no line numbers. Further editing of the text file is not needed.
- *.MIF : Please see chapter 7.
- *.PIN : Choose pipe model results, print summary and state Y to 6. print Summary, thus 2.4 to get to the right menu. Page shifts in the file should be erased.
- *.DFL : Choose pipe model results, print and plot(menu 2.5), select discharge data by first grid point in each pipe. Save the data and replace "!" and "-" signs by space.
- *.VFL : See *.DFL.
- *.WFL : See *.DFL.

CBFfile.txt : If special cross-sections from a cross-section database is used in the MOUSE model a converted version of the CBF files must be made named CBFfile.txt. This is done by choosing catchment data, thus go through menus A.9.6 and then export text file.

B.3 STSim with semi-automatic calibration

This version of the STSim model is very similar to the one described above. The main difference is that a number of events are processed by the model repeatedly. In this way the optimal combination of input values can be obtained.

In order to run the model, all the events used for the computations must be prepared as explained in paragraph B.2.2. All the files should be put in the same directory. The filenames (*.CAL), containing the measured data from the events, are given in the top of the *.MIF file. This file now states the events to be processed and how to vary the input parameters. An example of the *.MIF file is given below.

```
[CALIBRATION FILES]
5
D:\MARAISOP\STSimSM\Mop15.CAL
D:\MARAISOP\STSimSM\Mop05.CAL
D:\MARAISOP\STSimSM\Mop09.CAL
D:\MARAISOP\STSimSM\Mop12.CAL
D:\MARAISOP\STSimSM\Mop14.CAL

[DELTA T]
5 5 5 5 5 seconds

[NUMBER OF TIMESTEPS]
4902 8176 4122 6391 3601

[ENTITY]
                                       Lower limit Upper limit Step
=================================================================
SURFACE ACCUMULATION RATE       (kg/ha/day)  0.0      0.0      1.0
SURFACE DECAY RATE                  (1/day)  0.0      0.0      0.25
SURFACE WASHOFF COE. 1                  (-)  0.01     0.01     5.0
SURFACE WASHOFF COE. 2                  (-)  1.6      1.6      1.0
PIPE DEPOSITION RATE                (1/day)  0.11     0.11     0.05
PIPE STORAGE CAPACITY           (% of vol.)  2.0      2.0      1.0
PIPE EROSION RATE                   (1/m^3)  0.001    0.001    0.001
CRITICAL BED SHEAR STRESS DEPOSITION (N/m^2) 0.5      0.5      0.2
CRITICAL BED SHEAR STRESS EROSION   (N/m^2)  1.0      1.0      0.25
INITIAL SURFACE LOAD                (kg/ha)  70.0     70.0     50.0
INITIAL PIPE LOAD                  (% of SC) 0.01     2.0      0.01
INITIAL WASTEWATER CONCENTRATION   (kg/m^3)  0.15     0.15     0.1
```

```
COMPUTATIONAL SEDIMENT DENSITY      (kg/m^3)  1600.0  1600.0  200.0
DAILY PE DISCHARGE              (kg/day/PE)  0.1     0.1     0.1
====================================================================
(Please do not alter sequence of input parameters in the list above)
```

If the value for the lower limit or the upper limit values are the same the model hold this parameter constant. As seen in the example above all the values are the same and the model only runs the five events once calculating the error, the correlation coeficient R^2 and the objective function.

The *.CAL files containing the measured target values for the model, must state the location of the measurement by the upstream node and the downstream node. Additionally the maximum transport rate and the accumulated transport during the event must be stated. The layout of the *.CAL file is seen below.

```
[LOCATION]
RIV0140
OUTLET

[CALMTMAX]
0.163 kg/s

[CALMTACC]
1368 kg

Transport rate [kg/s] 97050604

[CAL MASS TRANSPORT]
23
06:13:30   0.027839
06:38:30   0.046889
...
```

The graphical interface for the model is very simple. A screen dump of the model is seen in figure B.4.

When executing this model none of the physical modelling results, such as sediment transport rates are saved. In order to get the actual modelled results it is necessary to run the original STSim model again for the specific combination of input parameters.

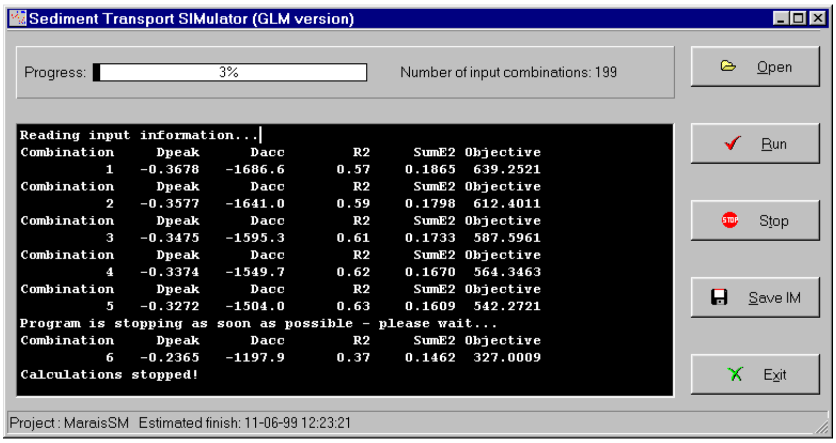


Figure B.4: *User interface for STSim with semi-automatic calibration.*

B.4 STNeuro Manual

The neural network model is completely different from the STSim models. This a black-box model tailored to the purpose of modelling sediment transport in sewer systems. It is, however, also applicable for other problems. The main architecture of the model enables the user to input one input time series which can be tabbed delayed as desired. Furthermore, a number of extra input neurons can be used. Subsequently, up till ten layers of hidden neurons can be added to the network layout. The model does, however, only estimate one output signal. As for the STSim model, the STNeuro model is also implemented in Delphi 4.0, which makes it very easy to change the graphical user interface, but also the computational models behind.

Figure B.5 shows a screen dump from the model.

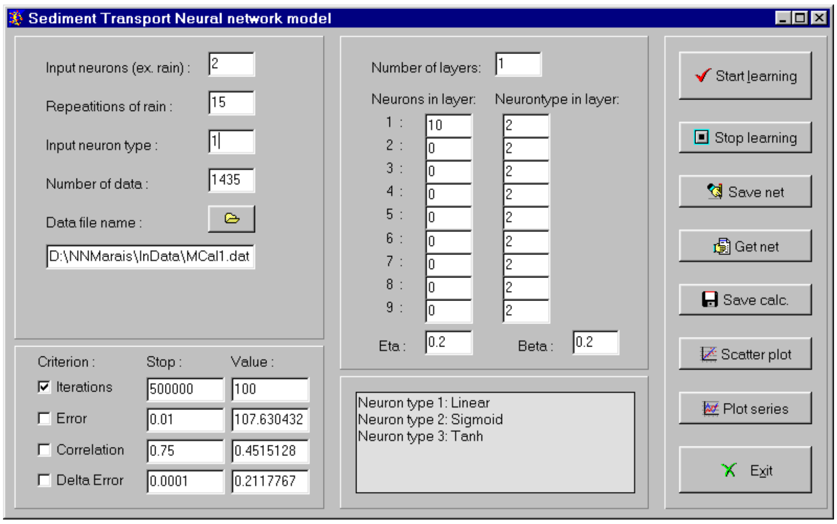


Figure B.5: *User interface for STNeuro.*

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Hydraulics & Coastal Engineering Group
Aalborg University
Series Paper No. 19
ISSN 0909-4296