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Sewer sediment production and transport modelling: A literature review

Modélisation de la production et du transport des solides en réseau d'assainissement: Etude bibliographique

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SUMMARY
This literature review presents the state-of-the-art concerning sediment sewer transport modelling. After a description of solid particles found in domestic sewage and in storm water, the different steps taken into account in models are described: build-up over the catchment, washoff by rainfall, transfer through gully pots, transport, deposition and erosion in sewer pipes. For each step, several modelling approaches are presented with their basic equations. The paper ends with a short presentation of some models, and with some remarks about the comparison between models and field experimental data.

RÉSUMÉ
Cette synthèse bibliographique décrit les principes de la modélisation du transport solide en réseau d'assainissement. Après une description des particules solides observées dans les eaux usées domestiques et les eaux pluviales, les différentes étapes prises en compte dans les modèles sont décrites: accumulation à la surface du bassin versant, lessivage par la pluie, traversée des avaloirs, transfert, sédimentation et érosion dans les collecteurs. Pour chacune de ces étapes, les différentes approches modélisatrices et leurs équations de base sont indiquées, en terminant par une présentation sommaire de quelques logiciels. Quelques remarques importantes sur la comparaison des modèles et les données expérimentales in-situ sont faites en fin d'article.

1 Introduction

With the increase of urbanization in cities, problems due to domestic sewage and storm water become more and more important: floods, overflows, pollution of receiving waters, high exploitation costs due to sediment deposits... [10, 14, 32, 40, 99].

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To try to understand and minimize these problems, a possible approach is the modelling one. A common distinction is made between hydrological and hydraulic models, which reproduce rainfall-runoff processes in sewer systems, and quality models which reproduce pollutant loads or concentrations at the sewer outlet.

Hydrological and hydraulic models are now well established, and are becoming more detailed. Generally, they are conceptual for their hydrological part, and conceptual or physically based (deterministic) for hydraulic transfer [19, 45, 65, 122, 141].

Quality models are more recent [70], and more difficult to establish and calibrate, particularly because of the lack of good and adequate field experimental data, and, of course, because the phenomena themselves are very complex and not very well understood. Many pollutants can be taken into account: chemical oxygen demand, nitrogen, total suspended solids, heavy metals... Only models for solid concentrations and loads will be presented hereafter. This choice is due to the major importance of sediment problems in urban hydrology. After a short description of the particles transported in sewers, we will present successively:

- the accumulation of sediments over an urban catchment;
- the washoff of these sediments by rainfall;
- their passage through gully pots;
- their transfer, erosion and deposition in sewer pipes.

For each phase, the distinction will be made, if necessary, between conceptual and physically based models.

This paper simply presents the basic equations of the models, but not their detailed working or comparative simulation results (see paragraph 8). Nevertheless, the accuracy of these models is an important question. The agreement between observed and calculated values is an usual criterion of estimation, which can be quantified thanks to objective functions like least squares method. Many urban hydrologists admit that a quality model gives "satisfactory results" if the overall curve of the calculated pollutograph is similar to the observed one. Such a similarity can include discrepancies for local points, peak values or time shifts. These discrepancies are generally accepted because:

- the measured values are themselves not accurate due to sampling techniques, with errors in a range of 20 to 50% or more.
- the equations are often rough approximate compared to the complexity of the physical phenomena.

However the discrepancies have to be appreciated in relation to the objectives of the user: does he want an accurate value at each time step, or an overall estimation of the total load for a storm event? Some quality models for sewer design just give an order of magnitude.

The expression "satisfactory results" is then partly subjective. From a scientific point of view, it shows that further research is obviously needed to improve basic knowledge and modelling approaches, even if some models are already employed by engineers to design sewer systems. The pollution of stormwater is now a so important problem that many solutions have to be used, even if the knowledge of the phenomena and the capabilities of the models are still not well defined.

2 Sediment and particles characteristics

2.1 *Origins and concentrations*

Many studies have been carried out about domestic sewage and storm water pollutions to evaluate total suspended solids (TSS) concentrations. For domestic sewage, the mean values of TSS concentrations are generally between 80 and 500 mg/l [15, 23, 50, 81, 119].

The pollutographs observed at the outlet of combined sewers during dry weather periods show variations of concentration which are well correlated with flow rate (see Fig. 1) [13, 34, 57, 132].

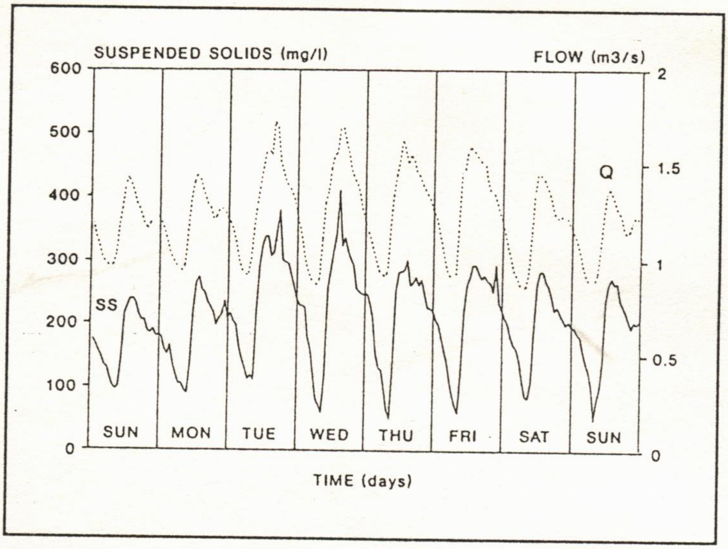

Fig. 1. Correlation between dry weather flow rate and TSS concentration [132].
/ Corrélation entre le débit par temps sec et la concentration en matières en suspension TSS [132].

Many authors consider that some of these particles deposit during the night, when the flow velocity is small, and are partly eroded by the flow increase at the beginning of the morning [34, 57, 82, 132]. The mass of deposits then increases during dry weather period [66, 110].
TSS concentrations in storm water present greater variations. The results obtained during the twenty last years show that these concentrations are not negligible. Some values are given in Table 1. It is important to notice that these concentrations can be much greater than dry weather concentrations.

Table 1. Values of TSS concentrations in storm water (mg/l).

minimum	mean	maximum	source
combined sewers			
200	341	700	[15]
176	425	647	[53]
	484		[115]
49	487	1863	[34]
	589		[81]
126	674	1760	[119]
seperate sewers			
21	190	2582	[53]
	235		[115]
	191-473		[64]
	496		[81]

Particles in storm water originate essentially from the impervious surfaces of the catchment (streets, roofs, kerbs, parking lots) and from the atmosphere (dust). Many parameters are required to explain the TSS concentrations in storm water: antecedent dry weather period, rainfall characteristics (especially rainfall intensity), land use, urbanization, geometry of catchment and sewers...[116] .

During a storm event, it can be observed that about 50% of the TSS load are transported by the first 30% of the flow volume: this phenomenon is called "first flush", and its amplitude can vary in a large extent [15, 16, 34, 52, 56, 58, 82, 83, 91, 92]. The first flush is not always observed and it is more important for small catchments.

Generally, peaks of concentration and peaks of flow rate occur at the same moment (see Fig. 2) [15, 34, 64, 90].

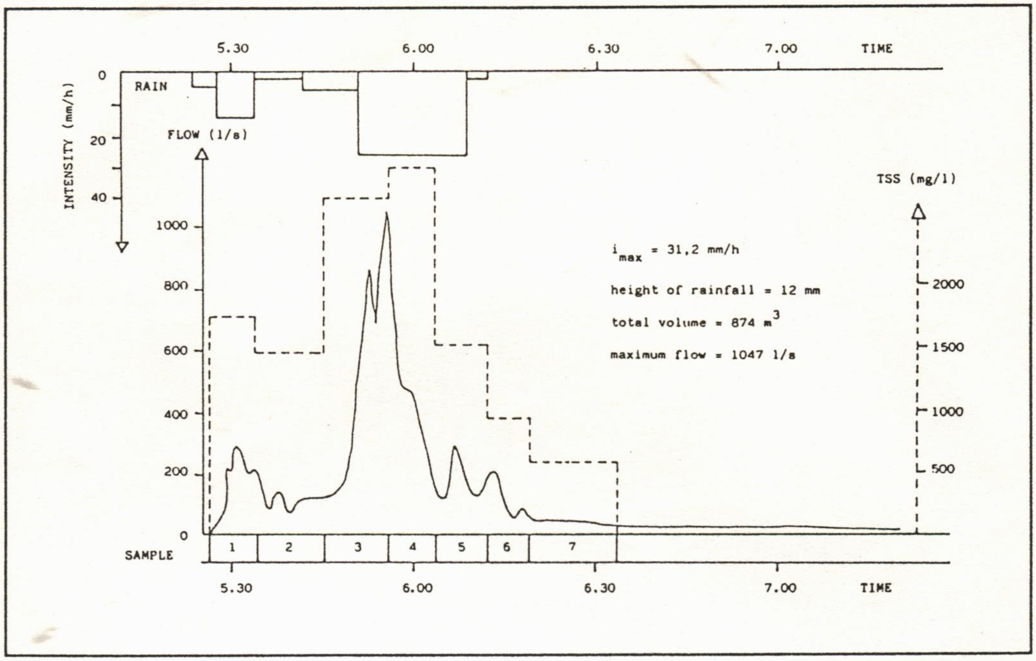

Fig. 2. Pollutograph for a storm event in a combined sewer [119].
Diagramme de pollution dans un réseau unitaire pendant un épisode d'orage [119].

2.2 *Organic and mineral fractions*

Particles transported by domestic sewage are essentially organic: they contain about 70–80% of organic matter, because of their domestic origin [15, 34, 56, 119]. Particles in storm water are more mineral (about 60% of mineral matter) and the mineral fraction increases with storm duration [34, 119]. These percentages are mean values: great discrepancies are observed depending on the catchment and on sampling locations.

2.3 *Size distributions*

Many authors have worked on particle size distribution [9, 27, 36, 37, 38, 40, 58, 85, 90, 97, 132] and, although a great variability according to the experimental site is observed, the following results can be given:

- *particles deposited along roads and kerbs:* their diameter is usually between 200 and 1000 μm, with a mean median diameter d_{50} of about 300–400 μm (specific gravity: 2.6).
- *particles in domestic sewage:* the d_{50} is about 30–40 μm (specific gravity: 1.5).
- *particles transferred in sewers by rain weather:* they are very fine, with a median diameter of 30–40 μm, and are transported essentially in suspension (specific gravity: 2.4) [39, 43].
- *particles deposited in sewers:* they are larger than those transported by suspension: their d_{50} is about 200–1000 μm [12, 42], and they are transported essentially by rolling and saltating along the sewer invert (specific gravity: 2.6).

Along a sewer trunk, the mean diameter of deposited particles diminishes from inlet to outlet: there is a granulometric sorting with distance [11, 86]. Some authors have observed an increasing median diameter of deposited particles at the same location with time, due to the consolidation of deposits by organic matter [9, 38, 79, 94] and/or by chemical precipitation [121].

2.4 *Pollutant load*

Many pollutants are attached to solid particles. The observations show that the smaller the particle, the more important the pollutant load, especially for heavy metals or toxics [33, 38, 52, 63, 123, 128] (see Table 2).

Table 2. Percentage of total particulate pollution load associated with the different particle size fractions [39].

particle size fractions (μm)	COD	BOD	TKN	hydrocarbon	Pb
>250	28%	28%	26%	69%	13%
50–250	4%	20%	58%	4%	34%
<50	68%	52%	16%	27%	53%

3 Accumulation of particles on the catchment

This is the first phase of every model. The build-up of sediments is usually supposed to be linear or exponentially asymptotic with time. Both approaches are proposed in the literature, and experimental results do not allow to choose definitely between them.

3.1 *The Storm Water Management Model*

The Storm Water Management Model (SWMM) is known worldwide. Its first version was created at the end of the sixties for the US EPA (Environment Protection Agency). The accumulation process is governed by the following equation which leads to an exponentiel asymptotic build-up [7]:

$$\frac{\mathrm{d}Ma}{\mathrm{d}t} = \mathrm{ACCU} - \mathrm{DISP} \cdot Ma \tag{1}$$

with

Ma	accumulated mass of particles at time t (kg)
t	time (d)
ACCU	daily accumulation rate (kg/d)
DISP	disappearing coefficient (d^{-1})

DISP is a coefficient representing particle removal due to wind, traffic, biological and biochemical degradation, street sweeping. *Ma* increases until an upper limit equal to ACCU/DISP is reached. Sartor et al. [123] found that this limit was reached after about 10 days, but this value is subject to great variations according to site and weather conditions.

The values of ACCU and DISP depend on urbanization, site, weather, and must be determined by calibration for each catchment. In such a model, the antecedent dry period seems to play an important role which is not easily explained.

Because the SWMM is well known, several models use the same basic relation, like the French model FLUPOL [30, 31], the German one THALIA [73], and others like NPS [93] or STORM [135].

3.2 *Servat's model*

Servat [124] studied different accumulation models (asymptotic, power, linear and parabolic functions), and he proposed to use a simple linear relation:

$$Ma_i = \mathrm{ACCU} \cdot \mathrm{DTS}_i \qquad (2)$$

with

Ma_i mass of particles over the catchment (kg/ha) accumulated during a dry weather period DTS_i (d)

ACCU accumulation rate ($\mathrm{kg \cdot ha^{-1} \cdot d^{-1}}$)

According to French field data for small catchments with separate sewers, this relation has given the best values of accumulated sediments for a long period simulation (about 1 year).

4 Washoff by rainfall

During a storm event, the accumulated particles on impervious areas (streets, kerbs, roofs) are washed off. Many parameters are involved in this phenomenon [124]: rainfall intensity, rainfall height, rainfall duration, runoff peaks and volume, topography, particles characteristics. The most important among these parameters is the rainfall intensity, and especially the square of the maximum rainfall intensity [22, 125]. According to different authors, the particles are washed off by rain drop impact, and transported into sewers by surface runoff [1, 124, 142].

4.1 *Deterministic models*

Little research has been carried out on the physical process of washoff over urban catchments. Unfortunately the models proposed for soil erosion in agricultural or natural sites [29, 119, 143] are not easily adaptable for urban hydrology.

4.2 *Conceptual models*

The washoff is such a complex process that physically based models are replaced by conceptual or global models in urban hydrology. They usually do not separate erosion by rain drops and transport by runoff. Sometimes, they are reduced to a simple "washoff coefficient".

4.2.1 The SWMM

The washoff process is supposed to be in direct ratio to the available mass of accumulated particles and to the rainfall intensity. This assumption is written [6, 77]:

$$\frac{dMa}{dt} = -Ke \cdot i(t) \cdot Ma \tag{3}$$

with

Ma = accumulated mass on impervious surfaces at time t (kg)
$i(t)$ = rainfall intensity at time t (mm/h)
Ke = washoff coefficient (mm^{-1})

The standard value of $Ke = 0{,}18\ mm^{-1}$. However it has been shown that Ke needs to be calibrated for each catchment [6]. If Me is the mass of particles entering into the sewer during a time step, the relation becomes:

$$Me(t+\Delta t) = Ma(t) \cdot [1 - e^{(-Ke \cdot i(t+\Delta t) \cdot \Delta t)}] \tag{4}$$

An adaptation to very small or very strong rainfall is possible thanks to the introduction of an "availability factor" Kd:

$$Me(t+\Delta t) = Kd \cdot Ma(t) \cdot [1 - e^{(-Ke \cdot i(t+\Delta t) \cdot \Delta t)}] \tag{5}$$

with

$Kd = 0.057 + 0.04.i(t)^{1.1}$ for suspended particles
$Kd = 0.028 + 0.003.i(t)^{1.8}$ for deposited particles

In the model FLUPOL [30], a lumped coefficient $Ke' = Ke \cdot Kd$ is used and the basic equation becomes:

$$\frac{dMa}{dt} = -Ke' \cdot i(t) \cdot Ma \tag{6}$$

Another refinement is the introduction of an exponent ω for rainfall intensity in relation (4), with $0.8 < \omega < 2$. With this last refinement, the model finally depends on the square of rainfall intensity and should allow a better calculation of peak values. In spite of all refinements and parameters introduced in relation (3), the results are not significantly improved.

4.2.2 NPS model

In the NPS (Non Point Source) model, Litwin and Donigian [93] proposed the following relation for impervious areas, where the washoff depends on the available mass and on the surface runoff:

$$\begin{array}{ll} Me(t) = Keni \cdot S_b^c & \text{if } Me(t) \le Ma(t) \\ Me(t) = Ma(t) & \text{if } Me(t) > Ma(t) \end{array} \tag{7}$$

with

$Me(t)$ = washed off mass at time t (ton/ha)
$Ma(t)$ = accumulated mass at time t (ton/ha)
$Keni$ = washoff coefficient
S_b = surface runoff on impervious area (mm)
c = numerical coefficient

In spite of great discrepancies, Litwin and Donigian considered that their results were satisfactory. Models using rainfall intensity are probably more realistic for the washoff process itself.

4.2.3 MOSQITO model

The MOSQITO model proposed by Hydraulics Research Wallingford [67, 105] uses the following equation:

$$\frac{dMa}{dt} = a_i \cdot i^{1.5} + a_e \cdot (\tau_0 - \tau_{ce}) + a_d \cdot (\tau_{cd} - \tau_0) - \frac{Ma \cdot q_b}{K \cdot q_b + Sm} \tag{8}$$

with

a_i, a_e, a_d = Price-Mance coefficients
i = rainfall intensity
τ_0 = mean shear stress
τ_{ce} = critical shear stress for erosion
τ_{cd} = critical shear stress for deposition
q_b = surface runoff
Sm = soil depression storage
K = linear reservoir parameter (the linear reservoir model is used to reproduce the rainfall runoff process)

The first calibrations [113] have shown that the second and third terms of relation (8) were negligible compared to the first and fourth ones. The relation can then be simplified and rewritten as:

$$\frac{dMa}{dt} = a_i \cdot i^{1.5} - \frac{Ma \cdot q_b}{K \cdot q_b + Sm} \tag{9}$$

This relation is the only one which distinguishes the washoff by rainfall and the erosion by surface runoff. However the results are not significantly better than those given by other models, probably because it needs more parameters whose calibration remains approximate.

4.2.4 Servat's model

Using several field data and statistical analysis, Servat [124] proposed the following expression:

$$Me = Ks \cdot Ma^{a} \cdot I_{\max 5}^{b} \cdot Vr^{c} \tag{10}$$

with

Me = washed off mass (kg)
Ma = accumulated mass (kg)
Ks = washoff coefficient
$I_{\max 5}$ = maximum rainfall intensity during a time step of 5 minutes (mm/h)
Vr = runoff volume (m^3)
a, b, c = numerical coefficients

The results have an overall accuracy of about 5% for a long period (several months), but the accurary decreases to 10–30% for particular events. Servat used the maximum intensity for 5 minutes, but the value for one minute (available with new devices) would be better, because the washoff phenomenon depends on instantaneous rainfall intensity peaks.

4.2.5 Other models

Many other models have been proposed [17, 26, 27, 55, 58], which are often a simple variation or an adaptation of the preceding ones. All of them need to be calibrated for each catchment before they can be used.

A last category exists: the statistical models. They can only give mean results, and are unable to reproduce satisfactorily a particular event. Their domain of validity is limited to the experimental sites which were used for their establishment [78].

A recent example has been proposed by Driver and Troutman [46]. One of the regression equations is (for areas in the USA with annual rainfall less than 500 mm):

$$Me = 14.374 \cdot H_t^{1.211} \cdot A^{0.735} \cdot d_p^{-0.463} \tag{11}$$

with

Me = washed off mass during the rainfall event (kg)
H_t = total rainfall height (mm)
A = catchment area (km^2)
d_p = rainfall duration (min)

Relation (11) gives very approximate results: the discrepancy can reach 200% or more. Such models are generally not enough precise for practical use or design: they can just give an order of magnitude.

5 Passage through gully pots

Washed off particles enter into the sewer system through street inlets along kerbs or in parking areas. These inlets are named gully pots when they have a storage capacity of some ten liters at their bottom. Particles are partly retained and accumulated in these devices. With a runoff increase, accumulated particles are eroded again and enter into the sewer pipes with storm water. These phenomena have been studied and it appears that they depend on runoff and particle characteristics, dry weather period, season, catchment slopes and imperviousness.

5.1 *Grottker's model*

After experimental studies, Grottker [61,62] proposed a model for both dry and wet gullies, which predicts the "passed load" (i.e. the load which cannot be retained by gully pot) as a function of the two main parameters he identified: the pollutant load and the flow rate through street inlets. This model is written:

$$Mp = Me \cdot a \cdot Q^b \tag{12}$$

with

Mp = mass of particles passing through the gully pot (kg)
Me = mass of particles washed off by rainfall (kg)
Q = flow rate through the gully pot (l/s)
a, b = numerical coefficients depending on the particle diameter

Grottker showed also that this relation was time independent. Relation (12) gives results with errors in a range of 10 to 30%. However the model should be tested for other sites.

5.2 *Fletcher and Pratt's model*

Fletcher and Pratt [54] proposed a model to reproduce the flushing mechanism of accumulated sediments in gully pots by wet weather. This flushing out is due to the stirring action of the inflow water. Their model was tested later by Wada and Miura [133, 134].

Two phenomena can be distinguished:

- for sediments already in suspension in the gully water, the concentration is given by the following relation:

$$C = C_0 \cdot \exp\left[\frac{-t \cdot Q \cdot P}{100 \cdot V}\right] \tag{13}$$

with

C = suspended sediment concentration in the outflow (mg/l)
C_0 = initial concentration in the gully pot (mg/l)
t = time (s)
Q = inflow (l/s)
V = gully pot volume (l)
P = percentage of gully pot fluid mixed (%)

if $Q < 0.12$ l/s then $P = 664 \cdot Q + 19.7$
if $Q \geq 0.12$ l/s then $P = 100$

- for sediments resuspended from bottom deposited sediments, with M total mass of released sediment (kg) and Kr rate of release (mg/s), the relations are:

if $0 < t < M/Kr$ (re-suspension is continued):

$$C = \frac{Kr}{Q}\left[1 - \exp\left[\frac{-t \cdot Q}{V}\right]\right] \tag{14}$$

if $M/Kr < t < \infty$ (re-suspension is finished):

$$C = \frac{Kr}{Q}\left[1 - \exp\left[\frac{-M \cdot Q}{Kr \cdot V}\right]\right] \cdot \exp\left[\frac{-t \cdot Q}{V} + \frac{M \cdot Q}{Kr \cdot V}\right] \tag{15}$$

Wada and Miura determined Kr and M with the following empirical equations:

$$Kr = (1.78 \cdot Q + 0.22) \cdot Mt \tag{16}$$

$$M = (57.1 \cdot Q + 0.83) \cdot Mt \tag{17}$$

with Mt the total mass of sediment in the bottom of the gully pot.

These numerical coefficients were experimentally determined by Wada and Miura.

Fletcher and Pratt observed that the gully pots were efficient: with the maximum inflow that they examined (1.0 l/s), only 0.2% of the bottom sediments was released. The results presented by the authors showed a good agreement with field data.

6 Sediment transport in sewer pipes

This phase is the most important one in models but it is also the most difficult one for researchers because of:

- the insufficiency of available data for sewer systems;
- the complexity of the physical process which involves bed and suspended loads, deposition, erosion;
- the great number of parameters: velocities, shear stresses, collectors geometry, particles characteristics.

Many researchers have worked on sediment transport, especially for fluvial hydraulics and solid transport in closed pipes. The transposition of their results and relations to urban hydrology is problematic because site conditions are very different: particles are smaller ($d < 100\,\mu\text{m}$) and partly cohesive due to organic matter, the flow is turbulent and unsteady, pipes are not filled up but present free surface flows.

Recent research has been undertaken for pipes with free surface flows, deposits and cohesive particles to reproduce as closely as possible the actual conditions observed in sewers [80, 107, 114]. However it is still too early to have definitive and practical results.

In the following paragraphs, the main definitions and relations for fluvial hydraulics and solid transport in pipes are briefly reminded. Some equations which are actually used in urban hydrology are then described with more details.

6.1 *Fluvial hydraulics*

The research in this field has been carried out for alluvial channels, where the bed material of the river itself can be eroded or deposited. Usually, bed load transport and suspended load transport are distinguished. The total load transport is, by definition, the sum of these two components, although real physical processes are interactive and not easy to separate.

6.1.1 Bed load transport

In bed load transport, particles are sliding, rolling and saltating, without to leave definitely the bed. A lot of models have been established since more than one century to describe this process. Among the most widely known relations, the following ones can be quoted: Meyer, Peter and Muller [87], Einstein [51], Van Rijn [130].

These models give similar results for non cohesive particles with homogeneous diameter in uniform flow. Such specific conditions are of course not those of sewer systems, although some adaptations are possible to account for large particle size distributions [104, 118, 127, 130].

6.1.2 Suspended load transport

In suspended load transport, particles from bed material or from other sources remain in suspension in the flow without definitive deposition. Among the most widely known relations, the following ones can be quoted: Rouse [24], Van Rijn [131], Velikanov [30, 41], Wiuff [137], Celik and Rodi [35].

These formulas are based on mechanical equilibrium, turbulence effects and energy considerations. Like bed load transport models, they are valuable for non cohesive particles with homogeneous diameters.

6.1.3 Total load transport

Much research has been carried out about total load, and the authors have proposed several models which generally give results with the same order of magnitude [25, 137, 139, 140]. The models of Van Rijn [130, 131], Yang [139, 140], Ackers and White [4, 5], based on different approaches, are interesting examples.

6.2 *Solid transport in closed pipes*

The transport of solids by water in closed pipes has been studied by several authors, due to its economical interest. Durand proposed a distinction between different transport regimes in relation to the particle diameter [47, 48]:

- homogeneous mixture:
  for $d < 25\ \mu m$: particles remain always in suspension without deposition and with a homogeneously distributed concentration.
- intermediary mixture:
  this is a transition domain, for d from 25 to 50 μm.
- heterogeneous mixture:
  - for d from 50 to 200 μm: particles are transported in suspension with heterogeneous concentrations;
  - for d from 0.2 to 2 mm: particles are transported with intermediate conditions;
  - for $d > 2$ mm: particles are transported by saltation.

Durand's relations have been established and verified for higher concentrations and for coarser particles than those observed in sewers.

A more specific model has been established directly for sewer pipes with free surface flows by Macke [95, 96].

6.3 *Common relations for channels and pipes*

Using dimensionless parameters, Graf and Acaroglu [2, 59, 60] have proposed a general model for both closed pipes and open channels. This model is valid for non cohesive particles with uniform diameters greater than 90 μm. The agreement with experimental and field data is correct.

An attempt to use the relation of Graf-Acaroglu for solid transport in sewer systems is in hand at the university of Karlsruhe (Germany), but the results are not yet published [Beichert, personal communication].

A model for bed load transport in both circular pipes and channels with fixed bed has also been proposed by Novak and Nalluri [108, 109].

6.4 *Relations used in urban hydrology*

The three following models are employed in current models for transport in sewers systems. This is the reason why it is interesting to present them with more details.

6.4.1 Macke's relation for total load

Macke [95, 96] proposed a relation which has been experimentally established for circular pipes and which is based on a "no deposition" criterion. This relation is written:

$$q_t = \frac{1.64 \cdot 10^{-4} \cdot \tau_0^3}{g(\varrho_s - \varrho) \cdot w^{1.5}} \tag{18}$$

with

q_t = volumic total transport rate (m^3/s)
τ_0 = shear stress (N/m^2)
ϱ_s, ϱ = density of sediments and water (kg/m^3)
w = settling velocity of particles (m/s)

This model has been successfully tested with field data in Hamburg (Germany). Its domain of validity is:

$0.1 < d\,(\mathrm{mm}) < 3$ particle diameter
$0.5 < \tau_0\,(\mathrm{N/m^2}) < 74$ shear stress
$0.05 < D\,(\mathrm{m}) < 2.4$ pipe diameter

6.4.2 Velikanov's model for suspended and/or total load

The Velikanov's model is based on turbulent energy analysis. It was first proposed for quality models by Combes [41], and it is employed in the model FLUPOL by Bujon [30]. Its main advantage is its simple mathematical expression:

$$\begin{aligned} C_{\min} &= \eta_{\min} \cdot \varrho_s \cdot \varrho_m \cdot (\varrho_s - \varrho)^{-1} \cdot \frac{U}{w} \cdot J \\ C_{\max} &= \eta_{\max} \cdot \varrho_s \cdot \varrho_m \cdot (\varrho_s - \varrho)^{-1} \cdot \frac{U}{w} \cdot J \end{aligned} \tag{19}$$

with

$C_{\min}$, $C_{\max}$ = limit sediment concentrations (g/l)
$\eta_{\min}$, $\eta_{\max}$ = efficiency coefficients
ϱ_m = density of mixture sediment + water ($\mathrm{kg/m^3}$)
ϱ_s = density of sediment ($\mathrm{kg/m^3}$)
ϱ = density of water ($\mathrm{kg/m^3}$)
U = mean flow velocity through pipe section (m/s)
w = sediment settling velocity (m/s)
J = energy line slope

$\eta_{\max}$ is the fraction of the total turbulent energy that keeps the particles in suspension.
Noting C the real concentration,
if $C < C_{\min}$, there is erosion until $C = C_{\min}$.
if $C_{\min} < C < C_{\max}$, there is sediment transport at concentration C without deposition or erosion.
if $C > C_{\max}$, there is deposition until $C = C_{\max}$.
Combes proposed $0.0005 < \eta_{\min} < 0.002$ and $0.002 < \eta_{\max} < 0.007$, whereas Bujon used $\eta_{\min} = 0.018$ and $\eta_{\max} = 0.022$.
This model is employed for total transport, but it seems that it gives better results for suspended load. Bujon, for FLUPOL, added an "efficiency coefficient" to fit the results as well as possible.
The Velikanov's model is easy to use, but it needs more experimental validations.
The models proposed by Wiuff [137] and Celik and Rodi [35] for open channels have similar expressions, with refinements to calculate the efficiency coefficients. Wiuff, for example, proposed an expression of $\eta_{\max}$ that depends on grain size and shear stress.

6.4.3 Ackers-White's model for total load

This model [4, 5] was first established for sediment transport in open alluvial channels. An empirical adaptation for circular pipes has been proposed to have a better agreement with laboratory data [2, 37]. The three main equations are:

$$Fgr = \frac{u^{*n}}{\sqrt{g \cdot d_{35} \cdot (s-1)}} \cdot \left[\frac{U}{\sqrt{32} \cdot \log\left[\frac{12 \cdot Rh}{d_{35}}\right]} \right]^{1-n} \tag{20}$$

$$Ggr = Caw \cdot \left[\frac{Fgr}{Aaw} - 1 \right]^{m} \tag{21}$$

$$q_t = Ggr \cdot s \cdot d_{35} \cdot \frac{1}{Rh} \cdot \left[\frac{U}{u^*} \right]^{n} \cdot \left[\frac{We \cdot Rh}{S} \right]^{1-n} \tag{22}$$

with

Fgr = dimensionless mobility particle number
Ggr = dimensionless solid flow number
q_t = total solid flow (kg particles/kg water)
u^* = friction velocity (m/s)
U = mean flow velocity (m/s)
Rh = hydraulic radius (m)
s = specific gravity of particles
d_{35} = particles diameter (35% in mass of particles have a diameter of less than d_{35})
Aaw, Caw, n, m = numerical coefficients depending on dimensionless particle diameter

The last term of equation (22), $(We \cdot Rh/S)^{1-n}$, is the main corrective term for the adaptation to pipes, with $We = 10 \cdot d_{35}$ the effective deposited sediments width (m), and S the flow section (m^2). This correction has an important effect for coarse particles (i.e. for bed load transport). These equations have been verified by other researchers who obtained good results for laboratory experiments [98]. The Ackers-White's model, with this adaptation, is used in the model MOSQITO.

6.5 *Cohesion of sediment*

All the above-mentioned equations were established, and are valid, for non cohesive particles. Nevertheless it is clear that sewer deposits are cohesive [106, 138]: the rheological properties of their fine fraction are similar to those of a clay, and especially an illite [9, 21]. This fraction has an important influence on the deposits behaviour.

Consequently, it should be necessary to take sediment cohesion into account, with some characteristics like erosion threshold shear stress, initial rigidity, sediment consolidation with time, percentage of organic matter.

In estuarine sediment field, some equations have been established [8, 100, 101, 102, 103, 112]. However, because of the lack of knowledge about sewer sediments and despite recent research [9, 21, 136], these relations have not yet been verified or adapted for urban hydrology. Quality models then usually ignore this important aspect.

7 Brief presentation of some softwares

7.1 *The SWMM*

It is one of the first quality models in urban hydrology. The first version [84] was fairly simple: all particles with a diameter greater than d_d were deposited, all particles with a diameter smaller than d_e were eroded; d_d and d_e were calculated with the Shields criterion, for each time and space step. The solid transfert was simply calculated too: the particles were supposed to be homogeneously distributed in water, and they were transferred at the same velocity as water, by applying the mass conservation law. All particles are described by a single size distribution.

This model is simple but some assumptions are unrealistic and lead to approximate results: same velocity for particles as for water, single and constant size distribution for deposits and suspended load despite erosion and sedimentation.

Some years later, an improved and more sophisticated version was established using Sonnen's transport model (1977). This model distinguishes bed load, suspended load and wash load, for 10 granulometric classes [41]. It considers separately particles from domestic sewage and from storm water.

The bed load is represented by a relation derived from those of Kalinske [87] which is approximated by straight line segments with the following expression:

$$q_s = a \cdot \varrho_s \cdot u^* \cdot d \cdot 10^{\left(\frac{-b \cdot \tau_{cr}}{\tau_0}\right)} \tag{23}$$

with

q_s = bed load flow per unit width (kg/s · m)
u^* = friction velocity (m/s)
ϱ_s = density of particles (kg/m^3)
d = particles diameter (m)
a, b = numerical coefficients depending on τ_{cr} and τ_0
τ_0 = mean shear stress over flow section (N/m^2)
τ_{cr} = critical shear stress for particles (N/m^2)

For suspended load, Sonnen uses the classical equation of Rouse.

The wash load is assumed to be the difference between particles entering into the sewer and particles depositing in sewer. There is deposition if the flow velocity is smaller than a critical value Uc which is given by the relation (derived from that of Durand [47]):

$$Uc = 0.9 \cdot (2g \cdot y \cdot (s-1))^{0.5} \tag{24}$$

with

Uc = critical flow velocity (m/s)
y = flow depth (m)
s = specific gravity of particles (kg/m^3)

This version of the SWMM can give satisfactory results but its calibration is not easy due to the great number of parameters. Rouse's and Durand's equations have been modified or adapted with assumptions which are not well established. In later versions, this sophisticated approach has been pulled out.

7.2 *THALIA model*

The THALIA model [73, 74, 75], established at the University of Karlsruhe (Germany), is initially a copy of the SWMM with Sonnen's model. The relation for wash load is replaced by another one derived from Bagnold's studies [95], where a critical shear stress τ_c is introduced. This modification should allow a better accuracy and is written:

$$\begin{aligned} \tau_c &= 0.4 \cdot w^2 \cdot (\varrho_s - \varrho) \quad \text{if } d < 2\,\text{mm} \\ \tau_c &= g \cdot d \cdot (\varrho_s - \varrho) \quad \text{if } d > 2\,\text{mm} \end{aligned} \tag{25}$$

with

w = settling velocity of particles (m/s)
d = particles diameter (m)

The solid transfer model is coupled with the hydrodynamic software HAMOKA which uses the complete St Venant's equations. It appeared that the calculation duration for solid transfer was three times longer than the time required for hydraulic calculations. To reduce this difference, the authors established a simplified version, which uses an empirical bed load transport equation from Shields [87]:

$$\frac{q_c}{Q} \cdot \left[\frac{\gamma_s - \gamma}{I_r \cdot \gamma}\right] = 10 \cdot \frac{\tau_0 - \tau_{cr}}{(\gamma_s - \gamma) \cdot d} \tag{26}$$

with

q_c = volumic bed load transport rate (m^3/s)
Q = flow rate (m^3/s)
I_r = invert slope
τ_0 = mean shear stress (N/m^2)
τ_{cr} = critical shear stress for erosion beginning (N/m^2)
d = particles diameter (m)
γ_s, γ = volumic weight of particles and water (N/m^3)

Contrary to the first version, the simplified one does not take the real deposits into account, but the time needed for calculations is 10 times shorter and the results are not significantly different: the divergence is about 10%.

7.3 *Combes's and FLUPOL models*

Both models use Velikanov's equations. Only FLUPOL [30, 31], established by the Compagnie Générale des Eaux and the Agence de l'Eau Seine Normandie (France), was employed to reproduce real data. It is coupled with an hydrological and hydraulic model using linear reservoirs and Muskingum schemes for flow calculations. FLUPOL does not use several granulometric classes, but it distinguishes between solids during dry and wet weather.

An "efficiency coefficient" has been introduced to improve calibrations, but it is more an artful device than a hypothesis in relation to the Velikanov's theory. The first calibrations for French catchments around Paris give results with a satisfactory overall agreement between calculated and observed values. However this recent model needs further validations for other catchments.

7.4 *MOSQITO model*

This model [71, 72], developed by Hydraulics Research (United Kingdom), is one of the most detailed at the present time but it is still in its development phase. MOSQITO, which is coupled with the hydrodynamical software Wallrus [28], uses the Ackers-White's transport model. For wash load, it makes calculations with the advection-diffusion model from Holly and Preissmann [69]. The most interesting aspect of MOSQITO is its separation of deposits into two layers:

- the upper layer, named "active layer", composed of non cohesive sediments, easily eroded, with an important organic matter fraction;
- the lower layer, named "storage layer", composed of consolidated deposits, with an important mineral fraction and a specific gravity greater than 2.

When the whole active layer is eroded, it becomes possible to erode the storage layer if the shear stress is greater than a critical value chosen by the user. With such a solution, MOSQITO is able to take the cohesion of sediments into account.

The first calibrations show that MOSQITO can give interesting results for some catchments, but it is too early to draw more precise conclusions. There is a need for further research and comparisons with field data. Nevertheless, it is important to note that [44, 111]:

- MOSQITO is very sensitive to deposit height in the sewer and to the particle characteristics (density and settling velocity);
- the storage layer can be eroded but its re-building is not very well defined;
- the dry weather part of calculations is too simplified and not sufficiently realistic.

7.5 *KOSIM model*

The KOSIM (Kontinuierliche Simulation) model [49, 117, 126] was established by the Institut für Technisch-Wissenschaftliche Hydrologie in Hannover (Germany) to design storm tanks. The hydrodynamic part is composed of linear reservoirs in cascade. The KOSIM model is not as deterministic or detailed as the above-mentioned ones. Its three main hypotheses are:

- the TSS concentrations in storm water are supposed to be constant in time;
- the TSS sources are domestic sewage, storm water from impervious areas and storm water from pervious areas;
- the TSS load is always in direct ratio to the flow rate.

The resulting TSS concentration at the outlet is written:

$$C(t) = \frac{Qeu \cdot Ceu + Qpp \cdot Cpp + Qpi \cdot Cpi}{Qeu + Qpp + Qpi} \tag{27}$$

with

$C(t)$ = TSS concentration at the outlet in mixed water
Qeu, Ceu = flow rate and TSS concentration of domestic sewage
Qpp, Cpp = flow rate and TSS concentration of water from pervious area
Qpi, Cpi = flow rate and TSS concentration of water from impervious area

This model takes the "first flush" phenomenon into account, but not the erosion and deposition in sewers. A second version [120] has been proposed with the following improvements:

- a first equation represents the growing of deposits in sewers:

$$P(t) = P_{\max} - (P_{\max} - P(t - \Delta t)) \cdot \exp(-K_1 \cdot t) \tag{28}$$

- a second equation represents the erosion of the deposited sediments:

$$P(t) = P(t - \Delta t) \cdot \exp(-K_2 \cdot (Q(t) - Q_{\text{lim}}) \cdot t) \tag{29}$$

with

$P(t)$ = solid load at time t
$P_{\max}$ = maximum mass of deposits in sewers
$Q(t)$ = flow rate at time t
Q_{lim} = critical flow rate under which there is no erosion
t = time step
K_1, K_2 = numerical coefficients

K_1 is determined assuming that 50% of the TSS entering into sewers during dry weather deposit in 24 hours. K_2 is determined assuming that Q_{lim} allows the erosion of the whole deposit in five minutes.

KOSIM is a simple conceptual model which tries to represent the main phenomena and to give a mean load for long periods. It should be evaluated with more experimental data, especially to evaluate the importance of the hypothesis of constant TSS concentration in storm waters. This model is already used in Low Saxony (Germany) as official software for every storm tank design project.

7.6 *Other models*

The present review does not pretend to be exhaustive, and other models for sewer sediment transport exist [19]. Among the recent ones, the following two may be quoted:

- Berndtsson's et al. model: it is a conceptual model which was originally established to predict overflows from combined sewers [18, 68]. It can take the erosion of deposited sediments into account, with a relation proposed firstly by Göttle [58] for surface wash off.
  The main assumption is that the deposited sediments are eroded only by storm water flow. This model was not initially adapted for whole sewer systems, but only for storm tank overflows. However, we think that its adaptation [88] could be interesting because of its simplicity.
- Le Guennec et al. [89] have recently developed a model to reproduce experimental data obtained in a trunk sewer in Marseille (France).

8 Difficulties for model comparison

After the above descriptions, it is evident that a comparison of the models and of their results would be very interesting. But such a task is not easy because:

- each model has been calibrated with its own data sets. A direct comparison is impossible.
- each model has been calibrated only with a few data, and, sometimes, it has not been verified with other data sets (or this verification has not been published in the literature). This problem is partly due to the lack of available field data. It is then difficult to have a great confidence in the model, to evaluate its domain of validity and its accuracy, and to determine its actual facilities. It is also difficult to know the relative importance of the different parameters because sensitivity analyses are not usual.

Finally, it is now impossible to give any serious criterions of choice: users have to remember that these models are new tools which are still in development stage. They need further calibrations and should be used with care.

Any valuable and efficient comparison would require the use of the same field data sets. Such a comparison remains difficult because:

- all models are not commercialized or public.
- the available models do not need the same data and/or the same format of data. Any comparison would require a long preparation to collect enough data and to adapt them for each model. The setting of a data base with existing field data will be the first step to begin this task [66, 110].

However, it is obvious that this comparison will have to be done in the future. An attempt is already in hand in Germany: the first results should be published in 1992 [76].

The last difficulty, but not the least, is the collecting of sufficiently precise field data. The modelling approach cannot now be achieved because data about bed load and deposits are insufficient: these data are determinative and their absence hinders calibration and verification of some parts of the models.

It is also necessary to develop conjunctively modelling and field experiments. In fact, many improvements are needed in sampling methodology, especially for bed load and deposits measurements, and in the field of metrology. Unfortunately, field experiments in sewers are very expensive. It is a no negligible drawback.

9 Conclusion

This literature review shows that sewer sediment transport models are growing in number, with recent developments. However they still remain difficult to establish and calibrate because:

- the solid transport theories have generally been developed for fluvial hydraulics or solid transport in pipes. Their transposition to urban hydrology is not straightforward because of important differences in sewers environmental conditions.
- cohesion and consolidation of sediments are important factors which have not been extensively studied and which are at the present time not well understood and described.
- experimental data for models calibration are insufficient, dispersed and of varying quality and accuracy. A homogeneous database would be an interesting and important improvement.

Deterministic and detailed models like Mosqito are useful to understand the phenomena involved in sewers and to predict accurately deposits and overflows. They are necessary for progress, but they remain too much complicated for practical use in management or design offices.

Until these deterministic models become more developed, there is a place for conceptual models. Their precision is lower, but they are easy to use and need less parameters and experimental data to be calibrated and employed. This is the reason why we have undertaken a research to establish a new conceptual model for sewer solids in small urban catchments [20].

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Notations

a	numerical coefficient
a_i, a_e, a_d	Price Mance coefficients
A	catchment area
Aaw	numerical coefficient
ACCU	daily accumulation rate of sediment
b	numerical coefficient
c	numerical coefficient
C	TSS concentration
Caw	numerical coefficient
Ceu	TSS concentration in domestic sewage
C_{min}, C_{max}	limit concentrations in Velikanov's model
Cpi	TSS concentration in water from impervious areas
Cpp	TSS concentration in water from pervious areas
d	particles diameter
d_{50}	median particles diameter
d_p	rainfall duration
DISP	disappearing coefficient
DTS	dry weather period
Fgr	dimensionless mobility particle number
g	acceleration of gravity
Ggr	dimensionless solid flow number
H_t	total rainfall height
$i(t)$	rainfall intensity
I_{max5}	maximum rainfall intensity during a 5 minutes time step
I_r	invert slope
J	energy line slope
K	linear reservoir parameter
K_1	numerical coefficient
K_2	numerical coefficient
Kd	availability factor in the SWMM
Ke	washoff coefficient in the SWMM
Ke'	lumped washoff coefficient in FLUPOL
$Keni$	washoff coefficient in NPS
Kr	rate of release in gully pot
Ks	washoff coefficient in Servat's model
m	numerical coefficient
M	total mass of sediment released in gully pot
Ma	accumulated mass of sediment on a catchment
Me	washed off mass of sediment
Mp	amount of particles passing through the gully pot
Mt	total mass of sediment in the bottom of the gully pot
n	numerical coefficient
P	percentage of gully pot fluid mixed
$P(t)$	solid load at time t in KOSIM

$P_{\max}$	maximum amount of deposits in KOSIM
q_b	surface runoff
q_c	bed load transport rate
q_t	total sediment transport rate
Q	flow rate
Qeu	domestic sewage flow rate
$Q_{\lim}$	critical flow rate in KOSIM
Qpi	flow rate of water from impervious areas
Qpp	flow rate of water from pervious areas
Rh	hydraulic radius
s	specific gravity of particles
S	flow section
S_b	surface runoff on impervious surface in NPS
Sm	soil depression storage
t	time
u^*	friction velocity
U	mean flow velocity
Uc	critical velocity
V	gully pot volume
Vr	runoff volume
w	settling velocity of particles
We	effective deposited sediment width
y	flow depth
Δt	time step
γ	volumic weight of water
γ_s	volumic weight of sediment
$\eta_{\min}$, $\eta_{\max}$	efficiency coefficients in Velikanov's model
ϱ	density of water
ϱ_s	density of sediment
ϱ_m	density of mixture (sediment + water)
τ_c	critical shear stress in THALIA
τ_{ce}, τ_{cr}	critical shear stress for erosion
τ_{cd}	critical shear stress for deposition
τ_0	mean shear stress

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Effects of hydraulic and solids loading on clarifier performance

Influence du débit et de la charge en matières solides sur le fonctionnement des décanteurs

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SUMMARY
A numerical model for an idealized circular tank with no baffle is used to investigate the relative importance of inlet solids concentration and inlet flow on the removal of solids field. Effluent concentration is very sensitive to the velocities in the withdrawal zone. The densimetric Froude number relates hydraulic and solids loading. It was found that the upward velocities in the withdrawal zone increase with decreasing densimetric Froude number for a constant discharge. Under constant solids loading there is an optimum densimetric Froude number for minimum effluent concentration. The performance of the clarifier is a function of the densimetric Froude number and the return activated sludge ratio. For the clarifier considered here, the minimum effluent concentration for a constant solids loading, occurred at a densimetric Froude number of about 0.5. The Reynolds number, when in the turbulent regime, did not significantly affect the tank hydrodynamics. The predicted solids distribution was in good agreement with the field data. The flow pattern was verified by comparison with thermal density current in a scale model.

RÉSUMÉ
Un modèle numérique a été utilisé pour étudier l'importance relative des débits d'apports solide et liquide sur les performances d'un réservoir circulaire idéal sans déflecteurs. La concentration à la sortie est très sensible aux vitesses dans la zone de soutirage. Le nombre de Froude densimétrique relie les apports liquide et solide. Il a été trouvé que les vitesses ascendantes dans la zone de soutirage augmentent lorsque, à débit constant, le Froude densimétrique diminue. A charge solide donnée, il existe un Froude densimétrique optimal donnant une concentration minimale à la sortie.Le rendement du décanteur est fonction du Froude densimétrique et de la concentration en boue activée (RAS). Pour le cas du décanteur envisagé ici, la concentration minimale en effluent correspondait à un nombre de Froude densimétrique voisin de 0,5. En régime turbulent, le nombre de Reynolds n'a pas d'influence significative sur l'hydrodynamique du réservoir. Les concentrations en matière solide calculées étaient en bon accord avec les mesures.
Le champ de courant a été vérifié par comparaison avec les courants de densité sur modèle réduit thermique.

Introduction

The study of clarifier performance has aroused considerable interest in recent years as indicated in a recent review by Stamou and Rodi (1984). In 1977 Larsen made an extensive study of rectangular clarifiers that were influenced by density current. His work included measurements in prototype and model tanks. Also, in his report, a numerical model was described by using a mixing length turbulence model. Imam et al (1983) following the work of Larsen used dye tests for model calibration. Recently Celik and Rodi (1985), Lyn and Zhang (1989) and Adams and

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