SOLIDS TRANSPORT IN COMBINED SEWERAGE SYSTEMS

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I certify that this thesis is the true and accurate version of the thesis approved by the examiners.

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ABSTRACT

Interest in the problems caused by the deposition of sediments in sewers, and by the transport of solids through and from sewer systems has given rise to a great deal of research both in the UK and internationally. A collaborative research effort has been initiated by the Water Research Centre (WRc) under the auspices of the UK Urban Pollution Management (UPM) programme to study all aspects of sediments in sewers. The work of the Wastewater Technology Centre (WWTC) of the University of Abertay Dundee (UAD) has made a major contribution to this aspect of the UPM programme. The work reported in this thesis constitutes a significant component of the WWTC research on solids transport in sewers.

The accurate prediction of solids transport in sewers is a problem which has been addressed by a number of workers using various approaches, none of which has currently proved wholly satisfactory. This thesis describes an investigation of the transport of solids in combined sewers during both dry weather flow periods and storm flows. The study is based primarily on measurements of flow conditions and suspended solids concentrations at two sites on the main interceptor sewer at Murraygate, Dundee City Centre and one other site on a trunk sewer in the Perth Road area of Dundee. In addition, information on the associated accumulation of sediment in sewer inverts was obtained.

The relationship between hydraulic conditions in these combined sewers and the transport of solids in suspension has been examined. The aim of the work was to arrive at a methodology by which an appropriate model could be selected or developed which would predict solids transport rates given information on hydraulic conditions.

It was anticipated that such models would be site-specific, requiring significant calibration. Site-specific regression equations were developed for dry weather and storm conditions respectively for the sites studied. However, by using the data from the two study sites in combination, and by incorporating a factor which related to the varying site conditions in terms of topography and geometry (the DAS factor), a non-site-specific model was also developed. This gave similar levels of performance to the site-specific models with the added advantage that this model was potentially suitable for more general application to other sewers, with little or no calibration requirement. This was demonstrated by application to one other sewer.

The work described led to the development of the non-site-specific model proposed, which was unique in form. More important however were the fundamental procedures developed by which the model type was selected and subsequently developed, ie the main objective of developing a methodology for model development had been achieved.

A method of sampling and measuring rates of transport of bed load solids using a specially constructed in-situ flume was developed which proved the worth of utilising such an approach. Data on bed load material and transport rates were obtained which were, however, too limited to enable the development of a predictive model of that mode of solids transport. In association with the main modelling work described here for suspended solids, this provided a means by which rates of transport in this mode could be estimated.

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To Elaine, Eve and Ellen.

THE DEPOSITION AND RE-ENTRAINMENT OF SEWER SEDIMENTS IN COMBINED SEWERAGE SYSTEMS

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Glossary of Terms

Apparent Yield Stress	Stress at which the rate of sediment sample deformation continually increases under constant applied stress during rheometrical testing
Ashed Residue	The remaining solid material in a sediment sample after placing in a furnace at 550°C for 30 minutes
Bed Load	Material transported near to bed by saltation
Biochemical Oxygen Demand	Oxygen consumed by bacteria as a result of the concentration of organics in a sewage or sediment sample
Chemical Oxygen Demand	A measure of organic material present in a sewage or sediment sample as a result of chemical oxidation
Cohesive Sediment	Sediment which exhibits cohesive properties (a tendency for the individual particles to adhere)
Combined Sewer	Sewer carrying both foul and storm sewage
Combined Sewer Overflow	A structure to relieve excess flow loading from a sewer
Competent Bottom (or Bed) Velocity	The bed velocity which is just able to move material of a given size and specific weight
Competent Mean Velocity	The mean velocity which is just able to move material of a given size and specific weight
Critical Velocity	Similar to competent velocity, and may also refer to either mean or bed velocities

Critical Yield Stress	Applied bed shear stress beyond which bed erosion commences
Deposition	Process by which suspended solids are deposited on the invert or sides of a sewer to form a sediment deposit
Deposit Free Conditions	Hydraulic conditions which maintain solids in suspension without sediment deposition over a clear invert
Detachment Velocity	The lowest average velocity at which individual particles from the bed continually become detached and at which the fluctuating lift force is equal to the weight of the particles in water
Dry Weather Flow	Sewer flow on days when there is no significant rainfall
Entrainment	Process of lifting near bed material into suspension
Erosion	Disturbance or lifting effect of flow causing the transfer of some or all of the material in a sediment bed into the near bed layer of transported material, or into suspension
First Foul Flush	Increased concentration of pollutants at the start of a storm event
Fluid Mud	Material transported close to the invert or sediment deposit
Flume Traction	Bed load without deposition

Foul Flow	Sewage in a combined (or separate) sewer originating from domestic and commercial premises which may include industrial effluent
Impervious Surfaces	Surfaces which allow virtually no rainfall to percolate through to lower subsurface layers
Incipient Motion	Hydraulic conditions which promote the commencement of erosion of a sediment deposit
Invert	Lowest part of the internal surface of a pipe or sewer
Limit of Deposition	Maximum concentration of transported material for given hydraulic conditions without deposition of solids on invert
Loss on Drying	measure of amount of water lost from a sediment sample when dried to a constant weight in an oven at 105°C
Near Bed Material	Material transported in a relatively dense layer close to the invert or sediment bed of a pipe or sewer
Non-Cohesive Sediment	Sediment which does not exhibit cohesive properties
Non Displacement Velocity	The highest average flow velocity at which bed particles are not displaced and at which the maximum value of the fluctuating lift force does not exceed the weight of the particles in water
Non-Foul Flush	Storm related flows/samples measured/obtained after the end of the first foul flush
Non-Site Specific	See "site specific"
Non-Volatile Solids	Solids remaining in a sediment sample after

	furnacing at 550°C for 30 minutes
Particle Size Distribution	Relative proportions of particle size in the ashed residue of a sediment sample measured by sieve analysis
Perceived Class	Sediment classification estimated by visual inspection of sediment characteristics and location
Permissible Canal Velocity	The maximum average velocity for which there is no objectionable scour in the bed of a canal
Pervious Surfaces	See "impervious surfaces"
pH	Measure of relative acidity/alkalinity of a sample on a logarithmic scale
Pseudo-homogeneous Flow	Particles are transported in turbulent suspension and are uniformly distributed through the depth of flow
Re-entrainment	Entrainment of deposited material
Saltation	Rolling and sliding motion of particles moving near bed (mainly river/canal)
Sediment	Accumulations of sewage solids on the bed or sides of sewers or sewer appurtenances
Sediment Classification	A means of differentiating between sewer sediments which display varying characteristics
Site-Specific	Pertaining only to one specific sewer location
Soffit	Highest part of the internal surface of a pipe or sewer
Storm Event	Hydraulic conditions associated with significant amounts of rainfall on a sewer catchment
Storm Flow	Sewage flowrates associated with significant

	amounts of rainfall on a sewer catchment
Stormwater Overflows	See "combined sewer overflows"
Suspended Solids	Solid material transported in overlying water/sewage as opposed to near bed material
Time Since Start of Storm	The time elapsed since a storm commenced in relation to the time at which a particular sewage sample was obtained
Total Solids	Mass of material remaining in a sediment sample (expressed in relation to original mass of sample) after drying at 105°C in an oven
Total Suspended Solids	Material in suspension in sewage, measured by filtration
Volatile Solids	See "non-volatile solids"
Volumetric Concentration	Concentration expressed in terms of relative volume
Wash load	Particles are transported in turbulent suspension and are uniformly distributed through the depth of flow

List of Symbols

Note: All symbols are in standard SI units unless otherwise stated

a_c = coefficient

a_e = the lower limit of y where suspended load begins

a_m = particle dimension

a_{n} = coefficient

a_{s} = a constant, specific to sewer section

a_{v} = reference height

a' = thickness of bed layer

A = service area (acres)

A_b = area corresponding to the bed

A_c = channel cross-sectional area

A_E = dimensionless depth

A_f = cross-sectional area of flow

A_p = projected area of particle

A_w = area corresponding to the sides

ADWP = antecedant dry weather period

b_m = particle dimension

b_{n} = coefficient

b = coefficient

B = channel width (for a rectangular channel)

B_e = effective width of water

B_f = breadth of flow

c = constant of proportionality

c_m = particle dimension

C_o = sediment concentration

C = concentration of sediment with fall velocity ω at level y

C_a = concentration of sediment with fall velocity ω at level a

C_{ae}= average concentration of the bed layer

C_{av}= reference concentration

C_b = Chezy coefficient

C_D = coefficient of drag

C_L = lift coefficient

C_V = volumetric sediment concentration

C' = Chezy coefficient related to grains

d = size of sediment

d_p = particle diameter

d_s = diameter of sphere

d_g = mean grain diameter

$\bar{d}$ = mean grain diameter

d_{mm}= sediment size in mm

d_n = diameter of particle such that $n\%$ of sample is finer

d_{35} = sediment grain diameter

d_{50} = median diameter of particles in a mixture, sediment size

D = pipe diameter

$\overline{D}$ = average equivalent diameter (inches)

D_{gr} = dimensionless grain size

D_* = dimensionless particle parameter

DAS = diameter, area and slope factor

f_b = bed form factor

F = viscous resistance

F_A = specified area taken from the graph of particle size distribution

F_B = specified area taken from the graph of particle size distribution

F_D = hydrodynamic drag

F_L = lift force

F_t = force normal to the angle of repose ϕ

F_n = force parallel to the angle of repose ϕ

g_s = bedload rate for a given size i_s

g_{sb} = mass in motion per unit width

g_{ss} = the suspended load rate in weight per unit time and width

$g_s i_s$ = bedload rate for a given size i_{ss}.

$g_{ss} i_{ss}$ = the suspended load rate in weight per unit time and width

for particle size i_{ss}

G = rate of bed load transport in weight per unit width

G_s = mass in motion

h = depth of flow

H = height of bed forms

i = channel slope

i_s = fraction of suspended load in a given size range

k = constant

k_s = equivalent roughness height

k_{sk} = a time coefficient possibly related to slope

k_1, k_2, k_3 = particle shape factors

k_4 = constant

K = Von Karman constant

K_v = correction factor for fall velocity

L = control volume length

M = grain distribution modulus

N = number of particles per unit area

p = factor which indicates the proportion of the bed taking the fluid shear

P = wetted perimeter of channel cross section

q = rate of flow per unit width

q_b = sediment transport rate per unit width

q_B = rate of bed load transport in weight per unit width

q_{CR} = the critical discharge per unit width for incipient motion

q_p = discharge per capita, including infiltration (gpcd)

q_s = transport weight per unit time and per unit width of stream

q_{sb} = volumetric rate of solids movement per unit width

Q = flowrate (m^3/s)

$Q(t_e)$ = flowrate at time t_e

r = radius of cylinder

r'_h = hydraulic radius with respect to grain size

r^2 = correlation coefficient

R = hydraulic radius

R_b = hydraulic radius of the bed

R_e = Reynold's number

R_W = hydraulic radius of the walls

$(R_*)_{CR}$ = critical shear Reynolds number

s = particle specific gravity, relative density

s_s = specific gravity of sediment

S = hydraulic gradient or energy slope

$\overline{S}$ = average pipe slope

S_b = bed load transport

S_c = longitudinal slope of channel

S_s = suspended load transport per unit width

S_v = shear strength of material

t = bed thickness

t_e = time elapsed since start of storm

t_r = relative bed thickness

T = transport stage parameter

TS = deposited solids loading (lbs/day)

TSS = total suspended solids (mg/l)

TSSS = time since start of storm (hours)

u = flow velocity component

$\overline{u}$ = the average flow velocity

u_b = fluid velocity at the bottom of the channel

u_i = instantaneous fluid velocity

$(u_i)_{CR}$ = critical fluid velocity

$(u_b)_{CR}$ = critical bottom velocity

$u_{*,b}$ = bed shear velocity

u_{CR} = characteristic velocity at incipient motion condition

u_{35} = velocity at a distance $0.35d_{35}$ from the bed

u_* = friction or shear velocity

u_{*s} = shear velocity

u_*' = the shear velocity due to grains only

u_*'' = effective bed shear velocity

$u_{*,c}$ = critical bed velocity

$(u_*)_{CR}$ = critical shear velocity

u_y = velocity at depth y

u_∞ = free stream velocity

$v(t_e)$ = cumulated volume runoff at time t_e

V = average velocity of flow (m/s)

V_c = critical velocity for incipient motion

V_{CS} = critical scour velocity

V_O = effective threshhold velocity

V_p = pipe velocity

$\overline{V_p}$ = average pipe velocity

V_s = settlement velocity

$\overline{V}_s$ = mean particle velocity

V_{sc} = self cleansing velocity

V_t = threshold velocity

V^* = shear velocity

w_a = terminal velocity of a sphere (actual)

w_f = particle fall velocity in clear still fluid

$\overline{w_s}$ = local mean vertical flow velocity

w_s = submerged weight of particle/grain

w_t = terminal velocity of a sphere (theoretical)

W = bed width

W_e = effective sediment bed width

x = ADWP in hours

X = correction factor

X_{calc} = calculated sediment concentration

X_{obs} = measured sediment concentration

X_r = regression coefficient

y = elevation above datum

y_n = depth of flow

y_o = uniform depth of flow

y_s = mass of solids in g /(m length of sewer)

Y = depth of flow

Z = suspension number

Z' = suspension number

α = inclination of the bed from the horizontal

β = ratio of sediment diffusion and fluid diffusion coefficient

χ = a coefficient dependent on sediment size

χ'' = sediment coefficient

ϕ = angle of repose of bed material

γ = specific weight of water

γ_f = specific weight of fluid

γ_s = specific weight of the particle

η = packing coefficient

φ = correction factor for concentration profile

λ' = particle shape coefficient

λ_s = overall friction factor with transport

λ_{sb} = bed friction factor with transport

μ = dynamic viscosity of fluid

μ_r = ripple factor

θ = angle of the pipe to the horizontal

ρ = density of water

ρ_f = density of fluid

ρ_s = density of solids

σ_s = geometric standard deviation

τ = shear stress

τ_b = bed shear stress

τ_0 = average value of tractive force per unit of wetted area, or unit tractive force, or tractive stress

$(\tau_o)_{cr}$ = critical shear stress for incipient motion

τ_y = shear stress at level y

υ = kinematic viscosity coefficient for clear fluid

Δ = relative density of sediment

Δ_r = the apparent roughness diameter

Δp = lift force per unit area of the particle

Φ = factor of probability that a particle will move in a given time step

Φ_b = transport parameter

Ψ = factor of probability that a particle will move in a given time step

$\in$ = sediment transfer or sediment diffusion coefficient

Θ_b = dimensionless bed shear stress

List of Abbreviations

BOD	Biochemical Oxygen Demand
BSI	British Standards Institute
CIRIA	Construction Industry Research and Information Association
COD	Chemical Oxygen Demand
CSO	Combined Sewer Overflow
DAS	Diameter, Area and Slope factor
DIT	Dundee Institute of Technology
DWF	Dry Weather Flow
FFF	First Foul Flush
LOD	Loss on Drying
MOSQITO	Modelling of Sewage Quality in Tanks and Overflows
NFF	Non-Foul Flush
NH_4-N	Ammoniacal Nitrogen
NVS	Non-Volatile Solids Content
pH	Negative log to base 10 of H^+ concentration
PSD	Particle Size Distribution

RBM	River Basin Management programme
SS	Suspended Solids
TRC	Tayside Regional Council
TS	Total Solids Content
TSS	Total Suspended Solids
TSS(FFF)	Total Suspended Solids (First Foul Flush)
TSS(NFF)	Total Suspended Solids (Non-Foul Flush)
TSSS	Time Since Start of Storm
UAD	University of Abertay Dundee
UPM	Urban Pollution Management programme
VS	Volatile Solids
WRc	Water Research Centre
WWTC	Wastewater Technology Centre

1 INTRODUCTION

1.1 Solids Transport in Sewers

Interest in the problems caused by the deposition of sediments in sewers, and by the transport of solids through and from sewer systems has given rise to a great deal of research both in the UK and internationally (Verbanck et al 1994).

In the UK, it has been estimated that some 10% of all sewers have permanent deposits of sediments (CIRIA 1987). The problems caused by the existence of these deposits include:

(i) reduction in the hydraulic capacity of the sewer, leading to possible flooding in surrounding areas during storms;

(ii) uncontrolled and highly variable washout of sediments and associated pollutants due to re-entrainment during storm events;

(iii) premature operation of stormwater overflows;

(iv) gas generation within sediment deposits, giving rise to the release of hydrogen sulphide into the atmosphere within the sewer. This gas may oxidise to form sulphuric acid within the airspace in the sewer, causing structural corrosion to the sewer.

The resultant maintenance costs incurred are considerable, estimated to amount to as much as £60m per annum in the UK in 1987 (CIRIA 1987).

Sediment build up occurs mainly in older sewers in central areas of cities which often do not have steep enough gradients for the efficient conveyance of low flows during dry periods. Deposition of solids from sewage flow therefore tends to occur during periods of lower flow. A proportion of the deposited material may subsequently be eroded from the sediment bed and re-entrained into the overlying sewage flow (Ashley et al 1992a). This effect may be particularly marked at the start of a storm event when a significant amount of readily erodible material may be resuspended over a short period of time, leading to an early peak in suspended solids concentration. The maximum pollutant load during storms has been found to be up to four times the theoretical maximum which could be attributable to a combination of surface washoff, gully pot storage and foul flow component (Berndtsson et al 1985). This phenomenon is known as a "first foul flush" (FFF) (Geiger 1987).

The origins of the solid material conveyed in sewage flows, and which form bed deposits in sewers, are many and of a highly variable nature. A survey by the Construction Industry Research and Information Association (CIRIA 1987) of sediment problems in the UK included attempts to gain an insight into the sources of these sediments, and their relative order of significance. This was done by assessing the responses given by UK sewerage authorities when asked by questionnaire to list what they perceived to be the main sources in order of importance. These were, in order of importance:

1) Winter gritting operations
2) Road surfacing materials and roadworks

3) Ingress of surrounding ground
4) Industrial/commercial processes
5) Construction work
6) Flooding
7) Sediment provided by run-off from impervious areas
8) Domestic sewage
9) Soil eroded from pervious areas
10) Windblown sand.

It was noted by CIRIA that the above list was a generalisation, and that local site conditions, whether temporary, permanent or seasonal, would be very significant in individual cases. Overall, this indicated a high degree of variability in the factors external to the sewer system which could affect the sediment type and quantities found in a particular sewer.

In the UK, a collaborative research effort investigating all aspects of sewer sediment deposition and erosion has been carried out since 1986 co-ordinated by WRc on behalf of the UK water industry (Crabtree and Clifford 1989), under the auspices of the UK water industry's Urban Pollution Management (UPM) programme. The work of the Wastewater Technology Centre (WWTC) of the University of Abertay Dundee (UAD) has made a major contribution to the UPM programme, particularly in the study of the nature, movement and polluting potential of sewer sediments in the combined sewerage system in Dundee, Scotland (Ashley et al 1992a). Data from field studies carried out by WWTC have been used to assist in the development of the sewer flow quality model MOSQITO (Modelling of Sewage Quality in Tanks and Overflows) (Moys and Henderson 1987). In addition, theories derived from these data will also aid in the development of new models for sewer sediment erosion and transport (Ashley et al 1990a). The various components of the work carried out in the UK as part of the UPM programme are tabulated in Table 1.

This thesis describes an investigation of the transport of solids in combined sewers during both dry weather flow periods and storm flows. As discussed in Chapter 3 of this thesis, there is a lack of good field data on in-sewer sediment movement. The study was based primarily on measurements of flow conditions and suspended solids concentrations at two sites on the main interceptor sewer at Murraygate, Dundee City Centre and one other site on a trunk sewer in the Perth Road area of Dundee. These data were utilised to examine the various methods by which total suspended solids (TSS) concentrations in the sewage flow at the study site could be predicted using a variety of combinations of recorded data. A comparison of the effectiveness of the methods used was undertaken by calculating the accuracy of prediction of TSS concentrations compared with measured values from sewage samples. Subsequently, a method by which the calculation procedure for predictions of TSS concentrations may be selected based on hydraulic conditions and other sewer data was proposed. This selection method was found to give satisfactory results.

TITLE	ESTABLISHMENT	DESCRIPTION	DATES
Transport of granular sediments in pipes	Hydraulics Research Ltd.	Lab study of transport of non-cohesive sediments in pipes including bed forms	1986 -
Sedimentation in storage tanks/CSO design and operation for self-cleansing	Universities of Manchester and Sheffield	Lab and field studies to optimise design with respect to minimisation of sedimentation	1986 -
Dundee Central Area Sewer Model	University of Abertay Dundee	Development of sewer flow simulation model for Dundee city catchments	1986 -
Influence of cohesion on sediment behaviour in sewers	University of Newcastle-upon-Tyne	Lab study to identify influence of cohesive additives on erosion threshold of non-cohesive sediments	1987 -
The nature and movement of sewer sediments in combined sewers	University of Abertay Dundee	Field study based investigation of sewer sediment origins, movement and polluting potential	1987 -
The rheology of sewer sediments and the development of a synthetic sediment for laboratory cohesive sediment studies	University College Swansea	Measurement of the shear resistance of sewer sediments. Development of a surrogate sediment for laboratory erosion sediments	1987 - 1992
Movement of cohesive sediment in a large combined sewer	University of Abertay Dundee	Field study based investigation of the fundamental mechanics of sediment movement	1988 -
Time - dependent changes in the characteristics of sewer sediments	University of Birmingham	Lab studies of real sewage and sediments	1988 -

Table 1 - Related Research Work in the UK

1.2 Research Aims and Objectives

The research had the following principal aims:-

1) To collect field data from specific sites on a combined sewer system. These data comprised flow data, cross-sectional data and sediment depths, together with the results of laboratory analyses on a series of sediment and sewage samples. The site used for this purpose was selected mainly for its ease of access for site work, and since it required the minimum disruption to the normal operation of the sewer system.

2) To devise a method by which rates and characteristics of solids transported as bed load material could be assessed.

3) To carry out the procedure proposed in 2) at a second suitable site on the same sewer system.

4) To identify the types of sediment present at the study sites with respect to the WRc five stage classification proposed by WRc (Crabtree 1989).

5) To examine different methods by which the rate of transport of solids at any given time for the specific site referred to in 1) could be predicted on the basis of the data available for the site.

6) To assess the accuracy of prediction of these methods by comparing recorded solids transport rates with predicted rates.

7) To develop a methodology of approach for the selection of the most appropriate model for prediction of solids transport rates in sewers.

The general objective of the research was to advance knowledge of ways in which rates of solids transport in combined sewer systems may be predicted. The specific objectives were as follows:-

1) To find out if a method of predicting instantaneous suspended solids concentrations in combined sewer flows during both periods of dry weather flow and storms, given flow data, could be developed using the field and laboratory data collected.

2) To propose a method by which an estimate of bed load transport rates may be made when the associated average suspended solids concentrations and flow conditions are known.

3) To develop a methodology by which predictive models of solids transport in combined sewers may be constructed for other similar site-specific applications.

4) To develop a non-site-specific model of solids transport in combined sewers by applying a similar analysis of data from more than one site and incorporating a factor which differentiates between conditions at the different sites.

1.3 Principal Results

The advancement of knowledge demonstrated in this thesis is in the following specific areas:-

1) The problem of measuring bed load transport rates was addressed by construction of an in-situ flume device. Data obtained using this apparatus allowed limited conclusions of a general nature to be drawn. It was concluded that on average, the rate of transport of bed load material during the periods for which measurements were taken was approximately 12% of the total solids (dry weight) transport load. It was also concluded that this information is indicative of further significant work that the use of similar devices could facilitate.

2) Ackers' model was found to be a viable proposition for the specified application for the prediction of suspended solids concentrations in a combined sewer. The form of Ackers model has been modified by the author for this application.

3) The methodology proposed for arriving at appropriate predictive models for suspended solids concentration was shown to give rise to a number of models suitable for various situations. The preferred options in each case (i.e. DWF and storm events for both site-specific and non-site-specific applications) were found to be various forms of regression analysis-based equations which related solids concentrations to various combinations of hydraulic and physical measurements. In all cases, the

models performed the allotted tasks adequately in comparison to the performance of various transport models tested in non-sewer applications elsewhere by other authors. The suitability of the methodology was thus demonstrated.

4) It is apparent from the non-site-specific model developed that, within the limits of variability of the data used, it is possible to develop a regression based equation using the proposed methodology which does not require further calibration for more general use. This finding, in conjunction with the vindication of the methodology used to achieve this stage, opens up the possibility of the ultimate development of a universally applicable model: it is logical to assume that a wider data base than used here for model development could achieve more reliable results. Alternatively, further work could lead to a fixed procedure for arriving at a site-specific model suitable for a particular application. These two approaches are complimentary, and the appropriate option could be selected based on the site data available in each particular application.

2 SOLIDS TRANSPORT MODELLING

2.1 Introduction

There are various types of model available in the literature for the prediction of solids transport in a number of different situations. Some of these are in common use. A model may be a "planning model" which predicts total quantities of material transported during a storm event or other period of time. Such a model may include a conceptual element to account for pollution accumulation, including solids, on the catchment surface during dry weather and subsequent erosion during storm runoff (Bertrand-Krajewski et al 1993, Hemain 1986). Alternatively it may be a model which predicts the variation in load or concentration of solids with time ("pollutograph") (Huber 1986). For the purposes of this study, the latter type only will be considered here. Also, the model may be stochastic or deterministic and be either statistically or physically based or some combination of both approaches. Hemain (1986) defines stochastic and deterministic models as follows:-

(i) Stochastic - models which comprise relationships between probability levels of the variables included in the processes.

(ii) Deterministic - models which comprise causal relationships between various variables controlling the processes modelled.

The latter group may be further subdivided into:

(a) Physically-based models which are derived from theoretical approaches.

(b) Statistically-based models, mainly developed from statistical analysis of experimental data. These may be purely empirical, but can also take advantage of physical theories.

The distinctions between the above groupings are somewhat blurred. Models may be a combination of different types, making the best use of the available techniques. In addition, alternative definitions and terminology are used by different authors. For example, Huber (1986) refers to regression rather than statistical modelling. Huber also makes the point that the opposite of "statistical" (or stochastic) does not necessarily imply "deterministic" since there are likely to be large errors in prediction by a purely deterministic model. The set of definitions as set out by Hemain is followed in this thesis.

Appendix B provides a detailed review of solids transport theory. The various "classical" approaches to the analysis of the problem are considered in detail, as presented in the literature. It is within the context of this backgound of theory that the various transport modelling approaches discussed in the following sections of Chapter 2 are considered. This thesis utilises the approach of suspended load transport modelling, although aspects of total load transport modelling are incorporated. This is discussed in Chapter 4. First it is appropriate to consider the various alternative modelling approaches that have been investigated by workers in this field.

The quantities or conditions predicted fall into a number of categories: models may predict the total load of solids transported (including or excluding wash load), bed load only, or suspended load only (again including or excluding wash load). A further type of model predicts certain limiting conditions under which either deposit-free transport of solids may be supported, deposition of solids commences, or at which the transport of solids commences. Each of these modelling approaches are discussed in turn in the following sections.

2.2 Limiting Conditions for Sediment Transport

This section considers modelling approaches which relate suspended solids transport to certain limiting conditions. In this connection, the work of May (1982, 1993), Hare (1988), Nalluri and Mayerle (1989) and of Novak and Nalluri (1984) are of note.

2.2.1 Deposit Free Conditions

Based on a series of experiments for the transport of non-cohesive sediments using pipe-full and part full flow through 158mm and 77mm diameter pipes, May (1982) developed a model to predict the minimum gradient required to produce a self-cleansing velocity. This velocity was shown to depend on a combination of pipe size, sediment concentration and sediment characteristics. The study was supplemented by data from field measurements in a 1.8m diameter sewer. The results of the study gave rise to the formula

$$C_v = \frac{2.05}{100}\left(\frac{D^2}{A_f}\right)\left(\frac{d}{R}\right)^{0.6}\left(\frac{V_s^{\,2}}{g(s_s-1)D}\right)^{\frac{3}{2}}\left[1-\frac{V_0}{V_s}\right]^4 \qquad (1.)$$

where C_v = volumetric sediment concentration

D = pipe diameter

A_f = cross-sectional area of flow

d = size of sediment

R = hydraulic radius

V_s = settlement velocity

V_o = effective threshhold velocity

g = acceleration due to gravity

s_s = specific gravity of sediment

Although the formula includes a term for specific gravity, all laboratory tests were carried out on single sized sands and gravels with a specific gravity of 2.65. The work was extended by Hare (1988) to include data from pipes of 300mm diameter. Hare's review of the available formulae showed that the May formulae gave best fits to the data for pipe-full conditions. However, for part-full flow, the formula proposed by Nalluri and Mayerle (1989) gave the best fit:-

$$V \Big/ \left(g(s-1)d\right)^{1/2} = 0.86 C_v^{-0.18} \lambda^{-1.06} \left(R/d\right)^{-0.2} D_{gr}^{-0.4} \qquad (2.)$$

where V = average velocity of flow

s = particle specific gravity

d = particle diameter

λ = friction factor

D_{gr} = dimensionless grain size proposed by Ackers and White (see section 2.5)

Other workers who have produced relationships which predict sediment-free flow conditions include Macke (1983) and Ambrose (1952).

2.2.2 Limit of Deposition

As discussed in section 3.1 of Appendix B, Robinson and Graf (1972) determined the limit of deposition at volumetric concentrations between 0.1% and 7%. By linear regression of the results of a series of tests they obtained

$$F_L = 0.928 \frac{C_v^{0.105} d_{mm}^{0.056}}{(1 - \tan\theta)} \quad (3.)$$

where F_L = lift force

d_{mm} = sediment size in mm

θ = angle of the pipe to the horizontal

This followed earlier work by Durand and Condolios (1956).

Nalluri et al (1994) conducted a series of tests in a 305 mm pipe channel over a fixed bed for transport of sand (d_{50} ranging from 0.53 mm to 8.4 mm) at limit deposit conditions. The resultant equation produced was as follows:

$$\frac{\tau_b}{\rho g \Delta d_{50}} = 0.47 C_v^{0.33} \left(\frac{W}{y_0}\right)^{-0.80} \left(\frac{d_{50}}{D}\right)^{-1.14} (\lambda_{sb})^{1.20} \quad (4.)$$

where τ_b = bed shear stress

ρ = density of fluid

Δ = relative density of sediment

d_{50} = median diameter of particles in a mixture

W = bed width

y_o = uniform depth of flow

D = pipe diameter

$\lambda_{sb} = 6.6\lambda_s^{1.45}$

λ_s = overall friction factor with transport

λ_{sb} = bed friction factor with transport

The equation was found to be suitable for prediction of limiting sediment transport concentrations for both clean pipes with no deposited beds, and rectangular channels with deposited beds.

2.2.3 Incipient Motion

The work of Novak and Nalluri (1984) was based on experimental data relating to incipient motion of single and grouped sediment particles. Measurements were carried out in flumes with fixed smooth and rough beds. Hence formulae in terms of critical velocity, critical shear stress and particle critical Froude number were developed.

A study carried out by Kleijwegt (1992) showed that incipient motion in sewers with a circular cross section occurred at values of critical shear stress of the order of 70 per cent of those predicted by the Shields curve (Bogardi 1978). This is discussed in more detail in Section 3.3 of Appendix B. Earlier studies of incipient motion were carried out by Ippen and Verma (1953). Also, Mantz (1977) extended the work of Shields to include the prediction of incipient transport of flakes, ie non-spherical particles.

Recently, attempts have been made to predict conditions for incipient motion of cohesive material. Work by Alvarez-Hernandez (1990) showed that the presence of cohesive material in the sediment bed significantly increases the magnitude of critical shear stresses for initiation of erosion. In cases where cohesive forces are significant, it is therefore likely that the cohesive properties of the sediment will have an overriding influence on the conditions under which the initiation of erosion occurs.

2.3 Bed Load Transport

A number of studies have attempted to predict bed load transport under varying conditions including alluvial channels and in full and part full pipes.

Following on from the earlier work of Du Boys, Meyer-Peter and Muller, Schoklitsch, Shields, Kalinske and Einstein (see Section 4.1 of Appendix B), Rottner (1959) used an approach similar to that of Shields in developing a dimensionally homogeneous equation for bed load transport. He stated that the transport rate can be predicted using the following dimensionless groupings

$$\frac{G}{\sqrt{\frac{\rho'-\rho}{\rho}}\sqrt{gh^3}} \quad , \quad \frac{h}{d} \quad , \quad \frac{V}{\sqrt{\frac{\rho'-\rho}{\rho}}\sqrt{gh}} \quad and \quad \frac{i}{(\rho'-\rho)/\rho}$$

where h = depth of flow

i = channel slope

G = rate of bed load transport in weight per unit width

Rottner (1959) developed a series of curves relating these groupings based on an analysis of approximately 2500 observations. Given sufficient information to evaluate any two of these dimensionless groupings, the other two can be determined using the resulting graph. In a comparative study of the performance of various solids transport theories against a large quantity of flume and field data (from natural watercourses) by White et al (1975), Rottner's method was found to be superior to the bed load formulae of Meyer-Peter and Muller (Section 4.1 of Appendix B). It was suggested by White et al that the Rottner method could be used as a total load theory, which is a reasonable suggestion given the performance of this model for this mode of transport as discussed in Section 2.5.

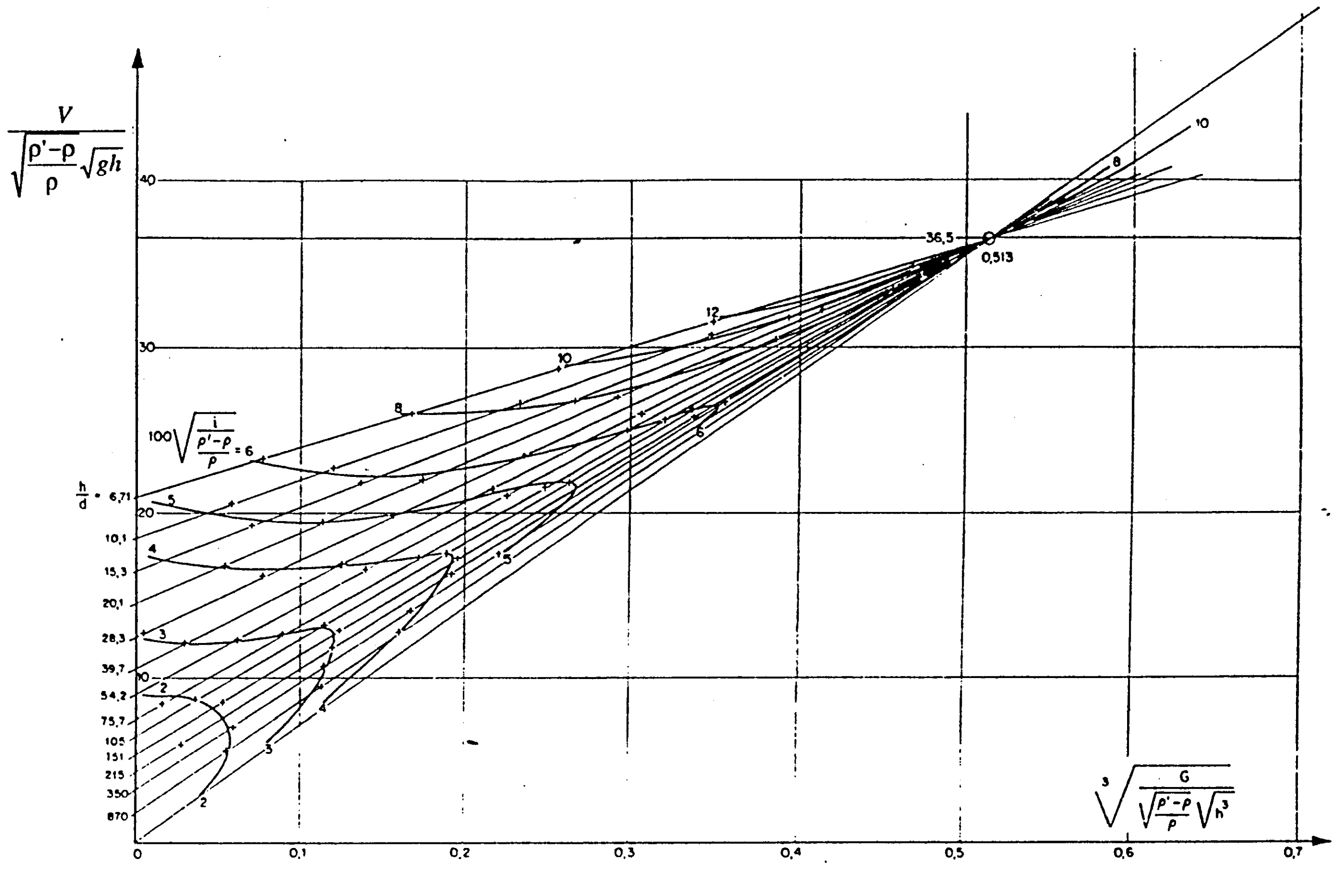


Figure 1 - Rottner's Bed Load Transport Chart

(Source: Rottner (1959))

Yalin (1963) developed a bed load theory based on the concept of saltation. Unlike Einstein's (Rouse 1950) statistically based method, however, Yalin assumed that the transport rate increased due to an increase in the length of jump made by particles in motion, rather than an increase in the probability that an individual particle will jump. Using dimensional theory, Yalin developed a transport equation based on excess shear. Although in the specific instance of flat beds, this relationship may perform better than Einstein's, it is not generally accepted to be superior to other available theories (Graf 1984).

Van Rijn (1984a) used a large set of flume and field data to develop a bed load formula for non-cohesive sediment. This formula includes the following dimensionless parameters:

$$D_* = D_{50}\left(\frac{\Delta g}{V^2}\right)^{1/3} \quad = \text{dimensionless particle parameter}$$

$$f_b = \left(\frac{C_b}{C'}\right)^2 \quad = \text{bed form factor}$$

$$T = \frac{\left(u_*''\right)^2 - \left(u_{*,c}\right)^2}{\left(u_{*,c}\right)^2} = \text{transport stage parameter}$$

where Δ = relative density of sediment

$= \left(\rho_s - \rho_f\right) / \rho_f$

C_b = Chezy coefficient

C' = Chezy coefficient related to grains

$u_*'' = \sqrt{f_b}\, u_*$

u_*'' = effective bed shear velocity

$u_{*,c}$ = critical bed velocity

The resulting formula includes coefficients based on a calibration to fit the data used:

$$S_b = \frac{0.053 B T^{2.1} (\Delta g)^{0.5} D_{50}^{\ 1.5}}{D_*^{\ 0.3}} \tag{5.}$$

where B = channel width (for a rectangular channel)

S_b = bed load transport

A similar approach to the above has been carried out by Perrusquia Gonzalez (1992). An experimental study of bedload transport in a part-full pipe was carried out in a concrete pipe, using quartz sands with a relative density of 2.65. A relationship based on dimensional analysis was proposed, expressed in terms of flow and particle parameters, and geometrical factors as follows:

$$\Phi_b = 46\times10^3 \Theta_b{}^{2.9} D_*{}^{-1.2} D_{50}{}^{0.7} Y^{-0.7} t_r{}^{-0.62} \qquad (6.)$$

where Φ_b = transport parameter

$$= \frac{q_b}{\sqrt{g(s-1)d_{50}{}^3}}$$

q_b = sediment transport rate per unit width

Θ_b = dimensionless bed shear stress

$$= \frac{R_b S}{(s-1)d_{50}}$$

$$D_* = \left[\frac{g(s-1)}{\upsilon^2}\right]^{1/3} d_{50}$$

t_r = relative bed thickness

$$= t/Y$$

t = bed thickness

$$s = \rho_s/\rho$$

$$R_b = \frac{\tau_b}{\rho g S}$$

S = hydraulic gradient

$$\tau_b = u_{*s}^2\,\rho$$

Y = depth of flow

u_{*s} = shear velocity

d_{50} = sediment size

D_* = dimensionless particle parameter

Recent work by Nalluri and Alvarez - Hernandez (1992) included attempts to model bed-load transport of cohesive sediments over a moveable bed. The test section of 154mm diameter flume contained a prepared bed of synthetic sediment (a mixture of laponite RD clay, sand and water). Under constant flow conditions, further synthetic sediment was fed into the flow at a constant rate at the upstream end of the flume, while bedload material was sampled at the downstream end by a sediment trap set into the invert. Measurement of transport rates was found to be impractical due to the rapid break up of the cohesive bed after commencement of erosion. However, the study showed that cohesive material, once in motion, behaves in a similar manner to non-cohesive material.

2.4 Suspended Load Transport

Apart from the relation for distribution of concentration proposed by Rouse (see section 4.2 of Appendix B), a similar relation was proposed by Ippen (1971) which differed only in that the Krey equation (Vanoni 1984) for the velocity profile was used in place of the Karman - Prandtl equation. Other workers who have given alternative concentration distributions include Hunt (1954).

Complementary to his work on bed-load transport, Van Rijn (1984b) developed a relation for the prediction of suspended solids transport. In common with Rouse, Ippen and Hunt, this was based on a relation for concentration profiles which related suspended solids to a reference level and which was intended for use in alluvial streams or canals. Unlike his predecessors, however, Van Rijn based his concentration profiles on measured data for height versus concentration. The reference height is defined as

$a_v = 0.5H$ or $a_v = k_s$ (use k_s if H is not known) (7.)

where a_v = reference height

H = height of bed forms

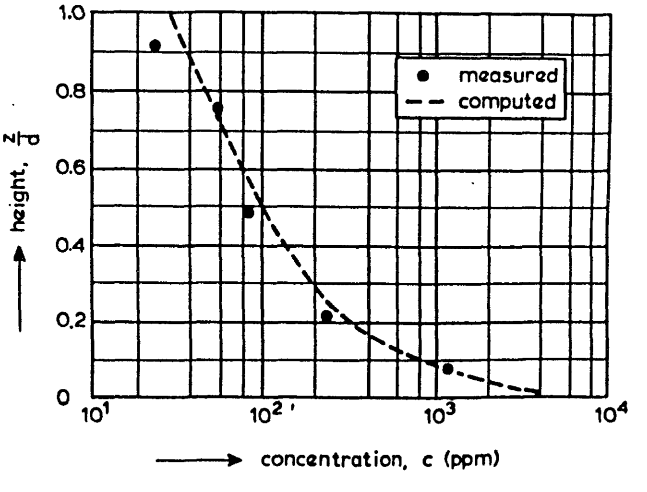
k_s = equivalent roughness height

The reference concentration at height a_v is given by

$$C_{av} = 0.015 \frac{D_{50} T^{1.5}}{a_v D_*^{0.3}} \qquad (8.)$$

where C_{av} = reference concentration

and D_* and T are as defined previously for bed load (see section 2.3).



Concentration Profile for Mississippi River (station 1100, Apr., 1963)

Figure 2 - Example of Van Rijn Concentration Profile

(Source: *Van Rijn (1984b)*)

$$D_s = D_{50}(1 + 0.011(\sigma_s - 1)(T - 25) \tag{9.}$$

where σ_s = geometric standard deviation

= 2.5 normally

Next, the fall velocity of suspended sediment is computed by one of the following formulae:

$$w_s = 1/18((s - 1)gD_s^2)\upsilon \qquad D_{50} < 100\mu m \tag{10.}$$

$$w_s = 10((\upsilon / D_s)((1 + (0.01(s-1)gD_s^3) / \upsilon^2))^{1/2} - 1) \qquad 100\mu m < D_{50} < 1000\mu m \tag{11.}$$

$$w_s = 1.1((s-1)gD_s)^{1/2} \qquad 1000\mu m < D_{50} \tag{12.}$$

where υ = kinematic viscosity coefficient for clear fluid

w_s = particle fall velocity in clear still fluid

Then the suspended load transport per unit width (S_s) is calculated:

$$S_s = F\,d\,C_a \tag{13.}$$

where S_s = suspended load transport per unit width

$$F = \frac{\left({a_v}/{d}\right)^{Z'} - \left({a_v}/{d}\right)^{1.2}}{\left(1 - {a_v}/{d}\right)^{Z'}(1.2 - Z')}$$

$Z' = Z + \varphi$

$$Z' = Z + \varphi$$

$$Z = \overline{w}_s/(\beta K u_*)$$

$$\beta = 1 + 2\left(\frac{w_s}{u_*}\right)^2$$

$$\varphi = 2.5\left(\frac{w_s}{u_*}\right)^{0.8}\left(\frac{C_a}{C_0}\right)^{0.4}$$

$$u_* = (gdS)^{1/2}$$

K = Von Karman constant (= 0.4 approximately)

φ = correction factor for concentration profile

Z = suspension number

Z' = suspension number

β = ratio of sediment diffusion and fluid diffusion coefficient

$\overline{w}$ = local mean vertical flow velocity

C_o = sediment concentration

Clearly, in order to predict suspended solids transport by this method, it is necessary to have a great deal of information regarding the sediment characteristics. Subject to this proviso, the Van Rijn formulae are widely accepted as a model which performs well (Kleijwegt 1992) and has been attributed with a greater level of accuracy of prediction than the Ackers and White model discussed in Section 2.5 (Robinson and Graf 1972, White et al 1975).

2.5 Total Load Transport

The well known combination of Einstein's bed load formula and Rouse's concentration distribution relation (Garde and Ranga Raju 1985, Graf 1984, Rouse 1950, Vanoni 1984) gave one of the earliest total sediment discharge relations. On the basis of observations, Einstein concluded that particles of a given size move in a series of steps of definite length and frequency, and that the rate of transport depends upon the number of particles in motion. The probability that a particle will move in a given time step is expressed in terms of transport rate, size and relative weight of the particles, and a time factor equal to the ratio of the particle diameter to its fall velocity. The probability of particle movement is also expressed in terms of the ratio of flow-related forces to the resistance of the particle to movement. These two probability relationships may then be equated

$$\Phi = f(\Psi) \tag{14.}$$

where

Ψ (shear intensity or flow parameter) $= \Delta \; d/\mu \; RS$

Φ (transport parameter) $= q_b/\sqrt{\;}\,(g\Delta \; d^3)$

μ_r = ripple factor

Δ_r = the apparent roughness diameter

q_B = rate of bed load transport in weight per unit width

Einstein investigated the form of the indicated function by plotting experimental measurements of Φ versus Ψ (see Figure 3), and thus developed a relationship for the prediction of bedload transport rates.

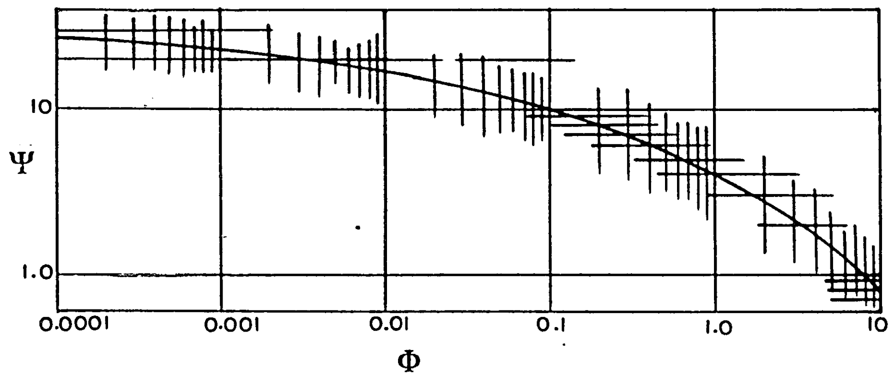


Figure 3 - Plot of Φ versus Ψ

(Source: Featherstone and Nalluri (1988))

Subsequently, Rouse (1950) proposed the following equation as a means of predicting the variation in concentration of sediment with depth

$$\frac{C}{C_{ae}} = \left[\frac{(Y-y)}{y}\frac{a_e}{(Y-a_e)}\right]^Z \qquad (15.)$$

where
C = concentration of sediment with fall velocity ω at level y
C_{ae} = average concentration of the bed layer
y = elevation above datum
a_e = the lower limit of y where suspended load begins

The evaluation of Rouse's distribution requires a sediment concentration at some reference level. However, Einstein designated a flow layer on top of the fixed bed as the bed layer, which was found to be of a thickness $a' = 2d$ (Graf 1984). The material in this layer is deemed to be the source of the suspended load, making the determination of the lower limit concentration C_{ae} possible. The value of C_{ae} is assumed to be the average concentration of the bed layer.

The development of the Einstein and Rouse equations is discussed more fully in Section 4 of Appendix B. A number of alternative solutions to the problem have been attempted since then with varying degrees of success. These are discussed below.

Engelund and Hansen (1967) presented a total transport relation based on a similarity hypothesis which implies that the dimensionless shear stress is a function of the dimensionless grain shear stress. Based on a series of experiments with different sand sizes, the formula for total solids transport was produced:

$$S = B\frac{0.05u^2 {u_{*,b}}^3}{(g\Delta)^2 D_{50}} \tag{16.}$$

where S = hydraulic gradient

$\Delta = {(\rho_s - \rho_f)}/{\rho_f}$

$u =$ flow velocity component

$u_{*,b}$ = bed shear velocity

Although relatively simple in form, this equation has been shown to perform well (White et al 1975) for flume and river data, although it must be stated that Engelund and Hansen specifically excluded wash load in their assessments of the data on which the formula is based.

One of the most widely used and best known methods of predicting total sediment discharge is the series of relationships developed by Ackers and White (1973). Since this model is discussed at a number of points in this thesis, the procedure for determination of the rate of solids transport in full or part - full sewers (Ackers and

White method), as set out in Project Report 1 by CIRIA (1987) is shown in Appendix C. Only an overall description is included here.

Although originally developed for prediction of solids transport rates in rectangular alluvial channels, the Ackers and White equations were subsequently modified to apply to the prediction of transport rates in sewers (Ackers 1984). By means of dimensional analysis, Ackers and White used physical arguments in deriving the form of a series of relationships which were calibrated using flume data, and compared with data from alluvial channels. Ackers proposed a modification which used these formulae along with the Colebrook - White friction equation. Following the analysis of laboratory data on sediment in pipes, Ackers found the effective transport width in a pipe without deposition was approximately equal to 10 times the particle size; when deposition occurred, the effective width was assumed to be equal to the pipe diameter.

The performance of the Ackers - White model for alluvial streams is comparable to that of Engelund and Hansen's and to Rottner's (see section 2.3) according to White et al (1975). It is also less limited in its applicability than Engelund and Hansen's, since the authors do not specifically exclude wash load. Also, unlike Rottner's method, it was not developed originally solely as a bed load model. (As stated in Section 2.3, White et al suggest that the Rottner method gives adequate performance in the prediction of total load for flume and alluvial channel data.) The limitations of the Ackers - White model are discussed in Section 7.3.1.

Sonnen and Field (1977) proposed a model for inclusion in the US EPA Stormwater Management Model (SWMM) (US EPA 1971) to predict total sediment discharge. As with the preceding Ackers - White model, it was deemed more appropriate to place the full description of the calculation procedures for this model in the appendices (Appendix O) since there is discussion of the model at more than one point in the thesis. The Sonnen and Field model uses the bed load relation proposed by Kalinske (see section 4.1 of Appendix B) and the Rouse equation for prediction of the vertical concentration profile. The resultant model, when used in early versions of the SWMM software, has been found to give satisfactory results provided it is carefully calibrated for site conditions (Bertrand-Krajewski 1993). However, this model has subsequently been removed from the SWMM software (Huber 1986).

The combination of the Van Rijn relations for bed load and suspended solids load (see sections 2.3 and 2.4 respectively) can also be used to predict total load, simply by summing the two quantities predicted. In a similar performance assessment exercise to that carried out by White et al (1975), Van Rijn (1984b) compared his total load model with those of Engelund and Hansen, Ackers - White and Yang. This comparison indicated that, for the data used (flume and alluvial channel data), the Van Rijn relation was superior in all cases.

Apart from the total load prediction methods discussed above, there are a number of alternative models which are less widely recognised. These include the work of Colby and Hembree (1955), Laursen (1958), Bogardi (1978), Bishop et al (1965), Bagnold (1966), Chang and Hill (1977), Graf and Acoroglu (1968), Toffaleti (1969), Wiuff (1985), and Celik and Rodi (1991). These are not discussed further here.

2.6 Conclusions

The purpose of this thesis is to investigate ways in which models may be selected or developed to predict solids transport in combined sewers. The fundamental mechanisms involved in the movement of sediment particles are discussed in Appendix B. These are not fully understood at the present time. Currently, a number of modelling procedures, including those discussed in Chapter 2, have been proposed by various workers. Some of these models have gained qualified acceptance for their intended purposes. In many cases the available models were first developed for non-sewer sediments, although modifications for in-sewer applications are available in some instances based on laboratory work. In other cases, models have been developed specifically to predict solids transport rates in sewers, but in the main these have been based on laboratory studies. It is not yet clear which of the various approaches available is most appropriate for the prediction of transport rates in 'real' combined sewers either in their original form or some modified form.

For the particular circumstances associated with the study conducted for this thesis, the selection of suitable models for consideration as a possible solution to the problem of prediction of solids transport rates is discussed in Chapter 4. The limitations of the models are critical, since there are a number of factors related to the conditions pertaining to sewers which complicate the issue. These factors are discussed in the following chapter.

3 SOLIDS TRANSPORT IN SEWERS

3.1 Introduction

There is a need for models which can predict the transport rates of suspended solids in combined sewers in order to assess the extent of sediment-related problems and to arrive at suitable solutions. The available theories and transport models which may be relevant are discussed in Chapter 2 with further information in Appendix B. In the context of modelling of solids transport in combined sewers, these models represent individual attempts to converge on a solution, with varying degrees of success. One of the main limitations in the development of the models, or in the assessment of their suitability for in-sewer applications is the data on which they have been based. The data used for model development and assessment have been either laboratory data, or field data from sources other than sewers. According to CIRIA (1994),

> "One area not covered by laboratory research to date is the transport of suspended sediment (or sediment which is carried partly in suspension and partly as bedload) over a deposited bed, at concentrations typical of those found in sewers."

Also, there is a lack of good field data on in-sewer sediment movement, including in-sewer sediment concentrations in relation to flow conditions. The fact that in-sewer data have not been used is an important limiting factor in achieving the required accuracy of prediction for sewer solids transport.

The added complexities which may affect solids transport in sewers are discussed in this chapter. These complexities may either invalidate the models previously discussed when applied to sewers, or at least limit the applicability or performance of such models if used in an unmodified form.

3.2 Sewer Types

A sewer system may be defined as a system of pipes and ancillary structures which are used to convey sewage from the points of origin to some disposal point remote from the sources of the sewage. The sources of sewage which are served by such a system may be large in number and varied, and distributed over a large catchment area. The sewerage pipes within such a system may be numerous and varied in configuration. The overall layout of the system normally comprises of a branched network of interconnected pipes serving a series of sub-catchments. Sewer systems may be generally categorised as either separate systems or combined systems. Since the work reported in this thesis utilises data obtained from a combined sewer system, only this category of sewer will be considered further here.

Within a combined sewer system, there may be a large number of sewer types located at various points in the pipe network. These differ in size and cross-sectional shape, (circular, rectangular, egg shaped, oval, etc) slope and construction (eg brick, clay, concrete). They may also differ with respect to age, condition, location - within the system (either peripheral or central areas) and in the functions they perform. All of these factors may have some influence on the likelihood, amounts and types of sewer sediment found at various points in a network, and on the suspended solids concentrations distributed temporally and spatially within a network during changing flow regimes (Crabtree et al 1991).

Attempts have been made to categorise sewer types based on relative sewer size, gradient, contributing catchment and propensity for sedimentation. Pisano et al (1979) proposed a relationship based on a regression analysis of field data for rates of solids deposition within a sewer network related to a number of variables including total sewer length, service area, slope and total solids input due to foul flows. The data used were from a large data base of information on deposition in small combined sewers (less than or equal to 0.39m) in Boston, USA. This approach assumed a critical pipe boundary shear stress of $0.19 N/m^2$, below which 40% of the dry weather suspended solids conveyed in each pipe length are deposited. The work led to the three alternative models classed as "elaborate", "intermediate" and "simplest" with decreasing numbers of variables and correspondingly lower reliability. The intermediate model for prediction of deposited solids loading was as follows.

$$TS = 0.001303 L'^{1.18} A^{-0.178} (\overline{S})^{-0.418} (\overline{D})^{0.604} q_p^{-0.51} \qquad (17.)$$

where TS = deposited solids loading (lbs/day)

A = service area (acres)

$\overline{S}$ = average pipe slope

$\overline{D}$ = average equivalent diameter (inches)

q_p = discharge per capita, including infiltration (gpcd)

Following on from this work, Ashley et al (1992b) proposed that there may be three categories of sewer for which both solids transport during dry weather and sedimentation potential could be differentiated. The categories of sewer were related to the location and physical characteristics of the sewers, and to the nature of the sediment deposits typically found in each sewer. The resulting three-fold categorisation of sewer types is as follows:

Collectors - small diameter, local sewers with the greatest relative range of flow variation, and requiring storm inputs to clean the sediment deposits which accumulate during dry weather.

Trunks - connecting the collector sewers to outfalls or interceptors, but with steeper gradients.

Interceptors - with the slackest gradients and greatest potential for sedimentation, dry weather flows having the least range of variability.

Generally, these categories are in order of increasing sewer size. In an attempt to classify the sedimentation potential of sewers based on physical characteristics, Ashley et al have produced a factor related to diameter, contributing catchment area and slope:

$$\text{DAS} = \text{Diameter (metres)} \times \text{Cumulative Area (ha)} \times \text{Slope (fraction)} \qquad (18.)$$

(ref. Ashley et al 1992b)

It has been found that this factor may be correlated with descriptions of proposed sewer classifications, giving the following ranges of values (Ashley et al 1992b, Jefferies 1991):

Collectors Sewers	DAS < 6
Trunk Sewers	6 < DAS < 8000
Interceptor Sewers	DAS > 8000

The above sewer classification method is utilised in Section 8.3 of this thesis.

A possible alternative to the above in differentiating between sewer types is to identify those sewers with a propensity for the deposition of sediment. Gent and Orman proposed a method for determining whether a section of sewer was susceptible to sedimentation during dry weather and also during wet weather. The method proposed was based on the results from a WALLRUS model using dry weather flow patterns and selected time series rainfall events (Gent and Orman 1991). This method has been employed to predict the location of sewer deposits for a number of sewers in Dundee and the nearby coastal town of Carnoustie (Ashley and Bertrand-Krajewski 1993, Oduyemi et al 1993). One of the Dundee sites where the method was applied was the Perth Road site. This was the only site at which it was possible to compare predicted and actual rates of deposition, giving inconclusive results (Ashley and Bertrand-Krajewski 1993).

3.3 Sewer Sediments

Bed deposits of variable amounts may be deposited at various points in a combined sewer system due to the flow-related processes of transportation, erosion and deposition of solids. Because of the miscellany of materials arising from the various sources, the sediments tend to be heterogeneous. However, solids found in combined sewers, either in suspension or in deposition, can be said to consist of combinations of material which conform with the following descriptive categories (Verbanck et al 1994):

1) Fine material

- always present in suspension in water flow
- less commonly present in the deposit (in tranquil zones, allowing settling)

2) Grit

- constituting the bed if there is one
- in true suspension only during exceptionally high flows/velocities

3) Sanitary solids, of domestic origin

- always present in suspension
- sometimes present in deposits

It is postulated by Verbanck et al (1994) that the apparent ubiquity of these three types of sediment may be due to the man-made character of sewer systems. Since sewer networks conform to different configurations and are managed according to national codes of practice, the nature and occurrence of sediments may differ from country to country.

According to CIRIA (1987), samples of sewer deposits obtained from eleven different areas in the UK showed that where deposits occurred, they were composed mainly of fine sands and coarser particles. The organic content of these samples averaged 18%, but values as high as 87% were found. The main conclusion from the study was that sediments can vary drastically within particular systems.

A continuous study of sediments in the No 13 trunk sewer in Marseille (Laplace et al 1990) has shown a gradual build up of deposits following cleaning. There is also evidence of an "evolution" of sediment both temporally and spatially. There is a tendency for grading of particle size with time, the largest particles being deposited at the upstream end of the sewer, and smaller sediment particles predominating further down the sewer (Crabtree et al 1991). Similar studies in Brussels indicated that with time, finer particles were "leached" out of deposits during peak dry weather flow periods (ibid).

Sediment particle size is particularly important in considering the development of bed cohesion, as well as changes in bulk density, particle nature and chemical/biological properties due to the consolidation of a bed deposit with time (Crabtree et al 1991). Studies of sewer sediment deposits which are of particular interest have been reported for three sewers in France, and a number of sewers in Brussels. The results of particle size measurements for these studies, along with typical ranges for UK sediments are summarised in Table 2 and Figure 4.

PARTICLE SIZE RANGE

COUNTRY	SITE	D10 (µm)	D50 (µm)	D90 (µm)
FRANCE**	Trunk sewer sediments	58 - 683	290 - 8400	2220 - 29000
	Trunk sewer sewage (Dry/Storm)	7.1 - 7.8	35 - 38	328 - 351
	Detention basin sewage (Storm)	3.4 - 8.5	29 - 36	436 - 1384
BELGIUM	Trunk sewer sediments	180	375	1200
UK	Sewer* sediments (various sewer types including trunk, interceptor and tanks)	<2000	<10000	<49000

* Ashed residue only
** Ignoring large (man-made) material

Table 2 - Particle Size Data - French, Belgian and UK Results

(Source: Crabtree and Clifford (1989))

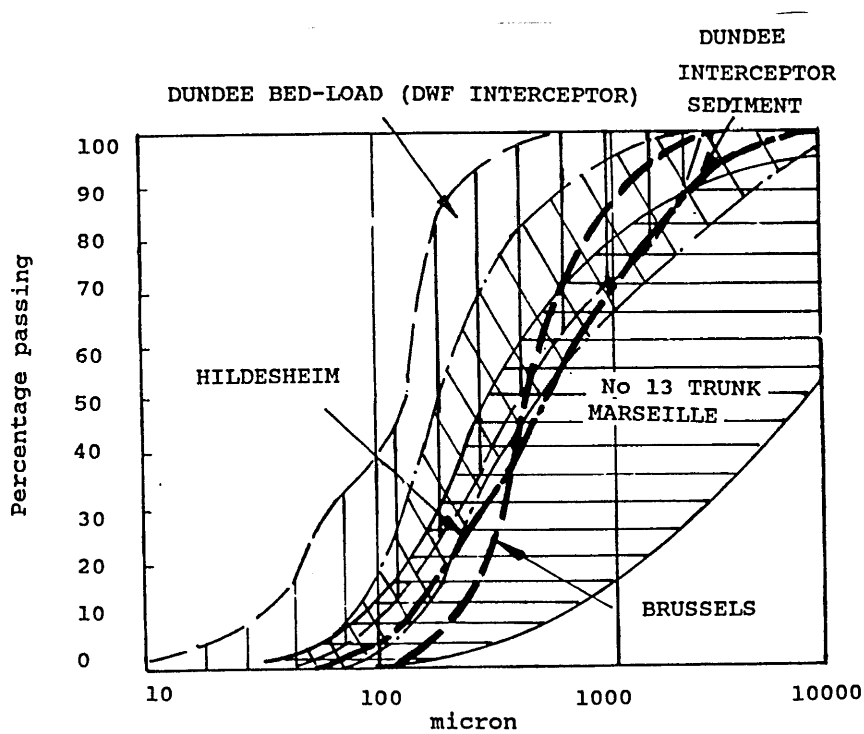


Figure 4 - Particle Size - Dundee/Marseilles/Hildesheim/Brussels

(Source: Ashley and Crabtree 1992)

The UK data are for samples with organic contents of between 3% and 96%, while the French and Belgian sediments are mainly granular material with low organic contents. There is thus a clear difference in the material which is deposited in the UK sewers compared with the other sewers referred to. This may be due in part to national differences in sewer network configuration and different sewage constituents, but the most likely reason for most of the variation in data is the range of sewer types and local practice in each case (Verbanck et al 1994). It should be noted, however, that these differences in sediment deposit may be minimal in some cases. For example, studies in Hildesheim in Germany on a sewer with similar characteristics to the Dundee interceptor sewer show that the two sewer systems contain similar sediments, although the Hildesheim sediments are slightly coarser (Ashley et al 1993c).

As well as the physical size of solid particles, data for the associated chemical pollutants are important in characterising and differentiating between different sewer sediments. These chemical parameters may be related to various physical characteristics of the sediment. Crabtree (1989) proposed a five-fold classification system for sediments based on their occurrence, appearance and nature, using data from sediment samples obtained from a number of sewers in the UK:

Type A - coarse, granular material

Type B - agglutinated type A deposits

Type C - mobile fine grained - usually overlying type A

Type D - organic wall slimes

Type E - deposits found in tanks

An alternative classification system, based on the above sediment classifications but accommodating the possibility of mixtures of different sediment types, was proposed as part of the work reported here (Ashley et al 1990b). This is discussed in Chapter 6.

There are some indications that there is a link between sediment classification, sewer type, and the propensity for sediment deposition/erosion (Ashley et al 1990c). Data collected in Dundee for chemical pollutants, obtained from samples which have been blended prior to testing are shown in Table 3. Data collected from elsewhere in the UK are also included for comparison.

There is a tendency for sewer sediments to exhibit cohesive-like properties to varying degrees. This is due mainly to the organic content of many sewer sediments, originating principally from foul flow. This cohesion has been attributed to agglutination and cementation effects of the organic substances contained within the deposits (Wotherspoon and Ashley 1992). Such deposits consequently have higher critical yield stress values than similar non-cohesive deposits and hence have measurably higher threshold values of bed shear stress before the commencement of erosion.

Field and laboratory studies have been conducted by Stotz and Krauth (1984, 1986) in order to investigate the flushing behaviour of sewer deposits. In the former case, the investigation involved flushing out deposits from a length of real sewer, while in the latter, effluent from a treatment works was passed through a channel in which primary sludge from a treatment works had been placed. The studies indicated that there was a marked increase in bed shear strength for deposits which remained undisturbed for at least 12 hours. Yield strength measurements of sewer sediments in the Dundee interceptor sewer have been shown to range from 10 N/m^2 up to in excess of 2500 N/m^2 (Wotherspoon and Ashley 1992). Related measurements showed that there was a strong correlation between moisture content and apparent yield stress. This was exhibited by a rapid reduction of yield strength with increasing moisture content. Since erosion of deposits frequently occurred at values of bed shear stress which were much lower than the measured values of apparent yield stress (typically at around 1 N/m^2 (Ashley 1993a)), it was considered likely to occur by the removal of large 'pieces' of the bed (WRc 1986). This may in part have been due to a change in the physical nature of the deposits immediately prior to erosion caused by an increase in moisture content (Wotherspoon and Ashley 1992). This caused a gradual liquefaction of the bed. Further discussion of limiting stress criteria are contained in Appendix B.

MEASURED PARAMETER	SEDIMENT ORIGIN / CLASS													
	DUNDEE INTERCEPTOR				DUNDEE TRUNK		UK AVERAGES							
	A/C		C - BED LOAD		A/C		A		C		D		E	
	M	R	M	R	M	R	M	CV	M	CV	M	CV	M	CV
BULK DENSITY (kg/cu.m)	1559	<2150	1070	<1450	1807	>474	1720	-	1170	-	1210	-	1460	-
TOTAL SOLIDS (%)	56	0.6 - 82.4	4.7	2.9 - 50.2	67	48 - 85	73.4	-	27	-	25.8	-	48	-
VOLATILE (%)	3.0	0.2 - 17.6	76	14.7 - 97.4	2.9	0.7 - 10.3	7	77	50	47	61	54	22	77
COD (g/kg)	16.2	0.6 - 52.2	1691	277 - 6353	2.5	<4	23	63	76	23	193	83	48	70
BOD_5(g/kg)	12.4	0.9 - 43.4	367	73 - 873	0.7	<1.6	4.2	91	20	51	103	95	13	61
Amm N (g/kg)	0.28	0.007 - 3.5	7.5	>0.1	0.03	<0.065	-	-	-	-	-	-	-	-

NOTES
M - mean, R - range, CV - coefficient of variation (%)

Chemical parameters in terms of g/kg of dry solids

Classes:
A - coarse granular material
C - mobile, fine grained - overlying Class A
D - organic wall slimes
E - deposits found in tanks

Table3 - Combined Sewer Sediments - Chemical Characteristics

(Source: Crabtree et al (35))

3.4 Dry Weather Flows

During periods of dry weather, the flow of sewage in a combined sewer is mainly due to foul inputs, with a possible contribution of groundwater from the soil surrounding the sewer pipe, which finds its way into the sewer through cracks and joints. The latter process is known as infiltration. Foul flows generally follow daily patterns which vary depending upon the location of a sewer within a network (Crabtree et al 1991) and also:

- nationally, particularly depending upon the state of development of the society;
- with the industrial/commercial enterprises connected, and the relative working practices prevalent;
- seasonally, particularly if infiltration is also considered;
- with the day of the week.

The variation of quality and flowrate of foul flows tends to be more pronounced nearer to the head of a sewer network (Pisano et al 1979). The variation in flowrates is referred to specifically by CIRIA (1987) as being dependent to some extent on the location of a sewer within the system:

- at the top end of a combined system with only a few houses served, even the dry-weather flow will consist of a series of sudden flushes as waste waters are released from households. For this reason BS8301 (British Standards Institution 1985) utilises a statistical approach to estimate typical flowrates.
- further down the system, the small peaks in the dry-weather flow are smoothed out but with typical daily variations of three to four times dry-weather flow.

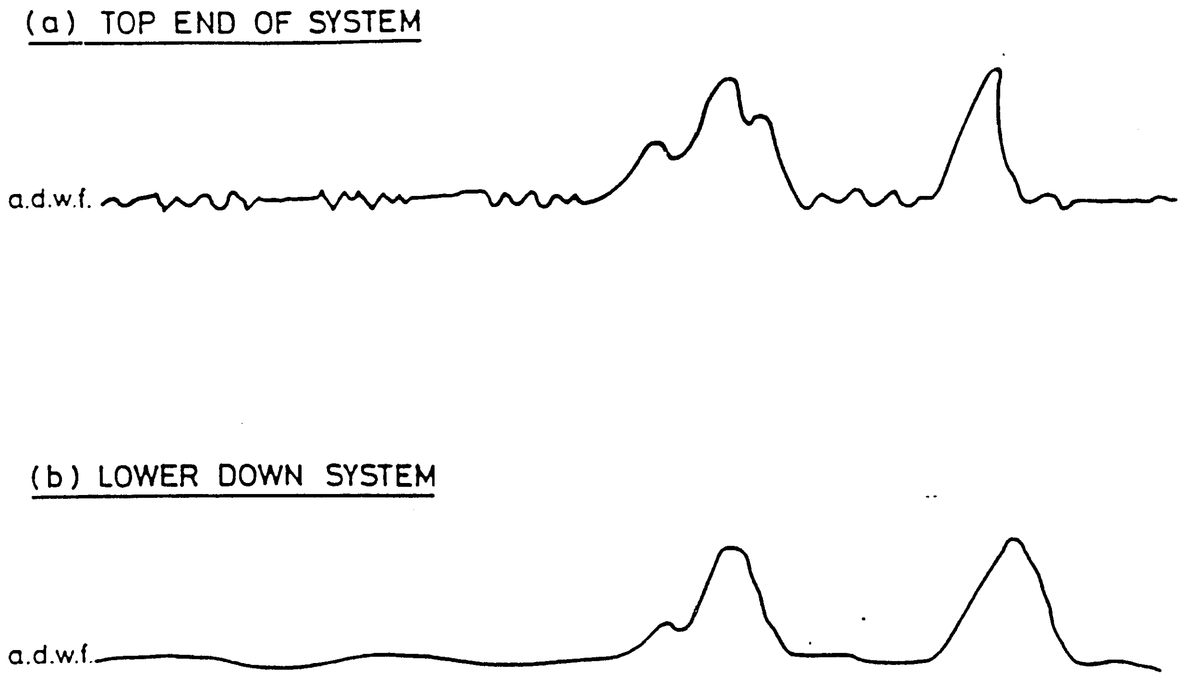


Figure 5 *Typical Dry Weather Flow Curve*

(Source: CIRIA, (1987))

The transport of solids in a combined sewer during dry weather periods is typically as illustrated in Figure 6:

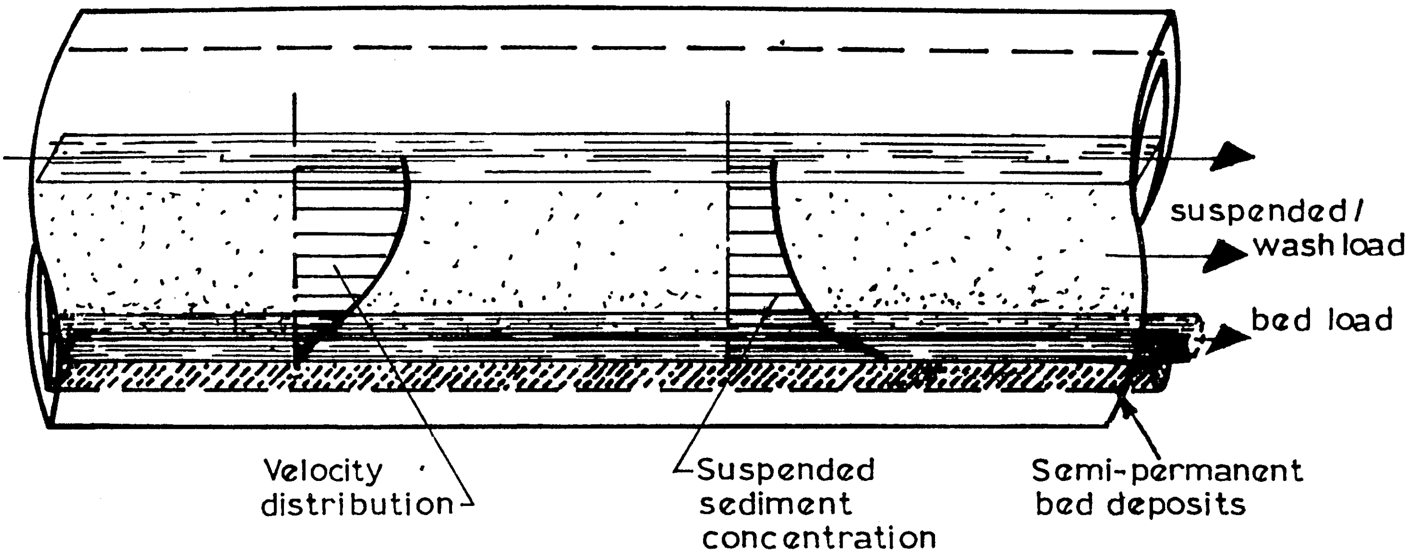


Figure 6. *Solids Transport in a Sewer (typical DWF condition)*
(Source: Ashley et al 1992a)

In a typical combined sewer, sewage flows over a quasi fixed bed deposit. Overlying this bed deposit is a moving layer of relatively concentrated solids. Above this layer, material in true suspension is conveyed in the sewage flow. This suspended material includes material which may settle out at some point downstream in suitable hydraulic conditions, plus a certain amount of material which is permanently in suspension, known as 'wash-load'. Gross solids consisting mainly of faecal matter, particles of paper, sanitary towels and tampons constitute another component of material in suspension (Jefferies 1992), although there is a tendency for these larger solids to move in the near bed layer, despite their near-neutral buoyancy (Ristenpart 1994). There is a tendency for the large organic solids to become progressively degraded during passage through a sewer system (Verbanck and Ashley 1992, Ristenpart 1994).

The bed load layer consists of a mixture of granular and organic particles which travel in a 'dense cloud' close to the surface of the more permanent bed deposits (Ashley et al 1992a). This layer does not necessarily correspond precisely to the traditional 'riverine' definition of bed load as a saltating layer of individual particles and hence alternative terms such as 'heavy fluid layer', 'dense undercurrent' and 'fluid sediment' have been proposed (Ashley et al 1993a, Verbanck 1990, Verbanck and Ashley 1992). For the purposes of differentiation from suspended load and wash load, the term "near bed material" or "material moving near to the bed" shall be adopted for the purposes of this report, apart from specific instances where more specific reference to the other terms referred to above are necessary.

Under dry-weather flow conditions, fine, mobile material tends to form a sludge-like layer on the surface of the pipe and on any fixed bed deposits, in the lee of bends, at manholes, and similar locations (CIRIA 1987). The manner in which this mobile, sludge-like layer builds up during DWF suggests that it is of fine particulate matter. This may be biological in nature, originating from flows which pass through the system during both dry weather and minor rainfall events. During major storms, this is supplanted by heavier, less organic material, as discussed in section 3.5. Concentrations of solids in this layer measured as part of the current study range from 25g/l (Laplace et al 1992) to over 87.5g/l (Ashley and Blackwood 1993). CIRIA state that there is some evidence that this mobile layer displays weak cohesive characteristics. This has been confirmed by rheological analysis of sediment samples obtained from a combined interceptor sewer as reported by Wotherspoon and Ashley (1992).

Maximum values of total suspended solids (TSS) for dry weather periods measured as a result of a study of the Hyndburn and Bolton catchments in the UK are reported to be around 200mg/l (WRc 1986). Similar measurements for mean daily TSS values in Dundee sewers range from 80mg/l in winter, to 173mg/l in summer (Ashley et al 1992a), while Laplace et al (Laplace et al 1992) report typical concentrations of 100mg/l for a trunk sewer in France. According to Verbanck and Ashley (1984), the median granulometric size of dry weather suspensions (excluding gross solids - sewer classification not specified) is in the range 0.03 - 0.04mm with median settling velocities of 0.4 - 0.5cm/s. The corresponding figure quoted by Laplace for a trunk sewer is 0.04mm. The material in suspension is highly organic with loss on ignition of 70%-80%. Densities are, as might be expected, close to unity, particularly for the larger size range of material in suspension.

It should be noted that in the above discussion, the suspended solids/wash load component of solids transport has been treated as if it were a uniform concentration of solids in suspension from the top of the near bed material layer up to the surface of the sewage. The various methods of sampling and analysis by which these figures have been produced give some measure of average TSS concentration in each case. In fact, this has been shown to be an over simplification of reality: indeed, there may be significant concentration gradients within the depth of flow (Ashley et al 1992a). Examples of sediment concentration with depth for dry weather are shown in Figure 7.

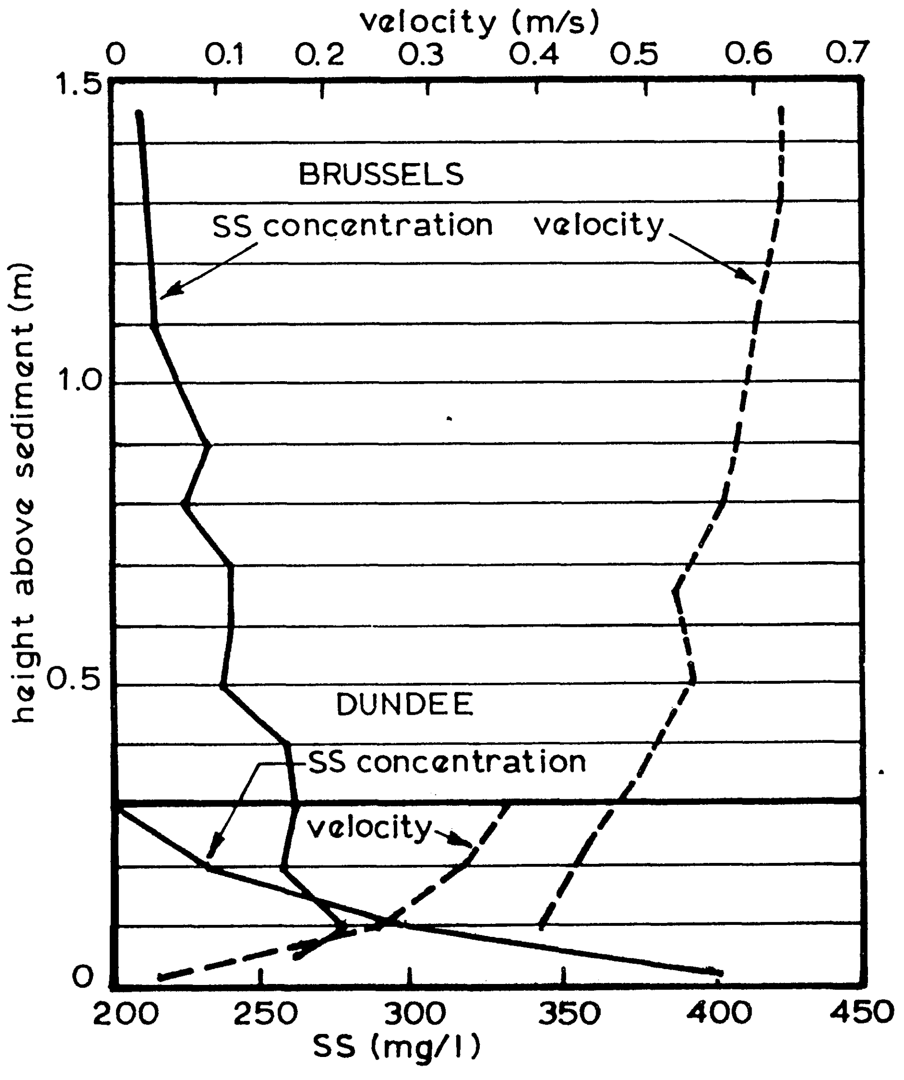


Figure 7. Suspended solids concentration variation with depth during DWF Brussels main interceptor sewer and Dundee interceptor. (Source: Ashley et al 1992a)

There are daily variations in both flowrate and in composition of sewage in combined sewers during periods of dry weather flow. These variations tend to follow regular 24 hour cycles. Typical dry weather flow and suspended solids concentrations for a trunk sewer in Brussels are shown in Figure 8 (Verbanck 1994).

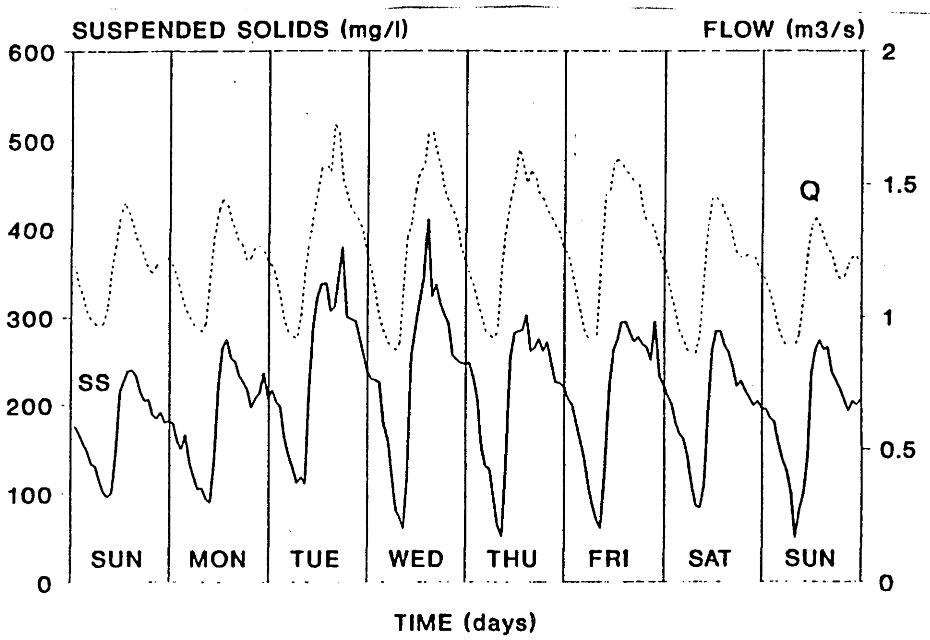


Figure 8. Dry weather flow pattern, Brussels Main Trunk, May 26 - June 2, 1985.

A feature of the daily cycle is the tendency for particles to settle out of suspension during low flows, particularly at night, followed by a resuspension of solids from material moving near to the bed or bed deposits during subsequent periods of higher flow. During night-time periods, there is thus often a build-up of sanitary solids followed by a "daily first foul flush" at the onset of daily peak flows (Verbanck 1994). This has also been reported by Geiger (1987), and has been observed in the Dundee studies (see section 6.7).

Studies of the yield strength of sewer sediments suggest that for flatter sewers (ie collectors and interceptors) during dry weather periods there is a weak layer of surficial deposits with low apparent yield strength which overlays a stronger layer of consolidating deposits (Wotherspoon and Ashley 1992). These layers have typical apparent yield strengths of <1 N/m^2 and up to thousands of N/m^2 respectively. Results of a series of artificial flushing experiments using primary sludge from a municipal wastewater treatment plant by Stotz and Krauth (1986) suggested a threshold value for erosion of consolidated sanitary solids of 1.8 N/m^2. This corresponds with observations in combined sewers in Dundee (Ashley et al 1992b) which indicated that significant deposition would only occur when shear stresses fell below 1.8 N/m^2, and no sedimentation at all when shear stresses exceeded 4 N/m^2. Since bed shear stresses during dry weather can exceed 4.8 N/m^2, this points to a mechanism of erosion which causes the daily first foul flush effect. This relationship between bed shear and the erosion/deposition of sediments at a point in the bed of a sewer has been examined by Ashley et al for sewers in Dundee during dry weather flows (see Figure 9).

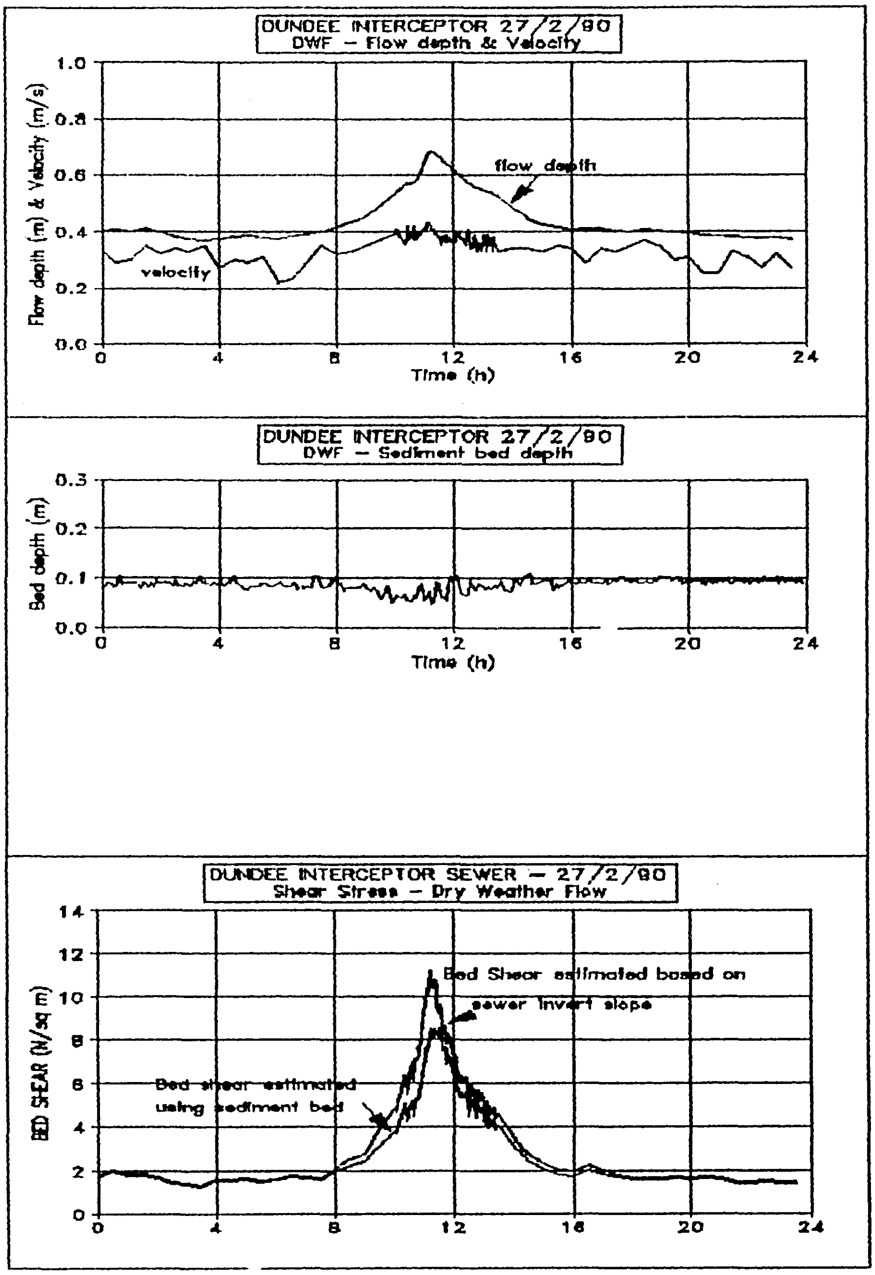


Figure 9. Dundee Interceptor Sewer - Dry Weather Flow

In addition to depth and velocity of sewage flow, and sediment bed depth, corresponding bed shear measurements were plotted. From Figure 9 it would appear that the daily peak flow, which causes the bed shear to exceed 1.8 N/m^2, is responsible for bed erosion. A subsequent reduction in shear allows sediment re-deposition.

Another consequence of the daily sedimentation/scouring cycle during dry weather periods is the opportunity for the solids deposited during the night to consolidate leading to the development of an apparent cohesive-like resistance which may increase the resistance to subsequent erosion (Verbanck et al 1994). As stated in Section 3.3, the work by Stotz and Krauth indicated that there was a marked increase in bed shear strength for deposits which remain undisturbed for at least 12 hours. This may in part explain the conclusion of Ashley et al (1993a) that erosion may or may not occur during dry weather flow peaks depending on the relative magnitude of the shear stress, the bed density, and flow history of a sediment deposit.

It has been suggested that some of the grit which enters the sewer system as wash load during storms may subsequently remain in motion as bed-load, whilst elutriation of finer material which has been deposited during the storm recession from the bed may also occur (Verbanck et al 1994). This may explain why the majority of sediments found on the inverts of some sewers are composed mainly of coarse, sandy material which will be moved only during extreme storm events (Verbanck 1994). Hence, much of the finer organic rich particles are washed out of the sediment bed due to the scouring action of dry weather peak flows. Another consequence of the presence of mobile layers of grit and organic material is the ready availability of material to settle out and form new deposits when hydraulic conditions change either temporally or spatially. A feature of this phenomenon is the tendency for near bed material to supply material which fills out discontinuities in the bed (Verbanck and Ashley 1992). Given sufficient length of dry weather flow period, the bed deposits in combined sewers could be expected to form a uniform gradient along the length of the sewer as exemplified by the sediment bed profile shown in Figure 10.

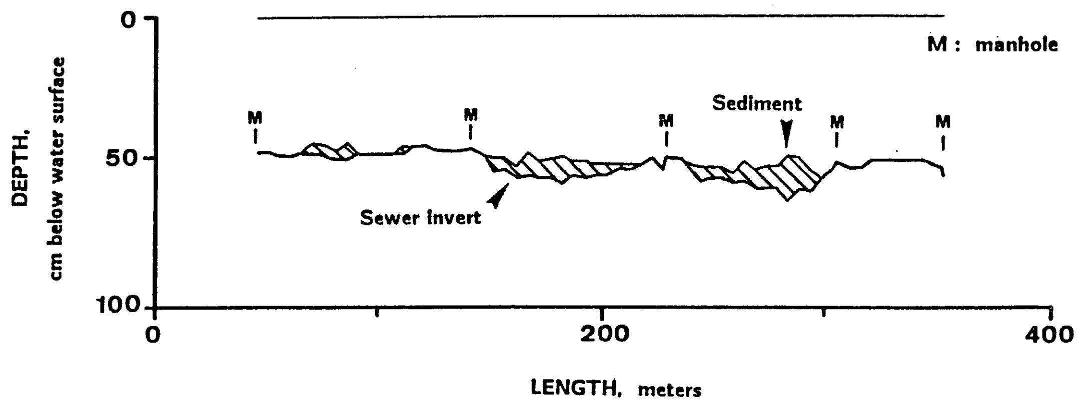


Figure 10. Typical Accumulation of Grits in a Sagged Sewer Profile

(Source: Verbanck et al 1994)

Although the daily pattern of dry weather flow includes periods of both erosion and deposition, these are not necessarily equal in terms of the quantity of material removed or deposited in any 24 hour period. Particularly for flatter gradient sewers, there may be a build up of sediment deposits over long periods of time (Ashley and Crabtree 1992).

According to CIRIA (1987) the process of building up of deposits in combined sewers depends on the shear stress conditions during dry-weather flow. Ashley and Crabtree (1992) note that in some sewers there is a declining rate of build up with time. A number of studies have shown evidence of a gradual decrease in deposition rates with length of dry weather period until an apparent equilibrium condition is reached typically within 12 to 20 days (Verbanck et al 1994). This may be the case for collector sewers draining small areas, although some studies have shown a similar effect for trunk sewers draining large areas (Laplace et al 1990).

3.5 Storm Flows

In combined sewers, flow originating as runoff on the catchment surfaces augments the foul flows associated with dry weather periods, resulting in an unpredictable and highly variable flow rate in the sewer due to the random nature of rainfall patterns.

During such storm flows, the concentration of virtually all particle sizes increases (Crabtree et al 1991). Most countries appear to experience a similar range of concentrations of suspended solids in combined sewer flows, of up to 720mg/l according to CIRIA (1987). For Dundee sewers, values of just under 2000mg/l have been recorded, which closely matches the figure of 2200mg/l more recently quoted for the UK as a whole (Ashley 1993b).

It is difficult to give a generalised description of the movement of solids during stormwater flows since this is dependent upon the duration and severity of a storm event and the point in time being considered, as well as site-specific characteristics such as the nature of the soil and local cleaning practices for urban surfaces.

The solids in suspension are a complex mixture of sediments from various origins on the catchment surface and within the sewer system.

Surface sediments tend to build up over time during dry weather providing they are not removed by street cleaning procedures. It has been suggested that the rate of build-up is non-linear (Sartor et al 1974), but subsequent studies do not necessarily support this hypothesis (Ashley and Crabtree 1992). The pollutants build up on roads, roofs and paved areas due to traffic, atmospheric effects, and various land use activities. As well as this gradual build up, an important source of surface sediments in winter in the UK is due to de-icing operations (Ashley and Crabtree 1992). The finest particles and dissolved pollutants are washed off at the start of the storm. There is evidence of a bimodal distribution of particle sizes, all below 250 μm . Gully pots, which connect road surfaces to drainage networks, are not effective at retaining this fine material which enters the sewer in two distinct peaks, the first at around 2 μm and the second (coarser) peak at around 20 μm . Separate studies of roof surface wash-in to sewer systems have shown significant quantities of total solids which can comprise up to 30% of total load in combined flow.

The settlement characteristics of solids originating from catchment surfaces has not been widely investigated. An estimate of the apparent specific gravity of particles in storm run-off has been ascribed an average value of 2.0 for the purposes of studies in combined sewer overflow performance assessment (Klemetson 1985). More recently, corresponding figures in the range 2.10 - 2.51 have been measured during field studies in London (Butler et al 1992).

The in-sewer processes which contribute to the resultant temporal variations in solids transport during storms include mixing of runoff from surface sources with foul flows. However, there is the added complication that the changing hydraulic conditions during the passage of storm flows also affects the processes of erosion and deposition of bed deposits within the system. The modes of transport of solids in the flow may also change during a storm, including the transfer of material moving near to the bed into suspension. The amount of sediment eroded from the bed to become part of the suspended solids load will depend on flow conditions and the resistance of the deposit to erosion and resuspension (Crabtree et al 1991). The potential release of pollutants from an eroding bed-load layer by the onset of storm flows has been studied by laboratory bed-load sampling procedures in Dundee in an attempt to distinguish what proportion of the pollutants are associated with different 'phases' of the sample. This has resulted in a proposed pollutant release mechanism as shown in Figure 11.

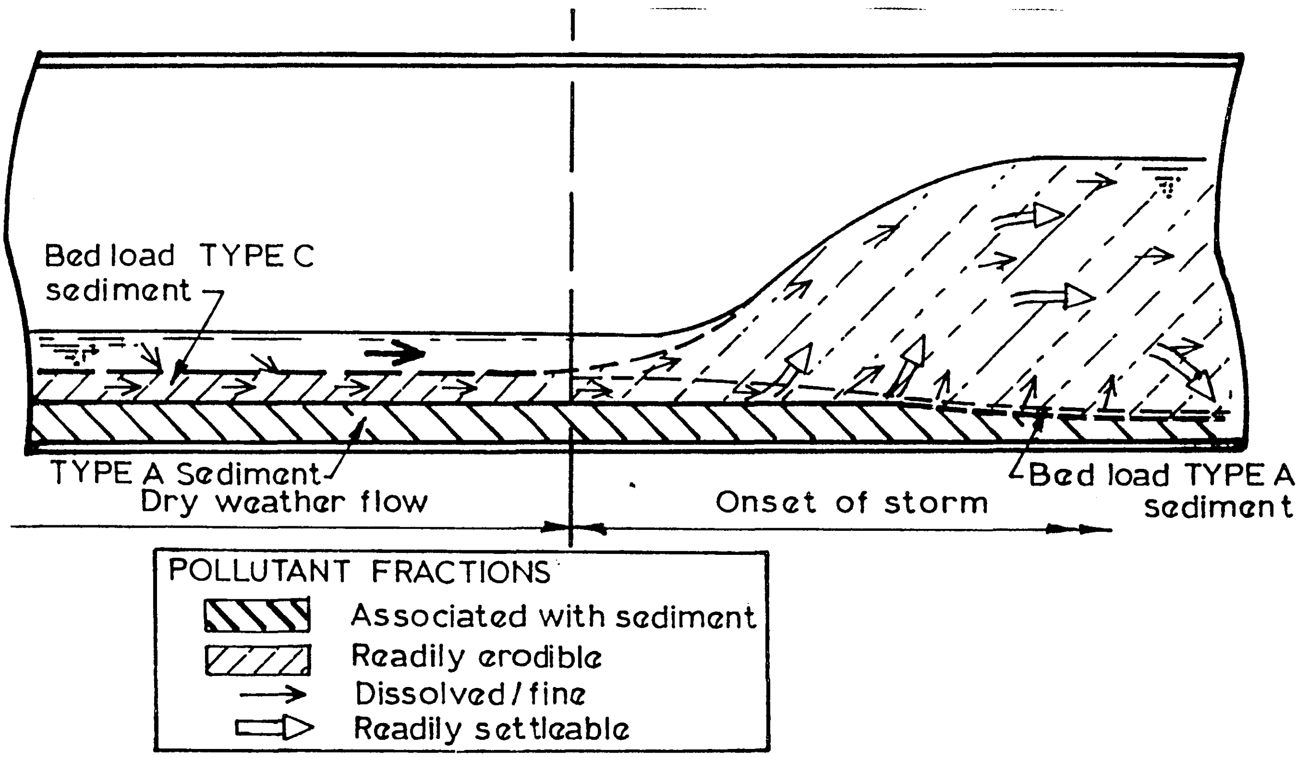


Figure 11.The release of sediments and associated pollutant load fractions by erosion - interceptor sewer.

(Source: Ashley et al 1992a)

Due to the entrainment of bed material into suspension during storm flows, a change in the nature of suspended solids is commonly observed during storms (CIRIA 1987, Verbanck 1994, Bertrand-Krajewski et al 1993). Also, the quantity of pollutants transported normally increases rapidly at the start of the storm. It should be noted that the elements of these changes are not necessarily universal in occurrence or nature. A typical picture of the sequence of events contributing to these changes is being built up as a result of a number of studies. First, there appears to be a short-lived increase in bed deposits due to deposition of solids eroded from the upstream sewers (Ashley 1993b). This is associated with the arrival of a sharp peak in suspended solids during the rising storm hydrograph, known as the first foul flush, caused by erosion of mainly near-bed material and possibly bed material in the upstream sewer due to increased shear stresses (CIRIA 1987, Ristenpart 1994). Other parts of the system which contain deposits which may be disturbed due to turbulence, such as gullies and sumps, may contribute to this peak. It is probable that there is also a simultaneous flux of surface sediments originating from roof surfaces which contributes directly to the first flush of resuspended in-sewer deposits (Crabtree et al 1991). The first foul flush may account for 50% of the solids transported during a storm in only 30% of the volume of flow (Bertrand-Krajewski et al 1993). Other issues relating to FFF are discussed in Section 3.6.1.

As the storm continues, the TSS concentrations may drop before rising to a secondary peak which coincides with peak flow. At this point, increased shear stresses associated with higher flows have started to erode the more permanent deposits, which are more resistant to erosion. These, plus the surface solids washed into the system due to surface transport, contribute to a change in the characteristics of the solids in suspension. These comprise of particles mainly of a larger size and density than are found in suspension during DWF or FFF (Crabtree et al 1991). The amount of this material moved and how far it is transported thus depend on a number of factors (CIRIA 1987) including:

- the extent of sources both outside and within the system
- the severity and duration of the storm
- the size, grading and specific gravity of the material
- the apparent cohesiveness and the degree of concretion or

agglutination of the deposits in the sewer

- the location of the material within the system and the physical characteristics of the system.

A number of the above factors may be influenced by the Antecedent Dry Weather Period (ADWP).

Following erosion, the bed often becomes reinstated to the level prior to the storm (Ashley et al 1993c). There is usually erosion of the bed deposits during significant storm events, but this does not necessarily entail the entire removal of the bed deposits.

Because of the entrainment of near bed material into suspension due to high degrees of turbulence, weaker differences in the vertical concentration gradient of solids have been observed during storms. This is in marked contrast with conditions pertaining to DWF. This pseudohomogeneous flow with relatively uniform solids concentration has been noted by a number of workers (Chebbo et al 1990, Ashley et al 1992a, Ristenpart 1994).

3.6 Complicating Factors Affecting the Modelling of Solids Transport in Sewers

In the preceding sections of Chapter 3, various aspects of flow and solids transport in combined sewers are discussed. There are a number of factors in addition which add to the difficulty in predicting rates of solids transport during different flow regimes. Section 3.6.1. discusses these factors while Section 3.6.2. considers the implications that these factors have with respect to the modelling of solids transport in combined sewers.

3.6.1 Additional Complicating Factors

Generally, the rate at which surface particulates are transported to the points at which they enter a combined sewer system are highly varied both spatially and temporally. The spatial variability is due particularly to variations in land use, while temporal variability is mainly associated with physical and meteorological parameters. Crabtree et al identified the following factors as being of particular influence on wash off characteristics from impermeable surfaces:

(i) Nature of surface - smooth surfaces give the greatest "yield".

(ii) Rainfall intensity and volume - particularly the former.

(iii) Particle size - lighter, organic particulates are washed off early in a storm.

(iv) Street cleaning - this only appears to be effective for removing the larger particulates and should be undertaken at a frequency commensurate with the average inter-storm period.

(v) Inter-storm dry period - if street cleaning is irregular or non-existent antecedent dry periods appear to be of most significance for dissolved, rather than suspended pollutants.

(vi) Traffic density - responsible for re-suspension and pollutant "losses", particularly during small storms when depression storage volumes are not exceeded.

(vii) A minimum depth of rainfall is required to initiate pollutant run-off, estimated at 0.35 to 1.5mm. Lower volumes on a rough surface allow particle aggregation and surface binding to occur.

Sediments arising from overland flow are deposited in the various appurtenances such as gullies at the surface-water entry pipes (CIRIA 1987). These sediments are subjected to considerable turbulence due to the configuration of the appurtenances. The finer material and lighter organics contained therein are thus likely to be easily disturbed and entrained, providing a significant contribution to the First Foul Flush. Larger material is carried into the system at higher flow thresholds. The effect of gullies in providing a store of readily resuspended sediment adds substantial unpredictability to sediment and pollutant loads in the system.

A major seasonal factor which affects the amount and type of material washed into sewer systems during rainfall events is the practice of winter de-icing of roads. Not all roads are treated and the materials used vary. The CIRIA (1987) study indicated that major highway routes and some 10% of minor roads are regularly treated during cold weather periods. The materials used vary, and may include rock salt, closely graded grit (1mm - 3mm), ethylene glycol and urea (Crabtree et al 1991). Where they are applied, the spreading of de-icing agents which contain insoluble solids adds further to the sediments and pollutant loads in sewer systems. In the case of rock salt, this may contain typically between 5 and 10% insoluble solids which appear in storm runoff some time after the washoff by rainfall of the soluble solids. It has been surmised that some 25-30% of annual solids loads could be attributed to the application of rock salt.

Summarising the movement of solids by overland flows, it can be said that the variability of various land types, land use and seasonal factors contribute to a highly complex series of mechanisms. Coupled with the random and non-uniform distribution of rainfall over a catchment both temporally and spatially during storms, these factors combine to make prediction of surface-generated solids input to sewers very difficult.

As previously indicated, foul flows in combined sewers tend to follow daily patterns which vary depending on the location of a particular sewer within a network. Crabtree et al (1991) have also identified the following factors which influence the type of daily flow pattern in each instance:

- National variations, particularly depending upon the state of development of the society;
- Variations due to industrial/commercial enterprises connected, and the relative working practices prevalent;
- Seasonal variations, particularly if infiltration is also considered;
- Variations with the day of the week.

Most reported studies of dry weather flows do not consider temporal factors other than time of day (Ashley and Crabtree 1992), let alone the influence of the other factors previously listed. In addition, the distance which foul flows travel through a sewer system to reach a particular sewer may reduce the relative proportion of large solids transported due to degradation in transport (Ashley 1993b).

Physical characteristics of the sewer network may have a profound influence on solids transport, and on the deposits found. A gradation of particle sizes, with different size fractions predominant in collector sewers, trunk sewers and interceptor sewers has been observed through studies in France and in Dundee (Crabtree et al 1991). Deposits in trunk sewers tend to be coarser than those found in collectors and interceptors, possibly due to the relatively steep gradients of these sewers. The cross sectional shape of the sewer itself also has a significant effect on the transport of solids (Verbanck et al 1992, Ashley 1993b).

In this context the configuration of sewer ancillaries is particularly significant, since the complex flow patterns around these features affect concentrations of sediment and how these are distributed in particular locations (CIRIA 1987). The material moving near to the bed, which is particularly important for filling discontinuities in the bed and smoothing the hydraulic boundary can be easily inhibited by relatively small obstacles (Verbanck and Ashley 1992), causing localised deficiencies of sediment available for deposition. The overall shape of the catchment has been cited by Thornton and Saul (1987) as another physical factor influencing the temporal variation of transport rates in both dry weather and storm flows since this will affect the flow patterns in the sewer system.

There appears to be some correlation between the length of antecedent dry weather period (ADWP) and the TSS concentration of the first foul flush (FFF) (Pearson et al 1986, WRc 1986), although there may be a time limit on this effect, depending on the nature of dry weather flows within the system. According to Stotz and Krauth (1986), the ADWP may be more important than runoff rates in some cases for determining likely concentrations of solids in the FFF. This dependency appears to be most marked for sewers with flatter gradients. The factors influencing the occurrence and nature of these flushes are not yet fully understood (Ashley et al 1992a), but according to Stotz and Krauth, they are less likely to occur for catchments with greater than 100ha of impermeable contributing catchment.

It has been shown by field measurements in sewers in Dundee (Ashley et al 1993c) that the onset of erosion may occur at different initial rates of shear stress depending on the history of sediment deposition, the length of antecedent dry weather period (ADWP), and the rate of change of shear stress. This work indicated that the greater rapidity at which bed shear stress rose, the greater the level of shear stress at which erosion was initiated. This may in part be explained by the fact that apparent yield strength of sediment is strongly correlated with moisture content, as has been demonstrated by Wotherspoon and Ashley (1992). Measured yield strengths of most sediment samples from combined sewers are far in excess of possible applied hydraulic bed shear stress, but a change in solid/liquid phase proportions at the surface of the sediment deposit may occur as turbulence increases due to increased bed shear stresses, leading to a sudden failure of the material to resist erosion. This condition would have more time to develop where the rate of increase of shear stress was less rapid. The relationship between moisture content and strength also implies that the strength of deposits of a higher density will be greater and therefore bed density increase with depth will be significant. Wotherspoon and Ashley also report that the structure of a sediment can very quickly recover significant rigidity following disruption. A further complication arises due to the fact that erosion of the surficial deposits (ie type C material including near bed material) has been observed in Dundee for boundary (fluid) shear stresses of only 1.8 N/m^2 (Ashley et al 1993c). This was previously discussed in Section 3.5 in the context of dry weather flows.

The onset of erosion of the sediment bed during storms and peak dry weather flows is of great significance to the quality of material transported in sewage flows. How fluid-induced shear stresses can be related to the resistance of the sediment bed to erosion is a question still to be answered. Also, there are significant variations in bed shear stresses with relatively small changes in hydraulic conditions, while the erosive potential of a flow field is drastically altered by changes in the concentration of solids already in suspension (Wotherspoon and Ashley 1992). Thus, apparently identical hydraulic conditions can result in very different rates of erosion.

Foul flushes are not observed in all sewers and are not always observed in the same sewer for different events (Ashley et al 1992a). Although the factors influencing the occurrence and nature of these flushes is not clearly understood, some general conclusions may be made (Ashley et al 1992b).

1) First flushes may be related to the size of the contributing catchment - flushes are less likely to occur for catchments which exceed 100ha of impermeable area.

2) There is evidence that the effect of antecedent dry period is important with regard to the laying down of a "store" of readily erodible sediments, and hence would be particularly important for systems, or parts of systems, where a significant rate of dry weather solids deposition occurred. Any rainfall would be expected to mobilise different amounts of the easily erodible deposits in different parts of a sewer network, and depending on the disposition of the solids, spatial variation of the rainfall and relative size of the system, the varying effects of detention and superposition of pollutographs would then result in different

observations of pollutant flushes both spatially and temporally.

The length of antecedent dry weather period appears to have two important effects. In addition to increasing the amount of material deposited, and hence available for entrainment during a subsequent storm, it also allows an increase in strength of sediments already deposited. Results of a study in Germany showed that during dry weather, erosion was initiated at an average bed shear of 0.7 N/m^2 whereas during wet weather this increased to an average of 2.27 N/m^2 (Ashley 1993a). A prolonged dry weather period of three weeks increased this figure from less than 2 N/m^2 to 3.3 N/m^2, presumably due to consolidation of the bed during the prolonged dry period. This may in part explain the apparent maximum length of ADWP which is reported as significant (see section 3.5).

A change in the characteristics of the material conveyed at different times during a storm event in combined sewers is reported by a number of authors (Bertrand-Krajewski et al 1993, Verbanck and Ashley 1992, Crabtree et al 1991). Laboratory studies using mechanical mixing appear to corroborate this observation (Crabtree et al 1991). Field data currently available on this subject are limited (Verbanck and Ashley 1992), but some tentative conclusions have been reached. After the initial first foul flush of fine particulates, a second wave of slightly coarser particles has been identified. Crabtree et al (1991) reported that a first modal peak of around 2 microns occurs during the first flush of in-pipe deposits, while the second modal peak of around 20 microns is attributed to the wash-in of surface particulates. Analyses of these particles show that agglomeration and cementation occur as particles are transported along the sewer, with some of the separate particles collecting into larger agglomerates along the length of a sewer. There is also a corresponding tendency for larger organics to be broken down. In storm sewers the cementation process may be caused by the precipitation of silica from degradation of organic particles and "weathering" and abrasion of quartz grains.

Very little is at present known about the bed-load layer in sewerage systems, however it has been hypothesised that during wet weather flows, bed deposits which may include overlying bed-load material are resuspended into the sewage flow (Crabtree et al 1991). The quantities of material transported as bed-load may therefore have a significant effect on the first foul flush. This material may be classed as being transitional between the suspended sewage particles and the bed deposits; broadly similar to class C sediments. This is illustrated by the fact that the Dundee Interceptor bed-load material has a very low average solids content (4.7%) compared with the class C material (27%), and average bulk densities of 1070 kg/m^3 and 1170 kg/m^3 respectively. In comparison to sewage flows in the same sewer, the average solids concentration is 23.4 g/l for bed-load material compared with values of less than 2 g/l for sewage.

Considerable work remains to be done if the occurrence, nature and polluting potential of sewer sediments are to be properly understood (Crabtree et al 1991). This work should particularly relate to the following:

- spatial differences in the nature of deposits found in collector, trunk and interceptor sewers;
- the bacteriological aspects of sewer sediments, and their influence on sediment physical and chemical characteristics;
- the release of pollutants in response to increasing shear stresses, and a fuller

explanation of the causes of foul flushes;

- temporal changes in bed shear strength and other polluting characteristics as a result of consolidation, biodegradation and prolonged dry periods.

3.6.2 The Implications for Solids Transport Modelling

There is a paucity of good quality, reliable data regarding the particle size and settling characteristics of sewage particulates (Crabtree et al 1991). This is important in the context of the modelling of solids transport in sewers. The effects of particle flocculation and agglomeration and the problems associated with taking representative samples for subsequent laboratory testing mean the data available are not necessarily reliable. This is compounded by the fact that noticeable gradients of suspended sediment concentration with depth mean that small volumes sampled from sewers during dry weather periods and storms must be considered as not necessarily representative of the "average" solids concentration (Ashley et al 1992a). Also, the non-conservative nature of sewage particulates and the dependence of settling characteristics on the surrounding suspension may have significant effects on laboratory or even in-situ tests (Ashley and Crabtree 1992). Due to the nature and the difficulties of measurement using standard techniques, the apparent particle size may not, therefore, be an appropriate determinant (Verbanck and Ashley 1992) for modelling purposes. Settling velocity information may be more reliable, although there are doubts about current techniques used to measure this. For these reasons, determination of the precise settling characteristics which truly represent the response of the solids in suspension is probably not possible (Crabtree et al 1991). The determination of such values may not be necessary however, since the development of analytical or simulation models of the processes may be developed, calibrated and verified using agreed test methodologies which are accepted as being representative standards.

The use of laboratory experiments in order to simulate the complex reality documented elsewhere by field observation should be done with great care (Verbanck 1994). Data collected in the Brussels Main Trunk sewer suggest that the effects of sewer deposits erosion on the quantities of solids transported cannot be completely assessed using a synthetic material which mimics the rheological properties of the deposits generally found on the bed of the sewer. To illustrate this point, Verbanck cites an example from field work where the daily deficit of particulate material in the catchment under consideration (a very significant 15 tonnes per day) would represent a layer of less than 0.1mm depth if the entire amount settled equally over the total length of the sewers in the system. For this reason, work in Brussels has been focused on experiments in real sewers. It is, however, worthwhile using a model based on data from the more controllable environment of the laboratory as a starting point since such models are likely to identify significant determinants which should be considered (WRc 1986, Hare 1988). In this context, the work previously carried out in the river engineering field may be of value, but does not necessarily account for the hydraulic conditions or material transported in sewers (Verbanck et al 1994). In an attempt to address these problems, experiments carried out in pipe channels using a synthetic cohesive sediment (Nalluri and Alvarez-Hernandez 1992) may help to bridge the gap between experimental work in the laboratory, and field measurement in sewers. This work in part fulfils the objectives set out by the Geotechnical Consulting Group (WRc 1986) in their report.

One of the conclusions arrived at by the Geotechnical Consulting Group was that the most productive approach to the improvement of current predictive techniques was likely to comprise two parallel programmes of work: the first (and most important) an empirical approach based on the measurement of actual performance of a sewer system, the second, laboratory studies of the mechanisms of accumulation of cohesive pipe deposits and their re-entrainment. It should be recognised, however, that the results of a laboratory study are unlikely to be usable directly to quantify the field condition: their value would be in helping to identify the significant parameters in the accumulation of deposits, and hence empirically relating one sewer system to another.

Further complications to consider include those set out by May (1982) in the section listing special factors to be considered with respect to modelling of the behaviour of sediment in pipes, as opposed to modelling of solids transport in alluvial channels:

(i) The mode of transport in a pipe is not uniquely determined by the rate of transport. In an alluvial channel the supply of material is effectively unlimited so that the transporting power of the flow determines both the rate and the mode of transport. In a pipe the rate of transport may be fixed independently by the rate of supply of material. Thus the same rate of transport can result from a high velocity flow carrying the material in suspension, or from a lower velocity flow carrying the material as bed-load.

(ii) The nature of the boundary over which the sediment moves changes with the mode of transport. At low rates of transport the bed-load particles move over the smooth surface of the pipe; at higher rates, when deposits form, the particles move over a stationary surface formed by other particles (as in an alluvial channel).

(iii) The width of the sediment moving along the base of the pipe does not remain constant, but increases as the rate of transport increases.

(iv) The head loss gradient along the pipe is determined by the composite roughness of the smooth pipe and the deposited sediment, with the proportions of the two surfaces varying according to the rate and mode of transport of the sediment.

(v) The presence of a sediment deposit has a more direct effect on the flow conditions in a pipe than on those in an alluvial channel. If the pipe flows full, the deposit reduces the area of the flow and thereby increases its velocity; in an alluvial channel the presence of the free surface prevents such a direct effect. However, in both cases, the bed can exert a more direct influence via the resistance which it provides to the flow.

In addition to the above points listed by May, observations have shown that in real sewers, deposition can occur during periods of high shear stress (Ashley et al 1993c), and indeed both sediment deposition and erosion can occur simultaneously within a particular sewer system at any one time (Ashley et al 1992a).

Thornton and Saul (1986) suggest that the relative dry weather flow concentrations of solids in a sewer have a significant effect on the likelihood of occurrence of a first foul flush. They also suggest that time of day at which the storm occurs will have an effect for similar reasons. It is equally possible that the reported variation between different sites in the occurrence of foul flushes may be largely attributable to differences in the sewer networks studied (Ashley et al 1992a). Several studies have shown that foul flushes may be predicted for small sized collector sewers using empirical relationships, while for larger sewers, other approaches may be appropriate (Ashley et al 1992b).

For the above reasons, it is unlikely that a "universal" sewer solids transport model such as those applicable to river and non-cohesive modelling problems is going to be achievable for combined sewer systems (Verbanck et al 1994). Empirical, deterministic and stochastic models have all been proposed for different aspects of interest, and it is still not clear how generally applicable the models eventually developed for the various processes will be (Crabtree et al 1991). The limited scale of site-specific calibration and verification required for models of the hydrologic and hydraulic processes may not be adequate for reliable sewer solids transport models.

The recommendations for further work which could be undertaken to solve the problems of solids transport modelling in combined sewers, made by the Geotechnical Consulting Group (WRc 1986), are that although it would be useful to improve the understanding of the mechanisms of sediment accumulation and re-suspension or movement, most benefit would be obtained by direct measurement of the performance of actual sewer systems. Since there may be practical reasons why the undertaking of extensive measurements may not be possible, a compromise may be reached by identifying some critical parameters indicative of the system performance which may be measured easily. In this context, Crabtree et al (1991) state that precise measurements truly representative of the response of solids in suspension are probably not possible. Such determinations may not be necessary for the development of models of these processes, provided the models are developed, calibrated and verified using agreed test methodologies which are accepted as being representative standards.

3.7 Conclusions

The complexities of modelling of solids transport in sewers discussed in this chapter mean that the sediment transport models available from the literature are either of limited validity when applied to sewers, or require modification in order to improve their applicability. The performance of such models once modified for sewer transport applications requires assessment using data from real sewers. If it is found that none of these models fulfil the required role adequately, alternative approaches to the modelling problem are required.

A modelling methodology has been developed in the present study which may be used to select existing models for modification for in-sewer applications, and to develop the required modifications. The methodology also includes the assembly of empirical models for comparison with the modified existing models. From the various models, the most suitable in terms of accuracy of prediction of solids transport rates in sewers can then be selected. This matter is considered in Chapter 4.

Data from actual sewers has been obtained, and criteria by which the performance of the models may be assessed have been developed. The methodology used for this work is discussed in Chapter 4, and the work involved in collecting the data for testing is described in Chapter 5. The development and utilisation of the methodology is detailed in Chapters 7 and 8.

4 MODEL DEVELOPMENT STRATEGY

4.1 Introduction

The strategy which was used to develop a model of solids transport for a combined sewer is discussed in this chapter. The aim of the modelling work was the prediction of solids concentration versus time, ie a "sedograph". The reasons for deciding upon this form of model output are discussed in more detail later in this chapter.

Central to this strategy was the development of a methodology by which the modelling objectives could be achieved. The methodology proposed in this chapter was used to construct site-specific models using data from two study sites in the Dundee sewer system. It was further tested by application to a combination of data from the two sites to successfully construct a non-site-specific model. This model was configured in such a way that it could be used for general application in the prediction of suspended solids concentrations for combined sewers at other sites.

By showing that this methodology worked when applied to the data available from the two study sites examined in this thesis, the potential of this approach was demonstrated. The methodology developed is therefore suitable for the construction of site-specific models for other sewers, or for the improvement of the non-site-specific model by the utilisation of a larger data base from more sites than used in the present study.

4.2 Methodology

From the discussion in Chapter 3 it is clear that data from field studies in combined sewers such as the Dundee sewer system could give some insight into the way such sewers perform under various conditions with regard to suspended solids transport. One of the objectives of the work was to use data from the field studies in a combined sewer system as a means by which the accuracy of prediction of selected currently available models of solids transport in sewers could be assessed, and possibly improved upon. This improvement was achieved by a calibration procedure which increased the goodness of fit of the model output to the data as discussed later in this section.

The majority of the models available for prediction of solids transport rates require information regarding average particle size and grading, specific gravity and settling velocities. This comes about due to the origin of most of the available solids transport models as the results of flume or river based studies where steady flow conditions predominate. Notwithstanding the difficulties in measuring these determinants, there is still the difficulty that they are temporally and spatially variable, changing with changing flow conditions in a sewer. When modelling the highly variable conditions found in sewers this adds to the uncertainties of achieving an accurate prediction (Ackers 1984). Clearly, if data on particle characteristics were to be used as part of the modelling procedure, these would have to be measured for every sewage sample obtained. This information is difficult to obtain for sewers, and for the field studies conducted as part of this work it was not considered practical to obtain such measurements. A model which can predict solids transport rates based on parameters related to site conditions which may influence particle characteristics, but which does not require information on sediment particle characteristics would be a useful way of avoiding this problem. This is the approach taken in the work considered here. It was considered essential to start with the available models based on river and flume data in the development of a sewer solids transport model since these models are likely to identify significant parameters which should be taken into account (WRc 1986, Hare 1988).

The approach that has been adopted for this study has therefore been to select from the models which are available those which are most suitable in terms of their original intended purpose, and for which the number of "unknown" variables are not too great. They should be models which have been shown to have some degree of accuracy in their use elsewhere. In addition, only models of solids transport over deposited beds were considered, as opposed to models of the limit of deposition, since these generally perform better (CIRIA 1994) and are in any case more appropriate to the site conditions observed in the field studies than models of limiting conditions.

The selected models were calibrated to achieve a best fit to the data from the study described in this thesis by calibration of the unknown variables. By this means, the best of the currently available models was chosen as one of the possible contenders for the ultimate selection of the "best" model for each particular application.

Other options which were explored included simple rating curves based on regression analyses, and also more complex regression formulae which were developed by a logical series of steps. The goodness of fit of these options were then assessed in the same way as for existing models.

The purpose of utilising these various modelling processes was to find the "best" model for each particular application. This must be a model which can be used within the constraints of the available data. Of those short listed, the obvious criterion for selection was performance in terms of accuracy of prediction. Where, however, there are a number of models with similar performance, the "best" model, according to Hemain, is the one which is simplest in terms of data requirements and ease of use (Hemain 1986). This criterion was therefore used as the ultimate deciding factor where more than one model performed to a similar standard.

Due to the variability of flows and concentrations of suspended solids in combined sewers, particularly during storm flows, no attempt was made in the modelling process to directly relate data on a temporal basis. Although it was possible in some cases to identify periods of rising or falling stage, each data point could only be deemed to represent the conditions prevalent at an instant in time. According to Stotz and Krauth (1986) the maximum desirable sampling interval during first foul flushes is of the order of 15 seconds. This was impractical given the constraints of equipment and resources available. It was possible, however, to relate the time at which a storm sample was obtained to the time at which the storm commenced. Hence the Time Since Start of Storm (TSSS) is one variable which was considered in the regression equations developed for storm data. Similarly, it was possible to assess the length of Antecedent Dry Weather Period (ADWP) prior to the start of a storm.

Samples of sewage were normally taken at only one depth in the flow, and it was assumed that these samples contained concentrations of solids which were representative of the average suspended solids concentrations (see Section 6.2). Therefore for both storm and dry weather flows, uniform concentration of flows with depth was by implication assumed, and all concentrations measured and predicted were construed to be average values. This approach was also taken by Moys and Henderson (1987) in the discussion of the proposed set up of MOSQITO, and agrees with the findings of Chebbo et al (1990) who noted for samples obtained just downstream of a junction of trunk sewers in Marseilles that the vertical partition of solids concentration was "relatively homogeneous". Due to the entrainment of bed material into suspension during storm flows, a change in the nature of suspended solids is commonly observed during storms (CIRIA 1987, Verbanck 1990, Bertrand-Krajewski et al 1993, Crabtree et al 1991). The calibrations considered later therefore were carried out separately for storm and dry weather flows.

Once suitable models had been selected as the best option for each particular application following the calibration procedure, validation of the models was carried out using separate data sets allocated to this purpose (see Chapters 7 and 8).

4.3 Consideration of Determinants to be Included in Modelling Procedures

There are a great many factors which could possibly influence the rates of solids transport in a combined sewer. Where these determinants can be measured, it is possible to assess the correlation between each determinant and the solids transport rates provided there are sufficient data on the corresponding variations between the two. Hence, a "league table" of ascending order of degrees of influence of determinants can be constructed. The assessment is based on values of r^2 (the correlation coefficient) from regression analysis, since r^2 is a statistical parameter which indicates the relative degree to which a correlation between two variables (ie the fit to a straight line when plotted on a graph) is likely if a relationship is anticipated.

Not all of the possible factors may have been measured for any particular study, and even where values are known, the variation in an individual determinant may be small or non-existent. Also, a number of simultaneous values of a particular determinant at different points in a system may mean that it is not possible to attribute a proportion of the cause and effect to particular parts of the system. In all of these cases, the determinant in question must be excluded from the study. The remaining determinants may give sufficient information about transport rates to achieve acceptable performance of the model. As discussed previously, the Geotechnical Consulting Group (WRc 1986) concluded that, where the measurement of all parameters is not practical, the measurement of key parameters may achieve a reasonable compromise.

The procedures used in field measurements to collect data for modelling purposes were arrived at through a consideration of what was possible given the practical constraints of field work. The determinants which could possibly be included in a solids transport model are listed in Table 4. In each case, the table indicates whether an individual determinant was measured, and if so, whether any variation in the data was observed. If no variation was observed in the particular determinant, or if the variation observed was not attributable to a particular part of the system, then the data for the determinant could not be used for modelling purposes. Similarly, if the data were very limited, the determinant was excluded. Therefore, for all site-specific modelling applications, only data with a tick in the "Measured" and "Variation Observed" columns and with no ticks in any of the other columns were used.

In addition to the above considerations, factors which were attributable, but which only showed limited variation between the sites on the Dundee combined sewer system for which data were obtained, were included in the modelling process in the case of non-site-specific applications. The study sites, which were located on the Murraygate interceptor sewer and the Perth Road trunk sewer are described in Chapter 5.

DETERMINANT	NOT MEASURED/ OBSERVED	MEASURED /OBSERVED	CALCULATED	VARIATION OBSERVED	VARIATION NOT OBSERVED IN INTERCEPTOR STUDY	VARIATION NOT ATTRIBUTABLE	INSUFFICIENT DATA FOR SUBDIVISION	LIMITED VARIATION IN DATA BETWEEN INTERCEPTOR AND PERTH ROAD SITES
Flow depth		✓		✓				
Flowrate		✓		✓				
Velocity of flow		✓		✓				
Hydraulic gradient			✓	✓				
Shear velocity			✓	✓				
Acceleration due to gravity	✓							
Fluid density	✓							
Dynamic viscosity	✓							
Sediment size	✓							
Density of particles	✓							
Sediment bed thickness		✓		✓		✓		
Sediment type		✓		✓		✓		
Sediment bed roughness	✓							
Sediment residence time	✓							
Sediment cohesive strength	✓							
Sediment moisture content	✓							

Table 4 (a) - List of Determinants to be Considered for Inclusion (In-Sewer)

DETERMINANT	NOT MEASURED/ OBSERVED	MEASURED /OBSERVED	CALCULATED	VARIATION OBSERVED	VARIATION NOT OBSERVED IN INTERCEPTOR STUDY	VARIATION NOT ATTRIBUTABLE	INSUFFICIENT DATA FOR SUBDIVISION	LIMITED VARIATION IN DATA BETWEEN INTERCEPTOR AND PERTH ROAD SITES
Sewer invert slope		✓			✓			✓
Sewer type		✓			✓			✓
Pipe diameter		✓			✓			✓
Pipe shape		✓			✓			✓
Pipe roughness	✓							
Particle size	✓							
Particle density	✓							
Particle shape	✓							
Settling velocity		✓		✓		✓		
Concentration gradients		✓			✓			
Antecedent dry weather period		✓		✓				
Time since start of storm		✓		✓				
Near bed material transport rates		✓		✓			✓	
Concentration of washload	✓							
Quantities of gross solids	✓							

Table 4(a) Continued List of Determinants to be Considered for Inclusion (In-Sewer)

DETERMINANT	NOT MEASURED/ OBSERVED	MEASURED/ OBSERVED	CALCULATED	VARIATION OBSERVED	VARIATION NOT OBSERVED IN INTERCEPTOR STUDY	VARIATION NOT ATTRIBUTABLE	INSUFFICIENT DATA FOR SUBDIVISION	LIMITED VARIATION IN DATA BETWEEN INTERCEPTOR AND PERTH ROAD SITES
Catchment area		✓			✓			✓
Catchment shape	✓							
Catchment slope		✓			✓			✓
Surface roughness	✓							
Soil type	✓							
Catchment usage		✓				✓		
Surface particle density	✓							
Surface particle size	✓							
Surface cleaning	✓							
Winter gritting	✓							
Rainfall intensity		✓		✓		✓		
Rainfall duration		✓		✓		✓		
Rainfall distribution	✓							
Overland flow depth	✓							
DWF patterns (seasonal)		✓		✓		✓		
DWF patterns (weekly)		✓		✓		✓		
DWF patterns (time of day)		✓		✓		✓		
DWF patterns (national variations)	✓							
Traffic density	✓							

Table 4(b) List of Determinants to be Considered for Inclusion (Other)

A summary of those determinants selected for modelling is shown in Table 5

DETERMINANT	SITE-SPECIFIC MODEL MURAYGATE INTERCEPTOR	SITE-SPECIFIC MODEL PERTH ROAD TRUNK	NON-SITE-SPECIFIC MODEL
Flow depth	✓	✓	✓
Flowrate	✓	✓	✓
Velocity of flow	✓	✓	✓
Hydraulic gradient	✓		
Shear velocity	✓		
Sewer invert slope			✓
Sewer type			✓
Pipe diameter			✓
Antecedent dry weather period	✓	✓	✓
Time since start of storm	✓	✓	✓
Catchment area			✓

Table 5 List of Determinants Selected for Inclusion

The modelling of solids transport processes can only be done for the mode or modes of transport actually measured. In the field studies described, the measurement of material moving near to the bed ("Bed Load Studies") was attempted, and resulted in limited but significant findings regarding the relative proportion of solids transported in this mode during periods of dry weather flow. Because these results could not be incorporated in the modelling process, these are not discussed in the main text of this thesis. The procedures used, results obtained and conclusions reached as a result of the bed load studies are therefore contained in Appendix A.

The data measured for near bed material were not sufficient to relate to other determinants in order to build a predictive model for near bed material transport rates. Therefore, the only model possible using the data from the field studies must relate to the mode of transport for which there is sufficient data, ie suspended solids concentrations. The models developed by this means could subsequently be used in isolation to predict TSS concentration in conjunction with a separate near bed material transport model (assuming a suitable model were available). Alternatively, further work which incorporated more detailed measurements of near bed material transport in association with data of the type used in this work could possibly facilitate the building of a model of total solids transport for a combined sewer.

It should be noted that it is possible to calculate suspended solids mass load values based on suspended solids concentrations and associated flowrates, but it is not advantageous to directly relate flowrates (and other determinants) to suspended solids mass loads in the modelling process, since this involves flowrate terms on both sides of a regression analysis. This could lead to spurious correlations, as discussed later.

4.4 Selection of Suitable Models for Assessment

There are a great number of solids transport modelling methods available in the literature as described in Appendix B and Chapter 2. The reasons why there are such a plethora of alternative models are several:-

1) Since the physical processes are not fully understood in any other than the simplest of cases, continual development of existing methods and

the testing of new approaches has carried on for a great many years.

2) There are a number of situations in which the prediction of solids transport rates is required (e.g. estuarine, pipe flow, riverine, canal flow, sewer flow). Some modelling methods are more suitable for one situation than another.

3) In any particular environment, such as sewer flow, there may be more than one type of situation for which a model is required (e.g. in the case of sewers, different sewer types, different timescales, relative size of network, level of detail required, different levels of input data requirement, prediction of transport rates for different transport modes).

From the preceding sections of this chapter, it may be concluded that the criteria for selection of models for site-specific calibration would appear to leave a wide choice. However, the feasibility of finding an optimum solution to a modelling problem dictates that a maximum of only two or at most three variables should be unknown. In addition, some models exclude wash load while others consider only material moving near to the bed (Engelund and Hansen 1967, Rottner 1959). There are also models which predict the transport rate of solids for deposit free conditions, ie self cleansing velocity (May 1982, Hare 1988, Nalluri and Mayerle 1989, Macke 1983, Ambrose 1952) which are therefore unsuitable for the sites at which field data for this work were measured, where there is evidence of considerable deposition. In addition, models which only consider the particular conditions of limit of deposition (Robinson and Graf 1972, Durand and Condolios 1956) and incipient motion (Novak and Nalluri 1984, Kleijwegt 1992, Ippen and Verma 1953, Mantz 1977, Alvarez-Hernandez 1990) are not intended to address the problem of predicting the rates of transport of solids for other hydraulic conditions.

From Table 4 it can be seen that the determinants for which data were available from the field work were flow depth, flowrate, velocity of flow, hydraulic gradient and shear velocity. This narrowed down the possible alternatives to those models discussed in Sections 2.4 and 2.5. Of these, only the Ackers model (1984) and the Sonnen and Field model (1977) (see Section 2.5) were suitable with respect to their range of applicability, accuracy, and in terms of data requirements. For each of these models, the number of determinants required which are not included in the available data are no more than three. Hence, in each case, the requirements discussed above in relation to the maximum number of unknown variables were satisfied. The selection of the Ackers model in particular is shown to be appropriate by the fact that CIRIA recommend the same model as the best option for modelling solids transport in sewers where at least part of the sediment is travelling in suspension (CIRIA 1994).

Each of the models selected estimates total load of solids transport, while this study is concerned solely with suspended solids concentration prediction. However, the Ackers model includes a transition exponent based on sediment characteristics which assigns the proportion of material transported as suspended solids. Given criteria which imply sufficiently turbulent conditions, it is possible that all predicted transport would be as suspended solids. Also, considering the Sonnen and Field model, this has separate calculations for suspended and near bed material load. Hence, only those calculations for suspended load are considered here.

4.5 Calibration Procedures

Calibration is necessary for the accurate use of all solids transport models. This is true of both SWMM (Hemain 1986) and MOSQITO (Moys and Henderson 1987). Jacobi (1990) demonstrated this fact by assessing the reduction in error achieved with a number of models applied to the same data by a number of different users when calibrated, compared with the performance of the same models applied "straight", ie without calibration. The approach to the use of the selected models and regression equations in the study described in this thesis was to produce models calibrated specifically for the main study sewer with the intention that the method used to arrive at the best model may be applicable to other sites. This could then be tested on data for a separate study site, and for a combination of data from both sites.

For the calibration of the Ackers, and the Sonnen and Field models respectively, the method used in each case was first to input all known variables relevant to the particular model, from the data sets used for calibration, on a spreadsheet. The "unknowns" were represented by columns into which values could be put based on decisions taken during the calibration procedure. These unknowns in the Ackers model were specific gravity and diameter of the particles, and sediment bed width. In the Sonnen and Field model the unknowns were specific gravity, diameter and settling velocity of the particles. For a discussion of the reasons why these determinants were not measured, see Section 4.2.

The lines of the spreadsheet each represented an instant of time. Each line of the spreadsheet thus contained a series of values contained within cells representing various measurements and assumed values, followed by cells containing cell operators which represented the formulae of the particular model. At the end of this row, the output in the form of predicted concentration was contained in another cell. One set of values for the unknown variables could be assigned to all rows of data simultaneously, and a regression analysis of the measured values of suspended solids concentration versus the predicted values carried out. The value of the intercept was set at zero, and by trial and error one or other of the unknown variables altered in order to obtain a value of 1.0 for X_r the regression coefficient (ie the slope of the regression line). When this was achieved, the values arrived at, plus the value of r^2 the correlation coefficient were noted. This procedure was repeated to build up a three dimensional grid comprising of the three variables as co-ordinates, each point associated with a value of r^2. In this way a "surface of fit" was built up, over which a pattern search arrived at the point on the grid corresponding to the values giving the optimum fit.

Since the object of the exercise was to calibrate the models in order to improve the accuracy of prediction, and not to gain information on particle characteristics, the fact that the values arrived at for specific gravity and particle size may not appear realistic does not preclude the use of the models in their modified form. The particle characteristics (and sediment bed width in the case of the Ackers model) arrived at may include inherent adjustments to compensate for errors in the original coefficients of the model, and for unknown factors which are not included in the model. As stated previously, the main advantage of using the models selected was that they were likely to identify significant parameters which should be considered, with the individual determinants and coefficients in an appropriate configuration. In the context of this study, the object was to modify them if necessary so that the optimum performance of each of the models was achieved in each of the particular applications.

The calibration of the rating curves was simply a case of carrying out a regression analysis of flow versus measured TSS concentration, with the value of intercept to be calculated rather than "forced" to remain zero. The resultant regression equation represented the rating curve.

In the case of the more complex regression equations, the procedure was similar to that for the rating curve. Once a particular set of variables for regression had been decided upon, a regression analysis as described above was carried out in order to calculate the various constants and indices of the regression equation.

4.6 Conclusions

From the preceding sections of this chapter it can be seen that, in order to build a model of solids transport for a combined sewer, field data are required to gain information on sewer performance.

A methodology for model building and performance assessment has been proposed, and the determinants to be measured in the field studies have been listed. Ackers, and Sonnen and Field models, plus rating and regression curves have been selected for testing/comparison. The calibration procedure to be used for these models has been described.

5 FIELD STUDIES

5.1 Objectives

The object of the field studies was to obtain data to be used to support the model development proposed in Chapter 4. In the previous chapter, the identification of the requirement for field data was made, and the determinants to be measured during the field studies were listed. These data were collected in order to fulfil the objectives of developing site-specific and non-site-specific models for combined sewers.

The directly measured determinants were flow depth, flowrate and flow velocity, all versus time, and also ADWP and TSSS. These were measured or recorded simultaneously at two sites on the main Dundee interceptor sewer, and independently at other times for the Perth Road trunk sewer in Dundee (see Figure 14 and Table 5). In association with these data, samples of sewage were taken for which TSS values were analysed. The measured or recorded parameters pertaining to these data were sediment type, sediment depth, sewer invert slope, pipe cross section, sewer type, pipe diameter, catchment area, catchment usage and catchment slope. Based on recorded data, associated values of hydraulic gradient and shear velocity with respect to time were calculated. Rainfall intensity and duration for the catchment areas were also recorded. In a separate exercise, sampling of near bed material was undertaken in the main interceptor sewer.

The work carried out to fulfil these objectives is described in the following sections.

5.2 Programme of Work

A programme of field work with associated laboratory work and data processing was carried out during a period from May 1987 to December 1989. The main focus of field work was on a length of combined interceptor sewer in the Murraygate area of Dundee (see Figure 15). At the two study sites at either end of this length of sewer, flow data were monitored, and suspended solids samples obtained. The depths of sediment along the sewer length were measured, and samples of fixed bed deposits at various locations along the sewer length were obtained periodically for analysis. This field work resulted in data which were subsequently used in modelling the prediction of TSS concentrations on a site-specific basis, and for more general application.

It was decided that, since the above work did not include any measurement of rates of near bed material transport, a separate study at another site upstream from the Murraygate site on the same sewer should be carried out to quantify and sample near bed material load. The data obtained on near bed material in the limited time available was useful in its own right. However, as discussed in section 4.3, it was not sufficiently detailed for incorporation in the main body of work. This near bed material or "bed load" work is discussed in Appendix A.

A number of parallel studies of various aspects of solids transport and erosion/deposition have been carried out in the Dundee area by the Waste Water Research Group at University of Abertay Dundee. As part of this other work, flow data and associated TSS concentrations had been measured for a site on a trunk sewer in the Perth Road area of Dundee. These data, although not measured specifically for the work discussed here, have subsequently been used for testing the more general application of the proposed methodology. Apart from an overall description of the Perth Road site, no details of the procedures employed for data collection are included in the following sections. The methods of obtaining the data from the Perth Road site used in the work described in this thesis are similar to those described in the following sections for the main interceptor sewer.

The timescales of the various field studies referred to in this section are shown graphically in the bar chart of Table 6.

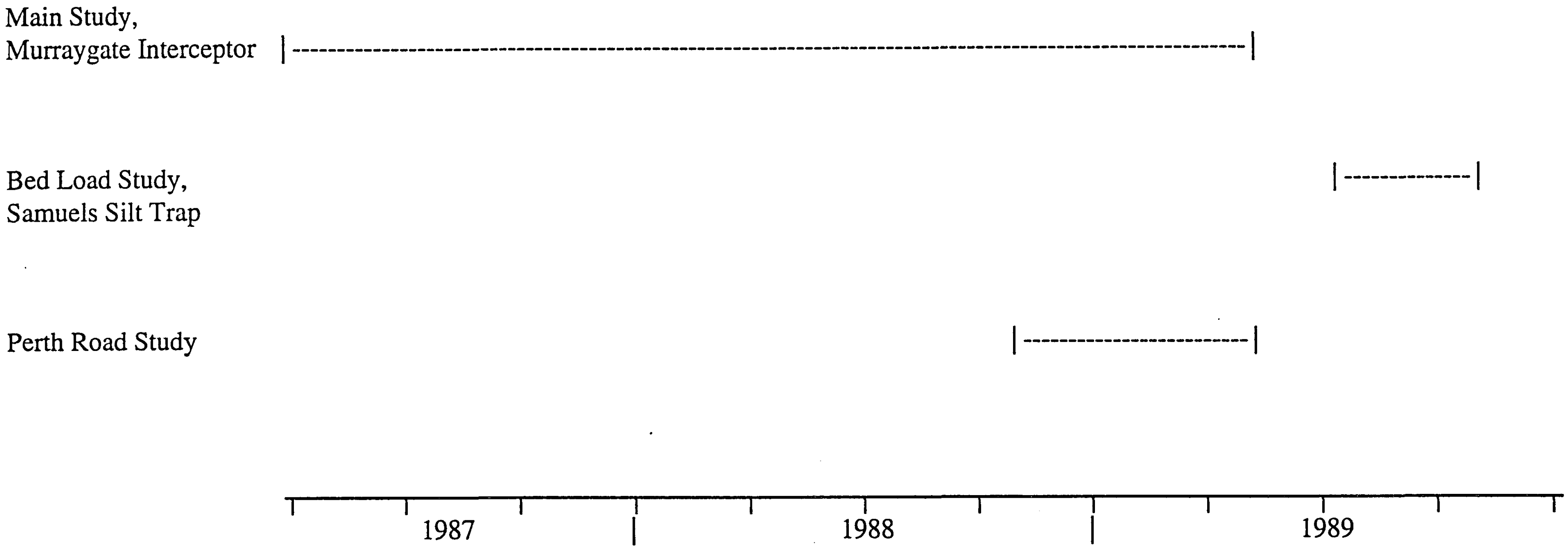


Table 6 - Programme of Field Study Work

5.3 Description of Sites

The city of Dundee is located on the North shore of the River Tay Estuary near to its confluence with the North Sea. Dundee is at the north-eastern end of the central industrial belt of Scotland and has historically been a centre of industry and commerce. The origins of the larger part of the current system of sewers in the city of Dundee date back to the mid 19th century, and its development is largely bound up with the growth of industry and population during the latter half of that century as a result of the industrial revolution.

This increase in population was facilitated by the use of the large estuary as a means of disposal of the city's industrial and domestic waste. The flax industry in rural areas surrounding Dundee gradually gravitated towards Dundee in the early 19th century where large mills were constructed, mostly along the lines of natural watercourses (Rennet and Charlton 1977). The flax industry was gradually replaced by jute spinning and weaving by the mid 1800's which continued to flourish until the early 20th century when a gradual replacement of the jute trade by light engineering and other manufacturing took place. Also, a tradition of shipbuilding, confectionery and printing industries have been an important element in the affairs of the City up to the present day.

Due to these changing patterns of industrial development, there was a rapid growth in population in the 19th century which accelerated between 1860 and 1870, and stabilised around the turn of the century with only a small percentage increase since that time.

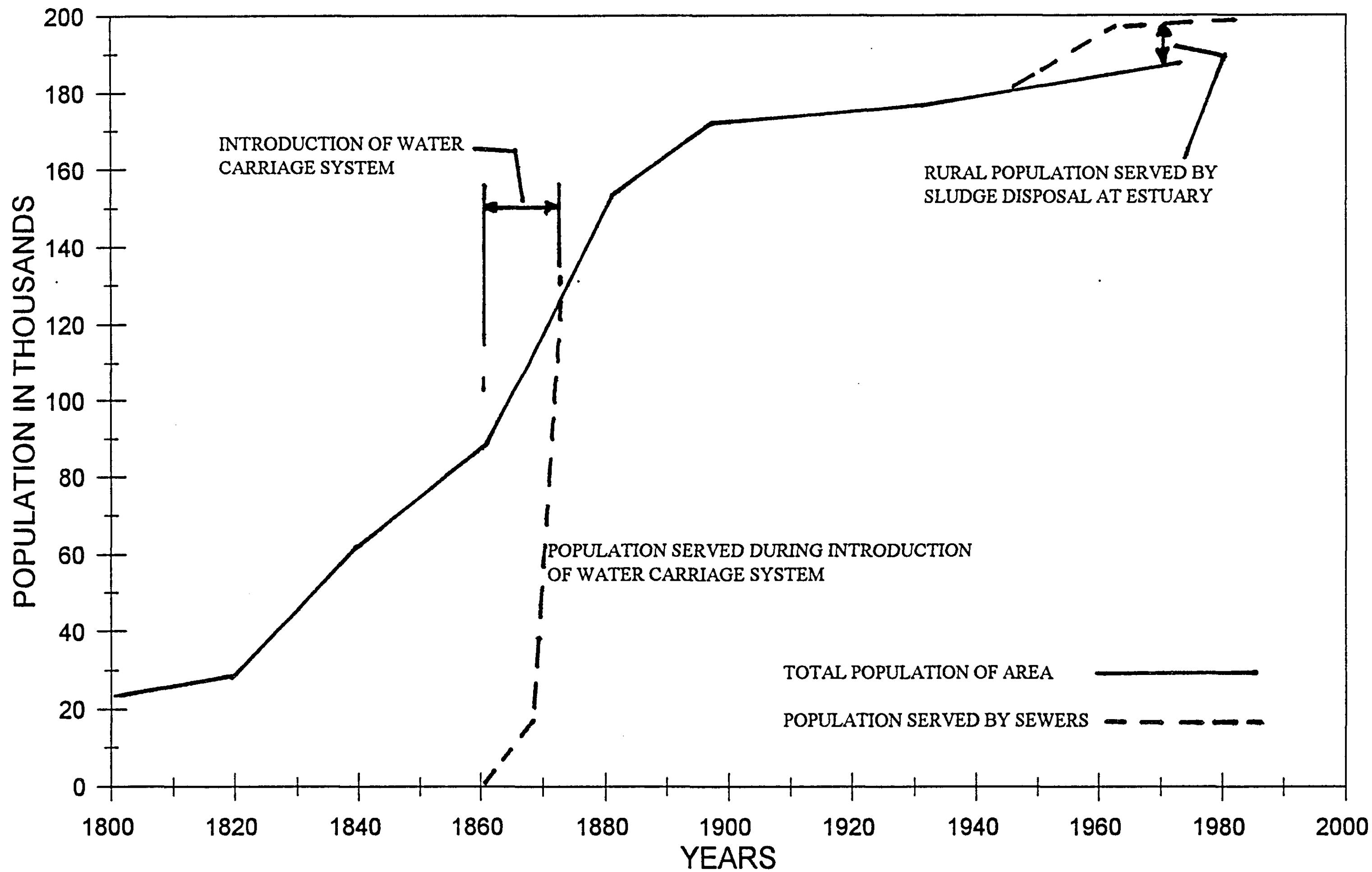


Figure 12. Population served by sewers discharging to the Tay Estuary at Dundee.

The current population of over 180,000 are served by a mainly combined sewer system comprising a mixture of stone, brick and concrete sewers some of which are up to 200 years old (Goodison and Ashley 1990). The majority of these sewers however, date from the period after 1860 during which a large number of sewers were built to accommodate the rapidly expanding population at this time. Prior to 1860, drainage consisted mainly of rubble drains with a few sewers in some of the main streets (Rennet and Charlton 1977). Due to frequent flooding in the 1870's in the central area of the city, it was decided to construct a large interceptor sewer from the High Street in the centre of the city to an outfall at Eastern Wharf in the city's dock area in order to alleviate hydraulic overloading during storms in the sewers of the central area.

The 2.3 kilometre long brick sewer of roughly circular cross section and approximately 1.5m to 1.8m in diameter was completed in 1883. Due to the very flat gradient of between 0.5% and 5% at which it was laid, this sewer has historically tended to accumulate sediment deposits typically of up to 250mm in depth along most of its length (Ashley et al 1990b). These deposits occur despite there being two silt traps along the sewer length, one at the head and another some 800m downstream. There are a number of similar traps within the rest of the sewer system which range in volume up to thirteen cubic metres (Goodison and Ashley 1990). A typical silt trap is shown in Figure 13. Traditionally the traps had sewer sediments removed by hand, although more recently, suction tankers have been used.

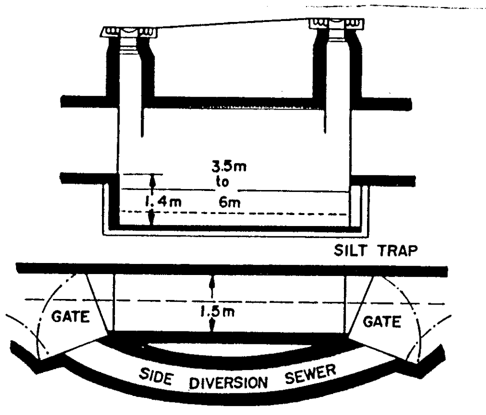


Figure 13 - Typical Dundee Silt Trap

The layout of the Dundee sewer system has changed little since the late Victorian era, and the present principal outfalls and major catchments are shown in Figure 14 (Rennet et al 1989).

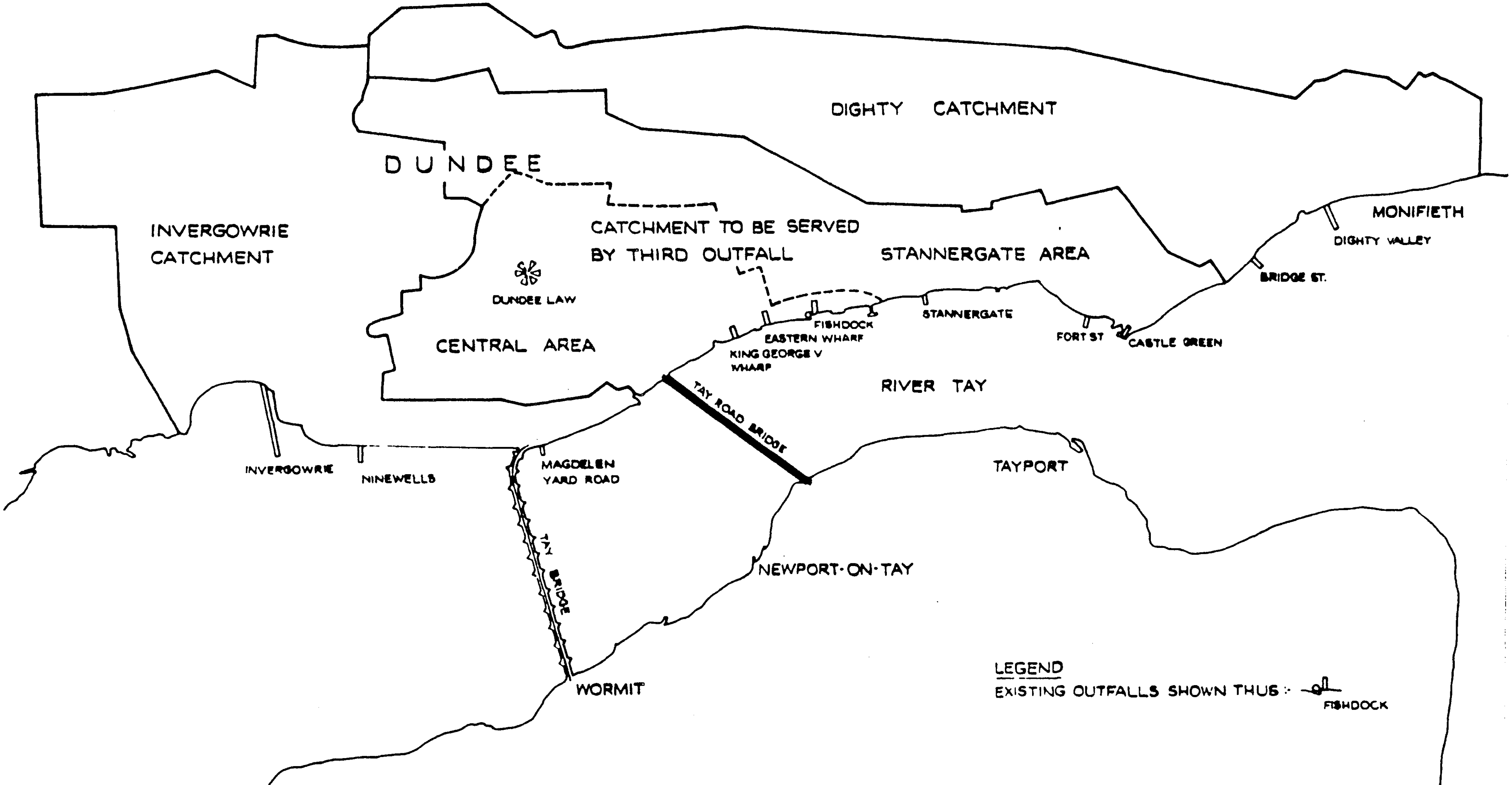


Figure 14 - Existing Principal Sewer Outfalls from Dundee Showing the Major Catchment Areas

A unique feature of the Dundee sewer system is the unusual control system, which is based on an extensive network of control gates (Au Yeung et al 1989). These have to be operated by man-entry and can be used to isolate parts of the system or to concentrate flows in individual sewer lengths if required (Goodison and Ashley 1990). Usually the 300 or more gates remain fixed in pre-set positions. A number of these gates are situated at various interconnections between the main interceptor sewer and the surrounding sewer network allowing the control of flows through the sewer or isolation of lengths of the main interceptor when required (Ashley et al 1990b). A plan of the sewers and the system of gates in the central area of the city is shown in Appendix D.

The main programme of field work on which this study is based was carried out in a 175m length of the main interceptor sewer in the Murraygate precinct of the city centre approximately 200m downstream of the first of the two silt traps on the sewer. This runs through the main retail area of the city and drains an area to the north west of 340 hectares with a resident population of 14590 (Ashley et al 1990b). The average gradient of this sewer length is one in 1446 and it is virtually straight in alignment. There are no major industries within the catchment drained by the sewer; commercial inputs comprise of motor trades, electronics and food processing, with one dye works and a hospital. Average impermeability of the catchment surface is approximately 40%, and contributing slopes range from 4% to 2%.

The cross section of the sewer at the upstream end of the study site is 1.530m high, roughly circular, with a maximum width of 1.415m. Due to particularly severe flooding at the time of construction of the sewer, it was decided to increase the cross sectional size of the length of sewer still remaining to be built. Consequently, there is an abrupt change in cross section 147m downstream of the upper end of the study site. The corresponding major dimensions from this point to the downstream end of the study site are 1.755m high by 1.415m wide. Almost immediately upstream of the upstream end of the study site and approximately 2m past the downstream end there are junctions between the interceptor sewer and major trunk sewers. Approximately mid way along the length of sewer, a gate can be opened to allow sewage to flow out of the interceptor sewer into a smaller sewer in Horse Wynd, thus allowing the main sewer to be drained down if the ends of the study length have been isolated (see Figure 15).

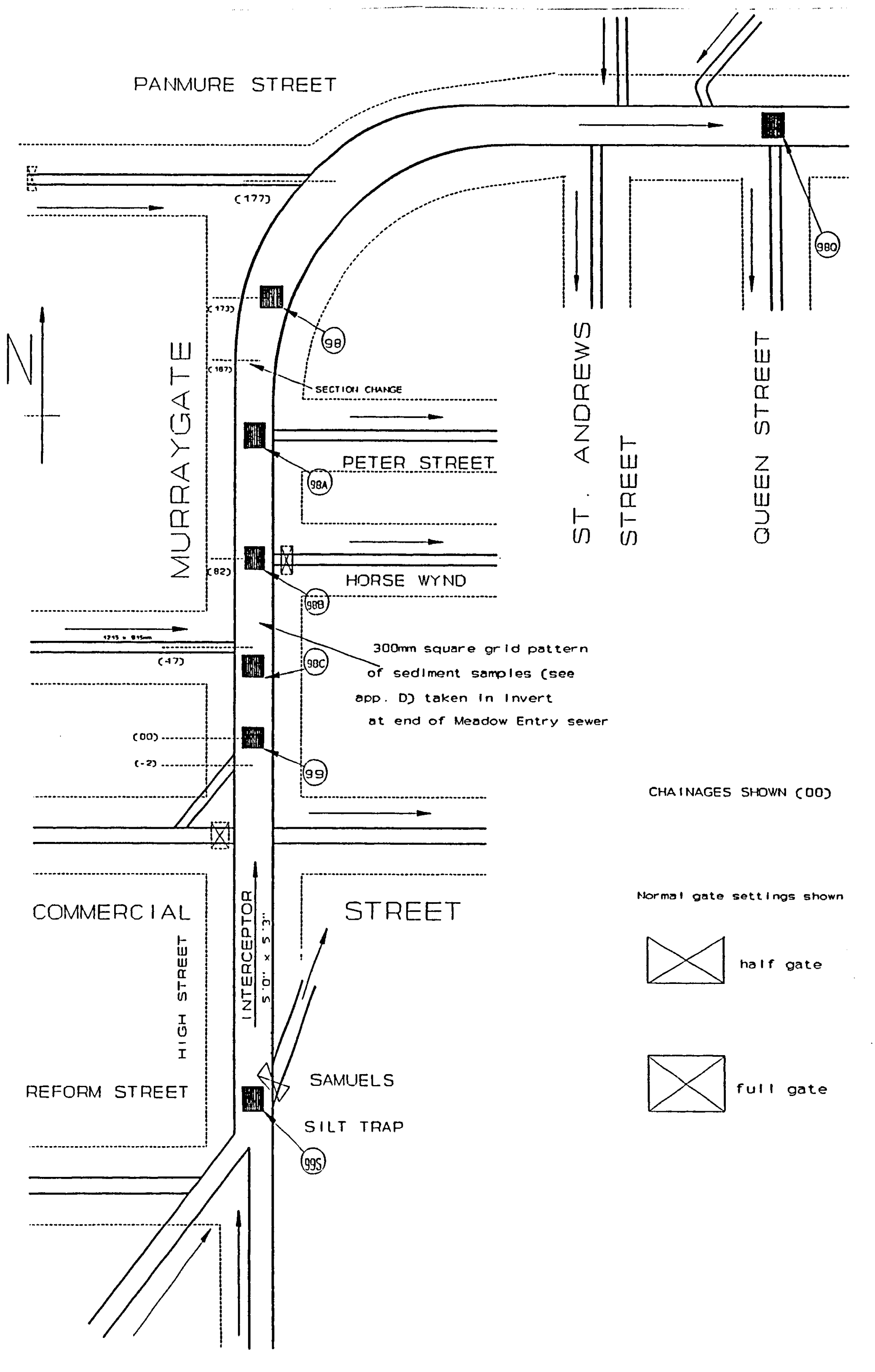


Figure 15 *Plan of Murraygate Sewer*

Plate 1 - Murraygate Area of Dundee, Looking North

Plate 2 - - Murraygate Area of Dundee, Looking South

Entry to the sewer could be gained via manholes at approximately 25m intervals, while the dimensions of the sewer allowed access to the entire length of the study site (Coghlan et al 1992). Two sampling and flow monitoring sites were established, one at the head of the length, and the other at the downstream end, the location of which are indicated by the position of sites 98 and 99 on Figure 15. These boxes were constructed at street level adjacent to the manholes at each sampling site in order to securely contain all automatic sampling and flow monitoring equipment. From the sampling box, connecting cables linked the flow loggers to the velocity and depth sensors, and flexible hoses connected semi-rigid sewage sampling tubes to the automatic sewage samplers. Details of a typical layout are shown in Figure 16.

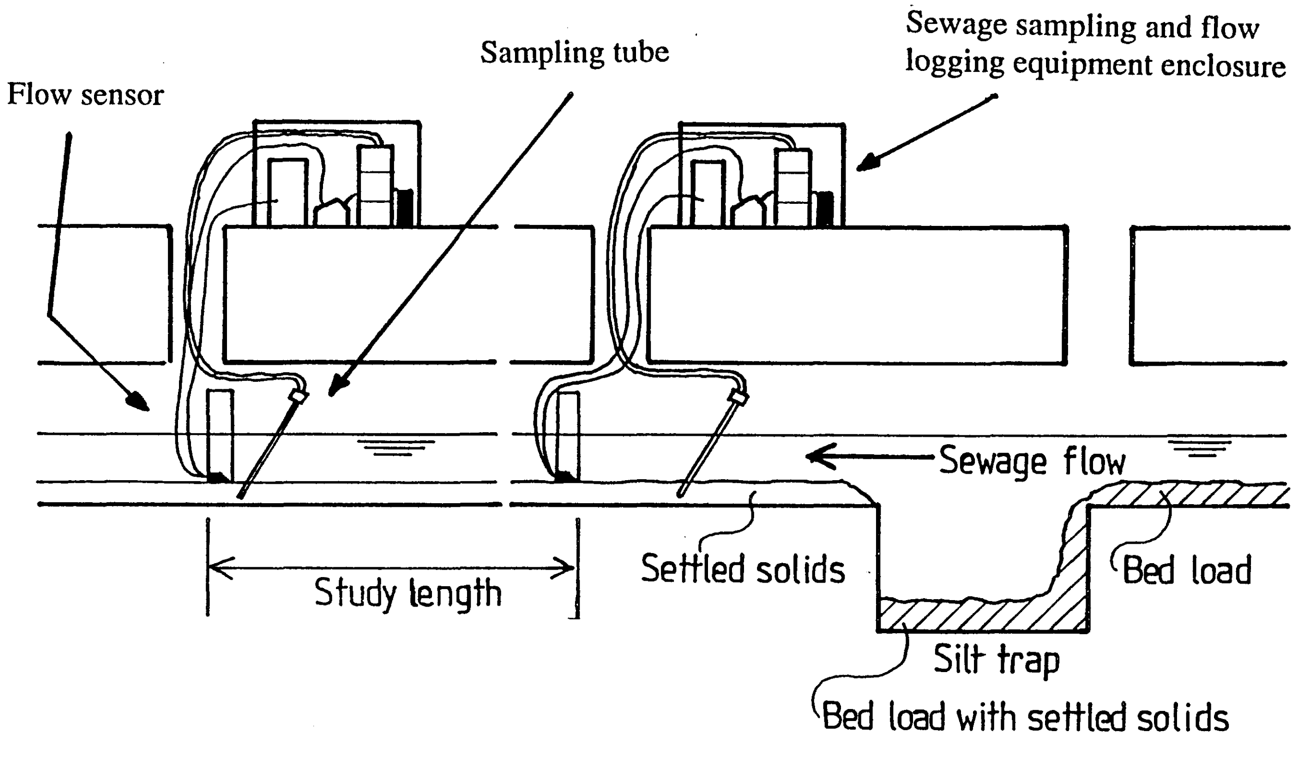


Figure 16 - Layout of Sampling Sites, Main Interceptor

Plate 3 - Sampling and Flow Logging Equipment Enclosure, Site 99

(see location plan, Figure 15)

Plate 4 - Sampling and Flow Logging Equipment Enclosure, Site 98

(see location plan, Figure 15)

The Perth Road site (Site 160) is at the outfall of a subcatchment of the Dundee Sewer System which is much smaller than the catchment served by the Interceptor Sewer. The main details are as follows:-

area	76ha
average slope	1:150
pipe diameter (subcatchment outfall)	1.05m

This sewer is classed as a trunk sewer according to the classification method proposed by Ashley et al (1992b), and as such is quite different from the Interceptor Sewer in the flow regimes and sediment types and quantities present. A schematic diagram of the catchment is shown in Figure 17.

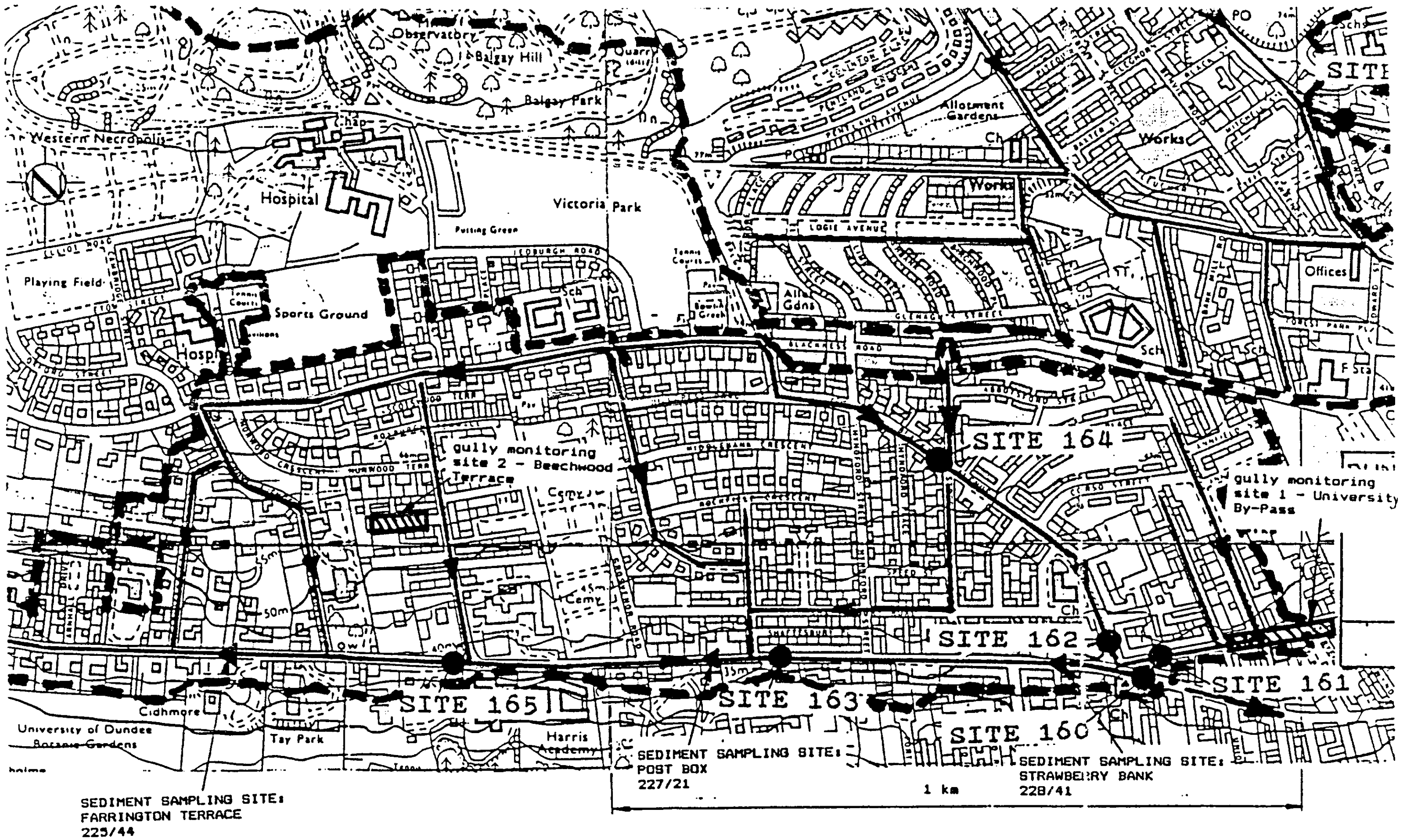


Figure 17. Perth Road Subcatchment Plan

Plate 5 - Perth Road, End of Main Sewer (Site 160)

(see location plan, Figure 17)

Plate 6 - Perth Road , Top of Main Sewer

5.4 Flow Measurements and Instrumentation

The following section relates to procedures used at the main interceptor sewer sites. Similar procedures were used at the Perth Road site.

The standard system used for in-sewer velocity measurement in the UK is the ultrasonic Doppler-shift system (Ashley et al 1993b). The Doppler-effect ultrasonic meter works by measuring the velocity of suspended solids particles or air bubbles and assumes that on average these are travelling at the same velocity as the sewage. The ultrasound is emitted as a "lozenge" which penetrates into the flow an unknown distance depending upon the turbidity of the fluid. The velocity measurement is based on the principle that the frequency shift between sound waves sent out by a transmitter and the reflected waves from the objects picked up by the receiver is proportional to the average velocity of the objects (Wotherspoon et al 1990). The principal limitations of this system are that the region in the flow field in which the velocity is measured is not precisely known.

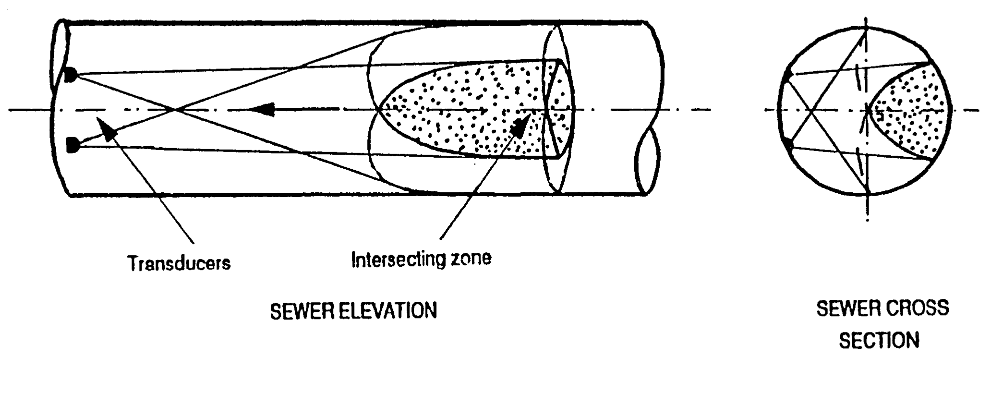


Figure 18 - Typical Ultrasound Envelope for 2 Transducers

Apart from the uncertainty about what is actually being measured by ultrasonic devices, further inaccuracies may be incurred due to variations in the speed of sound in varying densities and temperatures of liquid and the degree to which the portion of flow measured is representative of the total body of flow. Problems may occur due to the disruption of sound signals if the device becomes fouled with paper or rags. Despite these limitations, units incorporating ultrasonic twin sender-receiver crystals, together with pressure transducers to measure depth, have been used successfully in a number of applications (Wotherspoon et al 1990). The accuracy of these devices in ideal conditions is of the order of $\pm 5\%$ when expressed in terms of volumetric flowrate. In practice this may only be achievable for sewers less than 1200mm in diameter, although accuracy will be less for larger sewers (Ashley et al 1993b). According to Jefferies et al (1992) the relative accuracy of these instruments may be summarised:

(a) individual instruments have different error ranges;

(b) an accuracy of flow measurement to within 20% is attainable provided the flow depth is greater than 100mm and the velocity is between 0.3m/s and 2.5m/s;

(c) shallower sloping pipes give more precise results, non-uniform conditions and steep velocity gradients give poorest results;

(d) pressure (depth) measurement transducers are subject to zero drift errors.

At each of the two sampling sites on the main interceptor sewer, continuous measurement of velocities and depths of sewage was achieved using Detectronic ultrasonic flow monitoring equipment. This flow measurement package included an ultrasonic sender and receiver unit (or "mouse"), with an integral pressure transducer.

This was linked via cables to a metal box containing the solid state flow logging equipment. Selection of programme options for the recording and transfer of data (depth, velocity and time) was achieved by connecting an external control box when required. At appropriate intervals, the loggers were interrogated by connecting a small portable computer which stored the data for subsequent downloading to a P.C.

Plate 7 - Detectronic Flow Logger

During the early phases of the field work the logger mouse was mounted directly above the sediment bed by means of a fixing band so that the sensor could "look" upwards. This type of installation was found to be problematic because of fluctuating sediment levels and frequent "ragging up" of the sensor.

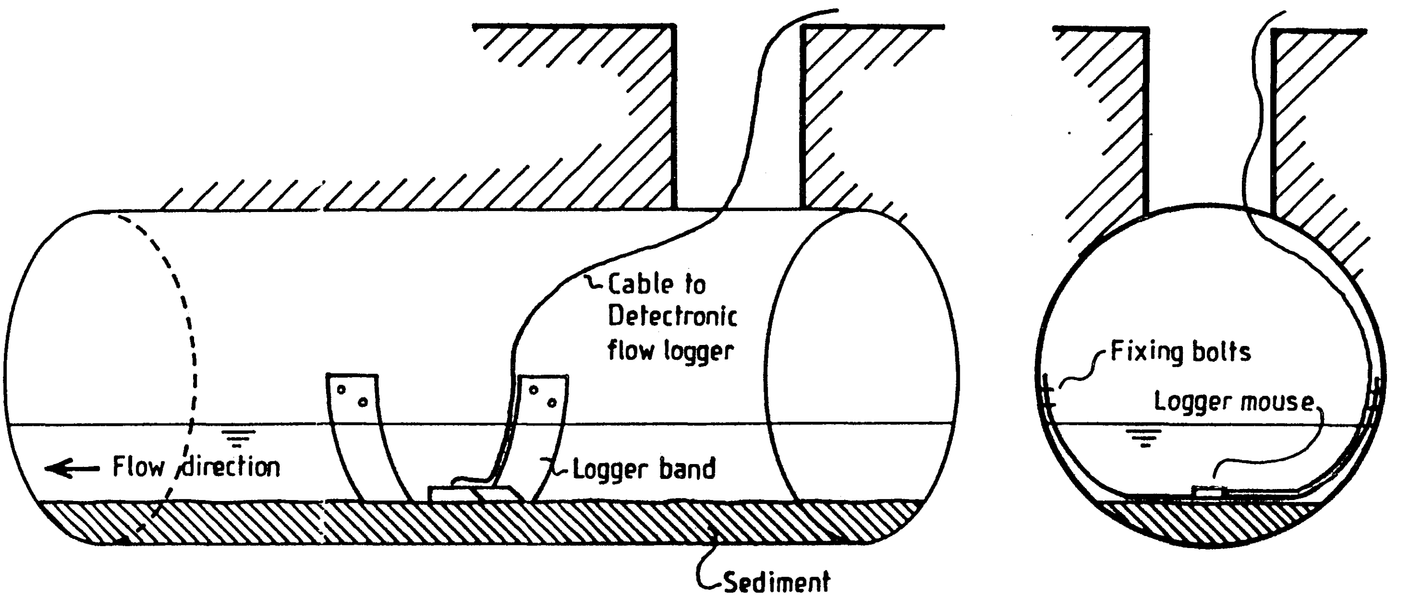


Figure 19 - Logger Sensor Mounting Band (Earlier Arrangement)

Plate 8 - Logger Sensor Mounting Band with Sensor (Centre)
and Sampler Float Switch Box (Left)

This was overcome by mounting the sensor at an angle , directly onto the sewer wall just above the normal sediment height. Although not ideal, this compromise was the only way to achieve reasonable results from the device in this situation.

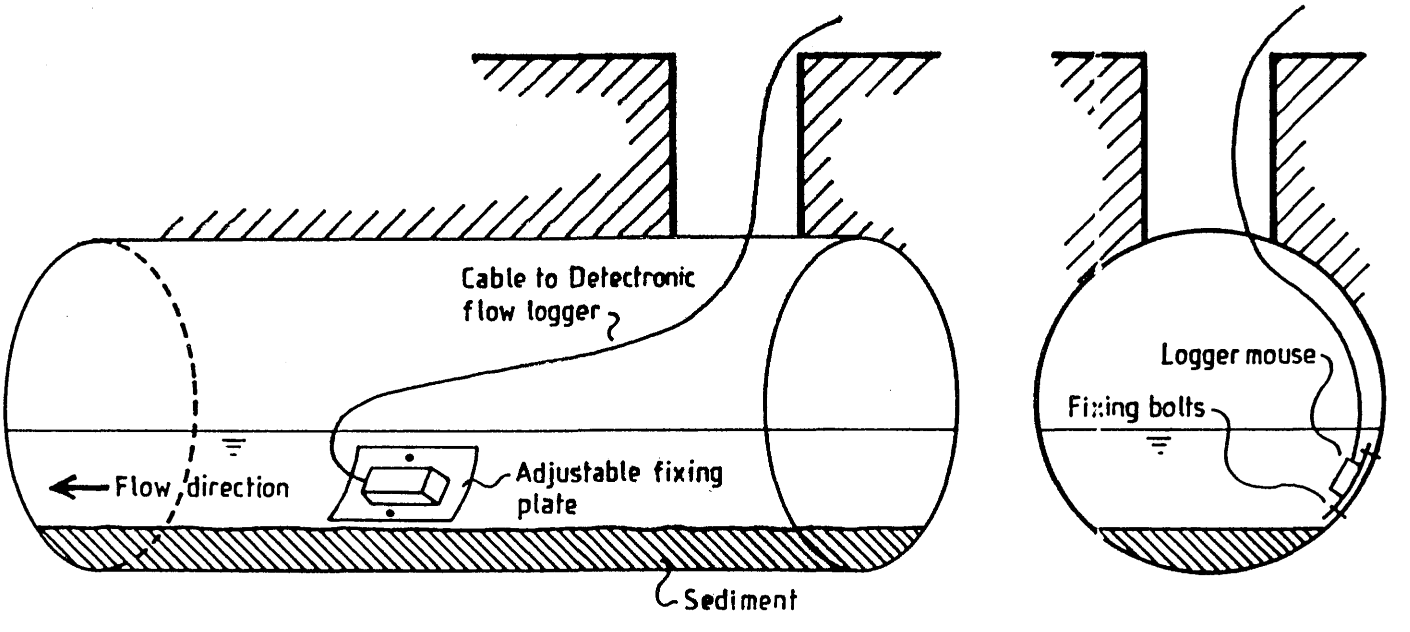


Figure 20 - Logger Sensor Mounting Plate (Later Arrangement)

Plate 9 - Logger Sensor Mounting Plate

At intervals during the field work, the loggers were removed from the sampling sites for calibration in the laboratory. This was done during periods when site work allowed, or when a problem with logger accuracy was suspected. Calibration consisted of two stages. Firstly, the logger sensor was placed at various depths in a tank of water, and depth readouts compared with actual depth to the pressure transducer measured with a metre stick. Secondly, the logger mouse was fixed to the invert of a 300mm wide flume, and various flowrates passed through the flume. This allowed the comparison of velocity readout from the logger with velocities measured simultaneously by a propeller meter. In the case of both depth and velocity readings, any apparent errors were corrected by appropriate adjustments to the instrument according to the manufacturers instructions, or if necessary the instrument was returned to the manufacturer for further investigation and/or repair.

Once downloaded on to a P.C. the recorded velocity and depth data were converted by appropriate software (Jefferies et al 1987) into a more useful form which included flowrates. This software, known as HYDROMASTER, utilised cross sectional data for the particular sewer for which data were being processed, along with any allowances needed for the height of the pressure transducer above the invert of the sewer in order to achieve this task. The final format of the output files from HYDROMASTER included information in a header showing the location and identification number of the logger, followed by lines of data. Each line of data consisted of the date and time of the reading, corrected height of flow, velocity, cross sectional area of flow, and computed flowrate.

The hydraulic gradients associated with the flow conditions measured and sewage samples obtained were calculated from measurements of flow depth at each sampling site, since the relative height difference between the inverts of the sewer at each end of the study length and the distance between the sampling sites were known. It was assumed that the hydraulic gradient at each point in time for both sampling sites was equal to the average slope of the flow surface between the sampling sites calculated at each time interval. This includes the inherent assumption that there is no significant change in specific energy from one end of the study length of sewer to the other - a reasonable assumption given that the 173m length of sewer has a shallow gradient, relatively low velocity of flow and similar depths/velocities at either end. No attempt was made to apportion part of the hydraulic gradient (and hence any boundary shear stresses) to the side walls of the sewer. Other authors have used Einstein's separation technique with some success (e.g. Kleijwegt 1992) for work with laboratory flumes, but this requires detailed knowledge of the relative roughness of the sewer walls and bed deposits, and of the relative proportions of bed width to depth of flow. Since the depth of sediment in a combined sewer has been shown to change rapidly, particularly during storm flows (Ashley et al 1992b), and flow depths constantly change, the proportion of bed width to flow depth was not ascertainable from the measurements taken for this study. Also the relative roughness values, particularly for bed deposits, would have been difficult to estimate. Taking the measurement of average flow surface slope to equal to the effective hydraulic gradient also fits in with the policy adopted in the methodology (see Chapter 4) of basing modelling on a range of measurements which are achievable on a practical level for real sewers. It should also be noted that it is possible that extra turbulence due to side wall boundary shear stresses may in any case contribute to the turbulence of flows which affect rates of solids transport within the sewer.

5.5 Sewage Sampling

The following section relates to procedures used at the main interceptor sewer sites. Similar procedures were used at the Perth Road site.

In order to obtain data on suspended solids concentrations versus time for the sewage flows in the interceptor sewer which could then be related to the recorded flow data, samples of sewage were required. The sewage samples were obtained by using up to three 24-bottle SIRCO samplers at both sites 98 and 99. These could be programmed in advance to operate automatically at predetermined intervals after a preset time delay. In addition the samplers could be started automatically by an external trigger device for storm flows. This was implemented by the simple expedient of mounting a flow switch on the wall of the sewer at an appropriate height which was linked to the samplers by cable. When the flow reached the required level, the switch closed, operating the samplers.

The samplers were mounted in the sampler box, and connected by flexible hoses to semi-rigid uPVC tubes of 10mm internal diameter mounted on the sewer wall at some 25 degrees from the line of the sewer. This angle plus the smooth surface of the tubes and a degree of flexibility allowed the tubes to remain relatively rag-free when submerged, yet located reasonably positively at the predetermined height. Three different depths of tube were employed, the lowest permanently submerged during dry weather flow, with the other two submerged in turn as depths rose in storm conditions (see Figure 21).

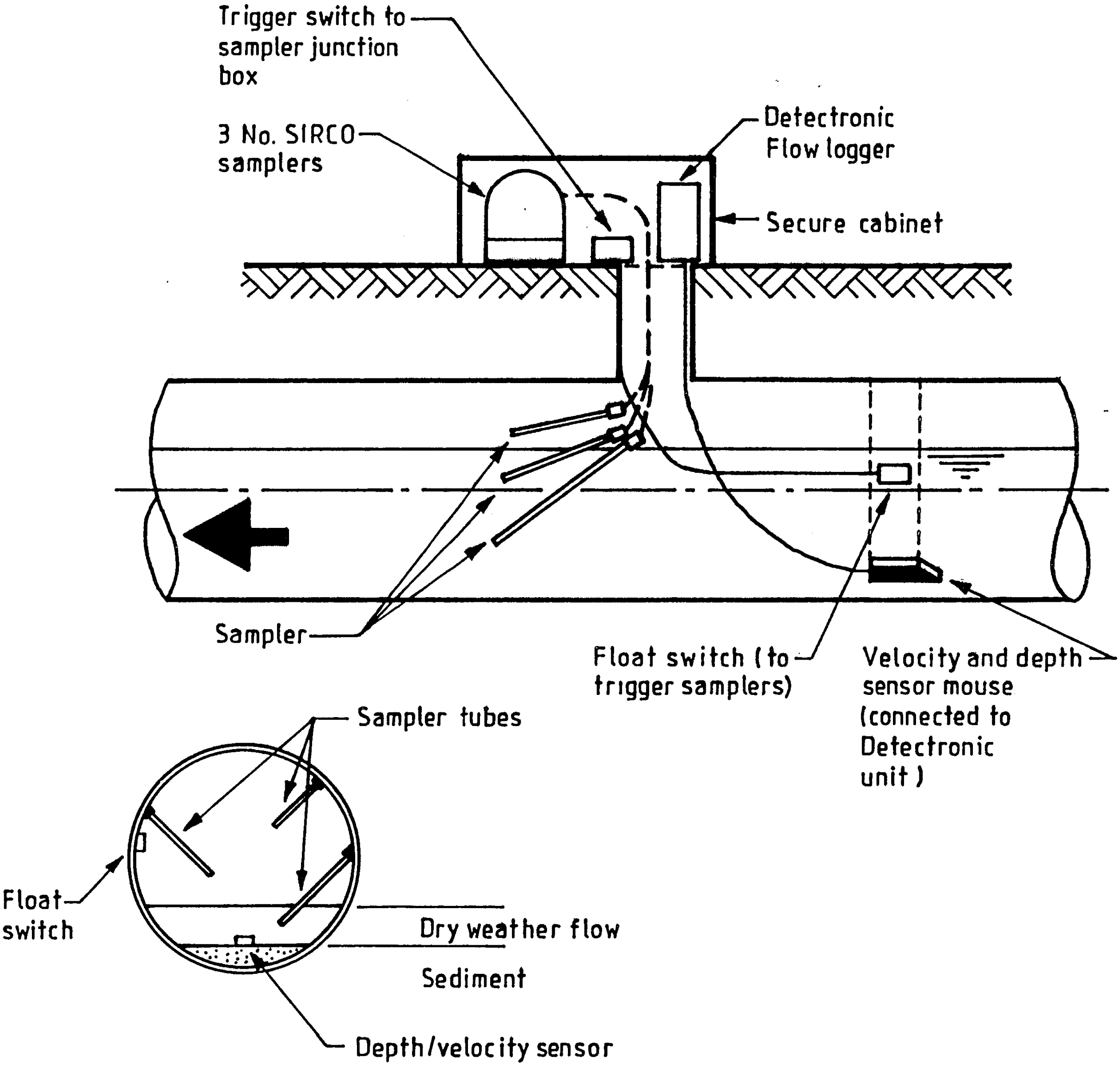


Figure 21 - Sewage Sampling and Monitoring Equipment Layout

Plate 10 - Equipment Enclosure

Plate 11 - Sampler Tube Location in Sewer

Plate 12 - Detail of Sampler Tube Mounting

Samples obtained by these devices were collected, numbered and subsequently analysed in the laboratory. Records of the time at which each sample was taken were kept so that these could subsequently be related to flow records. In the case of samples obtained as the result of a float switch being tripped, a record of elapsed time from a simple timing device connected to the float switch was related to actual time for this purpose. Hence a series of total suspended solids concentrations versus time were obtained at each location and depth.

Due to the limitations of operation of the samplers, it was not possible to take samples at more frequent intervals than once every 5 minutes. This has implications with regard to the uses to which the data may be put (see section 6.4.2). However, in practice the difficulties involved in anticipating the start of a storm meant that very few samples were taken at the fastest rate, since they are taken in such a short space of time that it is easy to "miss" the bulk of the storm. Of those samples that were taken at the faster rates, only those taken at intervals of ten minutes or greater were eventually used for modelling purposes (see section 6.4) due to mechanical problems with the samplers.

5.6 Sediment Bed Depth/Classification

This section considers the procedures used to assess sewer sediment in the main interceptor sewer. Data regarding sediments in the trunk sewer at the Perth Road site were not included for consideration as part of the work undertaken for this thesis.

In order to assess the quantity and type of bed deposits contained within the study sewer from time to time, a series of "walk-throughs" were conducted periodically to measure depths of sediment along the length of the sewer between sites 98 and 99, and to obtain sediment samples. This also allowed a visual inspection of the sediment bed at times when the sewer length had been isolated and drained down, although the majority of walk-throughs were conducted when the sewer flows were at normal dry weather flow levels.

At pre-marked chainage points indicating the distance along the sewer at 5m intervals, a measurement of depth was manually obtained by "feeling" the depth of deposit using rubber gauntlets and measuring using a metre stick. The depths were recorded along with a visual assessment of sediment type by inspection of a scooped sample according to the fivefold WRc sediment classification. The sampling/measuring interval of 5m is smaller than the 10m interval recommended by Laplace (Laplace et al 1989) as a reasonable compromise between accuracy of profile determination and practicality, but in view of the relatively short length of sewer and the number of sources of disturbance to the profile (e.g. junctions, manholes) the shorter interval adopted was deemed prudent.

Samples were also taken of sediment on occasion for subsequent laboratory analysis by a range of tests in order to confirm whether the physical and chemical characteristics of the sediment matched the classification arrived at by visual inspection. For this purpose a sediment sampling tool was developed to enable sediment samples to be obtained from beneath the sewage flow while minimising the possible contamination of the sample by sewage. Details of this sampling device and the associated procedures employed are given in Appendix E.

It is clear that the walk-through exercises were a necessary part of the field work in order to gain information on the bed deposits. However it must be stated that the physical act of walking on the sediments clearly caused a significant amount of disturbance to the sediments, no matter how much care was taken to minimise this. At the very least, some mixing of the layers of sediment was caused by the action of boots sinking into the sediment then being lifted. Also, some weakening and irregularity of surface at the site of each footprint would almost certainly have occurred at least temporarily. These effects are not quantifiable, but must be accepted as an unavoidable consequence of the fieldwork undertaken.

The purpose of categorising and quantifying the amount of sediment in the study sewer was primarily in order to compile data which could be used in association with the other data recorded during the study period to aid in the modelling work subsequently carried out (see Section 7). However, it was decided based on the results of the field and laboratory work that the data gathered with respect to types and quantities of sediment could not be related directly to the measured TSS concentrations due to the non-homogeneous nature of the sediments in the study length and the lack of information about upstream sediments (see section 6). The work was nevertheless useful in demonstrating the variable nature of the sediments both spatially and temporally within a combined interceptor sewer, and in confirming the usefulness of the WRc sediment classification system.

5.7 Near Bed Material Load Sampling

A procedure was devised in order to measure rates of transport and obtain samples of the material transported in the interceptor sewer as "bed load". The details of the procedures used and the results obtained are contained in Appendix A. As shown in

Table 4(a) of Chapter 4, however, the data on "bed load" transport obtained were not sufficiently detailed to be used for incorporation into the main modelling work. The importance and significance of what was achieved by this exercise is discussed elsewhere (see Sections 1.2,1.3,9.1 and Chapter 10).

5.8 Summary

A series of measurements were made using the various methods described in this section in order to obtain data for use in the modelling methods subsequently discussed in Chapters 7 and 8. Additional data were also obtained from other parallel studies for this purpose as described in this chapter. The actual values obtained are examined in the following chapter.

6 APPRAISAL OF DATA

6.1 Introduction

This chapter deals with the data collected over a two and a half year period from the Spring of 1987 to the Autumn of 1989 resulting from the study of the interceptor sewer in the Murraygate pedestrian precinct of Dundee city centre. These data consist of velocity, depth, flow and suspended solids concentrations at each of the two sampling sites for various periods of dry weather and storm flows, cross sectional data for each sampling site, longitudinal profiles of both the sewer invert and of the sediment deposits and sediment sample analyses. These data are contained in Appendices F, G H, J, K and L respectively, and are discussed in the following sections of this chapter.

The data are discussed in the following sections with respect to the inferences that can be made by inspection of the data where appropriate. A description is given of the methods by which data have been selected for specific purposes in the modelling process which is discussed in Chapter 7, and the way in which the data are arranged for this purpose.

Since the Perth Road trunk sewer data are "additional" data, not obtained as part of the field work undertaken as part of this thesis, these data are not specifically discussed in this chapter apart from Section 6.4.2 (Assignment of Data to Specific Purposes).

6.2 General Considerations

An initial period of preliminary trials was undertaken once the sampling sites were established in order to evaluate alternative means of sampling sewage. This phase was completed by September 1987 when sewage sampling using the automatic samplers at sites 98 and 99 commenced (see Figure 15, Chapter 5 for location of sites). The lowest of the sampler intake tubes were set at a height of 250mm above the invert of the sewer for the sample sets acquired between September 1987 and June 1988, following which they were raised to 360mm above the invert. In November 1988 these were again adjusted to 300mm above the sewer invert and remained at this height until the end of sampling in September 1989. The decision on the height at which sampling of dry weather flows was taken purely on practical grounds. If too low, the tubes invariably became blocked with rags, etc and if too high, they would draw air at low flows during the night. The adjustments from time to time were necessitated by changing sediment levels and consequent changes in the levels of sewage flows relative to the sewer invert. The height of sediment above invert was variable both temporally, and spatially along the study length of sewer. At some points in the sewer it was found that there was no sediment at certain times, while the maximum depth of sediment recorded ranged up to 250mm. The depth of overlying sewage flow above the sewer invert during dry weather periods was less variable, since this was influenced more by the average sediment depth along the sewer which tended to follow more gradual trends. Typically, during dry weather flow, the surface of the sewage flow was around 200mm to 300mm above the average sediment height.

Tubes mounted at higher levels to sample storm flows were installed in January 1989 so that, from that time until the end of the main study in the interceptor sewer on suspended solids transport in October 1989, there were tubes at 300mm, 600mm and 1100mm respectively above invert at both study sites. Due to mechanical problems with the automatic samplers and blockage of sampler tubes, many sewage sample sets had to be discarded as insufficiently complete to warrant analysis, or because of uncertainties over the time at which certain samplers had operated. Of the sewage sample sets that have been used, many are partially incomplete due to samples that have been "missed".

Flow measurement also suffered from a number of problems which affected the usefulness of data or samples collected. There were frequent instances when the velocity sensor heads became covered in rags, causing a reduction in the measured velocities, or zero velocities to be recorded. Since there were no inflows between the flow loggers at the two sampling sites, a comparison of calculated flows between the two loggers gave a useful method of checking for partial ragging of one logger. Where flows at both sites had not been properly recorded due to such problems, any sample data obtained could not be related to flow data. Since there were no significant inflows or outflows along the study length of sewer, the continuity equation for flow could be applied providing the flowrate was known for one of the study sites, and the cross-sectional area of flow for the same period of time was known for the other site (assuming flowrates are not changing rapidly). Hence, where flow measurements at only one site were affected by problems, flows at one site could be used along with depth information (and hence cross-sectional information) at the other site in order to calculate velocities at the site for which these had not been measured.

A number of the dry weather flow data sets obtained during the period from 11

December 1987 to 18 December 1987 have not been used for modelling purposes. This is because a plastic tray set into the invert of the sewer immediately downstream of site 99 at this time in order to obtain sediment samples was subsequently found to have caused a significant obstruction to flows because of accumulated debris, thereby invalidating any calculations of hydraulic gradients along the sewer length.

6.3 Sediment Deposits

A total of 16 sewer "walk-throughs" were carried out along the study length of sewer in the Murraygate interceptor sewer. From these, data on the quantity and type of deposits present along the length of sewer were obtained and longitudinal profiles of sediment depth were produced (see Section 5.6). These are shown in Figures G1 to G16 of Appendix G. These indicated that certain locations where there were local disturbances of flows leading to relatively high turbulence (ie at the confluence with the Commercial Street sewer gate chambers at Horse Wynd and Peter Street junctions) tended to have the shallowest deposits. Also, it is apparent that sediment build-up is greater in the upstream half of the study length than in the length downstream of Horse Wynd. The deposits were found normally to be continuous along the length with depths between 50mm and 250mm.

The interceptor sewer was partially cleaned of sediments in January 1987. The silt trap at Samuel's was cleaned out, but not the length of sewer along the Murraygate. This had filled again to the level of the sewer invert by the time of commencement of the study. The downstream sewer from Panmure Street as far as Constable Street was also cleaned (see Figure 15).

A second sewer clean-out was carried out from September 1988 to February 1989. This operation was much more extensive, as an additional length upstream, as far as the Nethergate, and the entire downstream length as far as the end of the Broughty Ferry Road were also cleaned (see Figures V1 and V2, Appendix V)). These lengths were found to be heavily silted, the downstream sewers containing deposits up to 500mm in depth. The study length itself was also cleaned out at this time. During the clean out period, boards were fixed in place over the Samuel's Silt Trap in order to stop it from refilling with sediment, and these remained in place until the bed-load study was undertaken in December 1989.

Average sediment depth values calculated from the depths along the longitudinal profiles for each survey date are shown in the histogram on Figure 22. These show a trend of gradual build up of sediments in the sewer until the cleanout in late 1988. After the cleanout, sediment depths are much lower initially, and appear to build up more slowly after the clean out. The rates of build up for these two periods are shown respectively in Figures 23 and 24. This change in the rate of sediment build up could be partly due to the removal of the downstream obstruction to flow caused by sediment deposits. It was believed that the point of downstream control was moved from a point approximately 1 km downstream of Site 98 to the end of the Broughty Ferry Road - approximately twice the distance - where the interceptor drops sharply down before outfalling into the River Tay. There would therefore have been a greater tendency for deposition to occur because of backwater effects prior to the 1988 clean out. The downstream changes in sediment deposit depth can be shown to have altered flow conditions in the study length by examining the stage/discharge curves for flow data before and after the clean out (see Figures 25 and 26). These figures show a decrease in depths for flowrates measured at Sites 98 and 99 following the cleanout.

Another possible factor influencing the slower build up of deposits after the clean out is the fact that the shallower deposits present at this time would have less tendency to promote further deposition than the deep deposits before the clean out (Ashley 1993b). Laursen (1958) has shown that the presence of a sediment deposit in a pipe increases the overall hydraulic resistance to flow, and therefore causes the depth of flow to increase and the velocity to decrease for any given flowrate. This decrease in velocity leads to reduced transport capacity of the flow and hence increases rates of deposition.

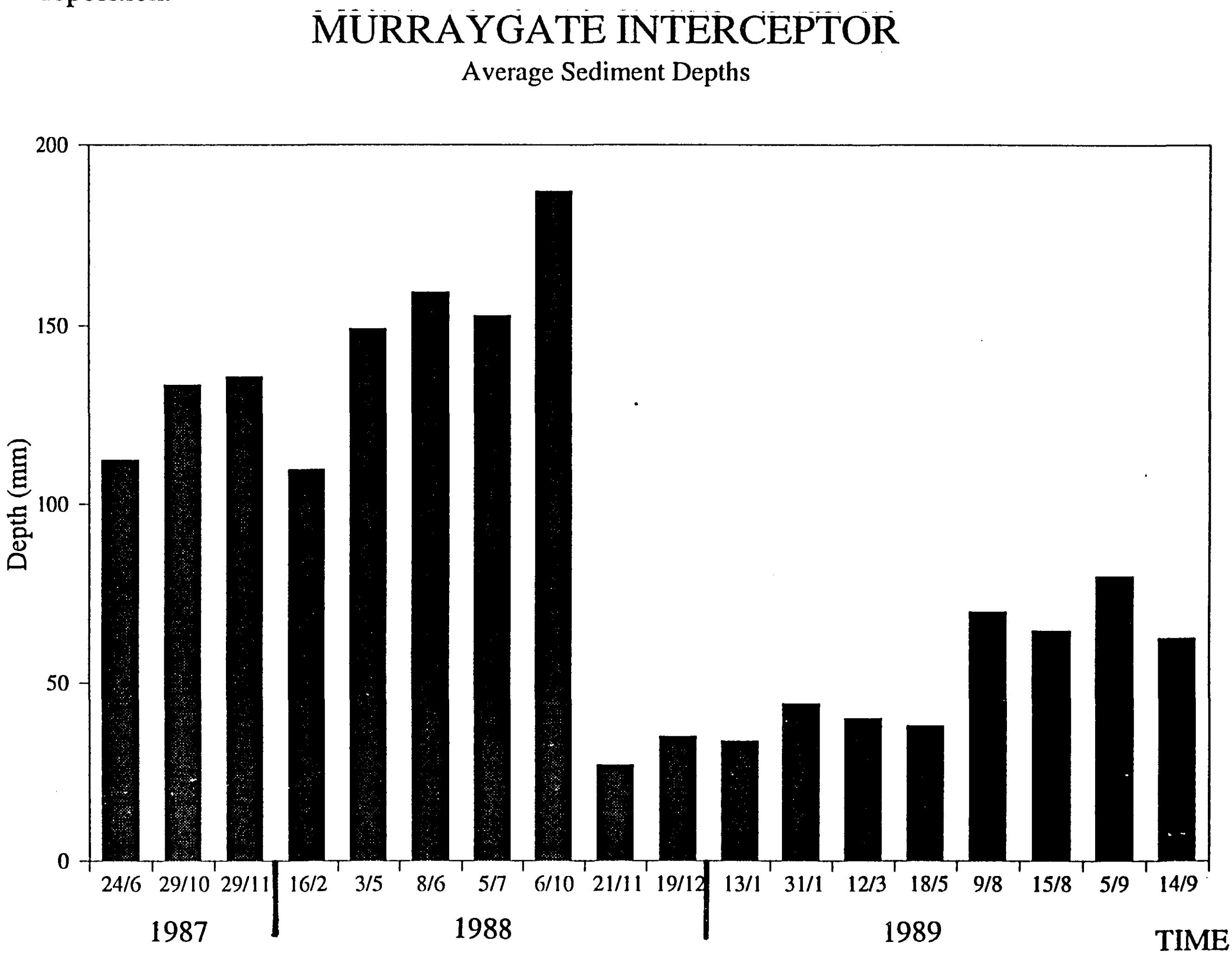


Figure 22 - Average Sediment Depths 24/6/87 - 14/9/89

Plate 13 - Sediment on Inside of Bend at Downstream End of Interceptor Study Length

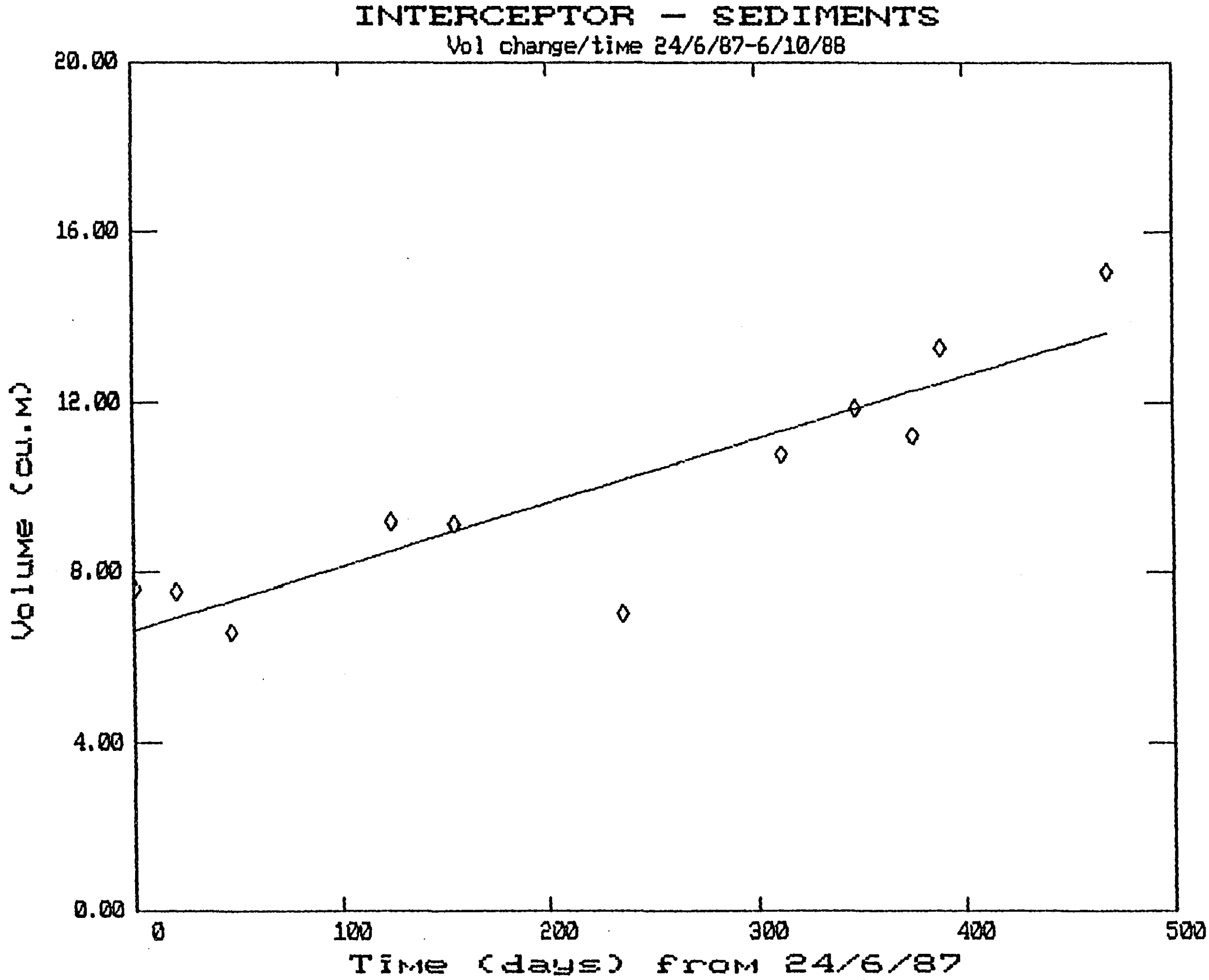


Figure 23 - Rate of Build-up of Sediments in Interceptor June 1987 - October 1988 (Source: Ashley 1993b)

Vol $= 6.623 + 0.015T\ m^3 \qquad r^2 = 0.79$

T $=$ time elapsed (days) since 24/6/87

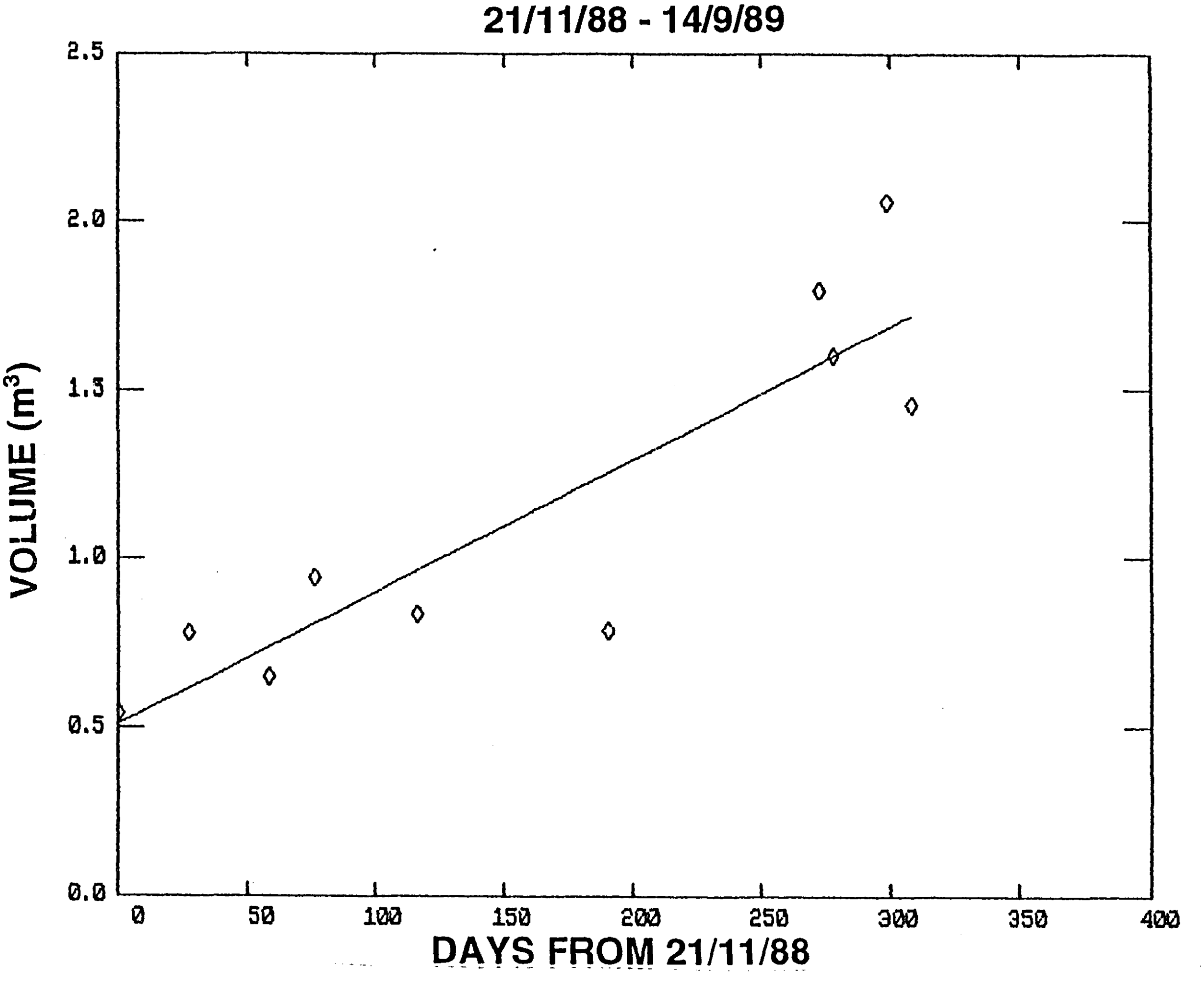


Figure 24 - Rate of Build-up of Sediment Deposits in Interceptor November 1988 - September 1989 Following Major Cleanout (Source: Ashley 1993b)

Vol $= 0.508 + 0.0039T$ m^3 $\quad r^2 = 0.786$

where T = time elapsed (days) since 21/11/88

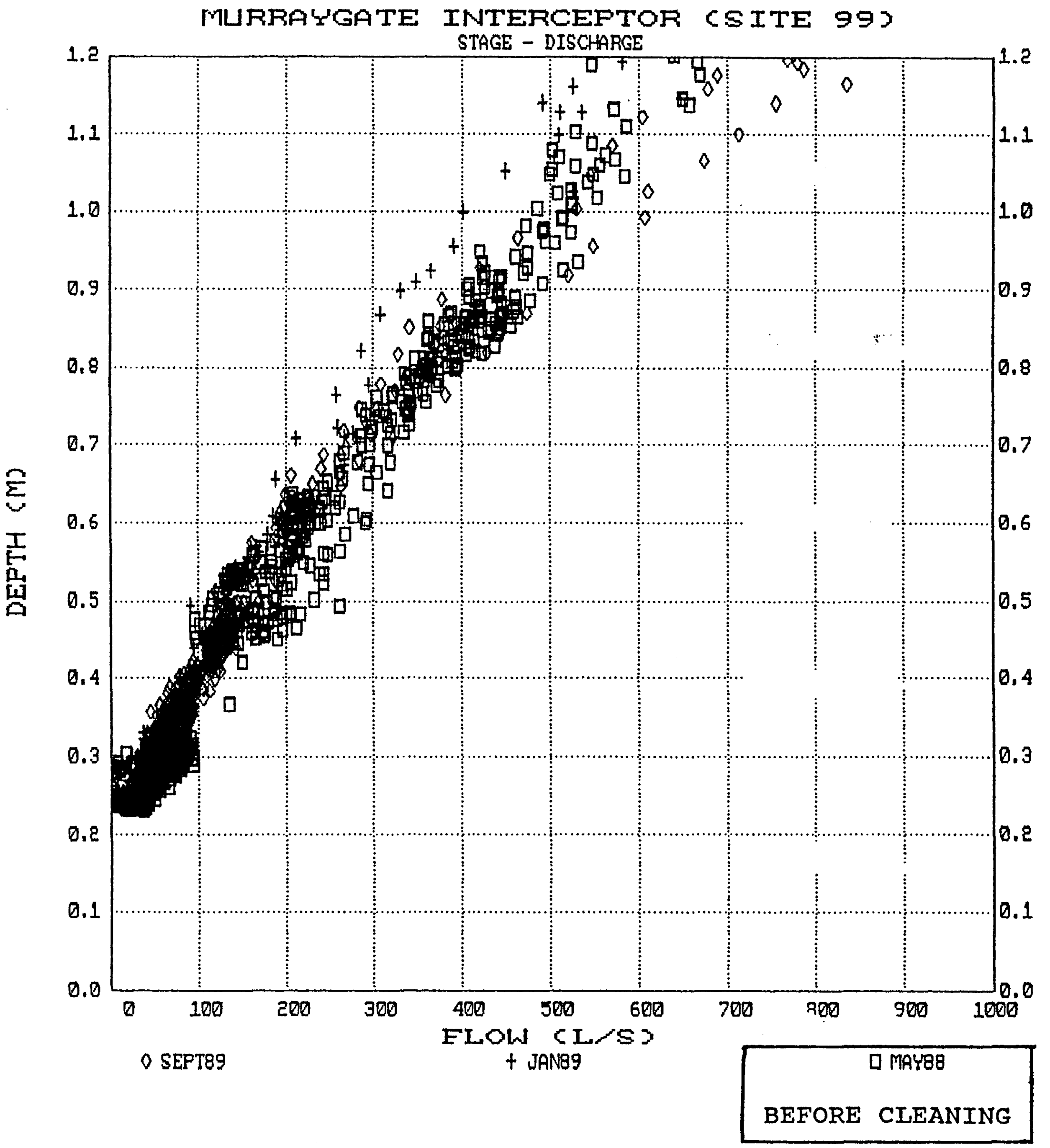


Figure 25 - Depth vs Flow, Murraygate Interceptor (Site 99)

(Source: Ashley 1993b)

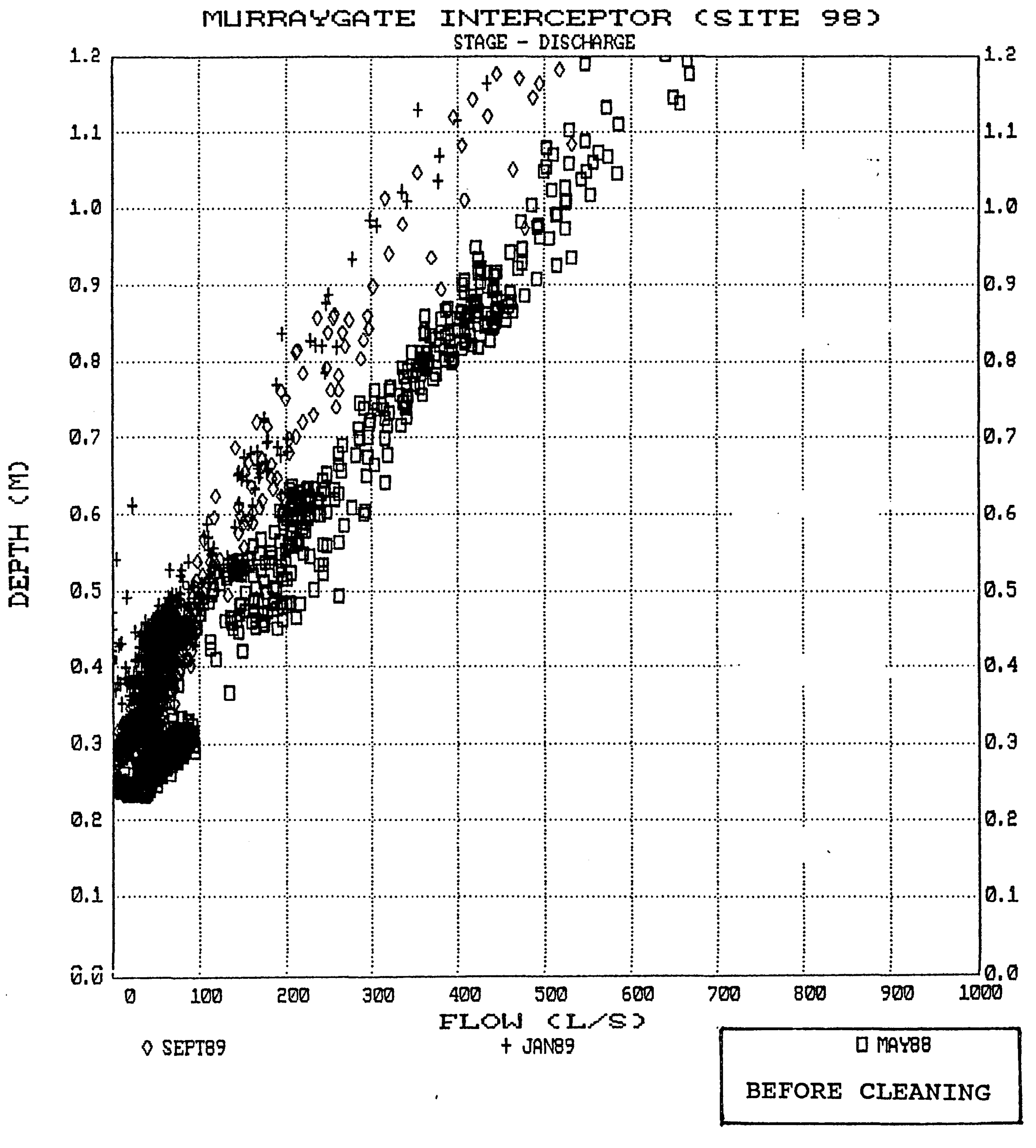


Figure 26 - Depth vs Flow, Murraygate Interceptor (Site 98)

(Source: Ashley 1993b)

A total of 108 samples of sediment were taken from the interceptor during the study period using a specially developed sediment sampling tool (see Appendix E). An initial visual inspection of the samples categorised the samples into one of six perceived classifications (Ashley 1993b, Ashley et al 1990b). These classifications corresponded to the 5 stage WRc classification (Crabtree et al 1991) as shown in Table 7.

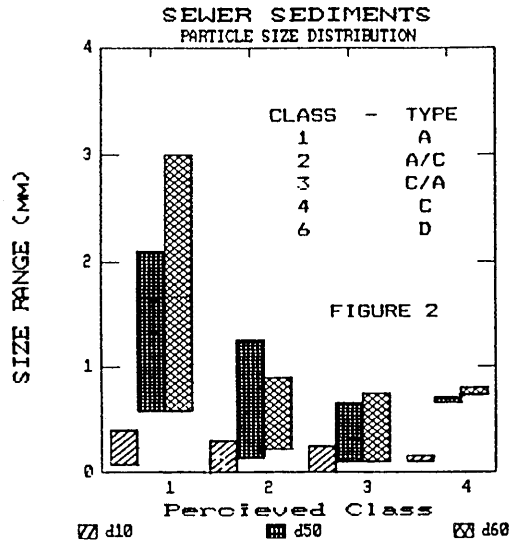
ER324E Class	Visual Class (Dundee)	Nature
A	1	Inorganic Sand/Gravel
A/C	2	Mixture mostly A with some C
C/A	3	Mixture mostly C with some A
C	4	Organic sludge
D	6	Pipe wall slimes
E	5	Tank deposits

Table 7 - Sediment Class

These were subsequently analysed for the following physical and chemical parameters:

perceived class, COD, ammoniacal nitrogen, pH, BOD, LOD, TS, NVS, VS, particle size distribution of ashed residue

The results of these analyses are shown in Appendix F. The results of the laboratory tests were compared with the perceived classifications, and showed reasonable correlation particularly with particle size distribution, volatile solids and ammoniacal nitrogen, but less so with COD (see Figures 27, 28, 29 and 30).



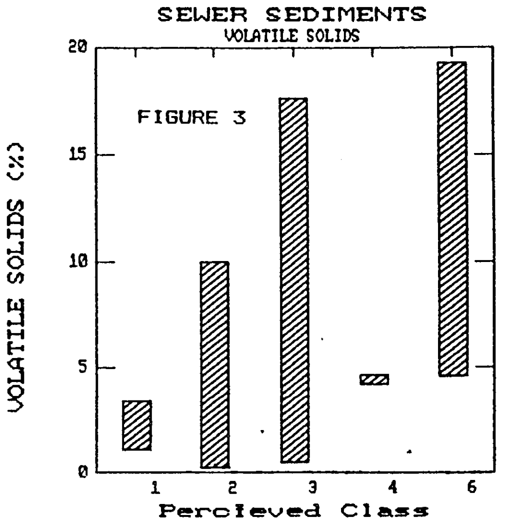
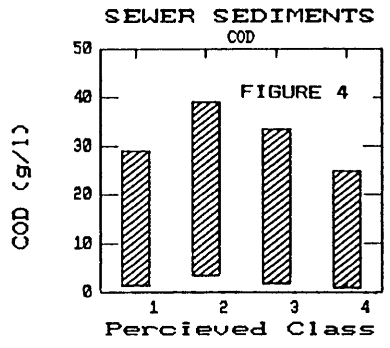


Figure 27 *Figure 28*

Source: Ashley et al (1990a)



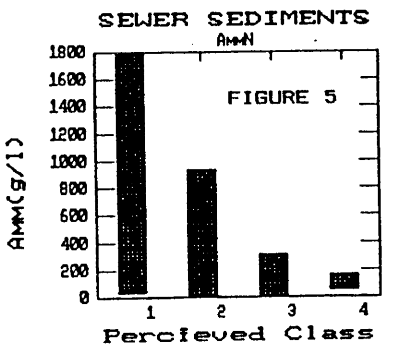


Figure 29 *Figure 30*

Source: Ashley et al (1990a)

These results indicated that a visual perception of sediment classification in terms of the descriptors given in Table 5 was justified both for particle sizes and chemical properties (Ashley et al 1990a). This numerical classification accommodated the overlap in sediment types between class A and class C caused by the variable mixtures of the two sediment classes frequently found. This method of classification was used during the surveys of longitudinal sediment profiles in order to classify sediment deposits at fixed points along the sewer on each occasion. The results of these visual sediment classifications are given in Appendix G. The data reveal that the sediment in the Interceptor study length tends to be a mix of type A and type C in various proportions which vary both temporally and spatially. At the junction of sewers upstream of Samuel's silt trap there is a preponderance of type A material, while the sewer length from Samuel's silt trap down to the study length contains similar deposited material to that found in the study length. This variability of material was confirmed by a study of sediment characteristic variability over a fixed sampling grid (see Appendix H), which demonstrated the high spatial variability in sediment type within the interceptor sewer. The temporal and spatial inhomogeneity of sediments has also been observed for a combined interceptor sewer in the city of Hildesheim as a result of studies by Ristenpart (Ristenpart et al 1994).

One possible reason for the mix of types is that type C bedload material moving over predominantly type A fixed deposits become mixed when sampled for laboratory analysis or visual inspection. It is also possible that some or all of the overlying type C material is stationary at certain times, depending on hydraulic conditions within the sewer at the time of sampling. The problem of differentiating between and sampling these layers separately has not been completely resolved at present, although sampling techniques which involve the freezing of sediment deposits have shown some success in work carried out in French sewers (Laplace et al 1990).

Because of the variable mix of sediment types found in and upstream of the study sewer as discussed above, it was not possible to ascribe characteristic sediment types to particular lengths of the interceptor sewer although it was possible to differentiate between sediment types in samples taken from various parts of the sewer. Also, the locations at which the individual sediment types were found changed with time. The sediment deposits have therefore been considered for the purposes of this study as a homogeneous material, for which there was no useful method of differentiating between types of bed material from one set of sewage and flow measurements to the next. However, allowances were made in hydraulic calculations for the interceptor sewer for changes in average depth of sediment at the sampling points, as calculated from the results of the sediment walk-throughs. The calculations affected were those for hydraulic gradient, flow depth and flowrate used as input to the models developed.

Plate 14 - Type A/C Sediment

Plate 15 - Type C/A Sediment

Plate 16 - Type C Sediment

6.4 Flow and Sewage Data Used for Model Development and Testing

A significant corpus of data were collected as a result of the field and laboratory work. Not all of these data were useful for the purposes of this study. In particular, flow data were collected virtually continuously for the whole of the field study period at time intervals ranging from 30 minutes down to 2 minutes. Also, flow and quality data may be suspect or faulty for various reasons, possibly invalidating whole data sets.

In order to make use of the data, those sets of data which were of use had to be identified. The uses to which the data selected were put had to be decided, and a method of storing these data in a readily accessible way had to be arrived at before model development and testing could be undertaken. These considerations are discussed in the following sections.

6.4.1 Rejection of Faulty or Spurious Data

As discussed previously in Section 4.2, the frequency of sampling of sewage was insufficient due to practical considerations for any detailed consideration of the continuity of variation in TSS concentrations with changes in flow conditions. Flow data other than at the times of sampling were therefore used only to assess such criteria as length of dry weather periods, time since start of storm and overall trends such as rising or falling stage. Therefore those flow data which related to individual samples were first identified from the total flow data records. At the same time, sets of data known to be faulty or suspect due to problems noted on site at the time were rejected. This immediately precluded the use of some sample sets which coincided with rejected flow data. Of the remaining data sets, others were then rejected where detailed inspection of flow data and comparison of flows between the two sample sites indicated further problems such as unexplained discrepancies between flow recorded at either end of the study length of sewer.

Concurrently with the consideration of validity of flow data, an appraisal of sample data was made, checking for sets against which problems had been noted during sampling or laboratory testing. These problems included notes of possible problems with sewage samplers (mainly mechanical) which could often be confirmed by checking the values obtained. Similarly, problems encountered with laboratory equipment or procedures would be noted at the time, and subsequent checks would look for signs of gross error from this source in the preceding laboratory results.

Once this exercise had been carried out, a set of reliable data sets were available for assignment to specific uses.

6.4.2 Assignment of Data to Specific Uses

It was necessary, after establishing which data sets were suitable for use in the modelling procedures discussed in Chapter 7, to decide which data sets should be used for model building, and which would be reserved for subsequent validation.

Firstly data sets were sorted into data associated with storm events, and data obtained during dry weather periods. In each case, the majority of data available were selected for use in model building and the remainder all used for model validation. The smaller subsets of storm and dry weather flow data to be used for validation were chosen on the basis that they should be as representative of the range of conditions as possible in terms of time of year, length and degree of extremity of event (for storms). Also it was decided that they should include data from both before and after the 1988 clean out in order to ensure that there was no inherent bias in the data overall due to this change in conditions. The same criteria of representativeness of the range of data was applied to the data remaining to be used for model building. This was easier in the latter case where more data were used, but in practice the data for validation consisted of only two data sets for dry weather flows and one data set for storms respectively. Key details of the data sets selected for each purpose are shown in Table 8 which also includes data for site 160 (see Chapter 8).

6.4.3 Arrangement of Data on Spreadsheets

The data produced by fieldwork and laboratory work in its raw form comprised printouts of flow data for each of the two flow loggers located at the study sites, and hand written results of sample times and concentrations from the laboratory work. In addition, notes made of sediment levels were required in order to compute hydraulic gradients (see section 5.4).

It was decided to assemble the data in the form of a spreadsheet, initially using individual files for each data set. This was done initially using the software package ASEASYAS and later the more powerful software QUATTRO PRO (Borland International Inc 1992). Since both of these packages are compatible with LOTUS 123, there were few problems in transferring data from one format to the other.

On each spreadsheet, data were arranged so that each line of data represented an instant in time, the first column indicating the date and location of sampling, subsequent columns containing numerical data representing time, flow, velocity, depth and TSS respectively.

Once data had been input to a spreadsheet in this form, it was a simple matter to combine data sets where required, to rearrange the order in which data appeared based on various criteria, or to add extra columns containing further information or the results of calculations performed by cell operators. Regression analyses could be carried out on data, macro commands could be used to perform complex sequential operations, and graphical data could be output in various forms. Extensive use was made of these facilities in the modelling process discussed in chapter 7.

Date	Site No	DWF	Storm	No of Samples	Time Interval Between Samples (mins)	ADWP	Calibration	Validation
15/7/88	99	✓		24	60	-	✓	
19/7/88	99	✓		15	60	-	✓	
27/11/88	99	✓		23	60	-	✓	
2/12/88	99	✓		23	60	-	✓	
19/12/88	99	✓		20	60	-		✓
14/9/89	99	✓		23	60	-		✓
19/9/89	99	✓		24	60	-	✓	
9/11/88	160	✓		17	60	-	✓	
16/11/88	160	✓		21	60	-	✓	
19/12/88	160	✓		24	60	-		✓
1/6/89	160	✓		24	60	-	✓	
5/7/89	160	✓		23	60	-	✓	
8/8/89	160	✓		23	60	-	✓	
13/7/88	99		✓	19	60	44		✓
16/7/88	99		✓	42	30	50.5	✓	
23/8/88	99		✓	24	60	5	✓	
10/8/89	99		✓	24	10	19	✓	
21/9/89	98		✓	24	60	18	✓	
21/9/89	99		✓	24	60	18	✓	
12/11/88	160		✓	7	4	51	✓	
18/2/89	160		✓	24	4	19		✓
24/2/89	160		✓	24	4	46	✓	
11/4/89	160		✓	18	4	25	✓	
5/6/89	160		✓	24	4	67	✓	

Table 8 - Key Details of Data Sets Selected for Model Validation/Building

An example of a typical spreadsheet layout is contained in Appendix I where first the numerical values and calculated figures are shown, and then the same spreadsheet is shown with the "hidden" cell operators revealed. Further discussion of cell operator and macro use is included in Chapter 7, while all graphical output from data in the current study are produced by this means.

6.5 Estimation of Errors in Data Recorded

The degree of control achievable for test results in a laboratory are not achievable using current technology for in-sewer measurement of flow and sewage quality. This, coupled with the high variability of conditions and the harsh environment in which field work must be carried out meant that a comparatively high degree of error had to be accepted in the data acquired.

Flow measurement by ultrasonic velocity sensors of the type used have been shown to be inaccurate by up to 10% in conditions similar to those prevalent in the interceptor sewer in dry weather flow, while storm conditions may increase the margin of error to as much as 20% (Ashley 1993a). This systematic error may be further compounded by the random effect of ragging up of the sensor from time to time.

The procedure by which sewage samples were obtained via sampling tubes connected to automatic sampling devices is also fraught with potential problems. The small diameter tube may obtain a sample which is not representative of the average concentrations present in the flow due to random fluctuations in solids transport or where significant concentration gradients exist. Larger objects such as gross solids and rags or paper may cause further problems. Firstly, they may be included in a sample, causing unrepresentatively large values of solids concentration. Secondly, they may be too large to pass up the sampling tube and hence be excluded entirely from the sewage samples. Lastly, they may block the end of the sampling tube causing either failure of the sampler, or in the case of rags they may act as a filter, causing unrepresentatively low solids concentrations in the samples obtained at such times.

In order to assess the accuracy of samples obtained in this way, a study was carried out comparing large "bulk" samples obtained by automatic sampler with samples obtained manually using a wide-necked container (Ashley 1993b). The results of these tests showed reasonably good agreement for dry weather samples, but underestimates of up to 50% in TSS concentration for automatic sampling during storm periods.

Compared with the figures quoted above for flow measurement and sewage sampling, the degree of error likely in the relevant laboratory procedures is negligible for systematic errors. Apart from random gross errors due to human error which cannot be quantified or predicted, it is contended that these errors are not significant in this context. Thus the errors estimated above for flow measurement and sewage sampling are the only errors considered further (see section 7.2).

6.6 Dry Weather Flows

The data measured during periods of dry weather flow at the two sampling sites which were selected as suitable for model calibration or validation as discussed in section 6.4 are shown graphically in Figures J1 to J14 of Appendix J. These comprise of "sedographs" showing temporal variation of TSS concentrations and associated graphs of the variation in the measured/completed variables of flow, velocity, depth and hydraulic gradient. General comments on these data based on visual inspection of the sedographs are given below.

All of the sample sets for dry weather flow which were successfully taken were from manhole 99 at the upstream end of the study length. The variable TSS concentrations measured exhibit regular daily variations, with the highest values appearing before midday and lowest values occurring in the middle of the night at around 0400 hours. Maximum values are as high as 300mg/l approximately while minimum concentrations are as low as 15mg/l. However, within this range of values, daily maxima are variable, being as low as 210mg/l in one case. The daily minimum values of measured concentrations are relatively stable at around 20mg/l, apart from the minimum value of around 10mg/l on 2 December 1988. It would appear that the lowest concentrations of TSS are associated with low values of velocity and flowrate, although the relationship to velocity is less direct due to frequent fluctuations in velocity (these may be partly due to inaccuracies in velocity measurement for reasons stated in section 5.4).

The plots shown for hydraulic gradient (see section 5.4) are shown in Appendix J. These plots show considerable variability from one data set to the next in overall values, and in the degree of variability in each data set. Some plots show periodic fluctuations which tend to stay around a mean figure for most of the 24 hour period, while in other cases they exhibit a virtually uniform value, with the occasional "spike". This may in part reflect errors in the measurements on which the values are based, but may also in part be due to changing conditions within the sewer or sewage flows. During times when values of hydraulic gradient (see Section 5.4) for the study length of sewer are consistently stable or low, this suggests the possibility of backwater effects due to the build up of downstream sediments. Banks of particularly deep sewer sediments (depths of sediment were not measured) had in fact been noted on occasional inspections of the sewer length downstream. This hypothesis is backed up by the fact that when the hydraulic gradients were stable, the depths of sewage flow measured tended to be greater in value and relatively less variable. Conversely, where depths of flow were in general less, both depths and hydraulic gradients displayed a greater degree of variability. Overall, it can also be said that, when velocities were low for a period of time, ie in the early hours of the morning, there was a tendency for hydraulic gradients to gradually reduce.

Depths of flow during DWF normally ranged in value from 0.35m to 0.45m, but were occasionally as low as 0.25m for some data sets. This again can be attributed to changes in sediment and flow conditions - partly by seasonal changes in inflow and infiltrations, but particularly due to the backwater effects as discussed above. It is noteworthy that the lowest depth values recorded coincide with the cleanout of sewer sediments in November 1988 and a number of storm events in August and September of 1989.

A particular feature of note is the increase in depth and flows up to a peak of around 0.7m and $0.12m^3/s$ respectively for a short period on 2 December 1988 and a similar flowrate with maximum depth of 0.5m on 27 November 1988. Although there is virtually no corresponding increase in velocity in the latter case and only random fluctuations in hydraulic gradient in both cases. No rainfall was noted for these time periods, but it is possible that these phenomena were due to isolated rainfall on outlying areas of the sewer catchment area. There is a corresponding peak in TSS concentration of around 190mg/l in the former case and 240mg/l in the latter at these times.

Overall it can be stated that the variations in DWF concentrations of TSS are not untypical of concentrations measured at other locations. Although foul flows follow daily patterns which vary depending upon location within a network and on country of origin (Crabtree et al 1991), the variations in concentration measured are similar to those measured for various sewers in Dunfermline, Fife (Jefferies 1992). These are shown in Figure 31 below.

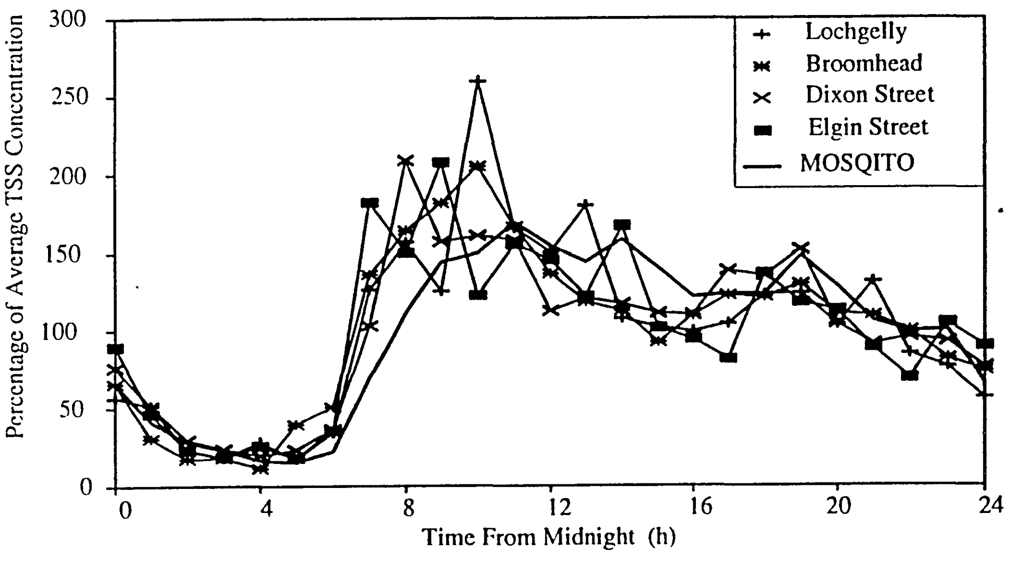


Figure 31 Variation of DWF Hourly Mean TSS Concentrations for Various Locations in Dunfermline, Fife (Source: Jefferies (1992))

Apart from intermittent peaks due to minor, unobserved rainfall events on upper parts of the sewer catchment (see earlier discussion in this section), the daily maxima of flowrate, depth and TSS tend to occur at around 0800 to 1000 hours. In general it can be concluded from visual inspection of the data that of those variables measured, flowrate is most closely linked with TSS concentrations, since TSS concentrations during dry weather flow reach maxima at times of peak flowrate. Therefore, flowrate is likely to be the most useful predictor of TSS concentration.

6.7 Storm Flows

Hydraulic measurements and TSS concentrations for storm flow periods are presented in a similar way to those for DWF in the previous section. These are in graphical form in Figures K1 to K18 of Appendix K. In addition, since the more extreme values of flowrate associated with storm events give rise to a relatively large range of dilution factors which could possibly mask the effects of these flows on transport rates, Appendix K includes a further set of graphs showing suspended solids mass load versus time. Additionally, values recorded for the largest storm event are included in Appendix L. Since it was deemed impractical in the current study to relate rainfall volume and intensity to sewer flows (see sections 3.6 and 4.3), these are not considered here. The graphs referred to above are discussed below.

All successfully obtained sewage sample sets for storm flows were from manhole 99 at the upstream end of the study length, except for data recorded for 21 September 1989 when sample sets and flow data for both study sites 98 and 99 were obtained simultaneously. Maximum depths recorded during each storm event ranged from 0.6m to 1.7m while minimum depths tended to be of the order of 0.4m with the exception of 21 September 1989 when the minimum depths at both sites were around 0.3m. Since the height of soffit of the sewer at manhole 99 is 1.53m, the largest storm event recorded on 30 August 1989 (Appendix L) is also the only one for which surcharge conditions apply, albeit only for a short time (around 20 minutes).

Velocities recorded ranged from 0.1 m/s to 0.8 m/s apart from a reading of 0.05 m/s on 21 September 1989 at manhole 99. This latter result is possibly due to a slight underreading of velocity towards the end of the storm due to ragging up of the sensor since flow rates predicted at this time for manhole 99 are slightly lower than those for manhole 98. This range of velocities is far greater than is seen in dry weather periods as would be expected. This is also reflected in the flowrates calculated which range from 0.01 to 1.3m^3/s.

Of those data sets available for modelling purposes, the range of antecedent conditions are somewhat limited, the maximum being 75 hours. This is not a particularly long dry weather period, and may limit the amount of information available from this source since longer dry weather periods may also have a significant effect on the availability of material within the system. In contrast, the values of time since start of storm (TSSS) range from 0 hours to 128 since some sample sets start well after the beginning of the storm event.

The range of flows recorded were as high as 1.3m^3/s down to 0.03m^3/s, neglecting the possibly artificially low values on 21 September 1989 for site 99 which are discussed below. This is a much greater range of flows than would be recorded in dry weather periods.

As a result of the above flow conditions, TSS concentrations range from 0mg/l to 560mg/l and suspended solids mass load as low as 0g/s up to approximately 220g/s. As might be expected at relatively high flowrate values, the hydraulic gradient plots show less signs of backwater effects than those for dry weather flows. The hydraulic gradients associated with storm flows generally cover a greater range of values and are more variable than the plots for dry weather flows. There is no hydraulic gradient plot for 30 August 1989 (Appendix L) since there were insufficient data on which to base the relevant calculations for that particular day.

Sampling of storm flows via tubes set at more than one height - referred to as multi-depth sampling - commenced in January 1989. The storms from this time onwards were sampled simultaneously at each study site by samplers connected to tubes set at 300mm (tube 1), 600mm (tube 2) and 1100mm (tube 3) above invert, samples being obtained from those tubes which were submerged depending on depth of flow at any particular time. Results from these multi-depth sample sets indicated that for storm flows, the concentration of suspended solids with depth is reasonably constant, as illustrated by data for individual tube heights for the storm on 30 August 1989 in Figures L1 and L2 of Appendix L.

It would in any case be difficult to relate varying concentration with height to overall suspended solids transport rates since only average velocities were recorded. Therefore the concentrations subsequently discussed here are based on average values, for sample sets where samples at more than one height were obtained. Since velocities are lower nearer the bed, where concentrations are higher, some of the errors inherent in using this assumption may cancel out, i.e. the overall mass transport at different elevations, as proposed by Rouse (Graf 1984), becomes sensibly constant. However there remains some indeterminate level of error which is unavoidable due to the limitations of the measurements taken.

The data on 21 September 1989 (Figures K13 to K18 of Appendix K), which include flow and quality data at sample sites 98 and 99, show good agreement for TSS concentrations and suspended solids mass load between the two sites. The small discrepancies are probably as much to do with inaccuracies in flow measurement as any other factor. Particularly at the end of the data sets, lower flows are indicated for site 99 despite the fact that there are no significant inflows between site 99 and site 98. This is most likely due to partial ragging of the sensor head at site 99 on the recession limb of the storm causing a reduction in the values recorded for velocities at this time.

Generally, peaks in TSS concentration are more erratic than is the case for dry weather flow. Peaks in flow, velocity, hydraulic gradient and depth are often before or after the peaks in TSS concentration although general changes in one are reflected in the other - if hydraulic gradient values are high, a peak or peaks in TSS concentration will be noted at that time, and generally low values of TSS concentration are prevalent afterwards. The rate of change of hydraulic conditions is much greater than for dry weather flow and this may partly explain the lack of direct correspondence between changes of flow conditions and changes in TSS. Also due to the unstable flow conditions during storm flow conditions, upstream flow conditions which do not necessarily show up in downstream flows (ie at the study sites) may cause subsequent peaks in TSS concentration to appear downstream.

First foul flush effects are not always observed in the data sets, partly because the early stages of each storm were not always observed in some cases, and also possibly because the sampling rate was too coarse to pick up very short-lived peaks in concentration in many cases (as discussed in section 5.5, the minimum interval between samples was in practice 10 minutes). It is also possible that a first foul flush does not always occur. The data on 21 September 1989 include the start of the storm, and both study sites gave a peak concentration of around 270mg/l corresponding to an 18 hour antecedent dry weather period. On 10 September 1989 the highest value of 450mg/l may be part of a first foul flush corresponding to a similar ADWP of 19 hours, showing that other factors are involved. On 16/7/88 the only other data set which starts near to the start of storm flows is associated with an ADWP of over 50 hours. This resulted in the highest recorded concentration of 560mg/l. This may indicate that an ADWP of up to 2 days or more may have some influence on first foul flush peaks, although more data of a similar nature would be required in order to confirm this hypothesis.

The varying dilution factors due to swiftly changing flowrates during storms also mask the consequent effects of storm flows on the rates of material transported in suspension at these times. Hence, suspended sediment load plots were also produced as shown in Appendix K.

From these plots, it can be seen that peaks in suspended mass load coincide with pronounced peaks in depth, velocity and flowrate after comparatively steady periods. These may occur on more than one occasion well after the beginning of the storm, suggesting periods of varying amounts of upstream deposition and erosion. The peaks in suspended mass load do not normally coincide with maximum TSS concentrations, suggesting that these are "secondary" peaks in suspended solids transport. Most of these are closely associated with velocity readings, although this does not always show a clear relationship. Some correlation with hydraulic gradient, depth and flowrate is evident in some but not all cases.

Maximum suspended mass loads of around 160g/s were recorded in two peaks near the start of the storm on 16 July 1988 at the time of highest flow, velocity and depth after a 2 day ADWP. Similar flow conditions on 10 August 1989 associated with the shorter ADWP of 19 hours resulted in a maximum of half this value at 80g/s. These results indicate some relationship between ADWP and maximum suspended loads. However, the maximum suspended mass load recorded of 480g/s during maximum flowrates on 30 August 1989 correspond to an ADWP of approximately 3 hours. Hence maximum suspended mass loads are associated with high flowrates, high velocities and steep hydraulic gradients commensurate with surcharge events, apparently regardless of ADWP.

There is a suggestion from these results that, apart from the influence of unknown upstream conditions previously discussed, the variables measured appear to have varying degrees of influence at different times. It is most difficult to see clear relationships at times when conditions change or fluctuate with greatest rapidity. This may partly be because the frequency of sampling is insufficient for these rapidly changing conditions. When changes are more gradual, and at a more constant rate, this is less of a problem as shown by data for 10 September 1989.

The shorter time period of around 2 days over which ADWP appears to have some influence on first foul flush (as opposed to 4 days for maximum suspended mass load) is consistent with the hypothesis that initial consolidation of deposited material during dry weather is relatively rapid. The weaker surface deposits which have been resident for 2 days or less are therefore relatively readily available for subsequent first foul flushes, while material that may have built up over a longer time period plays a more significant role in supplying material for resuspension during periods of higher shear stress and turbulence.

6.8 Conclusions

The data obtained have been assessed in terms of their relevance and accuracy for the purposes of modelling and the appropriate data assembly on spreadsheets. The data were categorised as either storm related or dry weather flow related. In each of these categories, the majority of data were selected for use in model building, while a smaller but representative portion of the available data were reserved for the purposes of model validation. Errors in flow data were assumed to range from 10% for dry weather flows up to 20% for storm flows. Errors in TSS concentration due to the sampling procedures used were assumed to range from lower indeterminate values up to as much as 50% for storm flows, although the latter figure is likely to represent the size of error in a small proportion of the total data used. Since the procedures used to obtain the data are typical of the methods currently employed for such studies, and in the absence of more reliable field data of this type, the use of this data is justified in attempting to develop a modelling methodology. Furthermore, the accuracy of prediction of this data by the model developed when tested for validation, as discussed in section 7.5, vindicates the use of the data for this purpose.

7 DEVELOPMENT OF A SUSPENDED SOLIDS TRANSPORT MODEL FOR THE MURRAYGATE INTERCEPTOR SEWER

7.1 Introduction

The rationale behind the approach taken in this study to the modelling of TSS concentrations was discussed in Chapter 4. This chapter describes the development of a model for estimating TSS concentrations for the Murraygate interceptor sewer. Subsequent work in extending this modelling approach to more general applications is described in Chapter 8.

7.2 Performance Assessment of Selected Currently Available Models

The models which were selected for this purpose (see Section 4.3) were the Ackers model (Ackers 1984) and the Sonnen and Field model (Sonnen and Field 1977) see Section 2.5). The object of this exercise in the case of both models was to optimise the fit of the model to the flow and quality data obtained for the Murraygate interceptor sewer. This was done by maximising the value of r^2, the correlation coefficient, from a regression analysis of measured versus predicted values of TSS. This optimisation process involved the systematic variation of the unknown values in each case for the data input to the model, and at each step carrying out a regression analysis. The details of the procedures used in this process for each of the models are discussed in the following subsections.

In order to arrive at a solution in which predicted values were of the correct order of magnitude, a "one to one" relationship (ie with a regression equation coefficient equal to unity) was required as output from the regression analyses, as shown in Figure 32. For this reason, a zero intercept on the vertical axis was "forced" for the relationship between measured data and predicted values. However, where a zero intercept is forced, r^2 has no absolute value and is only useful in such cases as a comparative figure when "tuning" a model to obtain the best possible fit for a particular data set. That is, where a change in the unknown variables has been made, the r^2 coefficient shows whether or not an improvement in fit has been made, or the optimum values reached (when the maximum value of r^2 is attained). Hence, a direct comparison of goodness of fit between the models is not possible using the r^2 value in this way. Therefore, some other criteria for the quality of fit was required for the proposed comparison.

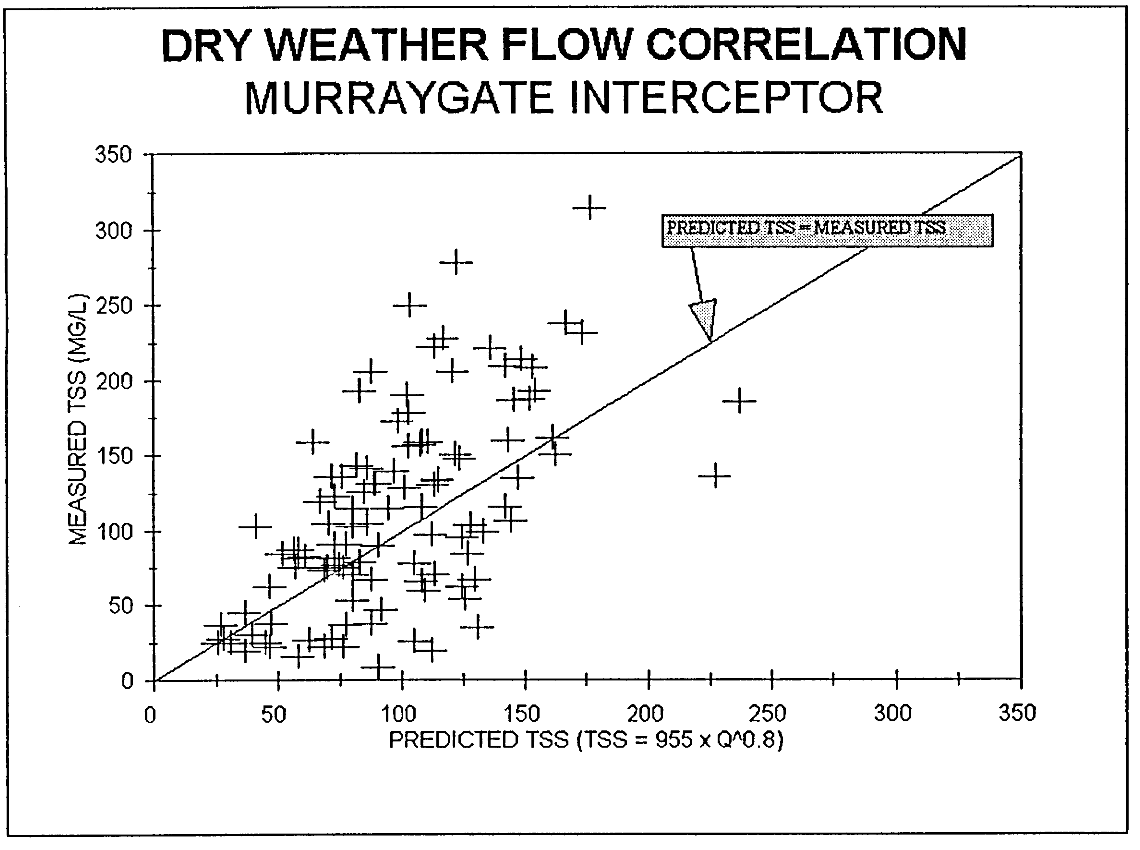


Figure 32 - Typical Regression Plot for Model Performance Assessment

White et al (1975) tested a series of solids transport theories against a large data bank of flume and river data in order to assess the relative performance of the models. So that a true comparison of model performance could be made, the basic parameter used to quantify the goodness of fit to the data in each case was the percentage of the data for which the ratio *Xcalc/Xobs* was greater than ½ and less than 2, where

Xcalc = calculated sediment concentration

Xobs = measured sediment concentration

This parameter can be applied to assess model performance, regardless of the form the model takes.

It was decided that the above method of model performance assessment be used in order to identify which of the various models tested was most suitable for the intended purpose. The use of this method of comparing model performance had the added advantage that a direct comparison could be made with the use of solids transport formulae for the type of data for which they were originally intended.

7.3 Development/Calibration of Models

The following sections describe in turn the work carried out in developing and/or calibrating each of the models selected as possible options for prediction of TSS concentrations in the Interceptor Sewer.

7.3.1 Ackers Model

The basic form of the series of formulae used in calculating the predicted solids transport concentration are as shown in Appendix C. The layout of spreadsheets used for the following procedure are given in Appendix M.

The procedure followed in each set of regressions was as follows:-

1 Assign values of W_e(the effective sediment bed width), s (particle specific gravity) and d_{35} (sediment grain diameter) based on trial and error

2 Carry out regression analysis of X_{calc} versus X_{obs}

3 Alter value of W_e in order to arrive at a value of X_r coefficient of 1.0 by trial and error

4 Record the values of the three variables plus the associated r^2 value

5 Alter one or more of the three variables

6 Repeat steps (1) to (5).

It was assumed that only settleable solids were to be considered, implying that the value of s must be greater than unity. However, the maximum value should not logically exceed the value of specific gravity of near bed material which, had been found to be not more than 1.05 normally (see Appendix A). Hence the limits of the range of values of s were defined as $1.0 < s < 1.05$.

Steps (1) to (5) were continued in a logical series of operations, to build up a series of r^2 values set out on a grid pattern as shown in Appendix N. It can be seen that the values of s and d_{35} formed two of the co-ordinates, and at each point, W_e gave the third co-ordinate at which the X_r coefficient equalled 1.0. Next to this is shown the r^2 value.

The initial results from this exercise for dry weather indicated convergence on a region where small variations of d_{35} and s caused large variations in the value of W_e for which a solution was arrived at, with little variation in the corresponding r^2 values. It was therefore decided to use an assumed value of W_e in order to arrive at an interim solution for which a sensitivity analysis could be carried out. The value of W_e chosen was 0.5m since this was a round figure of the order of magnitude of the actual sediment bed width in the study sewer. This is much in line with the original intended use of the value W_e, since Ackers (1984) suggests that the value of W_e approaches the pipe diameter if the depth of deposits is greater than 0.1 times the pipe diameter, and may have an assumed value if the solids concentration is being calculated.

This decision simplified further calibration work since there were only two remaining unknown variables. The values arrived at were therefore as follows:-

$W_e =$ 0.5m (i.e. 0.28 to 0.33 of the major dimension of the sewer section - see Section 5.3)

$s = 1.0078$

$d_{35} = 0.001338$m

$r^2 = 0.444658$

The r^2 value of approximately 0.44 (based on 108 data points) is not particularly high. This is to be expected given the large number of unmeasured or unknown parameters involved in such a complex situation as is found in a large combined sewer system. The d_{35} figure arrived at is not necessarily a true representation of the wide range of particle sizes present in the sewage flow (as discussed at the end of this section). The actual particle characteristics may include particles larger and smaller than this size and covering a range of densities. This statement holds true for the other results shown subsequently in this section.

At this point, a sensitivity analysis (Robinson and James 1985) was carried out on the model with the above values. The results were achieved by perturbing each of the unknown values in turn and noting the resultant change in X_{calc}. For a 10% change in each of the three values, the following percentage changes in X_{calc} were recorded:-

Parameter	% change in X_{calc}
d_{35}	-3.92
s	-89.5
W_e	+0.39

Table 9 Sensitivity Analysis for 10% change in Parameters

The above analyses revealed that W_e was not sensitive, and it was therefore decided that a fixed value was justified. In order to avoid using a completely arbitrary figure, it was decided to base further calibration work on a W_e value calculated from the actual average sediment bed width over the duration of the study period. This was found to be a figure of 0.53m.

The calibration for all DWF data was re-done with the new W_e value, which produced values of s and d_{35} as shown below:-

$$\left.\begin{aligned} s &= 1.0137 \\ d_{35} &= 0.0008m \end{aligned}\right\} W_e = 0.53m$$

The value of X_{calc}/X_{obs} between 0.5 and 2 calculated for the calibrated model was 68.8%

A similar procedure was used to calibrate the Ackers model separately for storm data using the same value of W_e. The values arrived at were as follows:-

$$\left.\begin{aligned} s &= 1.000285 \\ d_{35} &= 0.02m \end{aligned}\right\} W_e = 0.53m$$

The value of X_{calc}/X_{obs} between 0.5 and 2 was also found to be 68.8% for the storm data, although the similarity to the above figure for dry weather flow is entirely coincidental.

Validation of the storm and DWF models arrived at was achieved by applying the respective models to the storm and DWF data set aside for this purpose (see section 6.4.2). The values of X_{calc}/X_{obs} between 0.5 and 2 for each model validation are shown below:-

Storm	84.2%
DWF	60.5%

Table 10 X_{calc}/X_{obs} between 0.5 and 2 from Validation

The accuracy of prediction in the validations were higher than for calibration in the case of the storm data model, and lower in the case of the DWF model, but of a similar order of magnitude considering the small amount of data used for validation purposes (43 data points for DWF and 19 for storms).

In summary, the values arrived at for the site - specific calibration of the Ackers model for the study site were as follows:-

Parameter	DWF	STORM
d_{35}	0.8mm	20mm
s	1.0137	1.000285
W_e	0.53m	0.53m

where W_e = <u>actual</u> average sediment bed width

Table 11 - Values for site-specific Ackers-White Equation

The particle characteristics arrived at above are those of comparatively large and light particles in the case of both the DWF and storm data calibrations, when compared with typical values reported from the measurement of particle size and specific gravity for particles in suspension in sewers as shown in Table 12.

Parameter	DWF	STORM
d_{35}	0.04mm	0.04mm
s	1.5	2.4

Table 12 - Particle Characteristics

(Source: Bertrand-Krajewski et al 1993)

Clearly the values arrived at by calibration do not closely resemble the figures which might be expected based on current knowledge of the actual characteristics. However, apart from the doubts which have been cast over the accuracy of measurement of sewage particle characteristics (Crabtree et al 1991), it should be borne in mind that the Ackers model was based on laboratory data. Calibration constants arrived at in developing the original model may not be appropriate for the current study. Also, other unknown factors may have some influence on the results of in-sewer studies. This outcome is in line with the proposals for calibration procedures set out in Section 4.5 It should also be pointed out in this context that the original size range of particles for which the Ackers and White model was proposed was between the limits 2.5mm to 0.04mm (1973).

7.3.2 Sonnen and Field Model

The overall description of the model is contained in Section 2.5. The original version of this model (Sonnen and Field 1977) uses Kalinske's equation (Kalinske 1947) to predict bedload as g_S the mass in motion per unit width. This is detailed in Appendix O. This is then used to predict suspended solids concentrations at various heights in the sewage flow using Rouse's equation (Graf 1984), from which the total mass rate of movement may be calculated with the aid of the velocity distribution equation attributed to Vanoni (Sonnen and Field 1977). The details of this procedure are set out in Appendix O.

First attempts to obtain a solution using this model gave spurious results with extremely low predicted concentrations. This was traced to the fact that the equations used by the model for calculation of $(\tau_o)_{cr}$ the critical shear stress for incipient motion (Sonnen and Field 1977) were based solely on particle size, with no allowance for the value of specific gravity of the particles, i.e. Kalinske's equation is only valid for particles with a specific gravity of 2.65. These equations are an approximation based on a series of curves attributed to Lane (1955). From the same source an alternative set of equations for $(\tau_o)_{cr}$ attributed to Krey and Schoklitsch respectively (Lane 1955) which include terms for the specific weight of solids were used. These are shown below (equations 17 and 18). Using these in the model enabled a set of results to be obtained for calibration purposes.

$\underline{d \geq 0.002\text{ft}}$

$$\begin{aligned}(\tau_0)_{CR} &= 0.076(\gamma_s - \gamma)d \\ &= 0.0156d\gamma\,(s-1) \\ &= 0.9712d(s-1)\end{aligned} \tag{19.}$$

$\underline{0.00003 \leq d \leq 0.001\text{ft}}$

$$\begin{aligned}(\tau_o)_{CR} &= 0.000285(\gamma_s - \gamma)d^{1/3} \\ &= 0.003642d^{1/3}(s-1)\end{aligned} \tag{20.}$$

where $(\tau_o)_{CR}$ = critical shear stress (lb/ft^2)

γ = specific weight of water (= 62.4 lb/ft^3)

γ_s = specific weight of the particle (lb/ft^3)

d = median diameter of particles (ft)

There is a transition zone between $d = 0.001$ft and $d = 0.002$ft.

The equations for calculation of near bed material load and hence of suspended load are given in Appendix O. The unknown values in this case are d_{60}, s, and V_S (the settlement velocity).

The calculations involved are somewhat cumbersome, and use a mixture of metric and imperial units. The fact that the equation for the solution of R'_h (the hydraulic radius with respect to grain size) involves a trial and error solution which must be repeated for each data point every time the specific gravity or sediment size values are changed means literally hundreds of calculations had to be done before each regression analysis was carried out. This would not have been possible without the macro techniques (Borland International Inc. 1992) available on the spreadsheet used to set up automated loops to carry out "batches" of calculations.

The calibration carried out for DWF resulted in the following values for the unknown parameters:-

$$d_{60} = 1 \times 10^{-5}\text{mm}$$
$$s = 2.0$$
$$V_s = 0.024\text{m/s}$$

These values are clearly physically improbable since the value of s arrived at is well outside of the range of values suggested in Section 7.3.1, with a high settling velocity for such a small particle size. Also, the calculated accuracy of prediction X_{calc}/X_{obs} (0.5→2) was extremely low at 9.2%. Further calibration with this model was therefore abandoned.

7.3.3 Rating Curve

Since the simplest model that gives acceptable performance is the one to be preferred (Hemain 1986), it was decided to try a simple rating curve as a method of prediction of suspended solids concentration, based solely on flowrate variation. The proposed form of the equation was:

$$\text{TSS} = a_c Q^{b_c} \qquad (21.)$$

where a_c and b_c = coefficients

Q = flowrate (m^3/s)

Using results of a regression of log Q versus log TSS for 108 data points, the following formula was arrived at for dry weather flow data:-

$$\text{TSS} = 955\ Q^{0.8} \qquad (22.)$$

This gives X_{calc}/X_{obs} between ½ and 2 of 82.6%, which is better than the result for the Ackers model.

When this regression was repeated for storm data however, a stable solution could not be found. Examining the r^2 values for the two sets of data illustrates why this was the case:

	Dry Weather	Storm
r^2	0.353126	0.000115

Table 13 - r^2 Values

ie the amount of variation in TSS determined by variation in flowrate during storms is very low. In this case, the Ackers-White model performs much better since other variables such as velocity and hydraulic gradient are included in the model.

7.3.4 Regression Equation

There are many possible combinations of variables that may be included in a regression equation, and these equations may take a variety of forms. According to Hemain (1986) the most important variables in most regressions of this type are $Q(t_e)$, t_e and $v(t_e)$, where t_e equals time elapsed since the start of the storm and $v(t_e)$ = cumulated volume runoff at time t_e (ie this is only relevant for separate storm sewers). ADWP is not seen as significant in the data examined by Hemain.

Stotz and Krauth (1986), on the other hand, see ADWP as an important value, suggesting the following formula:-

$$y_s = a_s \, x^{k_{sk}} \tag{23.}$$

where y_s = mass of solids in g /(m length of sewer)

a_s = a constant, specific to sewer section

x = ADWP in hours

k_{sk} = a time coefficient possibly related to slope

This equation is intended for use in the prediction of FFF in separate storm sewers only.

Jewell and Adrian (1982) discuss various regression equations used for the prediction of instantaneous flux of solids transport. A logical sequence of equations may be built up by carrying out a regression analysis of the independent variable versus one or more dependent variables (linear regression), the independent variable versus the natural logarithm of one or more dependent variables (semi-log regression) or the logarithm of the independent variable versus the logarithm of one or more dependent variables (log-log regression). In each case (linear, semi-log or log-log) the first dependent variable to be tried is the one which has the highest correlation r^2 with the independent variable. Other dependent variables in descending order of correlation coefficient are added in sequence, and for each addition, a regression analysis is carried out.

One of the problems to be addressed in such regression exercises, is deciding which variables to include in the equation, and which are most significant. According to Pisano et al (1979), an increase in standard error of estimate by inclusion of another variable indicates that the additional information given by the extra variable is offset by the loss in degrees of freedom, ie the regression is better without the extra variable.

A strategy for development of regression equations was decided upon as follows

(1) Carry out regression of measured TSS values as independent variable against each individual possible dependent variable in turn.

(2) Construct a "league table" of variables with the highest r^2 coefficient having the highest rank.

(3) Carry out a series of regressions of independent variables versus dependent variables, adding progressively lower ranking variables. This should be

continued until either all variables are included in the regression, or until rejection of an extra variable by the Pisano criteria.

(4) The resultant regression equation is input into the spreadsheet as a cell operator, and used to predict TSS values. Hence, the percentage of X_{calc}/X_{obs} between 0.5 and 2 is calculated.

The above steps were also carried out for log TSS versus log (variable a), log (variable b) .... log (variable n) and also for TSS versus ln (variable a), ln (variable b), ... ln (variable n).

The individual steps in the above procedure were carried out for the Murraygate sewer. Separate regression procedures were carried out first for DWF data and then for storms. In each case, the best equation in terms of accuracy of prediction was selected based on model performance by the X_{calc}/X_{obs} criteria. The step by step tabulation of regressions carried out and resulting equations are tabulated in Appendix Q. The final selected equations for DWF storms are presented here. These were as follows:-

DWF:

$$TSS = 955 \times Q^{0.8} \qquad (24.)$$

$$(X_{calc}/X_{obs}\ (0.5 \text{ to } 2) = 82.6\%)$$

STORM:

$$TSS = 104.4 + 416.4\ v - 0.8\ TSSS - 3.124\ ADWP \qquad (25.)$$

$$(X_{calc}/X_{obs}\ (0.5 \text{ to } 2) = 78.3\%)$$

Since the best fit for storm flows was obtained by linear correlation, a linear correlation was tried separately for data associated with first foul flush (FFF), and for other data ie non- first foul flush (NFFF) to see if this gave a worthwhile improvement in accuracy of prediction.. In order to achieve this some method of selection of FFF data is required. The simplest definition of FFF is data at the start of a storm event during the time period when there is a continuous increase in velocity and depth. Any data whether during a rising or falling limb of a storm which is subsequent to a drop in depth or velocity is deemed to be NFFF data. This regression exercise resulted in the following two equations which in combination could be considered as alternatives to the equation for storms shown above:

$$\left.\begin{aligned} TSS(FFF) &= 376.3 + 224.6\,v - 5x10^5\,S \\ TSS(NFFF) &= 104.9 + 434.4\,v - 0.91\,TSSS - 3.245\,ADWP \end{aligned}\right\} \quad (26.)$$

$$(X_{calc} / X_{obs}(0.5 \text{ to } 2) = 79.0\%)$$

Units:- TSS (mg/l), Q (m^3/s), V (m/s), TSSS (hours), ADWP (hours), S (dimensionless)

Note that the regression equation for DWF turned out in fact to have reverted to a form of the simple rating curve discussed earlier. Tables of regression output leading to the above formulae are given in Appendix Q. The relative merits of the of the various regession equations presented above, plus the other models discussed in the preceding sections of this chapter are discussed in Section 7.4.

7.4 Selection of Best Options

The choice of a model for a site-specific application should be based on both accuracy of prediction and ease of use as discussed in Section 4.2. The results of the calibration exercises for the various modelling approaches can be summarised in Table 14.

Model	Storm		DWF
Ackers	68.8%		68.8%
Sonnen	-		9.2%
Regression Equation	78.3%	*(1)	82.6%
	79.0%	*(2)	

(1) Single equation

(2) Combined FFF/NFFF equations

Table 14 - Summary of X_{calc}/X_{obs} (0.5 to 2) values

Clearly, the choice was very straightforward, since the regression equations were the simplest to use, particularly in the case of DWF, are also the most accurate and are thus the preferred option. In the case of the storm flows, the single equation for all storm flows was almost as accurate as the combined equations for FFF/NFFF flows (see Appendix Q) but was much simpler to use, and was thus selected as the preferred option. The selected equations were therefore tested by a validation procedure.

7.5 Validation of the Best Option Models

As discussed previously in Chapter 6, a certain amount of storm and DWF data were reserved for validation of the model after calibration. Data from two DWF periods and one storm event were available for this purpose (see Table 8). The validation was carried out, giving the following results:-

DWF	81.4%
STORM	79.0%

Table 15 - X_{calc}/X_{obs} (0.5 to 2) Values for Validation Data

Graphs showing examples of measured concentration versus predicted concentration for the validated models are contained in Appendix U. One of the these graphs is repeated below (Figure 33) for illustrative purposes.

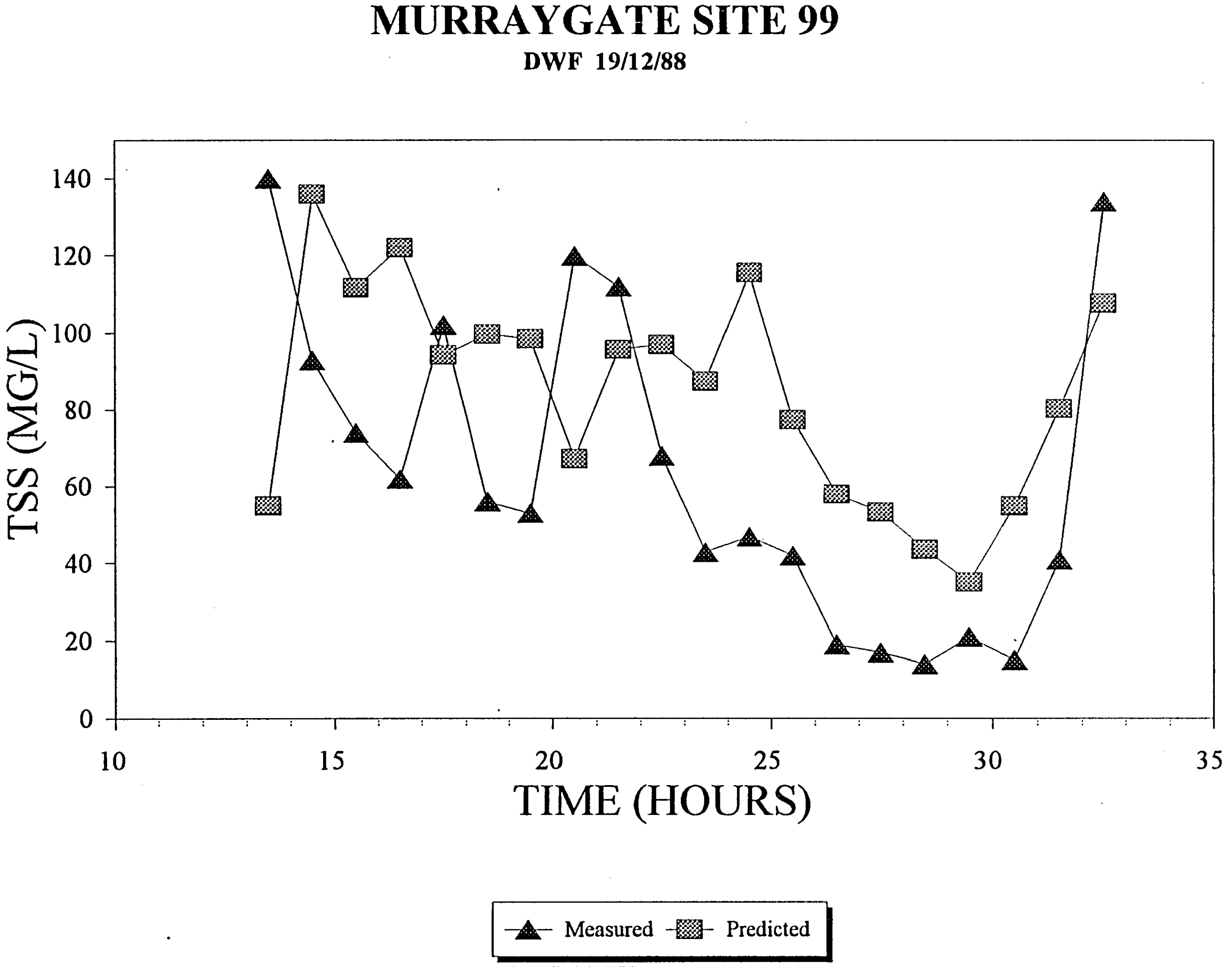


Figure 33 - Comparison of Measured and Predicted Values of TSS

7.6 Conclusions

In the preceding sections of this chapter a number of alternative models as proposed in Chapter 4 for prediction of TSS concentrations in the study sewer were considered.

By a process of calibration, selection of the best options, and validation, the following equations (equations 24 and 25) were selected:-

1 For dry weather flow

$$TSS = 955\ Q^{0.8}$$

2 For storm flows

$$TSS = 104.4 + 416.4\ V - 0.8\ TSSS - 3.124\ ADWP$$

Units:- TSS (mg/l), Q (m^3/s), V (m/s), TSSS (hours), ADWP (hours)

The methodology as discussed in section 4.2 has been shown to be satisfactory in developing the models required for the Murraygate interceptor sewer. It is therefore appropriate to consider the more general application of the modelling methodology used, as discussed in Chapter 8.

8 UTILISATION OF THE MODELLING METHODOLOGY DEVELOPED FOR MORE GENERAL APPLICATION

8.1 Introduction

The intention of the study was primarily to develop a methodology by which a site-specific model could be developed given a certain amount of data for that site. In order to test the approach developed, it was decided to apply it to data from a site on the Perth Road in Dundee which were available as a result of a separate programme of work to the field studies discussed here. It was also decided to attempt to develop a non-site-specific model which would be applicable to both sites. This work is described in the following sections.

The data available for the calibration and validation for this site are contained in Appendix R. Key details of these data are shown in Table 8 of Section 6.4.2. For a description of the Perth Road site, see section 5.3.

8.2 Site-Specific Calibration/Validation for the Perth Road Site

This was carried out in a similar manner to the procedure for the regression equations in section 7.3.4. It was not possible to apply the other approaches (i.e. Ackers and White or Sonnen and Field models) for this site due to lack of data required to calculate hydraulic gradients. The step by step tabulation of regression analysis results leading to the development of the regression equations are shown in Appendix S.

The following formulae were developed:-

DWF: $TSS = 1930\ Q^{0.48}$ (27.)

Calibration $X_{calc}/X_{obs}(0.5 \text{ to } 2) = 80.6\%$

Validation " / " " = 75.0%

Storm: $TSS = 769.1 + 9134.4\ Q - 1661.7\ Y - 1162.5\ V - 0.6247\ TSSS$ (28.)

Calibration $X_{calc}/X_{obs}(0.5 \text{ to } 2) = 95.9\%$

Validation " / " " = 87.5%

Units:- TSS (mg/l), Q (m^3/s), Y (m), V (m/s), TSSS (hours),

Details of the above calibrations and validations are contained in Appendix S .

Clearly, the equations arrived at were very similar in form to those arrived at for the Interceptor Sewer despite the apparent differences in catchment and sewer type. Also, the modelling approach used was, if anything, more successful in this case.

Graphs showing examples of measured concentration versus predicted concentration for the validated models are contained in Appendix U. An example of one of the graphs for the verification data is shown below (Figure 34):

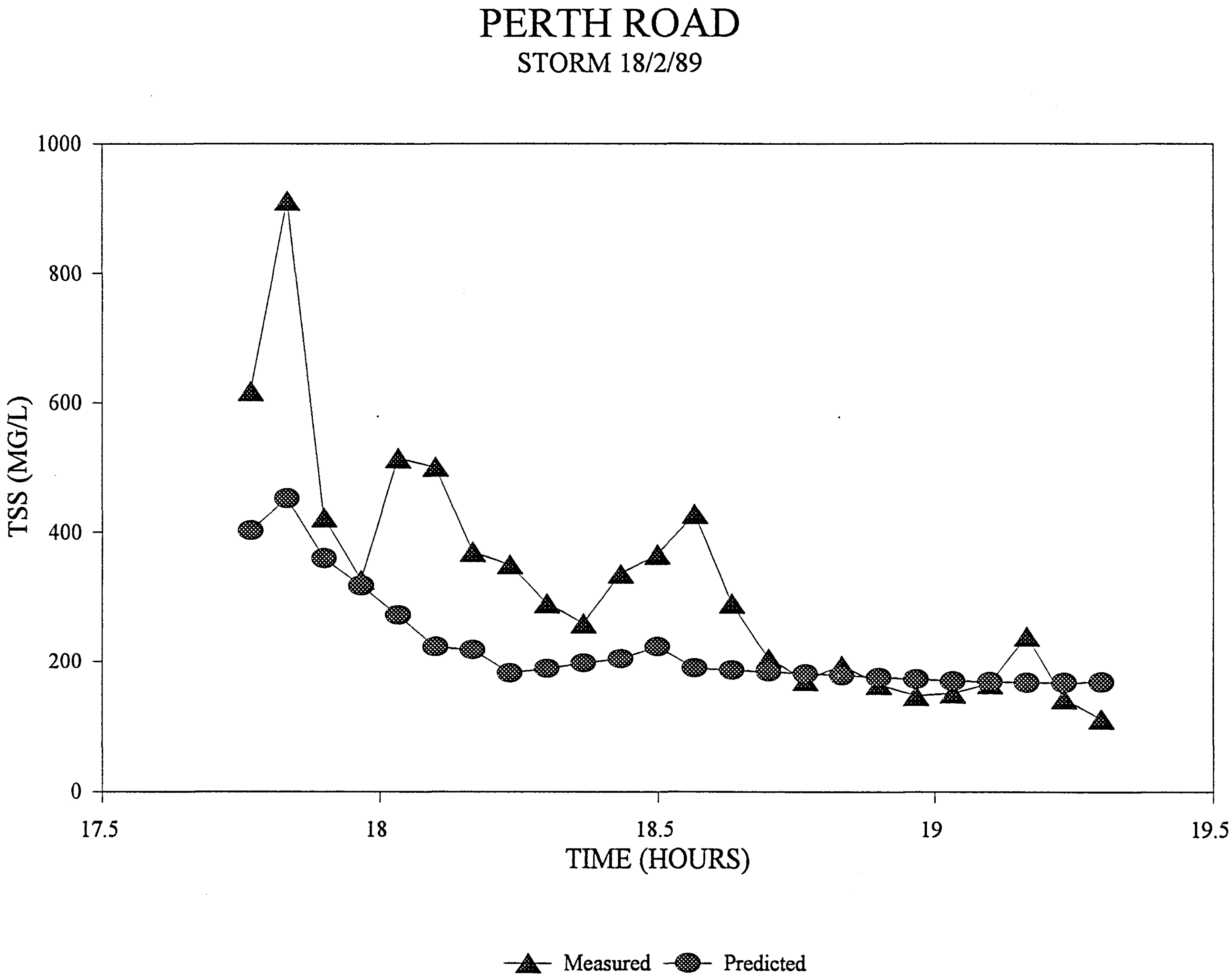


Figure 34 - Comparison of Measured and Predicted Values of TSS

A selection of data from this catchment for 8 storm events were also used with the French HYPOCRAS model (Ashley and Bertrand-Krajewski 1993). This is a conceptually based model of sewer sediment transport, which predicts flow (based on rainfall input), TSS and bed load. The HYPOCRAS model represents principally the surface derived sediment transport, with only limited application to the in-sewer sediment erosion/deposition processes. It requires site specific calibration of a number of model parameters. This model gave satisfactory results for the Perth Road catchment although it was not possible to compare the performance of this model based on the published results with the above model for storm flows.

8.3 Non Site-Specific Calibration/Validation

Since the formulae for each site were similar in form, it was decided to attempt an overall calibration of data from both sites in order to develop models which could be applied to either site, and be more generally applicable.

This was done by combining data from both sites, but in compiling the spreadsheet, some means of accounting for the different properties of the two sites was required. The details of the procedure are contained in Appendix T. Since only two sites were included, it was not possible to explore the effects of individual parameters explicitly. Indeed, it would have been possible to simply "flag" the different sites by assigning an arbitrary number to each site, eg all data for the Interceptor numbered "1" and all data from the Perth Road numbered "2". It was decided, however, that it would be more useful to use some factor which could be related to quantifiable site parameters to facilitate comparisons with data from other sites (such comparisons were not made as part of the work carried out).

The DAS (Diameter, Area and Slope) factor for classification of sewer types (Ashley et al 1992b) was selected as ideal for this application, since it uses measurable properties of particular sewer locations in order to classify the sewer type by a simple numbering system which corresponds to the following categorisation of sewer type:

- Collectors - small diameter, with the greatest relative range of flow variation, and requiring storm inputs to clean the sediment deposits which occur during dry weather
- Trunks - connect the collector sewers to outfalls or interceptors, and have steeper gradients
- Interceptors - with the slackest gradients and greatest potential for sedimentation; dry weather flows having the least range of variability

The DAS factor is calculated as follows:

$$\text{DAS} = \text{Pipe diameter (m)} \times \text{catchment area (ha)} \times 1/(\text{pipe slope}) \qquad (29.)$$

The resulting factor gives:

Collector sewers DAS < 6

Trunk Sewers 6 < DAS < 8000

Interceptor sewers DAS > 8000

For the Murraygate interceptor study sites, DAS = 7522 (i.e. approximately interceptor according to the above factor). For the Perth Road site, DAS = 120 (i.e. a trunk sewer). Generally, for the sewer system contributing to the flows in the Murraygate interceptor as a whole, the number and relative proportions of sewers for each category according to the DAS factor are as shown in Table 16 below.

DAS	SEWER TYPE	NUMBER OF SEWER LENGTHS	% OF TOTAL
< 6	COLLECTOR	50	15
6 - 8000	TRUNK	262	79
> 8000	INTERCEPTOR	20	6

Table 16 - Relative Proportions of Sewer Type in the Dundee Sewer System Contributing to The Main Interceptor at Murraygate

The DAS factors were included in the calibration procedures discussed in Appendix T, and were found to be significant for DWF prediction only. This would indicate that sewer type has less influence on suspended solids transport rates during storm flows than during DWF. The resultant equations are shown below:-

DWF:-

$$TSS = 2.47 \times 10^{4} Q^{0.55} \times DAS^{-0.45} \qquad (30.)$$

Calibration Xcalc/Xobs (0.5 to 2) = 80.6%

Validation Xcalc/Xobs (0.5 to 2) = 75.0%

STORM:-

$$TSS = 42 + 272.3 V \qquad (31.)$$

Calibration Xcalc/Xobs (0.5 to 2) = 78.2%

Validation Xcalc/Xobs (0.5 to 2) = 76.7%

Units:- TSS (mg/l), Q (m^3/s), V (m/s),

Details of the above calibrations and validations are contained in Appendix T. Graphs showing examples of measured concentration versus predicted concentration for the validated models are contained in Appendix U. An example of the comparison between measured and predicted values for the validation data is shown in Figure 35:

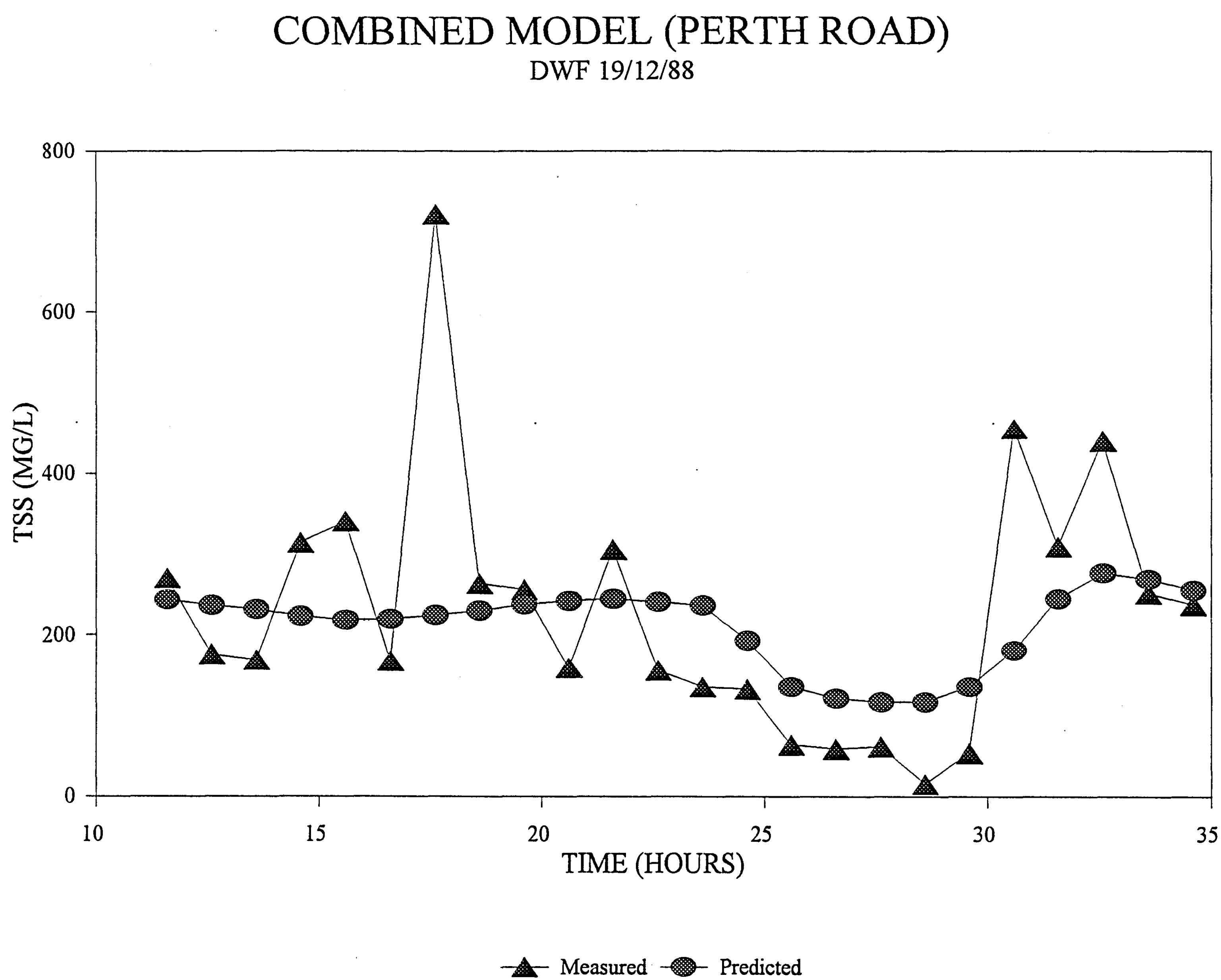


Figure 35 - Comparison of Measured and Predicted Values of TSS

8.4 Conclusions

It has been shown that the methodology proposed is applicable not only to the Interceptor study site, but also to a trunk sewer.

A non-site-specific model which is applicable to both of the sites for which data were available has been developed and has been shown to be satisfactory, despite the apparent differences between the two sites. Due to the relative simplicity and wider applicability of the non-site-specific models, and the fact that the accuracy of prediction of the site specific models was not significantly greater, the non-site-specific approach is preferable on the basis of the current study.

9 APPLICATION OF THE MODELLING METHODOLOGY

9.1 Utilisation of the Modelling Work

There are three main ways in which the work described here might be utilised:-

1) The models proposed based on the data collected in Dundee could be used directly. In the case of the sites used in the study and for which site-specific models were developed, the relevant site-specific model would obviously be the most appropriate. For other trunk or interceptor sewers either in the Dundee sewer system or elsewhere, the non-site-specific models could be used, although confidence in the accuracy of prediction would be less as a lack of similarity in terms of sewer system layout, size, areas and land use between the study sites on which the models were based and the site under examination may introduce doubts about the validity of the models in their current form. Ideally some rudimentary comparison between the values predicted and the actual TSS concentrations would be advisable.

2) Another site specific equation or equations could be developed. This would involve site data collection in order to carry out a regression analysis which would produce another site-specific equation or equations. The procedure used in order to do this is contained in Section 9.2.

3) Further development of the non-site-specific model. This approach has the greatest potential for further progress. Since the current model is based on a

limited amount of data for only two sites in one particular sewer system, the reliability of a similar model based on a much larger data base from more varied sources is likely to be greater. The model so produced could be used in situations where collection of site-specific data was not practical or desirable. Because the model would be developed from a wider range of data, a higher level of confidence could be placed on the predicted values.

9.2 Guidelines for the Use of The Modelling Methodology

The previous section suggests possible ways in which the work described in this thesis may be used. The following guidelines are applicable for the development of a site-specific model. For further development of the non-site-specific model, the procedure would be similar, but should include some parameter for variation in sewer type such as the DAS factor.

1) List the determinants which can be measured (or are available if data have already been collected) for the sewer under consideration, given practical constraints, and specify the determinant which is to be predicted by the model.

2) If field data are not available, carry out required field study, and assign data for model building/calibration and for model validation. It may also be useful to separate data into different categories (eg storm and DWF flows) and develop separate models for each category.

3) Select any existing models which require data which include a maximum of 3

"unknown" determinants (ie variables for which values must be input to the model, but for which there are no data from the sewer site). These "unknown" determinants should be constant for any likely application of the model.

4 Calibrate the chosen models from (2) using the field data available, by varying the values of the unknown determinants until an optimum fit of predicted values to measured data is achieved.

5 Carry out a regression analysis of the field data using the procedure set out in Chapter 4 in order to obtain a regression equation which predicts the required determinant.

6 Select the best option from (4) or (5) based on accuracy of prediction (best fit to field data used for model building).

7 Carry out a validation of the best option model, using remaining data assigned for this purpose.

10 CONCLUSIONS AND RECOMMENDATIONS

10.1 Main Results

The main conclusions overall that can be drawn from this research are as follows:-

1) The WRc five-fold classification for sewer sediments is satisfactory by comparison with laboratory analysis of samples. It is possible to estimate sediment type based on appearance and location, and this was shown to correlate well with chemical and physical analyses of samples. An alternative numerical "perceived" classification which takes into account the possibility of mixtures of different sediment classes has been proposed based on the sediment types found in the Dundee main interceptor sewer (see Section 6.3). This classification method was an original concept as an extension to the WRc classification.

2) An initial study of the relative proportions of near bed material transported in combined sewers during DWF suggests that the rate of near bed material transport is typically 12% of the value of the corresponding rate of transport of suspended solids. There were no figures available in the literature relating to the relative proportions prior to this study.

3) The modelling methodology proposed was used in order to select and develop the most appropriate models for suspended solids transport prediction for application at two individual sites, and for non-site-specific applications. In all cases, regression equations were selected in preference to the other options listed in

Section 10.1. These equations were as follows: equations 24 and 25 (Section 7.3.4); equations 27 and 28 (Section 8.2); equations 30 and 31 (Section 8.3). Neither the methodology proposed nor the equations developed appear in the literature. The equations developed do not require information on the existence, depth or type of sediment present.

10.2 Limitations of the Models Developed

The methodology proposed has been developed for one particular site, and tested at one other. This does not confirm that the methodology is therefore universally applicable. Equally, the non-site-specific model is not necessarily suitable for other sites. It is, however, important to state that there are no indications from the information available in this study that the methodology or the non-site-specific model developed using the methodology could not be applied to other sites. The incorporation of the DAS factor allows scope for further trials by direct application to data from other sites.

10.3 Further Work

It should be possible to extend the work described to develop a more generally applicable model by applying the approach outlined here to a much larger number and variety of data sets. In doing so, it is likely that more information would be gained from a regression of the individual determinants of diameter, area and slope rather than the combination of DAS used here. Also, the possibility of incorporating near bed material load data with data similar to the data base used in this work would be worthy of consideration since this could possibly lead to a model of total load transport.

a short time before final production of this thesis, further work by Ackers (1993) proposed a simplified version of the Ackers-White theory for solids transport. The formulae considered, however, are for the case of flow in a wide open channel and do not include allowances for the case of non-rectangular sections. There would appear to be potential for more work on the use of the Ackers-White theory for sewer flow applications based on a combination of laboratory and in-sewer data.

Finally it is important to point out that work reported here has in no way reached an end point. It is hoped, however, that the results presented will be of use in the form given, or as a small building block in the body of work that continues to be produced in relation to this topic.

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Appendix A

Bed Load Studies

CONTENTS

INTRODUCTION

It was decided that, in order to gain a more complete picture of solids transport within the interceptor sewer, some assessment should be made of the quantities and characteristics of material transported as bed load. One of the purposes of the exercise was to establish whether or not this was a significant mode of transport which could be considered to be distinct from suspended solids transport.

The time and resources available to this part of the field study were limited, and this component of the work should be considered as supplementary to the main body of the work rather than an integral part.

1 EXPERIMENTAL PROCEDURES

A novel method was evolved in order to assess bed load transport rates. This involved the construction of a temporary wooden structure in the upstream silt trap. This in-situ flume structure included panels which approximated to a continuation of the normal sewer section along the entire length of the silt trap. A series of bed-load sampling traps were set into the invert of the flume at the downstream end. Hence, bed load samples could be obtained by removal of the sampling traps following closure of an upstream gate, enabling an assessment of both the quantity and quality of the bed load material entrapped (Coghlan et al 1992).

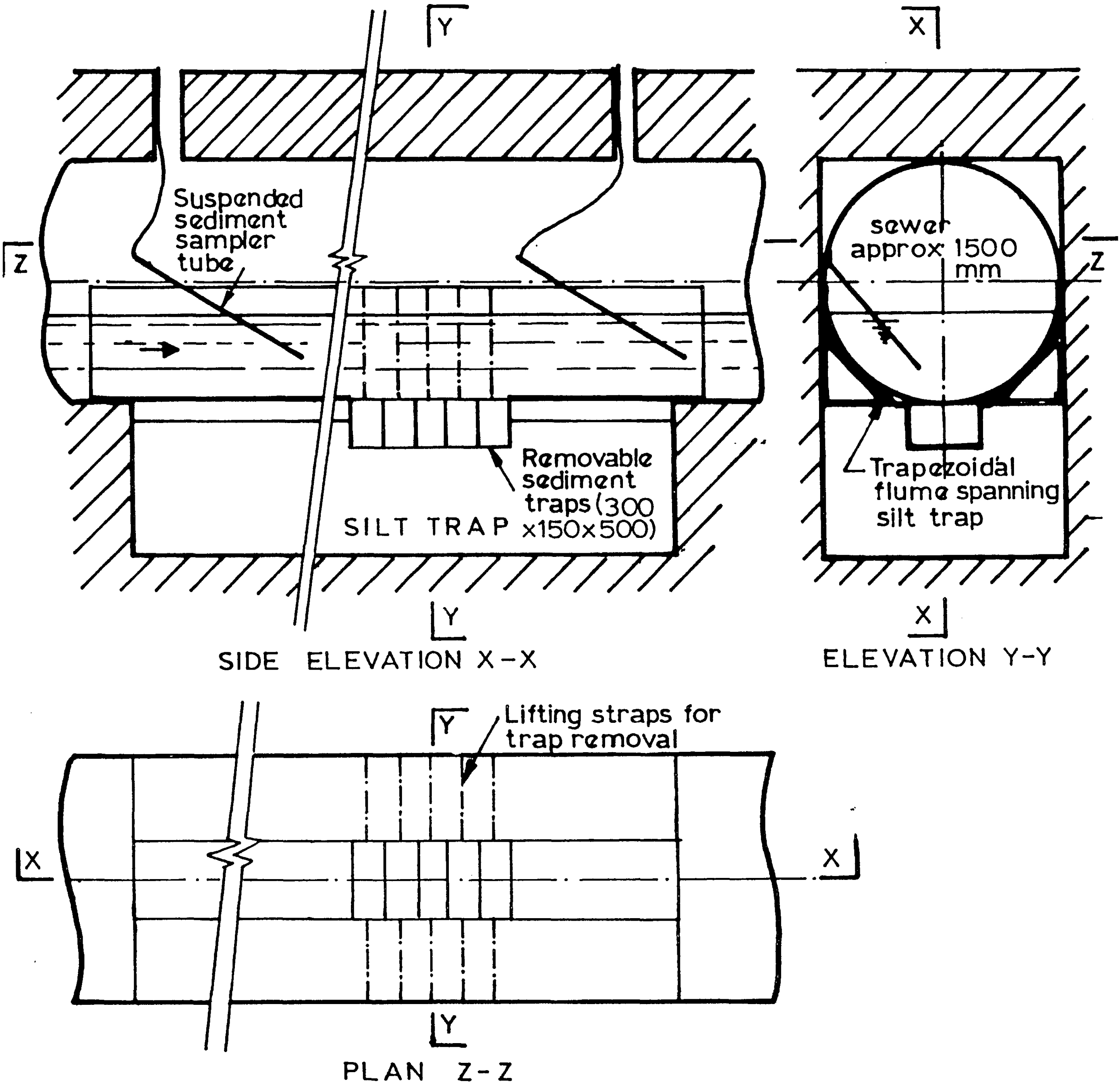


Figure A1 - Bed-Load Flume

(Source: Ashley et al (1992a))

Simultaneously with the bed load sampling, flow monitoring and sewage samples were obtained so that the bed load measurements could be seen in the context of overall flow conditions and TSS concentrations. A laboratory analysis of the bed load samples obtained was subsequently carried out. This comprised of measurement of total and volatile solids content, and concentrations of total solids, COD, BOD_5 and NH_4 - N. Bulk density was also measured.

It was possible in the case of some of the dry weather flow bed load samples to estimate the rate at which material was transported. This was done by calculating the total mass of solids collected, and relating this to the length of time over which the samples were obtained. This was only possible in the case of sample sets which did not completely fill the containers, and hence was not possible for longer dry weather flow periods, or for any of the storm periods monitored. It was noted that a higher proportion of gritty material of higher density was found in the samples associated with storm events.

Unfortunately, the amount of time available for this exercise was extremely limited due to the constraints of sewer management by TRC Water Services. However, the lack of previously reported similar measurements of this type elsewhere makes the limited information gained particularly valuable.

2 RESULTS

The data relating to the bed load studies are contained in Appendix A. Unfortunately, due to mechanical problems with the sewage samplers installed above the in-situ flume at the time of the study, no useful data on suspended solids were obtained. The bed load results were summarised by Ashley (1993b) as shown in table 1.

Class (Perceived)	Bulk Density (kg/m^3)	Total Solids (%)	Volatile Solids (%)	COD of Wet Solids (mg/kg)	AmmN of Wet Solids (mg/kg)	BOD of Wet Solids (mg/kg)	
	1.070	4.7	76.0	180821	137	46398	ave
4	1.448	2.9	97.4	336356	571	71846	max
(C)	0.972	50.2	14.7	54591	34.2	16833	min
	63	63	58	38	37	32	No
	0.079	9.67	17.9	17.9	109	16361	STD

Table 1 - Averages/Ranges Sediment Sample Results - Bed Load Class 4

The bed load data have also been summarised in a form for comparison with dry weather and wet weather maxima for sewage flows in the interceptor sewer (Ashley et al 1992a).

		DRY WEATHER SEWAGE		BED-LOAD	WET WEATHER Pollutogragh
		Summer	Winter	Average	Maxima
SOLIDS					
Total	(mg/l)	173	80-195	85880	124-1955
	(g/hd)	54	21-50	3.7	
VSS	(%)	40-60	<90	74	<98
COD					
conc	(mg/l)	517	41	193000	410-11475
	(g/hd)	163	10.4	80.8	
NH_4-N					
conc	(mg/l)	21	27-91	1700	10-116
	(g/hd)	6.6	7-23	0.71	
BOD_5					
conc	(mg/l)	143	-	49500	103-267
	(g/hd)	44.6		20.7	
FLOW	(l/hd)	315	254	0.1	7698

Table 2 - Sewage Quality: Dry and Wet Weather Compared with Bed-Load Concentration - Dundee Sewers (Source - Ashley (1993b))

Plate 1 - Silt Trap Full of Sediment Prior to Cleaning out

Plate 2 - Installing Support Beams

Plate 3 - Consruction of Framework

Plate 4 - Fitting Covering Boards in Place

Plate 5 - Installing Sample Containers

In addition to the test results indicated above, settlement velocity tests were conducted on bed load samples and compared with typical settlement velocities for suspended solids in sewage sampled from the interceptor (Ashley et al 1992a). The bed load tests were carried out using an inverted tube technique, whereas the sewage tests utilised a standard settling column. The results of these tests showed a considerable variation in particle settlement velocity with between 50 and 80% of the particles having a settlement velocity greater than 15mm/s (McGregor and Ashley 1990). In comparison only some 10% of DWF sewage particles have a settling velocity greater than this rate (Ashley 1993b).

3 DISCUSSION

The average solids load conveyed as bed load is 3.7g/head per day (maximum 9.8g/head per day), which when compared with average winter rates of transport for suspended sediment of 28g/head per day, represents some 12% of the total solids in transport (Asley et al 1992a). The bed load phase of solids transport thus constitutes a large contribution to the total sediment and pollutant transport.

From assessment of the particle size ranges of the inorganic bed-load fraction compared with data for other sewage and sediment particles, and also from inspection of the relatively high values of COD, BOD and Amm N concentration, it has been proposed that the bed load material originates from foul inputs rather than from surface or groundwater sources (Ashley 1993b). This contention is based on the fact that the bulk of gully input material is inorganic and has a larger typical particle size.

The bed load material corresponds to type C deposits which overlay type A material in the interceptor sewer (Ashley et al 1992a). These have been found to have low yield strength due to their dilute nature and are likely to be readily eroded as a first flush.

4 CONCLUSIONS

The bed load studies described here were necessarily limited in their scope due to time constraints but have provided useful information on the nature and quantities of material transported as bed load.

The initial work on the technique of utilising an in-situ flume for bed load measurement described here has subsequently been the subject of development work in University of Abertay Dundee. Further refinements have been made and similar field work has been carried out on other sewers in Dundee (Ashley et al 1993a).

Appendix B

Sediment Transport Theory

CONTENTS

1 INTRODUCTION

A great deal of fundamental research has been carried out by many respected authors over the years with the purpose of predicting sediment transport due to fluid motion in a number of situations. These situations include rivers, estuaries, canals and conduits. There is a considerable amount of information relating to sediment transport rates in these applications which has facilitated the development of models which have gained wide acceptance (CIRIA 1987, Bertrand-Krajewski 1992).

This is not the case, however, for sediment transport prediction in sewers. Much work has been done in recent years to rectify this, including the utilisation of existing models previously intended for other applications, either in their original or modified forms (Kleijwegt 1992). Also, the development of new models specifically for sewer applications has been undertaken. Where models have been developed or adapted, this has been done largely on the basis of laboratory studies.

These studies have normally been carried out using granular synthetic sediments in pipes or flumes under controlled conditions. Tests have usually been carried out under steady flow conditions. The sediments used are normally homogeneous, often single sized material. In most cases the material used is non-cohesive, with some notable exceptions (Alvarez-Hernandez 1990). Whilst this work is important in improving the understanding of the basic mechanisms which should be considered in in-sewer sediment transport processes, there are a number of factors which limit the applicability of models which are adapted or developed solely on the basis of laboratory tests. These are as follows:

1) The synthetic sediment does not necessarily behave in the same way as sewer sediments. As stated previously, the synthetic sediment is homogeneous, often single sized and usually non-cohesive, whilst the real sediment in a combined sewer is normally highly variable in nature with well graded size fractions and often exhibits cohesive properties (CIRIA 1987). Also, real sediments may be highly organic, giving rise to temporal changes in the physical properties exhibited due to biological activity, while surrogate sediments are inert. The term "combined sewer" and the implications of sewer type for the properties and transport of sewer sediments is discussed in Chapter 3 of the main text.

2) Flow regimes within a combined sewer are unsteady and non - uniform, particularly during storm events. In contrast, laboratory studies tend to concentrate on steady flow conditions. Not only does this affect circumstances hydraulically, but in the case of a sewer with deposits upstream, the nature of the material being transported under different flow conditions will alter (Verbanck 1990). Hence there is an effect on the rates of transport due to the "flow history" both temporally and spatially: if flowrates are decreasing or increasing with time, deposition or erosion may occur upstream of a point in a sewer network being considered, affecting the quantities of material arriving at the point being considered. Similarly, if the gradient or geometry of a sewer changes as storm flows travel downstream, there will be a corresponding change in the transport capacity of the sewage flow, again determining the amounts of material arriving at a particular point

in the system.

3) Models originally developed for applications other than in-sewer work generally assume that the specific gravity of the transported material is close to that of quartz, i.e. 2.65 approximately (Verbanck and Ashley 1992). Frequently, much of the material suspended in sewer flows has a specific gravity close to unity.

4) Natural channels have erodible boundaries, while sewers have fixed boundaries within which there may be an erodible bed deposit.

5) As a consequence of 4), a natural channel may have an unlimited source of erodible material. This is not the case for sewers.

The following literature review considers the classical theories of sediment transport in as much as they may have some bearing on sediment transport modelling in this particular application. An appraisal of the various sediment transport models available is made in Chapter 2 of the main text with a view to selecting models which may be appropriate for assessment in terms of their performance using the data collected for the study site. The selection of suitable models is undertaken in Chapter 4 of the main text.

2 SEDIMENT DEPOSITION

The origins of particles in suspension, the processes involved in the transport of solids through the sewer system upstream of a particular point, and the flow conditions prevalent at that particular point in a sewer system, all have an influence on the behaviour and constituents of material in suspension in the sewage flow (CIRIA 1987). The degree to which suspended material may settle out depends upon a combination of the physical characteristics of the material and the flow conditions present.

The simplest case that can be considered is that of spherical particles falling through infinite fluid in quiescent conditions. For any given combination of fluid properties, sphere density and diameter, there is a corresponding terminal velocity at which the sphere will fall through the fluid. At this velocity, the viscous drag on the sphere is balanced by the submerged weight of the particle. Stokes Law is applicable to fine particles with relatively low fall velocities, where $R_e < 0.1$, and can be used as an approximation up to $R_e = 1$ (Garde and Ranga Raju 1985):

$$\omega_t = \frac{d_s^2}{18\mu}\left(\gamma_s - \gamma_f\right) \qquad \text{(B1.)}$$

where d_s = diameter of sphere

w_t = terminal velocity of a sphere (theoretical)

μ = dynamic viscosity of fluid

Above the range of Re values stated by Stokes, it is not possible to apply a purely analytical solution, therefore an empirical constant, the so called coefficient of drag

C_D is introduced.

According to Isaac Newton (Garde and Ranga Raju 1985):

$$F = \frac{1}{2} C_D A_p \rho_f \omega_a^2 \tag{B2.}$$

where F = viscous resistance

A_p = projected area of particle

w_a = terminal velocity of a sphere (actual)

ρ_f = density of fluid

Also, according to Stokes

$$F = 3\pi d_s \mu \omega_t \tag{B3.}$$

Therefore

$$3\pi d_s \mu \omega_t = \frac{1}{2} C_D A_p \rho_f \omega_a^2 \tag{B4.}$$

$$\Rightarrow C_D = \frac{24\mu}{\omega_t d_s \rho_f} = \frac{24}{R_e} \tag{B5.}$$

(for $R_e < 1$ approx)

Hence, since $$\omega_t = \frac{24\mu}{C_D d_s \rho_f} \tag{B6.}$$

then combining with equation (B1.)

$$\omega_t = \sqrt{\frac{4d_s(\gamma_s - \gamma_f)}{sC_D\rho_f}} \tag{B7.}$$

For higher values of R_e, equation (B7) may be used provided the value of C_D employed takes into account inertia effects (Graf 1984).

A number of approximate solutions to the value of C_D are given in the literature. These may be theoretically based (Stokes, Oseen, Goldstein, Proudman et al) or empirical formulae based on experimental results such as those proposed by Schiller et al, Dallavalle, Langmuir et al and Olsen (Graf 1984). Each of these is applicable within specific ranges of Reynolds number. Generally for $0.5 < R_e < 10000$, Garde and Ranga Raju (1985) suggest the following expression as a suitable compromise:

$$C_D = {}^{24}\!/_{R_e} + {}^{3}\!/_{\sqrt{R_e}} + 0.34 \tag{B8.}$$

The above discussion considers only spherical particles. If the particles are not spherical, then the shape of the particles must be taken into account since this will have a significant effect on settlement velocities. The effect of particle shape should be considered separately for $R_e < 0.1$ and $R_e > 0.1$.

If $R_e < 0.1$, then

$C_D = 24/R_e$

for a sphere

and

$C_D = 20.37/R_e$

for a circular disc (falling with its maximum cross sectional area normal to the direction of motion).

These two shapes represent two extremes of particle shape, and as such are very close in value. McNown and Malaika (1950) give a range of values between these extremes based on the dimensions a_m, b_m and c_m of an equivalent ellipsoid. The dimensions a_m, b_m and c_m are taken along three mutually perpendicular axes. The longest or major axis is a_m, b_m is the intermediate axis and c_m is the shortest or the minor axis.

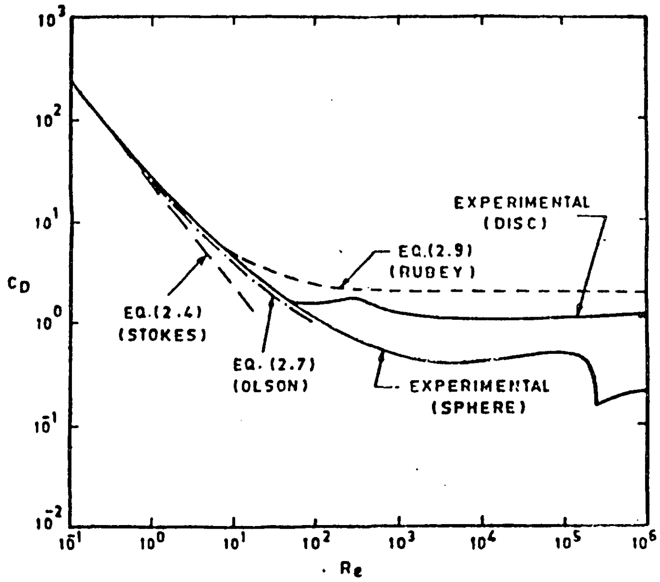


Figure B1 - Variation of C_D with R_e for a Sphere and a Disc

In the case where Re > 0.1, the shape of the particles is very important, and the only stable orientation of a particle is that with the maximum cross sectional area normal to the direction of motion. Hence

$$c_m \big/ \sqrt{a_m b_m} = \text{shape factor} \tag{B9.}$$

is considered a suitable shape factor for use in the prediction of fall velocity (Garde and Ranga Raju 1985). Experimental results attributed to Albertson relating C_D to R_e for various values of equation (B9.) therefore give improved accuracy compared with calculations based on the assumption of spherical particles (Graf 1984). In the absence of data relating to natural particles, it is recommended that a shape factor of between 0.6 and 0.7 be used in conjunction with sieve diameter to estimate fall velocity (Garde and Ranga Raju 1985). It should be noted that the shape factor used

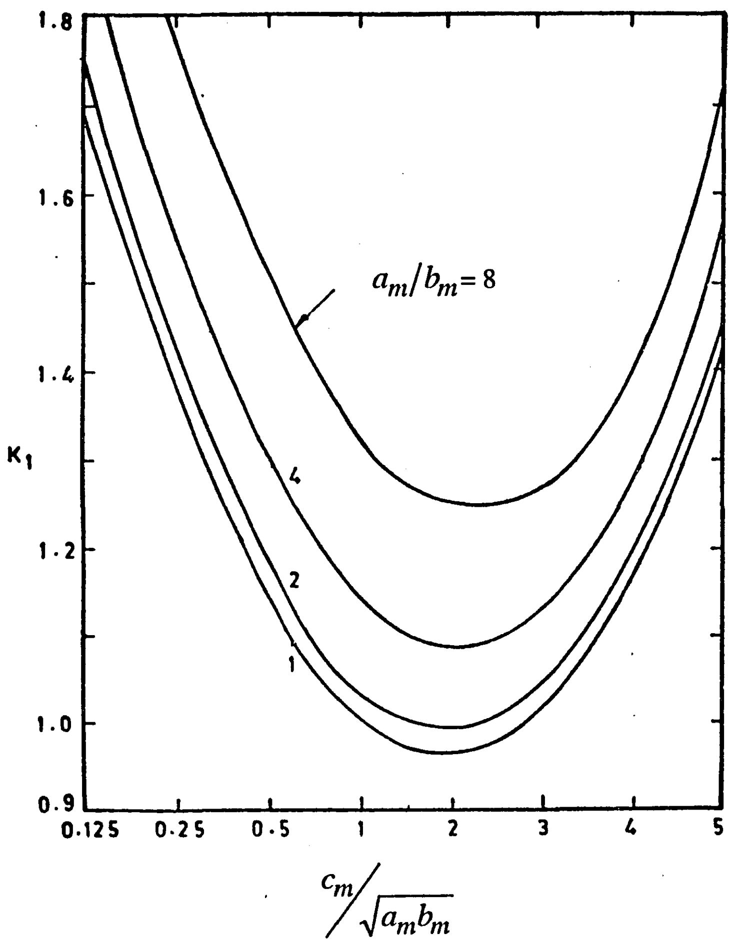


Figure B2 - Variation of K_1 with $c_m/\sqrt{a_m b_m}$ and a_m/b_m for Ellipsoids

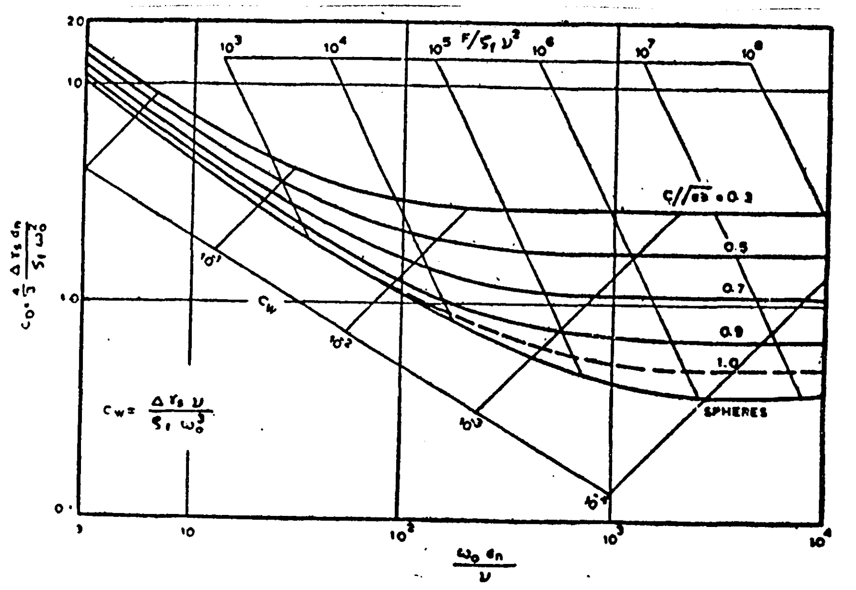


Figure B3 - Relationship between C_D , R_e and Shape Factor

does not take into account the distribution of surface area and volume of the particle, since a cube of side d and a sphere of diameter d will both have a shape factor of 1, but will have different types of motion and different rates of fall. However, the effects of this type of variation are not yet fully understood, and in any case are unlikely to be important when the particles under consideration are of random shape, as is found in sewage flows.

It is not always possible to assume that settlement takes place in an infinite fluid. This is certainly not the case for in-sewer applications. Boundary proximity must therefore be considered in the prediction of the fall velocity of particles. The actual fall velocity (ω) may be less than the theoretical value (ω_0) . Various formulae for the calculation of the correction factor

$$K_v = {}^{\omega_0}\!/\!_{\omega} \qquad \text{(B10.)}$$

have been proposed by Bremmer, McNown and Happel et al for various situations (Graf 1984).

In situations where more than one sphere is falling through a fluid medium, a mutual interaction will be observed. If the extent of the fluid is effectively infinite, as is the case when there are a few closely spaced particles situated well away from boundary effects, then a downward motion will be induced in the fluid immediately surrounding the particles, thus increasing downward velocity of the particles. However, if the particles are dispersed, then downward movement of fluid in close proximity to the particles is balanced by upward movement of fluid at some distance from individual particles. Thus, the settling velocities of the particles in this case will be decreased. The latter phenomenon is known as hindered settlement (Graf 1984). Various factors which account for the respective decreases or increases in velocities are proposed by Smoluchowski, McNown and Lin, Maude and Whitmore and Happel et al among others, (Garde and Ranga Raju 1985, Graf 1984).

The above discussion considers the fall of particles in a still fluid and is therefore only of limited interest in relation to sewage flow in sewers. No account is taken of the effects of turbulence, particle roughness or particle rotation, all of which are present due to the flow conditions in a sewer. Very little work has been done on assessing the influence of these factors, and the work which has been undertaken is inconclusive. Also, in the context of combined sewers, the potentially cohesive nature of the material is likely to give rise to flocculation in the velocity gradients present, which in turn will alter the settling properties of the material (Krone 1986). In addition, although each factor has been considered separately in this discussion, all or some of the factors could act simultaneously.

Finally, since the material in suspension in a combined sewer is highly variable in nature, with a number of unquantifiable effects (at present) such as concentration gradients and cohesion, a purely theoretical solution to the problem of prediction of rates of settlement in this context is not feasible given the current limits of knowledge of the mechanisms involved. There is therefore a need for empirical or stochastic methods of prediction.

3 ENTRAINMENT OF SOLIDS

The term entrainment refers to the process by which stationary particles located on the bed of a channel are caused to move and are subsequently picked up into the main flow. This occurs as a result of forces exerted on the particles due to the movement of the overlying fluid (e.g. (CIRIA 1987)).

The point in time and space at which entrainment commences will be dependant on the magnitude of hydrodynamic forces exerted by the fluid increasing until a situation is reached where these forces are greater than the restoring forces acting on the particles. Hence the particles become dislodged and start to move with the direction of flow (Graf 1984). The increase in the hydrodynamic forces corresponds to an increase in velocities near the bed, although this may not be directly related to the forces on the particles (Ashley et al 1992b). Other possible measures of the point of commencement of entrainment which may be considered are bed shear stress and mean velocity of flow respectively. The advantage of the former is that it is a more direct measure of the forces exerted on the particles, while the latter can be measured more easily, but is indirect. In all of these cases, the reason for further discrepancies between changes in hydraulic conditions and changes in forces actually exerted on individual particles may in part be due to unmeasured local fluctuations and turbulence (Graf 1984).

There are a number of terms used to describe the commencement of entrainment, with various definitions of the conditions they represent. As previously stated, CIRIA (1987) simply use the term entrainment, which is defined as the process of initial movement of sediment and its pick up into flow. This corresponds to some initial erosion velocity or critical shear stress. Garde and Ranga Raju use the term critical or incipient motion, which may refer to any of the following criteria:

(i) A single particle moving
(ii) A few particles moving
(iii) General motion on the bed
(iv) Limiting condition when the rate of sediment transport tends to zero.

May (1982) lists a number of possible critical velocities for movement, scour, deposition and transport of particles. Of these there are two which may be considered as possible indications of the commencement of entrainment:

(i) Threshold velocity V_t - the average flow velocity at which isolated particles just start to move.

(ii) Critical Scour Velocity V_{CS} - the average flow velocity required to scour a deposited bed.

Of these, V_{CS} is difficult to define, since it is also necessary to specify the rate at which deposits are scoured.

Other terms mentioned in the literature which define the conditions under which entrainment is deemed to have commenced include critical condition and initial scour (Graf 1984).

According to CIRIA (1987), the disturbing hydrodynamic forces acting on the particles fluctuate with time, due to the production and decay of eddies within the flow. It is suggested that the prevalent forces are due to a combination of the following factors:

(a) Pressure Gradients - in turbulent flow conditions, more exposed particles on the bed create wakes behind them which create a pressure difference across the grains. Depending on the intensity of the turbulence and the stability of the grain on the bed, this may dislodge the grains.

(b) Viscous Forces.

(c) Forces due to seepage into or out of the bed.

(d) Impacts due to other particle motions.

When the material in the bed is non-cohesive, it is mainly the submerged weight of the particle and any interlocking with other particles which resists motion. However cohesion, or effectively cohesive bonding, may occur due to the presence of fine material. This cohesion, together with any cementation and agglutination due to the presence of organic materials and chemicals can have a major effect on the ability of the bed deposits to resist erosion (Stotz and Krauth 1986, Garde and Ranga Raju 1985). This is discussed in Section 5. The simpler case of non-cohesive bed deposits is dealt with in this section.

There are three main predictive methods for entrainment which are referred to in the literature; critical velocity, lift force criteria and critical tractive force. Each of these is dealt with in turn.

3.1 Critical Velocity

Various terms have been used in the literature to refer to some measure of velocity of flow which corresponds to conditions of incipient motion, including the following (Garde and Ranga Raju 1985, Featherstone and Nalluri 1988, Graf 1984):

(i) Competent mean velocity - the mean velocity which is just able to move material of a given size and specific weight;

(ii) Competent bottom (or bed) velocity - the bed velocity which is just able to move material of a given size and specific weight;

(iii) Critical velocity - similar to competent velocity, and may also refer to either mean or bed velocities;

(iv) Permissible canal velocity - the maximum average velocity for which there is no objectionable scour in the bed of a canal;

(v) Non displacement velocity - the highest average flow velocity at which bed particles are not displaced and at which the maximum value of the fluctuating lift force does not exceed the weight of the particles in water;

(vi) Detachment velocity - the lowest average velocity at which individual particles from the bed continually become detached and at which the fluctuating lift force is equal to the weight of the particles in water.

This concept of relating velocity of flow to particle size has been applied in various forms by different investigations since it was first used by Du Buat in 1786.

In the case of cohesionless particles on a loose bed, in terms of the forces acting on an individual particle, the condition of incipient movement is described by

$$\tan\phi = \frac{F_t}{F_n} \qquad \text{(B11.)}$$

where F_t = force normal to the angle of repose ϕ

F_n = force parallel to the angle of repose ϕ

If these forces are considered to be the resultants of the hydrodynamic drag F_D, the lift force F_L and the submerged weight of the particle w_S, then the condition of incipient movement under the action of these three forces becomes

$$\tan\phi = \frac{w_s \sin\alpha + F_D}{w_s \cos\alpha - F_L} \tag{B12.}$$

where α = inclination of the bed from the horizontal.

The drag force can be expressed by

$$F_D = C_D k_1 d^2 \frac{\rho_f {u_b}^2}{2} \tag{B13.}$$

where k_1 = particle shape factor

u_b = fluid velocity at the bottom of the channel

Similarly, for lift force

$$F_L = C_L k_2 d^2 \frac{\rho_f {u_b}^2}{2} \tag{B14.}$$

where k_1 = particle shape factor

C_L = lift coefficient

Also, if

$$\omega = k_3(\rho_s - \rho) g d^3 \tag{B15.}$$

where k_3 is another shape factor, then according to Graf (1984), by introducing equations (B13.), (B14.) and (B15.) into equation (B11.):

$$\frac{\left({u_b}^2\right)_{CR}}{\left(\rho_s/\rho - 1\right)} = \frac{2k_3(\tan\phi\cos\alpha - \sin\alpha)}{C_D k_1 + C_L k_2 \tan\phi} \tag{B16.}$$

where $(u_b)_{CR}$ = critical bottom velocity

α = inclination of the bed from the horizontal

ϕ = angle of repose of bed material

In practice, it is very difficult to confirm the validity of equation (B16.) experimentally due to its complexity, and thus far this has not been achieved.

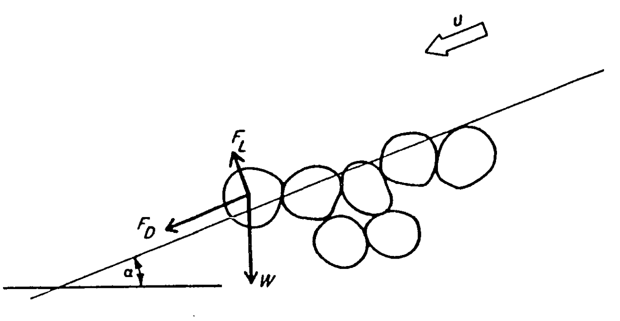


Figure B4 - Force Diagram on Particles in a Cohesionless Loose Bed

A number of investigators have attempted to obtain empirical relationships between velocity and incipient motion. Because of the difficulty in defining or measuring bottom velocity, these relationships frequently relate to mean velocity of flow or even surface velocity. In 1926, Fortier and Scoby published the results of a report based on the results of questionnaires sent to a number of engineers experienced in the design and construction of unlined channels. Based on the assumption that the experience of the engineers qualified them to make authoritative estimates of the maximum mean velocities allowable in canals of various materials, the report included a table of permissible canal velocities for a range of bed materials (Featherstone and Nalluri 1988).

Other investigators have attempted analyses which are based more directly on the characteristics of the particles of bed material. Hjulstrom presented the results of a detailed analysis of data on erosion, transportation and deposition of "mono-disperse material on a bed of loose material of the same size of particles" in graphical form, as shown in Figure B5. The plot of $\bar{u}$, the average flow velocity, versus d_p, the particle diameter, shows a narrow band denoting a zone of demarcation between transportation and erosion which corresponds to conditions of incipient motion (Graf 1984). The results of this study indicate that $\bar{u}$ decreases with decreasing particle size until a value of $d_p \cong$ 0.25mm. Below this value of d_p, resistance to erosion increases with decreasing particle size, suggesting that cohesive forces become significant in the smallest particle range.

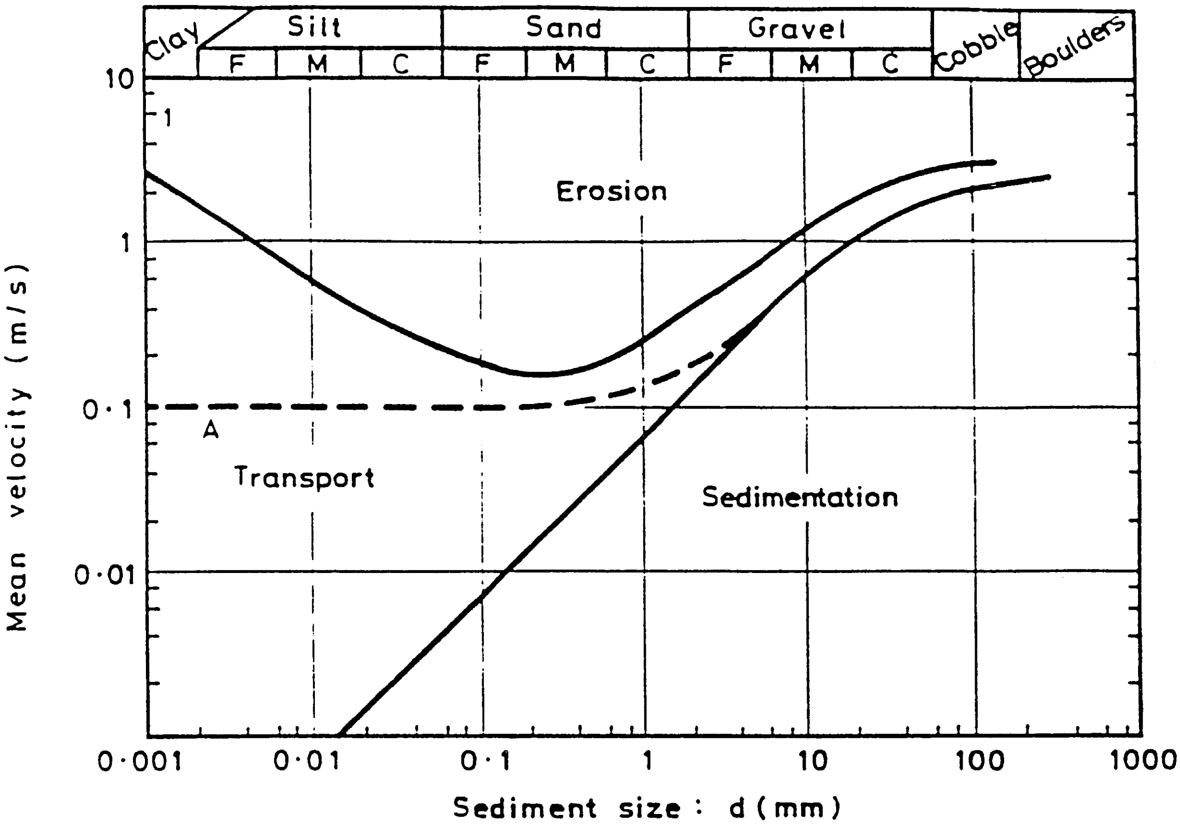


Figure B5 - Critical Erosion/Sedimentation Boundaries (after Hjulstrom and Postima)

Many researchers have proposed empirical equations which predict initial velocities. As early as 1753, Brahms advanced an equation relating the critical bottom velocity to the weight of the particle (Garde and Ranga Raju 1985). This takes the form

$$(u_b)_{CR}{}^6 = k_4 w_s \qquad \text{(B17.)}$$

where k_4 = constant

which may be considered as a simple form of equation (B16.). This type of equation which relates to particle weight is proposed by a number of authors, while others choose to relate critical velocity to particle size. An example of this approach is the relationship proposed by Sternberg (Graf 1984).

$$(u_b)_{CR} = \xi\sqrt{d} \tag{B18.}$$

More complex equations which take into account a combination of particle size, specific gravity and other factors such as depth of flow, bed slope and angle of repose of bed material are referred to in the literature. (Graf 1984, Garde and Ranga Raju 1985). Typical of these is the equation developed by Garde which is in the form

$$\frac{u_{CR}}{\sqrt{(\gamma_s - \gamma_f)\, d/\rho_f}} = 0.50\log\left(Y/d\right) + 1.63 \tag{B19.}$$

and the formula put forward by Carstens, where

$$\frac{\left(u_b^{\,2}\right)_{CR}}{\left(\rho_s/\rho - 1\right)gd} \cong 3.61(\tan\phi\cos\alpha - \sin\alpha) \tag{B20.}$$

where ρ_s = density of solids

Novak and Nalluri (1984) analysed data based on the results of a series of experiments examining incipient motion of single and grouped particles for flow in flumes over fixed beds. A general equation of the form

$$\frac{V_c}{\sqrt{gd(s_s - 1)}} = a_n\left(d/R\right)^{b_n} \tag{B21.}$$

was proposed. A tabulated range of values for the coefficients a_n and b_n was included which covered the situations of rough, smooth and moveable beds with single or groups of particles.

While many of the published relationships between incipient motion and critical velocities have shown reasonable results for particular applications, there are a number of simplifications inherent which limit their uses. More importantly they all rely on the ability either to measure directly the near bed velocity, assuming this can be adequately defined, or an implied relationship between the bed velocity and mean velocity is assumed. Neither situation is wholly acceptable. According to CIRIA (1987) a relationship between mean and "near-bed" velocities is not fixed in practice and in any case, a direct relationship between bed velocity and entrainment of particles has not been conclusively shown.

3.2 Lift Force Criteria

Although a component of lift force was included in the considerations for the basic equation of scour (equation (B9)), the subsequent derivations do not refer explicitly to lift, and the magnitude of these forces were not specifically discussed.

When a particle rests on a fixed bed, the velocity at the bottom is zero, while the velocity at the top of the particle is greater than zero. Using classical hydrodynamics, Jeffreys showed that a two dimensional cylinder resting on the bed in an infinite ideal fluid with its major axis perpendicular to the flow will be lifted (Graf 1984) if:

$$\left(\tfrac{1}{3}+\tfrac{1}{9}\pi^2\right)u_\infty^{\,2} > \left(\frac{\rho_s-\rho_f}{\rho_f}\right)gr \qquad \text{(B22.)}$$

where r = radius of cylinder

u_∞ = free stream velocity

For a spherical particle of the same diameter, there will be a smaller lift force for the same velocity because some of the fluid will pass under and around the particle. This reduction in forces could be accounted for by some modification factor, but the major drawback of this theoretical model is that it does not take into account the effects of turbulence and fluid viscosity, and hence drag. A similar model proposed by Reitz includes the effects of circulation and viscosity, while Lane et al strongly emphasise the role of turbulence in the determination of lift. Lane's work led to the further development of this model by Kalinske (1947) as part of his work on the movement of bed load (see section 4).

A series of experiments by Einstein and El-Samni (1949) in which the average lift force measured directly from beds composed of large diameter (d = 69mm) particles resulted in the equation

$$\Delta p = C_L \rho_f \, {u_{35}}^2 / 2 \qquad \text{(B23.)}$$

where Δp = lift force per unit area of the particle

u_{35} = velocity at a distance $0.35 d_{35}$ from the bed

which is similar in form to equation (B11.). The lift coefficient C_L was found to be a constant value of 0.178. The results of this study were used by Vanoni to calculate the ratio $\Delta p / \tau_0$ which he found to be approximately 2.5 (Graf 1984). This gives a strong indication that lift forces are significant in the initial movement of particles. Conversely, in a theoretical analysis of critical tractive force by Iwagaki, the inclusion or omission of lift force does not have any significant effect on the calculated value of critical tractive force (Garde and Ranga Raju 1985).

Once a particle has been displaced and moves upwards due to lift forces, the velocity difference between top and bottom of the particle will be reduced. As the particle rises, therefore, lift forces will tend to diminish and drag forces to increase (Chepil 1961). After reaching a certain height, the lift acting on a particle becomes less than its submerged weight, and the particle starts to fall. During the period of rising and falling, the particle is transported forward due to drag forces. Once the particle comes to rest on the bed once more, the whole sequence can be repeated. The particle therefore travels along in a series of curved paths (Garde and Ranga Raju 1985).

The results of studies which attempt to evaluate the contribution of lift force to incipient motion of particles from a sediment bed have been inconclusive to date. There has been no critical lift criterion thus far established which could be used in a similar way to critical velocity or critical shear stress criteria (Graf 1984). Moreover, since it is clear that where lift forces are present there are also drag forces acting on the particles, equation (Einstein and Krone 1961) from which the critical velocity equation (French 1985) was derived, seems to include all the important forces. Hence, present knowledge of the separate role of lift in the transport of sediment may be limited, but the critical shear stress equation discussed in the previous section implicitly includes the effects of lift. It will be seen in the following section that this is also the case for similar reasons in the use of critical stress equations.

3.3 Critical Shear Stress

Of the three approaches to the problem of defining the conditions under which entrainment of sediment particles commences, it is the critical shear stress or critical tractive force approach which has been most widely adopted (Garde and Ranga Raju 1985). This approach considers the system of drag and lift forces due to the flowing fluid acting on a sediment particle resting on the bed, and the submerged weight of the particle. It should be noted that it is not necessary in this approach to consider the drag and lift forces separately, since these are known to depend on the same dimensionless parameters involving flow, fluid and sediment characteristics.

In practice, the tractive force measured or predicted acts on an area of the sediment bed rather than on a single particle. This has been stated by both du Boys and by Lane (Featherstone and Nalluri 1988). This tractive force can be equated to the gravitational force due to the weight of overlying water resolved in the direction parallel to the channel bed. Hence for a given control volume

$$F_T = \gamma_f A_c L S_c \qquad \text{(B24.)}$$

where A_c = channel cross-sectional area

L = control volume length

S_c = longitudinal slope of channel

The unit tractive force is therefore

$$\tau_0 = \frac{\gamma_f A_c L S_c}{PL} = \gamma_f RS_c \qquad \text{(B25.)}$$

where τ_0 = average value of tractive force per unit of wetted area, or unit tractive force, or tractive stress

P = wetted perimeter of channel cross section

The above equations assume that shear stresses are uniformly distributed over the wetted perimeter. This is not necessarily the case. For unlined trapezoidal channels, where the material of which the side slopes is formed is similar to that of the bed, Lane (1955) suggests that the maximum tractive stress on the bottom is approximately $\gamma_f\, y_n$ (where y_n = depth of flow) while on the sides it is $0.76\gamma_f\, y_n S_c$. More generally, Einstein (1942) proposed a procedure in which the hydraulic radius of the bed R_b can be calculated. The method assumes that velocities are uniform over the whole cross section, and that the total flow can be divided into areas corresponding to the bed and sides, hence

$$A_c = A_w + A_b \tag{B26.}$$

where A_b = area corresponding to the bed

A_w = area corresponding to the sides

If the roughness of the walls is known, then using Manning's equation, R_w (the hydraulic radius of the walls) can be calculated. Hence, R_b is calculated. The unit tractive force acting on the bed is then calculated as

$$\tau_0 = \gamma_f\, R_b S_c \tag{B27.}$$

Many researchers have attempted to establish experimentally relationships which predict the critical shear stress at the point of incipient motion $(\tau_0)_{CR}$. Schoklitsch (Graf 1984) proposed the following equation

$$(\tau_0)_{CR} = \sqrt{0.201\gamma_f\left(\gamma_s - \gamma_f\right)\lambda'\bar{d}^3} \qquad \text{(B28.)}$$

where $\bar{d}$ = the mean grain diameter

λ' = particle shape coefficient

A number of similar equations were subsequently presented by others including Krey, Eisner, Nemenyi and O'Brien et al (ibid).

As an improvement to the above approach, Kramer suggested that the grain composition of the bed be described by a combination of mean grain diameter d and a grain distribution modulus M, given by the ratio of specified areas F_A/F_B taken from the graph of particle size distribution (Graf 1984). On the basis of data from a number of experiments using quartz grains of relative density 2.7, this approach resulted in the formula (Garde and Ranga Raju 1985):

$$(\tau_0)_{CR} = \frac{10^{-4}}{6}\left(\gamma_s - \gamma_f\right)^{d_g}\!\!\Big/_{M} \qquad \text{(B29.)}$$

where d_g = mean grain diameter

This was subsequently modified by Tiffany et al (Graf 1984) in the light of further work. Further equations of this type followed, notably from the United States Waterways Experiment Station and Chang, while others incorporate some method of accounting for the effects of variation of size distribution in similar equations (Garde and Ranga Raju 1985).

There are a number of similarities between the various empirical formulae developed for prediction of critical tractive stress. They all indicate an increase of critical tractive stress with increasing $(\rho_s - \rho)/\rho$, and also that $(\tau_0)_{CR}$ is proportional to d_n although the value of n varies widely (Garde and Ranga Raju 1985). However, although the physical characteristics of the bed material are to a certain extent taken into account, the formulae do not explicitly account for the effects of viscosity. Also, the results produced using different formulae differ from one another considerably. This may partly be explained by the difficulty in defining the conditions of incipient motion. Some workers defined the beginning of motion as the condition of isolated movement of particles, while others have based measurements on the condition of appreciable or general movement of particles.

Further advancements in fluid mechanics have suggested the desirability of relating turbulent flow conditions to the friction or shear velocity u_* , where

$$u_* = \sqrt{\tau_0 / \rho} \tag{B30.}$$

The term u_* is a measure of the intensity of turbulent fluctuations in the flow. This term was first used by Shields (Graf 1984). Shields also introduced a dimensionless entrainment function

$\frac{\tau_0}{\rho g \Delta d}$ or $\frac{\tau_0}{(\gamma_s - \gamma_f)d}$ as a function of shear Reynolds number (Featherstone and Nalluri 1988):

$$R_* = \frac{u_* d}{\upsilon}$$

where u_* = friction velocity

Hence, for critical values of τ_0 ,

$$\frac{(\tau_0)_{CR}}{(\gamma_s-\gamma)d} = f\left(\frac{(u_*)_{CR}d}{\upsilon}\right) \qquad \text{(B31.)}$$
$$= f(R_*)_{CR}$$

where $(u_*)_{CR}$ = critical shear velocity

$(R_*)_{CR}$ = critical shear Reynolds number

The results of a series of experiments performed by Shields in 1936 for the determination of the critical tractive force were plotted on a graph of

$$\frac{(\tau_0)_{CR}}{(\gamma_s-\gamma_f)d} \quad \text{versus} \quad (R_*)_{CR}$$

along with a line representing the mean of the experimental scatter by Rouse (Bogardi 1978). The Shields diagram, as it is generally referred to, has proved to be a useful method of prediction of incipient motion of particles and is now widely accepted (Graf 1984). Later refinements to the curve by Yalin and Karahan do not show large departures from the original results, although recent work by Kleijwegt et al (1990) on non-cohesive sediments in circular pipes with fixed beds has found shear stresses to be only 70% of those predicted by the Shields criterion. In this context it is stated that incipient motion of particles is caused by peak values of fluctuating bed shear stress which can be double the mean value in a rectangular channel and even greater in a circular channel due to the complex flow structure. Likewise, empirical formulae which relate to dimensionless shear stress produced by White, Kurihara and Iwagaki all show close agreement with the Shields curve (Garde and Ranga Raju 1985). Recent discussion, furthermore, has led to agreement by a number of authors that, of the possible parameters for defining erosive or depositional criteria, bed shear stress is the most appropriate (Verbanck et al 1994). One of the main reasons for this conclusion was that big fluctuations of shear have been observed to occur in sewers under apparently identical average flow velocities. Related work in this field by Nalluri and Alverez-Hernandez is discussed in Chapter 2 of the main text.

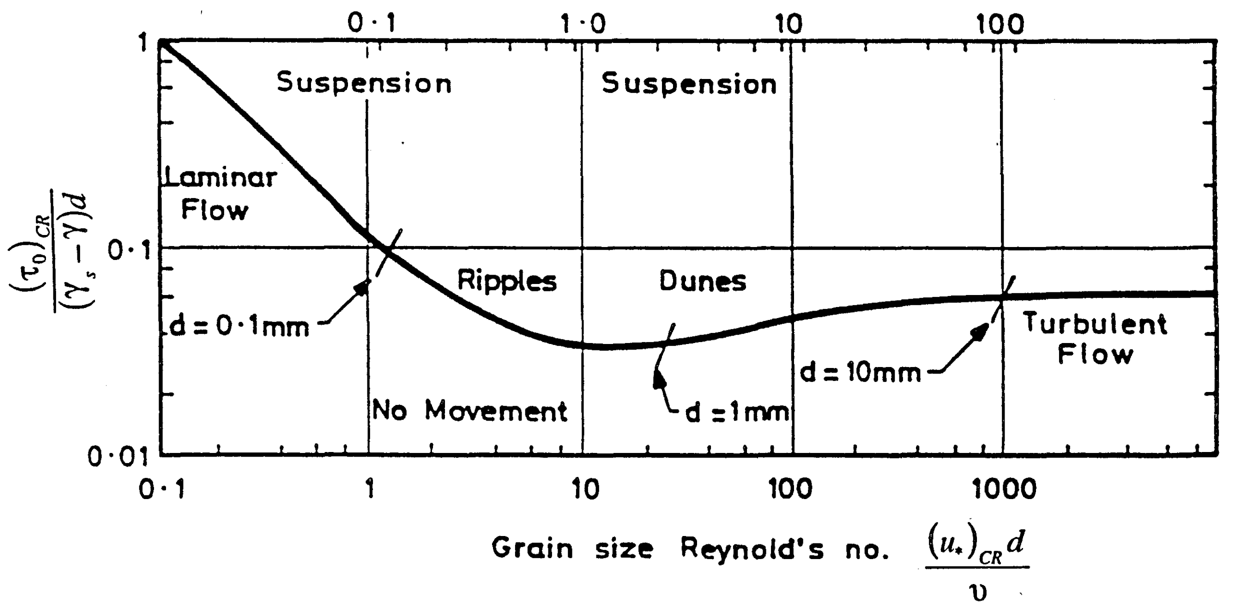


Figure B6 - Shields Diagram (Graf 1984)

The studies discussed in this section so far have only considered uniform sediment. Further work by various researchers (Verbanck et al 1994) has resulted in more generally applicable relationships, but at present these have not been satisfactorily confirmed by experimental data.

4 TRANSPORT PROCESSES

Once shear stresses on a bed deposit exceed a critical value, particles on the bed may begin to move in the direction of flow (Garde and Ranga Raju 1985). However, the mode by which they are transported (and the distance and speed at which they travel) depends upon the flow conditions, ratio of densities of the fluid to the sediment particles and the size of particles. The motion of the particles may be a combination of rolling, sliding, and occasionally jumping along the bed (saltating) whereby the particle loses contact with the bed for some time. This mode of sediment transport is known as the transport of bed load or contact load (Graf 1984). Alternatively, the entire motion of the solid particles may be such that they are surrounded by fluid. This is known as suspended load transport.

Finer lighter materials tend to travel in suspension, while heavier materials are transported as bed load. However, the mode of transport is strongly influenced by the level of turbulence (CIRIA 1987). Sub divisions of these broad descriptions are listed below in order of increasing turbulence and flow velocity for flow in a pipe with uniform non-cohesive material:

a) Transport over a deposited bed:

- As velocity increases the surface of the stationary bed becomes rippled or duned. Further increases in velocity result in the surface reverting to being plane, or possibly only a series of isolated dunes separated by clear pipe.

b) Bed-load without deposition (or flume traction):

- Particles move along the pipe invert in continuous rolling/sliding or intermittent bouncing contact.

c) Suspended load (heterogeneous flow):

- The sediment is maintained in suspension by turbulence, but is more concentrated towards the pipe invert.

d) Wash load (pseudo-homogeneous flow):

- All of the particles are transported in turbulent suspension and are uniformly distributed through the depth of flow.

In the following discussion, the simpler cases of pure bed load and pure suspended load are in turn discussed.

4.1 Bed Load

Early research in this field dealt with the development of relations for transport by streams (Vanoni 1984). Early work by Du Boys (Graf 1984) resulted in the development of the formula

$$q_s = \chi \, \tau_0 \left[\tau_0 - (\tau_0)_{CR} \right] \tag{B17.}$$

where q_s = transport weight per unit time and per unit width of stream

χ = a coefficient dependent on sediment size

This simple equation, developed using the assumption that the bed load moves as a series of sliding layers, has been criticised since the inherent simplifications are in strong disagreement with observations. Particles in the bed layer tend to move individually or in groups rather than en masse in layers. However, the fact that it has been shown to give reasonable results which at times show good agreement with field data have led to its wide use. Values of χ and $(\tau_0)_{CR}$ for equation (B29) were published by Straub (Vanoni 1984).

A number of investigators working after Du Boys have proposed empirical equations of a form similar to the Du Boys equation. Of these, the most commonly used is the one proposed by Meyer-Peter and Muller (Garde and Ranga Raju 1985). For uniform sediments this is in the following form (Rouse 1950)

$$q_s = \left(A_f q^{2/3} S_c - B_f d \right)^{3/2} \tag{B18.}$$

where B_f = breadth of flow

q = rate of flow per unit width

The next major development in the early thirties was the formula suggested by Schoklitsch based on laboratory experiments (Graf 1984)

$$q_s = \chi'' S^k \left(q - q_{CR} \right) \tag{B19.}$$

where χ'' = sediment coefficient

q_{CR} = the critical discharge per unit width for incipient motion

This was later modified by Schoklitsch based on field data

$$q_s = 2500 S^{3/2}(q - q_{CR}) \tag{B20.}$$

As an extension to his work on critical stress, Shields presented a model for sediment transport based on "excess" shear stress (Graf 1984). This semi-empirical tractive force equation was given in the form:

$$\frac{q_s}{\gamma_q}\left(\frac{\gamma_s - \gamma_f}{S\gamma_f}\right) = 10\frac{\tau_0 - (\tau_0)_{CR}}{(\gamma_s - \gamma_f)d} \tag{B21.}$$

The fact that the specific gravity of the sediment is taken into account makes the equation very useful for various applications.

Kalinske (1947) sought to improve upon the purely empirical formulations of the Du Boys type by incorporating relationships between bed shear and turbulence. On the basis that the volume rate of sediment movement per unit width must be equal to the product of the particle volume $\pi d^3/6$, the mean particle velocity $\overline{V}_s$ and the average number of particles per unit bed area $p/(\pi d^2/4)$ where p is a factor which indicates the proportion of the bed taking the fluid shear (Graf 1984):

$$q_s = \left(\frac{\pi d^3}{6}\right)\frac{p}{\left(\pi d^2/4\right)}\frac{1}{\left[u_i - (u_i)_{CR}\right]} \tag{B22.}$$

where p = factor which indicates the proportion of the bed taking the fluid shear

u_i = instantaneous fluid velocity

$(u_i)_{CR}$ = critical fluid velocity
$\overline{V}_s$ = mean particle velocity

This led to the dimensionless bedload equation

$$\frac{q_s}{u_* d} = f_{ct}\left[\frac{(\tau_0)_{CR}}{\tau_0}\right] \tag{B23.}$$

The shape of this function was predicted analytically with knowledge of turbulent behaviour gained from experimental results.

The most radical departure from the Du Boys type of analysis was that of Einstein's (Rouse 1950). On the basis of observations, he concluded that particles of a given size move in a series of steps of definite length and frequency, and that the rate of transport depends upon the number of particles in motion. The probability that a particle will move in a given time step is expressed in terms of transport rate, size and relative weight of the particles, and a time factor equal to the ratio of the particle diameter to its fall velocity. The probability of particle movement is also expressed in terms of the ratio forces by the flow to the resistance of the particle to movement. These two probability relationships may then be equated

$$\Phi = \mathrm{f}(\Psi) \tag{B24.}$$

Einstein investigated the form of the indicated function by plotting experimental measurements of Φ versus Ψ where

$$\Phi = \frac{q_s}{\sqrt{g(s_s - 1)F d^{3/2}}} \tag{B25.}$$

$$\Psi = \frac{\gamma(s_s - 1)d}{\tau_0} \tag{B26.}$$

and $$F = \sqrt{2/3 + \frac{36V^2}{gd^3(s_s - 1)}} - \sqrt{\frac{36V^2}{gd^3(s_{s-1})}} \tag{B27.}$$

This approach to bedload prediction has the advantage that the definition of critical values for initiation of sediment motion is avoided, and that bedload transport is related to fluctuations in velocity rather than an average velocity (Rouse 1950).

4.2 Suspended Load

Where flow conditions cause particle motion to occur, if the entire motion of the particles is such that they are surrounded by fluid, they are said to be in suspension (Graf 1984). The weight of the particles causes a tendency for settlement which is counterbalanced by the irregular motion of fluid particles. Therefore, the hydraulic conditions prevalent will determine whether or not a given size fraction will be in suspension.

Suspended load transport is an advanced stage of transport. At low shear stresses it is to be expected that only bed load transport would occur, while at higher shear stresses both modes of transport may occur (Garde and Ranga Raju 1985).

As early as 1858, Dupuit surmised that the power of suspension depends on the velocity gradient of the flow (Vanoni 1984). O'Brien presented the fundamental equation of sediment suspension by fluid turbulence (Rouse 1950).

$$C_o \omega_s = -\in \frac{\mathrm{d}C_o}{\mathrm{d}y} \tag{B28.}$$

where C_o = sediment concentration

ω_s = fall velocity of sediment

$\in$ = sediment transfer or sediment diffusion coefficient

Von Karman showed that the diffusion coefficient for momentum was related to the velocity gradient

$$\frac{\mathrm{d}u}{\mathrm{d}y}$$

and the shear stress at level y τ_y by the equation

$$\tau_y = \rho \in \frac{\mathrm{d}u}{\mathrm{d}y} \tag{B29.}$$

Hence, by integrating equation (B43.):

$$\ln \frac{C}{C_a} = -\rho \omega_s \int_a^y \frac{\mathrm{d}u}{\mathrm{d}y} \frac{\mathrm{d}y}{\tau_y} \tag{B30.}$$

where C_a = concentration of sediment with fall velocity ω_s at level a_e

(ref: (Vanoni 1984))

Hence, in order to calculate a value of C , the value of C_a must be known.

Rouse (1950) integrated equation (B42.) with the velocity gradient determined from the Karman - Prandtl logarithmic velocity law. This resulted in the equation

$$\frac{C}{C_a}=\left[\frac{(Y-y)}{y}\frac{a_e}{(Y-a_e)}\right]^Z \qquad \text{(B31.)}$$

in which $Z=\omega_s/ku_*$

This equation has come into general use as a means of predicting the variation in concentration of sediment with depth, despite unrealistic concentrations of zero at the surface and infinity at the bed (Vanoni 1984). Subsequent modifications have been suggested by Einstein and Chien (ibid).

A similar formula to equation (B46.) was proposed by Hunt (Garde and Ranga Raju 1985) which takes into account the space occupied by the suspended particles. This formula is not used due to its complexity.

Einstein and Chien have suggested a series of modifications to equation (B46.) which result in a more complex version of the equation. Although the modifications are a logical development, they have not been adequately tested against laboratory or field data (Garde and Ranga Raju 1985).

The evaluation of Rouse's distribution requires a sediment concentration at some reference level. However, Einstein designated a flow layer on top of the fixed bed as the bed layer, which was found to be of a thickness $a'=2d$ (Graf 1984). The material in this layer is deemed to be the source of the suspended load, making the determination of the lower limit concentration C_{ae} possible. The value of C_{ae} is assumed to be the average concentration of the bed layer.

If the total suspended load per unit width of the channel is given by

$$g_{ss} = \int_{a}^{D} Cu \, \mathrm{d}y \tag{B32.}$$

where g_{ss} = the suspended load rate in weight per unit time and width

and if C and u are functions of y then by introducing the suspension distribution equation (B46.) and expressing the velocity with the logarithmic velocity distribution, the following equation is obtained (Graf 1984):

$$g_{ss} = \int_{a}^{Y} C_{ae} \left(\frac{Y-y}{y} \frac{a_e}{Y-a_e} \right)^{Z} 5.75 u_*' \log\left(\frac{30.2y}{\Delta_r} \right) \mathrm{d}\, y \tag{B33.}$$

where u_*' = the shear velocity due to grains only

Einstein based values of Δ_r on the equation

$$\Delta_r = \frac{k_s}{X} \tag{B34.}$$

where $k_s = d_{65}$ approximately.

X = correction factor

Values of X were obtained from a graph derived by Einstein from experiments by Nikuradse (Graf 1984) of X versus k_*/δ where $\delta = 11.5\upsilon/u_*$. If the lower limiting depth a_e is replaced by the dimensionless argument $A_E = a_e/Y$ then

$$g_{ss} = \int_{A_E}^{1} CuY \, \mathrm{d}y \tag{B35.}$$

hence equation (B48.) becomes

$$g_{ss} = Yu_*' C_{ae}\left(\frac{A_E}{1-A_E}\right)^Z 5.75 \int_{A_E}^{1} \left(\frac{1-y}{y}\right)^Z \log\left(\frac{3.02y}{\Delta_r/Y}\right) \mathrm{d}\,y \qquad \text{(B36.)}$$

or

$$g_{ss} = 5.75 C_{ae}\, u_*' Y\left(\frac{A_E}{1-A_E}\right)^Z \log\left(\frac{30.2Y}{\Delta_r}\right)\left[\int_{A_E}^{1} \left(\frac{1-y}{y}\right)^Z \mathrm{d}\,y + 0.434 \int_{A_E}^{1} \left(\frac{1-y}{y}\right)^Z \ln y\, \mathrm{d}y\right]$$

(B37.)

The numerical integration of the two integrals for various values of A_E and Z was performed by Einstein (Graf 1984) by evaluating the following arguments:

$$I_1 = 0.216 \frac{A_E^{Z-1}}{(1-A_E)^Z} \int_{A_E}^{1} \left(\frac{1-y}{y}\right)^Z \mathrm{d}\,y \qquad \text{(B38.)}$$

$$I_2 = 0.216 \frac{A_E^{Z-1}}{(1-A_E)^Z} \int_{A_E}^{1} \left(\frac{1-y}{y}\right)^Z \ln y \,\mathrm{d}\,y \qquad \text{(B39.)}$$

I_1 and I_2 were given graphically versus A_E for a series of values of Z .

Equation (B52.) can thus be written in the form

$$g_{ss} = 11.6 C_{ae} u_*' a_e\left[2.303 \log\left(\frac{30.2Y}{\Delta_r}\right) I_1 + I_2\right] \qquad \text{(B40.)}$$

The value of the reference concentration C_{ae} was evaluated by Einstein, from experimental results as

$$C_{ae} = \frac{1}{11.6} \frac{g_s i_s}{u'_* a'} \qquad \text{(B41.)}$$

where $g_s i_s$ = bedload rate for a given size i_{ss}.

Hence, by substituting equation (B56.) for C_{ae} in equation (B55.), the following formula is obtained:

$$g_{ss} i_{ss} = g_s i_s (P_E I_1 + I_2) \qquad \text{(B42.)}$$

where $g_{ss} i_{ss}$ = the suspended load rate in weight per unit time and width for particle size i_{ss}

$$P_E = 2.303 \log\left(\frac{30.2Y}{\Delta_r}\right)$$

Thus for a mixed sediment, the transport rates of different sizes can be determined, then summed to obtain the total rate of suspended load transport (Garde and Ranga Raju 1985).

An approximate method of suspended load determination was proposed by Lane and Kalinske (Garde and Ranga Raju 1985) based on similar principals to Einstein's method. In this case, however, the value of the sediment diffusion coefficient $\in$ is assumed to remain constant with depth. The relationship so formed has been found to be sufficiently accurate for practical purposes in the case of wide rivers (Graf 1984).

5 EFFECTS OF COHESION

The preceding sections 2 to 4 have considered the movement of cohesionless particles as a result of fluid motion. None of the work discussed in these sections takes account of the influences on the behaviour of such particles that cohesive forces will exert, where these are present.

Research on cohesive materials is still in an early stage of development (Nalluri and Alvarez-Hernandez 1990) due to its greater complexity. Some of the more significant findings so far in this field are discussed below.

Where the material in suspension in a body of moving fluid or which forms a bed deposit has a significant amount of fines, cohesive forces may play an important role. The forces which resist motion are in this case the submerged weight of the particles and the cohesive forces between the particles (Garde and Ranga Raju 1985). Because of these cohesive forces, the corresponding critical velocity and critical tractive stress will be greater for such material than it would for a similar non-cohesive material. The motion of cohesive particles is also different from that of cohesionless particles. There is a tendency in the presence of cohesion for lumps of particles to move as a unit. This also has a significant effect on the settlement characteristics of cohesive material in suspension, as will be discussed later in this section.

Where fine particles contain clay minerals, these clay minerals have residual electric charges which may be positive or negative. These charges give the particles the ability to hold certain cations or anions. The clay minerals are usually crystalline with a sheeted structure (Graf 1984), and all clay minerals have a flake-shaped appearance.

Both repulsive and attractive physicochemical forces act on the particle due to the residual electric charges on the surface of the particle. The flat surfaces carry negative charges, while the broken edges of the sheets may carry both negative and positive charges. This gives the particle the ability to absorb cations and anions, although the latter is of lesser importance. The common exchangeable cations are calcium, magnesium, potassium and hydrogen (Garde and Ranga Raju 1985).

The cations held are retained in an exchangeable state (Graf 1984). The rate of ion exchange depends on the concentration and valency of ions present in the surrounding water. The properties of cohesive materials are therefore influenced by the chemical composition of the water, since ions in the surrounding water may replace those already attached to the particles which in turn affects the strength of interparticle forces.

Repulsive forces will dominate as long as the residual electric charges of the particles are not satisfied. As a greater proportion of the residual charges are satisfied, attractive forces assume more importance. Materials such as sewer sediments which contain significant amounts of organic matter may exhibit cohesive-like properties. These are discussed further in Chapter 3 of the main text.

When particles which exhibit cohesive properties are in suspension, the interaction of the particles and fluid is affected significantly by the cohesive forces. A dispersed suspension of very small particles may or may not settle out, even after a considerable amount of time has elapsed. However, if many of the very small particles come together to form flocs, the effective weight of the resulting agglomerate would increase, and sedimentation will occur (Graf 1984). This process is often referred to as flocculation. The flocs so formed are loose, irregular clusters of particles.

Formation of flocs or aggregates of cohesive particles may be due to interparticle collisions caused by Brownian motion, velocity gradients or differential settlement velocities (Krone 1986). This process may be aided by a reduction in the repulsive forces due to a chemical change in the surrounding fluid.

According to Burt (1986) the greatest influencing factor on rates of flocculation is the concentration of suspended particles. Krone (1986) also cites the strength of interparticle cohesion and the hydraulic stress imposed on the aggregates as factors which affect the rate of aggregation. Repeated collisions can produce aggregates having much larger sizes than those of the individual particles. The strength of interparticle cohesion within the floc depends in part upon the intensity of collisions. Floc formed in high velocity gradients therefore have stronger structures than those formed in less turbulent conditions. However, not all collisions result in the formation of a larger floc (Graf 1984). At one extreme, an elastic collision causes the particles to rebound. If this is general in its occurrence, the result will be a stable suspension. On the other hand, plastic collision leads to rapid floc formation. An intermediate stage between these two extremes result in slow flocculation.

Flocs formed from individual particles are described by Krone (1986) as first order aggregates. Second order aggregates are formed as a result of collisions between first order aggregates and so on. Also, as the size of a floc increases, it also becomes relatively less dense (Burt 1986). At the same time, the settling velocity of the particle increases with size. There is therefore a limit to the size of floc which may be formed in a particular situation since the aggregate ultimately breaks up due to its settling velocity. Because of this mechanism and the fluctuations in turbulence normally evident in a moving fluid, aggregates can be repeatedly broken and reformed during transport (Krone 1986). Apart from extensive work on settlement of cohesive sediments in estuarial waters, mainly by Einstein and Krone (Graf 1984), very little work has been done on predicting rates of settlement for cohesive particles. Estuarial waters have high salinity, leading to a high degree of adhesion of particles since the presence of the high concentration of salts suppresses the tendency for particles in supension to repel each other. The results of work done in estuarial waters are therefore not directly applicable to other situations. There is thus a lack of information on the sedimentation of sediments flocculated in channel flow.

Once flocs have formed and settled out, the likelihood of adherence to the bed under any given flow conditions depends primarily on the strength of aggregate-bed cohesion, which in turn depends on the number and strength of intermineral particle contacts (Krone 1986). The settling aggregates form a structure one order higher than the aggregates. The critical shear stress of recently deposited aggregates is therefore correspondingly low. However, the aggregates which form and settle out in high velocity gradients have low order structure leading to a relatively high critical shear stress following recent deposition.

As overburden pressure on a layer of deposited aggregates increases due to continuing deposition, voids between the higher order structure are reduced or eliminated. This increases the inter-mineral particle contacts, thus increasing the strength and density of the sediment. The voids ratio decreases rapidly with each level of aggregate structure. There is thus a much smaller increase in strength and density proportionally as the order of aggregate structure decreases with consolidation. For this reason there is a marked difference in strength between recently deposited cohesive material and the underlying layers. The rate of change of density and strength decreases with increasing depth below the surface of the bed deposits.

Further stabilisation of the bed may occur due to time-dependant creep consolidation (Krone 1986). Other factors such as gelling of interstitial water and the action of micro-organisms in the presence of organic materials may also alter significantly the strength of bed deposits. The stress history of the sediment is also important, since a deposit which has previously been compressed by an overburden which has since been removed will have much higher resistance to erosion than one which has recently been deposited.

As velocity of flow over an established bed of cohesive material increases, a point is reached when the material begins to be eroded. Du Buat first investigated critical velocity of erosion in 1786 (Graf 1984) by establishing the velocity at which "pottery clay" starts to be eroded. The mode by which erosion occurs in a cohesive sediment bed is different to erosion of cohesionless material (Graf 1984, Krone 1986). This mechanism of erosion is known as surface erosion. At higher shear stresses, large pieces of the bed may be dislodged by continuous body erosion (Partheniades 1965). The latter mechanism will be prevalent particularly when there has been a rapid increase in shear stresses to a value greater than the bulk shear stress of the deposit (Krone 1986). The bed fails instantaneously to a depth where shear strength is sufficient to resist erosion.

Investigations have been made by a number of workers in an attempt to determine the factors which influence the critical tractive stress of cohesive materials (Alvarez-

Hernandez 1990). Smerdon et al carried out flume studies on cohesive soils (Graf 1984). This resulted in the proposed equation

$$(\tau_0)_{CR} = \alpha\eta\frac{\pi}{6}d(\gamma_s - \gamma)\tan\phi + c\,S_v \tag{B43.}$$

where η = packing coefficient (= N)

N = number of particles per unit area

c = constant of proportionality

S_v = shear strength of material

For coarser sediments the second term of the above equation is negligible, and for fine sediments the first term is small (Garde and Ranga Raju 1985). Smerdon also noted a correlation between plasticity index and critical tractive force, while the research by Lyle et al suggests that both the plasticity index and the void ratio (and hence sediment compaction) are of importance (Graf 1984). Based on tests using fine materials ranging from 0.014mm to 0.319mm, Dunn (1959) suggests that the critical tractive shear stress is dependant on the vane shear strength and plasticity index of the material. This finding was contradicted by Partheniades (1965) who stated that critical shear stress is independent of vane shear strength.

Design criteria for the assessment of permissible canal velocities based on experience and personal deduction were proposed by Fortier et al (Graf 1984), with similar work subsequently carried out by Chow, based on Russian research reported in 1936. A number of field investigations, notably by Enger et al, Thomas et al, Gibbs, and Simons use a similar approach, but based on more detailed observation.

The investigations referred to in the literature generally indicate that the mechanisms involved in the entrainment of cohesive material involve physical, chemical and electrochemical processes which are at present not fully understood (Garde and Ranga Raju 1985). Hence, there is at present no reliable method by which critical tractive stress values for cohesive sediments can be predicted.

Recent work by Nalluri and Alvarez (1990) investigating the influence of cohesion on sediment entrainment was based on a series of experiments using a synthetic cohesive sewer sediment composed of a mixture of sand and clay gel. Problems were encountered with the use of the synthetic sediment due to rapid break up of the bed following initial erosion. It was concluded that at the stage reached it was impossible to investigate the problem in depth because of the complexities that cohesive sediments in the real sewer environment present.

There is at present very little information on the transport of cohesive sediment (Graf 1984). Einstein and Krone (1961) stated that the transport of cohesive sediment is essentially determined by the flocculation characteristics of the suspended material. The modes of transport may be either as particles which remain in suspension permanently due to very low settling velocities, or particles which may at times settle out to form a fluid mud which may move as connected fluid mud or be resuspended to move once more in suspension. The flow of fluid mud may be shear-induced or by a combination of shear and gravity flow.

Current knowledge of the effects of cohesion on the movement of sediment particles is very limited. Most of the studies so far conducted are not directly applicable to the prediction of sediment transport rates in sewers. Until these mechanisms are fully understood, a full analysis of sediment movement in sewers will not be possible.

6 CONCLUSIONS

The preceding sections of Appendix B deal in turn with theories describing the different components of transport of particles as a result of fluid motion. The mechanisms involved interact with each other, while the theories discussed tend to deal with the different aspects separately. To further complicate matters, even in the case of single-sized non-cohesive particles in uniform flow conditions, knowledge of all the mechanisms involved is incomplete. The problem is compounded by other complications which arise in the case of sediment transport in sewers. These include the availability of sediment (variable), the heterogeneity of the sediment, the divers nature of the sewer system, and the highly variable nature of the flow conditions (these factors are discussed in Chapter 3 of the main text). In such circumstances, a more pragmatic approach to the methods by which transport rates are predicted may be applicable.

Appendix C

Procedure for the Determination of the Rate of Sediment Transport in Full or Part-Full Pipes. (Excerpt from CIRIA Project Report 1, 1987 - see references in main section)

THE ACKERS-WHITE EQUATIONS APPLIED TO SEDIMENT TRANSPORT IN SEWERS

This Appendix presents in outline the Ackers-White Equations modified for determination of the rate of sediment transport in full and part-full sewers. The Ackers-White Equations may be used in combination with the Colebrook-White Equation for computing the equilibrium depth of sediment deposit that may occur in a sewer, as well as estimating the rates of sediment deposition and erosion. The methodology described applies only to non-cohesive sediments.

1 Nomenclature

A	-	cross-sectional area
Agr	-	value of mobility number Fgr at nominal initial movement
C	-	coefficient in the sediment transport function
d, d35, d50	-	sediment grain diameters
Dgr	-	dimensionless particle size
D	-	pipe diameter
Fgr	-	sediment mobility number
g	-	acceleration due to gravity
Ggr	-	dimensionless sediment transport rate
ks, kss, ksp	-	linear measures of effective roughness
m	-	exponent in sediment transport function
n	-	transition exponent dependent on sediment size
P, P_1, P_2	-	wetted perimeter
R	-	hydraulic mean depth = A/P (also known as hydraulic radius)
s	-	specific gravity of sediment
S	-	hydraulic gradient
v	-	mean velocity of flow
v*	-	shear velocity = $\sqrt{gRS}$
X	-	sediment transport, mass flux per unit mass flow rate
ν	-	kinematic viscosity of the fluid

2 Ackers-White Equations

The Ackers-White Equations were originally derived for determining sediment transport rates in alluvial channels, and the equations have been modified subsequently to apply to sediment transport in full and part-full sewers (Ref. 2).

Sediment transport is described by the ratio of the appropriate shear force on the sediment bed to the immersed weight of a layer of sediment particles. A general mobility number is defined:

$$Fgr = \frac{v*^{n}}{\sqrt{g(s-1)d}} \qquad \frac{v^{1-n}}{[\sqrt{32}.\log\ (12R/d)]^{1-n}} \qquad \text{-(C1)}$$

For coarse sediments (n = 0) the expression reduces to the form:

$$Fgr = \frac{V}{\sqrt{g(s-1)d}} \qquad \frac{1}{\sqrt{32}\ \log\ (12R/d)} \qquad \text{-(C2)}$$

For fine sediments (n = 1), Equation (1) may be written:

$$Fgr = \frac{V*}{\sqrt{g(s-1)d}} \qquad \text{-(C3)}$$

For transitional sizes of sediment, n may take a value between 0 and 1, depending on the dimensionless expression for grain diameter:

$$Dgr = d \left[\frac{g(s-1)}{\nu^{2}} \right]^{1/3} \qquad \text{-(C4)}$$

The efficiency of sediment transport is dependent on the mobility number Fgr, and there is a critical value of Fgr, denoted Agr, below which no sediment will be moved, and the transport efficiency is zero. A general transport parameter may be defined:

$$Ggr = C \left[\frac{Fgr}{Agr} - 1 \right]^{m} \qquad \text{-(C5)}$$

The non-dimensional transport parameter is related to the primary variables:

$$Ggr = \left[\frac{XR}{sd} \right] \left[\frac{A}{WeR} \right]^{1-n} \left[\frac{v*}{V} \right]^{n} \qquad \text{-(C6)}$$

Thus, for coarse sediment (n = 0):

$$Ggr = \frac{XA}{sdWe} \qquad \text{-(C7)}$$

For fine sediment (n = 1)

$$Ggr = \left[\frac{XR}{sd} \right] \left[\frac{v*}{V} \right] \qquad \text{-(C8)}$$

The exponents n and m and coefficients Agr and C are related to the dimensionless grain diameter for $1.0 < Dgr < 60$

$$n = 1.00 - 0.56 \log Dgr \qquad \text{-(C9)}$$

$$Agr = 0.14 + \frac{0.23}{\sqrt{Dgr}} \qquad \text{-(C10)}$$

$$m = 1.34 + \frac{9.66}{Dgr} \qquad -(C11)$$

$$\log C = 2.86 \log Dgr - (\log Dgr)^2 - 3.53 \qquad -(C12)$$

For coarse sediments, i.e. Dgr > 60:

$$n = 0.00 \qquad -(C13)$$

$$Agr = 0.17 \qquad -(C14)$$

$$m = 1.50 \qquad -(C15)$$

$$C = 0.25 \qquad -(C16)$$

For typical values of ν and s, a coarse sediment (Dgr > 60) has a grain diameter of 2.6 mm whereas a fine sediment (Dgr = 1) has a grain diameter of 0.04 mm.

3 Estimating sediment transport rates

Application of the Ackers-White Equations requires a knowledge of the mean velocity of flow, the hydraulic gradient, the sewer cross-section properties, the sediment grading (d50 for narrow gradings, d35 for wide gradings), the mass density of sediment and fluid, and the viscosity of the fluid. If the mean velocity or hydraulic gradient is not known, the unknown variable may be computed by use of a suitable friction equation (e.g. the Colebrook-White Equation described in Section K.4). The method of determining the sediment transport rate as a proportion of the fluid flow (by mass) is as follows:

(a) Determine the value of Dgr from known values of d, g, s and ν using Equation (4).

(b) Determine the values of n, m, Agr and C using Equations (9) to (12).

(c) Compute the value of the particle mobility Fgr from Equation (1).

(d) If Fgr < Agr, no sediment transport may take place.

(e) For Fgr > Agr, determine the transport parameter Ggr from Equation (5).

(f) Convert Ggr to sediment flux X, which is the mass ratio of sediment flux to fluid discharge. Use a suitable value of effective sediment width We (Section K5) in Equation (6).

4 Colebrook-White Equation

The Colebrook-White Equation is the friction equation most commonly used in the UK for calculating hydraulic conditions in sewers.

For part-full sewers the equation may be expressed in terms of the hydraulic mean depth:

$$v = -\sqrt{32gRS}\ \log\left[\frac{ks}{14.8R} + \frac{1.255\nu}{R\sqrt{32gRS}}\right] \qquad -(C17)$$

For sewers flowing full, the hydraulic mean depth may be replaced by the pipe diameter/4 for circular pipes:

$$v = -2\sqrt{2gDS}\ \log\left[\frac{ks}{3.7D} + \frac{2.51\nu}{D\sqrt{2gDS}}\right] \qquad \text{-(C18)}$$

The linear measure of effective roughness ks may be a composite value if part of the invert is covered by a sediment deposit. A simple method of combining the sediment roughness kss and the "clean pipe" roughness ksp is by straightforward perimeter weighting:

$$k_s = \frac{P_1 kss + P_2\ ksp}{P_1 + P_2} \qquad \text{-(C19)}$$

In Equation (19), P_1 is the wetted perimeter of the sediment deposit (= We) and P_2 is the "clean pipe" wetted perimeter.

Assessment of the value of sediment roughness should account for any bed forms that may occur. In general, the value of kss may be considerably greater than the sediment grain diameter d.

5 Equivalent width of sediment deposit

Referring to Equation (6) it may be seen that the sediment flux X is related to the value taken for the sediment width We. In the case of coarse sediment, the sediment flux is directly proportional to the sediment width (Equation 7). For sediment transport when a finite sediment deposit exists in the sewer, the equivalent width may be taken as the actual sediment width. For "clean pipe" transport calculations, it is necessary to select a suitable value of We. Research has shown that a suitable value may be obtained from:

$$We = 10 \times d \qquad \text{-(C20)}$$

However, the transport rates computed will be low for large diameter pipes. There is a case for taking the "clean pipe" situation to be represented by a small depth of deposit. For example, taking a sediment deposit equal to 1% of the pipe diameter, produces an effective width of approximately 20% of the pipe diameter.

An alternative approach for design is to permit a finite depth of sediment deposit (for example 10% x D) in the sewer. This provides a reduction in velocity and hydraulic gradient required to transport a specific quantity of sediment.

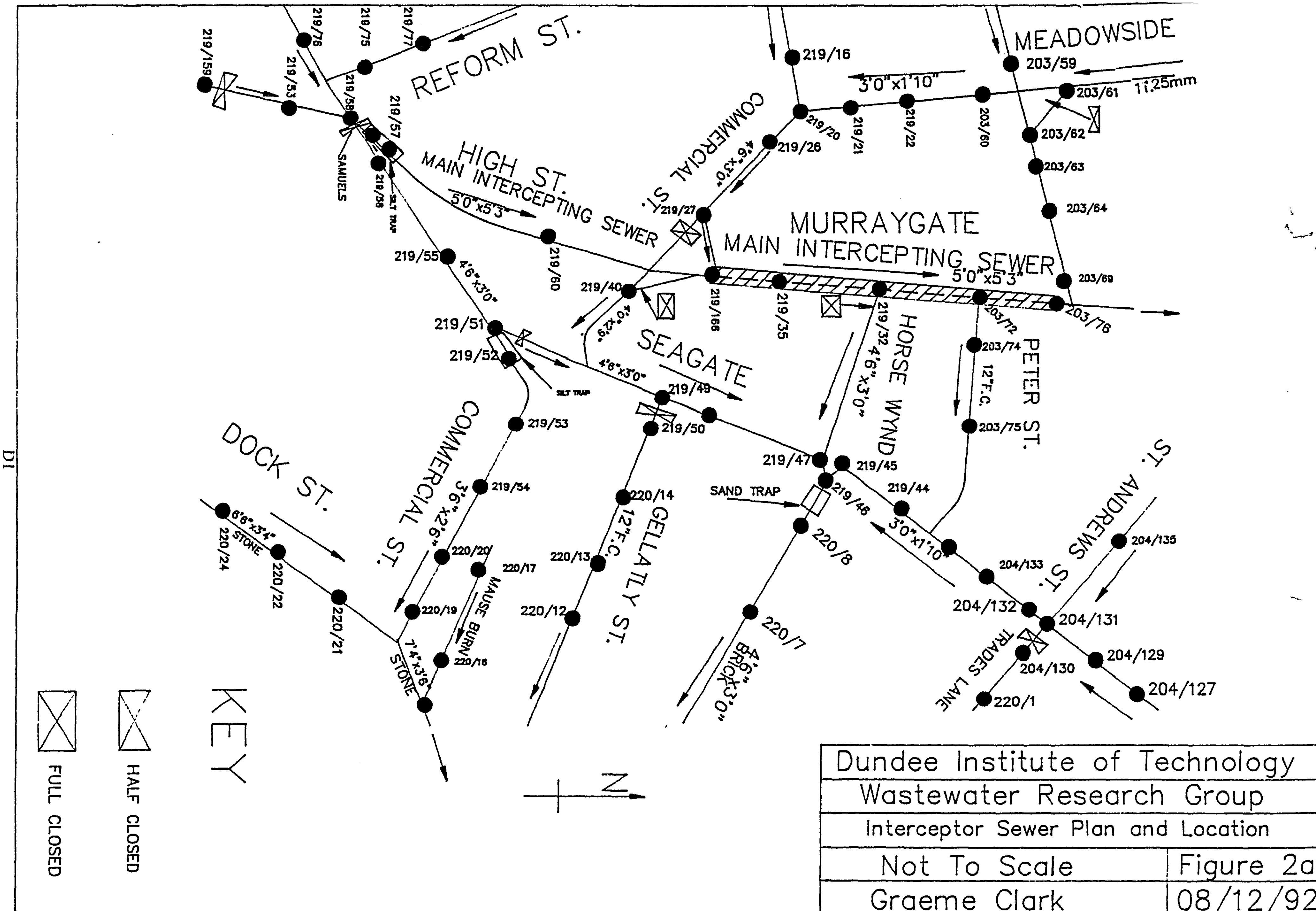
6 Equilibrium depth of sediment deposit

The Ackers-White Equations and Colebrook-White Equation may be used in combination to determine the equilibrium depth of a sediment deposit that would form under steady conditions of fluid discharge and sediment flux. As the depth of sediment deposit increases, the mean velocity of flow and effective sediment width increase, both leading to an increase in the potential sediment flux, and an increase in hydraulic gradient. The equilibrium depth of sediment occurs when the rate of sediment supply is equal to the calculated sediment transport rate.

As conditions in sewers are rarely steady over a long period, sediment may be depositing or eroding at a specific time. The rate of deposition or erosion may be determined from the difference between the sediment supply rate and the potential sediment transport rate.

Appendix D

Plan of The System of Gates in Dundee Central Area



Dundee Institute of Technology

Wastewater Research Group

Interceptor Sewer Plan and Location

Not To Scale | Figure 2a

Graeme Clark | 08/12/92

Appendix E

Sediment Sampling

PROCEDURE FOR THE SAMPLING AND PREPARATION OF SEWER SEDIMENTS

INTRODUCTION

The following procedure has been developed for the sampling and analysis of sewer sediments as part of the work carried out at Dundee College of Technology on the Dundee Sewer Sediment Research project (DSSR) under contract to the Water Research centre (WRc) and to Tayside Regional Council (TRC). The methods stated have been evolved to suit the requirements of the project, since there are no "standard" procedures for dealing with samples with a high solids content in the way that there are for aqueous solutions.

The objectives of the procedures shown are:-

(a) to obtain a sample which is representative of the "whole" sediment - not only the solid phase but including the liquid phase;

(b) to ensure that the sample contains insignificant amounts of raw sewage from the flow above the sample;

(c) to ensure that the sample is reasonably homogeneous before a sub-sample is taken for analysis in the laboratory;

(d) to remove as much grit as possible while leaving the majority of the organic material in suspension/solution, in a consistent manner where it is necessary to obtain an aqueous solution for testing purposes.

PROCEDURE

1. The samples are normally obtained from the bed deposits on the invert of a live sewer, over which there is a significant depth of flowing sewage at all times. The samples are obtained using a purpose built sampling tool designed in such a way as to exclude sewage. This is done by ensuring that the tool is completely filled with sediment and closed before withdrawing the sample. (See Fig 1.).

2. The closed sampler is lifted clear of the sewage to be placed in a broad necked polythene sample container. The container is closed and the sample transported to the laboratory for analysis.

3. The sample is mixed thoroughly in its container using a spatula, until a uniform appearance is achieved. At this point, the sample may be split into subsamples if required. The minimum size of subsample retained for the remainder of this procedure is approximately 200 g (this is a reasonable quantity for blending in a single operation).

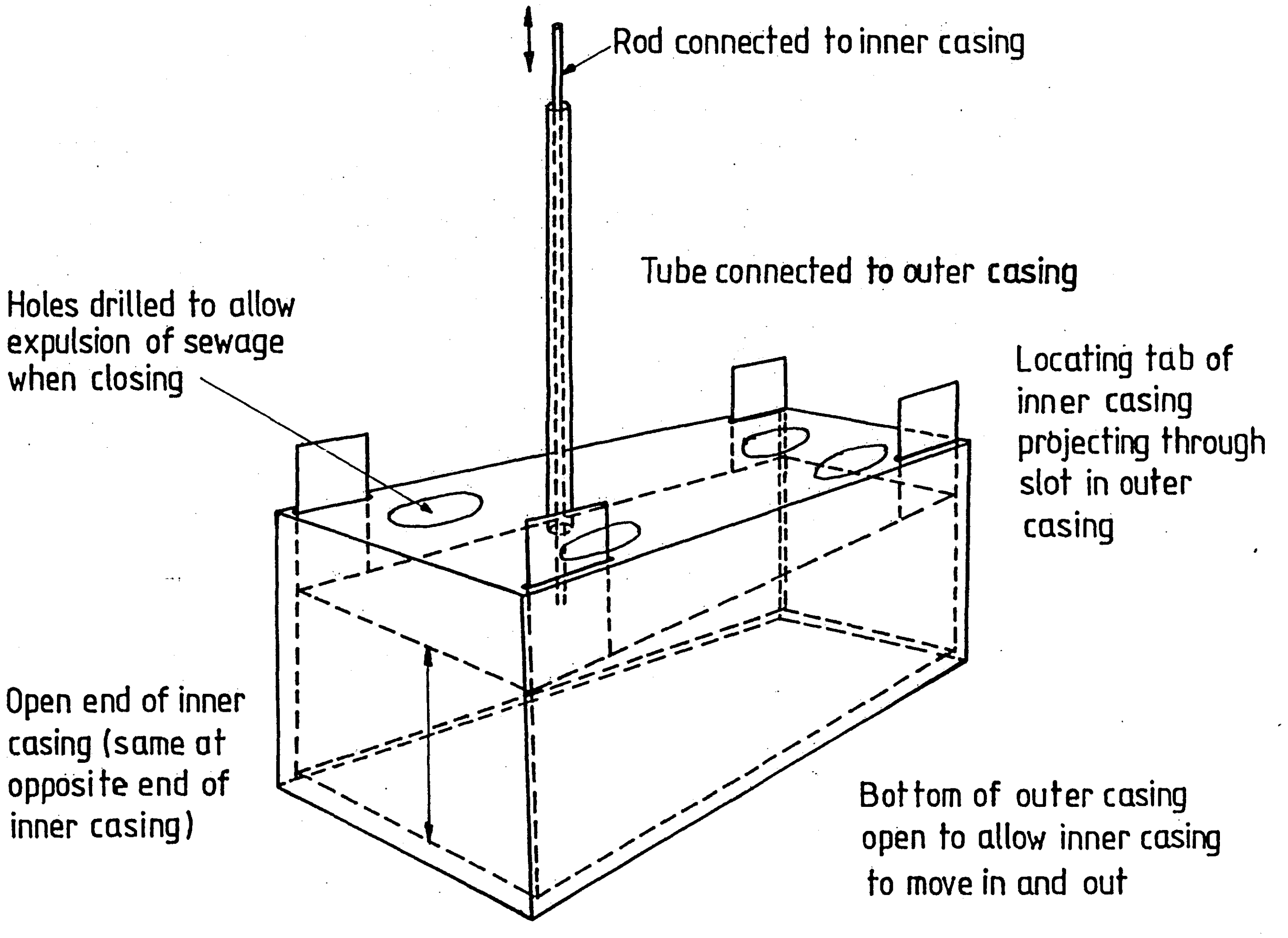


Figure 4. Sediment Sampling Tool.

4. The subsample is placed in an industrial food blender (see below for specification), and blended for a minimum of 30 seconds at low speed. In some cases, certain highly viscous samples and those with a high fat content will not initially circulate freely in the blending jar. In such cases, the blender is stopped at intervals and the sample mixed by hand to bring more of the sample into contact with the blender blades until a completely homogeneous appearance and consistency is obtained.

5. After blending, the sample pH may be directly measured by inserting a pH electrode. If the sample is sufficiently fluid, an amount may also be taken directly for measurement of ammonia content. If this is not practical, steps 6, 7 and 8 are completed before the ammonia test is carried out, with the results being factored up to take account of dilution.

6. The blended sample is diluted with distilled water. Dilution proportions are 10% sample to 90% distilled water by volume.

7. The diluted sample is blended once more for 30 seconds at high speed.

8. The resultant suspension is stored at 5 °C overnight to allow grit particles to settle out, and the supernatant liquid is used for testing purposes (eg COD,BOD) where an aqueous solution is required - the results of these tests are then factored up.

SPECIFICATION FOR BLENDER

The details given below are the manufacturers specifications of the particular model of blender used in the procedures referred to:-

Waring Bar Blendor BB1050
Model No 34BL57
240 volts, 1.5 amps
"Hi" and "Lo" Speed Settings - rpm not specified

Manufacturer - Waring Products Division
Dynamics Corporation of America
New Hartford
Conneticut 06057

Stockists - Europa Ecosse, 26 Exchange Street, Dundee

DISCUSSION OF PROCEDURE

The discussion points listed below are numbered to coincide with the part of the procedure to which they refer:-

1. It is assumed that, since the sampling procedure is carried out in a consistent manner, the negligible amount of sewage which may become mixed with the sediment will give a small and reasonably constant error. No attempt has been made to quantify this error, and all subsequent procedures have been developed on the basis that the sample contains only sediment, including interstitial liquid.

2. Some interstitial fluid will separate and be above the sample on receipt at the laboratory. The blending procedure ensures that the degree of such settlement is not relevant.

3. It is possible that the particle size distribution of a sample could be affected by blending since the metal blades could break down some of the particles. A subsample is therefore kept aside for such purposes if this test is required. Splitting of the sample into subsamples is done by emptying the mixed contents of the first container into two new containers, the transfer being done in small amounts which are alternatively placed into each container. If a smaller quantity than the 2 subsamples obtained is required, the procedure is repeated on one of the subsamples, etc.

4. The 30 second blending period is an abitrary figure, but is a reasonable amount of time for most samples. Also, the low speed seems to be more effective for undiluted samples, with less risk of burning out the blender motor. This time period is specified for consistency in preparation, but the proviso stated is a necessary practical consideration since some samples do not blend without a certain amount of "priming".

5. Many sediments become more liquid after blending, and take on the appearance of a smooth dark grey emulsion. In this state they may well be suitable for ammonia testing. Some, however, have too high a solids content to allow this test to be carried out without dilution.

6. Dilution proportions are arbitrary to change the physical state of the

sediment and to enable direct readings to be taken.

7. The reason for blending the diluted sediment is to separate organic material from grit as much as possible. Hence the higher blender speed is used.

8. At the end of the blending period, a mixture of grit and organic material is in suspension. Allowing to stand overnight lets the grit particles settle out, leaving a supernatant liquor containing organic material in suspension/dilution. If the liquor has been diluted the results will be factored up by the appropriate ratio.

CONCLUDING REMARKS

The procedure stated has been evolved to meet the needs of the research work being carried out. It is of necessity an empirical method of arriving at results which may not have absolute accuracy, but relies on consistency to give comparative results. However, the logic used in arriving at the procedure gives a reasonably representative set of values.

Appendix F

Sediment Sample Analysis Results

Table F1

SUMMARY - SEWAGE/SEDIMENT SAMPLES TAKEN FROM INTERCEPTOR SEWER May 1987 - September 1989

ALL DATA IN CHRONOLOGICAL ORDER

Date	Sample Sediment Class/ Profile (P)	Type Sewage	DWF/ ADWP hrs	Storm vol (mm)	Location 99	Site 98	No. of Samples	Laboratory tests Sediment/ Sewage-peak values COD mg/l	BOD mg/l	SS mg/l	pH	NH4 mg/l	DO mg/l	Cond	Density kg/l		Remarks
12/5/87	C/A				X	X	2	5029	-	-	6.2	-	-	-	1.02	Preliminary trial	
12/5/87		X	D		X		1	340	-	-	-	-	-	-	0.99		
20/5/87	A/C				X		1	11306	3151	-	6.7	-	-	-	0.99		
27/5/87		X	D		X		1	-	88	-	-	-	-	-	-	"	"
27/5/87	E				X		1	15604	4869	-	6.2	214	-	-	1.23		
27/5/87	D					X	1	35550	-	-	-	-	-	-	1		
3/6/87	A/C				X		1	22659	18720	-	6.2	-	-	-	1.78		
9/6/87		X	D		Ch 85m		1	227	358	19	7.2	17.6	-	-	0.98	Also LOD, TS, NVS, Temp	
9/6/87	C/A				X	X	2	8145	4030	-	6.7	96	-	-	1.16		
24/6/87	(P)	X	D		Queen Stre		1	555	349	-	6.9	42.4	-	-	1	"	"
24/6/87	A				"		2	2900	5030	-	5.8	385	-	-	1.68		
2/7/87	C/A				X		1	16656	6452	-	-	230	-	-	1.32		
8/7/87	A				X	X	2	8234	-	-	-	153	-	-	1.06		
14/7/87	(P) A&A/C				X	X	2	12179	10502	-	5.9	1790	-	-	1.6		
21/7/87	A/C & C/A				X	X	2	17821	-	-	6.1	210	-	-	1		
12/8/87	(P)							-	-	-	-	-	-	-	-		
2/9/87		X	D/28		X	X	22,23	153.8	-	414	7.6	84	5.7	-	-	Sample tubes freely suspended	
8/9/87		X	D/171		X	X	22,22	512	218	512	7.6	100	5.5	-	-	"	Flows mismatch
9/9/87	C/A & C/A				X	X	2	9213	1432	-	-	224	-	-	1.47		
29/9/87		X	D/141			X	23	715	870	256	7.4	42	3.1	-	-	"	Samples at one end
7/10/87	C				X		1	4670	-	-	4.9	158	-	-	1.42		
13/10/87		X	D/8		X	X	19,22	1578	-	209	7.6	41.3	2.25	-	-	"	Flows mismatch
29/10/87	(P)							-	-	-	-	-	-	-	-		
4/11/87		X	D/73		X	X	24,24	>1500	-	131	-	51.8	5.75	-	-	Rigid tubes 250mm above invert	
5/11/87		X	D/96		X	X	24,24	1101	-	349	-	44.1	4.9	-	-	"	
29/11/87	(P) A,C/A,				Along	length	5	>3000	-	-	6.57	770	-	-	1.97		
8/12/87	A/C & E				Ch 85 &	X	4	61153	-	-	-	252	-	-	1.82		
11/12/87		X	D/131		X	X	24,22	889	-	603	7.3	224	8.3	-	-	Rigid tubes 250mm above invert	
12/12/87		X	D/155		X	X	24,19	1333	562	210	7.3	168	9.8	-	-	"	Frost
13/12/87		X	D/178		X	X	23,21	1078	-	174	7.8	206	5.9	-	-	"	"
14/12/87		X	D/203		X	X	24,24	928	371	510	7.4	169	9.8	915	-	"	"
15/12/87		X	D/227		X	X	24,22	790	-	280	7.4	93	9.2	4650	-	"	"
16/12/87		X	D/S(5,15)	3	X	X	13,24	668	257	107	7.8	20.3	9	995	-	"	Site 98 (0940-0840) /99 (2001 - 0820)
17/12/87		X	D/S/13	1.6	X	X	24,24	916	-	153	7.4	182	8.9	768	-	"	
18/12/87		X	D/17		X	X	24,24	823	143	791	7.6	266	7.4	695	-	"	

Table F2

SUMMARY - SEWAGE/SEDIMENT SAMPLES TAKEN FROM INTERCEPTOR SEWER May 1987 - September 1989

ALL DATA IN CHRONOLOGICAL ORDER

Date	Sample	Type	DWF/	Storm	Location	Site		Laboratory tests									Remarks
			ADWP	vol				Sediment/									
	Sediment	Sewage	hrs	(mm)	99	98	No. of	Sewage-peak values									
	Class/						Samples	COD	BOD	SS	pH	NH4	DO	Cond	Density		
	Profile (P)							mg/l	mg/l	mg/l		mg/l	mg/l		kg/l		
16/2/88	(P)							-	-	-	-	-	-	-	-		
23/2/88	A/C& A				X	X	3	32139	-	-	6.1	217	-	-	1.6		
22/3/88	C/A				X		2	25391	-	-	6	189	-	-	1.66		
29/3/88	A,A/C,C/A				X	X	4	19200	-	-	6.2	67.9	-	-	1.94		
19/4/88	C				Ch -3.5m		1	870	251	-	4.9	72.9	-	-	1		
27/4/88	A/C,C/A				Along	length	9	-	-	-	5.9	16	-	-	1.46	Sewer drained down	
3/5/88	(P) C/A,A/				Along	length	9	29400	14685	-	6.5	55.6	-	-	1.76	"	
11/5/88	C/A,A/C,A				Along	length	9	33488	34894	-	7	24.2	-	-	1.81	"	
26/5/88			D/24		X		1	-	-	109	-	-	-	-	-	Rigid tubes 250mm above invert	{Settlement
1/6/88			D/30		X		1	-	-	297	-	-	-	-	-	"	{velocity
3/6/88		X	S/15	0.8	X	X	23,24	1300	-	787	6.8	18.9	1	-	-	"	Flow mismatch
8/6/88	(P)							-	-	-	-	-	-	-	-	"	
13/6/88		X	D/204		X	X	20,23	720	-	497	7.9	32.9	5.4	-	-	Rigid tubes 360mm above invert	{Settlement
27/6/88		X	D/31		X	X	24/24	1490	204	314	7.7	40.6	5.3	-	-	"	velocity
1/7/88		X	S/82	0.5	X	X	24/24	2000	267	630	7.3	25.9	5.7	-	-	"	
4/7/88		X	S/55	3.3	X	X	24/23	765	103	519	7.5	14.4	3.3	-	-	"	
5/7/88	(P)							-	-	-	-	-	-	-	-	"	
6/7/88		X	S/33	3.7	X	X	39/38	1800	261	1955	7.5	13.9	7.3	-	-	"	
12/7/88		X	D/S/44	9.2		X	15	-	195	166	7.2	19.4	7.1	-	-	"	DWF initially then rain -
13/7/88		X	S/0	12.2	X	X	23,24	410	107	175	7.4	14.7	6.9	-	-	"	Storm started on 12/7/89
14/7/88		X	D/9		X	X	7,15	-	-	284	-	-	-	-	-	"	
15/7/88		X	D/23		X	X	24,24	-	244	231	-	21.6	-	-	-	"	
16/7/88		X	S/54	11	X	X	41,40	450	-	561	7.6	9.8	-	-	-	"	
17/7/88		X	D/3		X	X	24,24	-	-	145	-	-	-	-	-	"	
18/7/88	(P)							-	-	-	-	-	-	-	-		
19/7/88		X	D/3		X	X	23,15	675	329	362	7.5	23.3	2.8	-	-	"	
11/8/88		X	D/S/10	4.8	X	X	24,24	780	180	375	7.6	35.1	6.9	-	-	"	
15/8/88		X	D/17		X	X	24,24	1325	236	334	7.6	22.8	5.1	-	-	"	
17/8/88		X	S/63	12.9	X	X	24,24	1055	152	639	7.5	20	8	-	-	"	1 hourly intervals
18/8/88		X	S/0	14.8	X	X	24,22	455	182	882	7.8	116	7.6	-	-	"	1 hourly intervals
23/8/88		X	D/S/2	2.4	X	X	24,24	975	193	196	7.6	17.9	5.9	-	-	"	
29/8/88		X	D/46		X	X	24,21	660	229	290	7.5	16.3	6.1	-	-	"	No flow data 99
6/10/88	(P) C				Ch 45m		5	24900	-	-	6	139	-	-	-		CLEANOUT
17/11/88		X	S/112	3.2	X	X	19,19	630	118	398	7.5	11.5	7.1	-	-	Rigid tubes 300mm above invert	Bac
21/11/88	(P)							-	-	-	-	-	-	-	-		
27/11/88		X	D/192		X		23	725	198	238	7.6	30.7	7.6	-	-	"	Bac
29/11/88	A/C					X	2	1495	-	-	6.8	-	-	-	-		Bac
2/12/88		X	D/24		X	X	23,24	1600	-	185	7.6	22.4	10.7	-	-	"	Bac
12/12/88		X	D/109		X	X	24,11	1500	-	614	7.8	139	6.8	-	-	"	
19/12/88	(P)	X	D/40		X		24	1180	-	169	7.6	26.7	6.2	-	-	"	Bac

Table F3

ALL MURRAYGATE/INTERCEPTOR SEDIMENT DATA 12/5/87 - 28/9/89

SEDIMENT DATA MAY 1987 - September 1989

																	wet solids			dry solids
SAMPLE		NUMBER	PERCEIVE	PERCEIVE	CHAINAGE	COD	AMM	pH	TEMP	DENSITY	BOD	BOD	LOD	TS	NVS	VS	COD	AMM	BOD	COD
<———	———	———>	CLASS	CLASS	(M)	(MG/L)	(MG/L)		(DEG C)	(KG/L)	(MG/L)	(g/l)	(%)	(%)	(%)	(%)	(MG/KG)	(MG/KG)	(MG/KG)	(G/KG)
870512	99	S	A/C	2	0	4352	-	6.2	-	0.968			91.4	8.6	4.1	4.5	4495.868		-1000	50.60465
870512	98	S	A/C	2	173	4970	-	6	-	1.012	-		87.1	12.9	5	7.9	4911.067		-1000	38.52713
870520	99	S	C/A	3	0	11394		6.7		0.992	3151	3.151	27.6	72.4	44.3	28.1	11485.89		3176.411	15.73757
870527	98	S	C/A	3	173	14536	168.5	5.7		1.227	4070	4.07	66.8	33.2	27.3	5.9	11846.78	137.3268	3317.033	43.78313
870527	99	S	C/A	3	0	12928	177.6	6.2		1.207	4034	4.034	61.6	38.4	32	6.4	10710.85	147.1417	3342.171	33.66667
870527	98	S	D	6	173	35550														
870603	99	S	A	1	0	12730		6.2		1.78	10517	10.517	38.8	61.2	46.6	14.6	7151.685		5908.427	20.80065
870609			E	5	125	8145	82.8	5.3		1.159	3474	3.474	18.2	81.8	19.9	61.9	7027.61	71.4409	2997.412	9.957213
870609			C	3	48	8735	52.3	6.7		0.932	2672	2.672	48.3	51.7	46.7	5	9372.318	56.11588	2866.953	16.89555
870616	99	S	C	3	0	17195	201.4	5.7		1.467	9153	9.153	42.9	57.1	53.2	3.9	11721.2	137.287	6239.264	30.11384
870616			C	3	85	12014	195.4	5.8		1.468	9221	9.221	39	61	57.9	3.1	8183.924	133.1063	6281.335	19.69508
870624			A	1	250	1726	229.1	5.8		1.677	2994	2.994	25.6	74.4	73.2	1.2	1029.219	136.613	1785.331	2.319892
870624	Samuels		A	1	-250	1077	73.1	6.3		1	898	0.898	99.4	0.6	0.4	0.2	1077	73.1	898	179.5
870702	99	S	C/A	3	0	16656	229.91		15.5	1.321	6452	6.451908	56.5	43.5	40.0	3.5	12606	174	4883	38.29036
870708	98	S	A	1	173	6889	138.55		16	1.005			94.0	6.0	4.4	1.6	6857	137.9		114.8205
870708	99	S	A	1	0	8234	153.09		16	1.056			53.3	46.0	43.7	2.3	7799	145		17.9004
870714	98	S	A	1	173	12179	1790.32	5.93	16	1.598	10502	10.50214	34.7	65.3	63.0	2.3	7619	1120	6570	18.6508
870714	99	S	C/A	3	0	11930	311.85	5.87	16	1.499	14545	14.54471	36.7	63.3	60.6	2.7	7957	208	9701	18.84665
870721	98	S	A/C	2	173	6986	209.94	6.12	17	1.000			94.1	5.9	4.9	1	6988	210		118.4051
870721	99	S	C/A	3	0	17821	121.37	5.14	16	0.970	.		90.9	9.1	4.6	4.5	18368	125.1		195.8311
870819	98	S	A/C	2	0	5748	931.77	5.6		1.071	7214	7.214256	39.1	60.9	57.6	3.3	5367	870	6736	9.438517
870909	98	S	A/C	2	173		231.55		16.25	1.206	7710	7.71	32.0	68.0	67.4	0.6	-1000	191.9983	6393.035	
870909	99	S	C/A	3	0	9213	224.15		15.75	1.466	1432	1.432	48.0	52.0	49.8	2.2	6284.447	152.899	976.8076	17.71731
870909		LAGGAN G	C/A	3		2441	154.18		15	1.758	1238	1.238	17.6	82.4	81.2	1.2	1388.51	87.70193	704.2093	2.962379
871007	99	S	C	4	0	4670	158.47	4.94	12	1.420			44.2	55.8	51.2	4.6	3288.732	111.5986		8.369176
871129	98	CS(A)	C/A	3	40	9000	78.38	5.66	11	1.650			33.5	66.5	66.0	0.5	5454.545	47.50303		13.53383
871129	98	S	A	1	185	1000	56.15	6.57	12.5	1.970			20.6	79.4	78.5	0.9	507.6142	28.50254		1.259446
871129	98	BS	C/A	3		5500	74.27	5.16	12.5	1.404			49.7	50.3	48.7	1.6	3917.379	52.89886		10.93439
871129	98	CS(B)	C/A	3	40	14100	311.17	6.56	11	1.630			76.7	23.3	20.7	2.6	8650.307	190.9018		60.51502
871129	99	AS	D	6		>30000	770.53	5.64	12.5	0.878			57.0	43.0	23.7	19.3		877.5968		
871208	98	AS(INV)	A/C	2	173	22256	217.16		12	1.820			25.4	74.6	74.4	0.2	12228.57	119.3187		29.83378
871208	98	AS(CHAM	E	5	173	61153	147.11		11	0.963			94.4	5.6	1.7	3.87	63502.6	152.7622		1092.018
871208	98	BS	A/C	2		31579	161.12		12	1.440			40.3	59.7	56.1	3.6	21929.86	111.8889		52.89615
871208	99	S	A/C	2	0	39198	252.19		12	1.708			30.5	69.5	68.5	1	22949.65	147.6522		56.4

Table F4

SUMMARY - SEWAGE/SEDIMENT SAMPLES TAKEN FROM INTERCEPTOR SEWER ALL DATA IN CHRONOLOGICAL ORDER

May 1987 - September 1989

Date	Sample	Type	DWF/	Storm	Location	Site		Laboratory tests									Remarks
			ADWP	vol				Sediment/									
	Sediment	Sewage	hrs	(mm)	99	98	No. of	Sewage-peak values									
	Class/						Samples	COD	BOD	SS	pH	NH4	DO	Cond	Density		
	Profile (P)							mg/l	mg/l	mg/l		mg/l	mg/l		kg/l		
13/1/88	(P)							-	-	-	-	-	-	-	-		
20/1/89		X	D/168		X	X	14,20	1700	-	306	7.3	121	7.5	-	-	"	Bac
27/1/89		X	S/31	6.4	M	M	30,27	11475		1529	7.4	15.5	7.6	-	-		300, 600 & 1100mm
31/1/89	(P)							-	-	-	-	-	-	-	-		
2/2/89	A/C				X			1080	-	-	5.5	74.5	-	-	-		Bac
13/2/89		X	S/21	2.2	Samuels		22	5000	240	1728	-	16.6	-	-	-	Flexible tube near invert	Bac
27/2/89	A/C				"		1	230	66	-	6.8	48.4	-	-	-		Bac
9/3/89		X	S/18	6.0	"		24	570	183	144	-	14.5	-	-	-	Flexible tube near invert	Bac
12/3/89	(P)	X	S/38	3.8	"		24	1250	-	482	-	-	-	-	-	"	Bac
12/3/89	A/C				"		1	245	60	-	7.2	33	-	-	-		"
16/3/89	A/C				"		1	340	54	-	7	38	-	-	-		"
4/4/89	A/C				"		1	225	-	-	6.9	61	-	-	-		"
5/4/89		X	D/24		"		17	1150	154	190	-	27.9	-	-	-	"	"
27/4/89		X	S/12	3.6	X		24	1795	244	614	-	13.2	-	-	-	rigid tube 300mm	"
27/4/89	A/C				Samuels		1	340	28.6	-	7.3	149	-	-	-		"
	A/C				X		1	520	222	-	6.1	50	-	-	-		"
2/5/89	A/C				X		1	1200	190	-	5.5	85.6	-	-	-		"
11/5/89		X	S/312	5.2	X		40	1450	239	804	-	11.9	-	-	-	rigid tube 300mm	"
18/5/89	(P) A/C				X		4	1380	219	-	6.2	37.7	-	-	-		
9/8/89	(P) A/C				Along leng		4	22000	7365	-	8.2	153	-	-	1.93		
10/8/89		X	D/2		X		22	-	-	452	-	-	-	-	-	Tubes 300, 600, 1100 above invert	
15/8/89	(P) A/C				Along leng		4	29000	7450	-	8	375	-	-	1.8	"	
15/8/89		X	S/62	1.0	M	M	31,32	790	237	371	7.8	11.5	7.7	-	-	"	flows only at 98
20/8/89		X	S/2.5	1.6	M	M	24,26	-	-	124	-	-	-	-	-	"	flow at 99 erroneous
30/8/89		X	S/4.5	14.4	M	M	99,107	-	208	601	8.2	103	8.6	-	-	"	
5/9/89	(P)							-	-	-	-	-	-	-	-	"	
7/9/89	A/C				Along leng		5	11200	3888	-	8	198	-	-	1.8	"	
13/9/89		X	S/D/28	0.2	X	X	24,24	1900	263	237	8.4	41	4.9	-	-	"	No flow data for 98
14/9/89	(P) A/C				Along leng		5	27200	5940	-	7.7	90.1	-	-	2.15	"	
14/9/89		X	D/18		X	X	24,24	890	-	259	8.4	22.2	5.5	-	-	"	
19/9/89		X	D/9		X		24	-	-	278	-	-	-	-	-	"	
21/9/89		X	S/D/16	1.6	M	M	24,26	-	-	276	-	-	-	-	-	"	
22/9/89		X	S/23	13.2	M	M	56,63	-	-	269	-	-	-	-	-	"	
23/9/89		X	D/21		X	X	24,24	-	-	250	-	-	-	-	-	"	NO flow data for 99
26/9/89		X	D/90		X	X	24,24	790	-	234	8.5	30.4	4.5	-	-	"	NO flow data for 99
27/9/89		X	D/114		X	X	24,24	-	-	225	-	-	-	-	-	"	NO flow data for 99
28/9/89	(P) A/C				Along leng		4	25000	-	-	6.9	73.8	-	-	1.88		

Table F5

ALL MURRAYGATE/INTERCEPTOR SEDIMENT DATA 12/5/87 - 28/9/89

SEDIMENT DATA MAY 1987 - September 1989

															wet solids				dry solids
SAMPLE		NUMBER	PERCEIVE	PERCEIVE	CHAINAGE	COD	AMM	pH	TEMP	DENSITY	BOD	BOD	LOD	TS	NVS	VS	COD	AMM	BOD	COD
<------	------	------>	CLASS	CLASS	(M)	(MG/L)	(MG/L)		(DEG C)	(KG/L)	(MG/L)	(g/l)	(%)	(%)	(%)	(%)	(MG/KG)	(MG/KG)	(MG/KG)	(G/KG)
880223	98	S	A/C	2		32139	70.06	5.4	9.5	1.521			36.2	63.8	60.8	3	21130	46.0618		50.37418
880223	99	S(B)	A	1		10210	189.13	6.1	9.5	1.597			37.0	63.0	62.0	1	6393	118.4283		16.20575
880223	99	S(A)	A/C	2		12271	217.15	5	10	1.458			37.5	62.5	60.8	1.7	8416	148.9369		19.63284
880322	99	S(B)	C/A	3	0	4730	189.13	6	.	1.663			37.0	63.0	62.5	0.5	2844	113.7282		7.507257
880322	99	S(A)	C/A	3		25391	161.12	5.3		1.588			38.4	61.6	59.6	2	15989	101.461		41.2184
880329	98	S(A)	A	1	172	19200	67.94	5.3	10.5	1.519			38.4	61.6	58.2	3.4	12640	44.72679		31.16909
880329	98	S(B)	A	1	174	12000	28.71	6.2	10.5	1.940			21.0	79.0	77.1	1.9	6185.6	14.79897		15.18995
880329	99	S(A)	C/A	3	0	9600	61.64	5.7	10	1.662			35.1	64.9	63.5	1.4	5776	37.08785		14.79154
880329	99	S(B)	A/C	2	-2	14399	67.95	5.4	10.5	1.606			34.2	65.8	64.0	1.8	8966	42.31009		21.88358
880419	99	S	C	4	-4	870	72.90	4.94	11	1.000	251	0.251					870	72.9	251	
880427	98	CS(C)	C/A	3	50		6.09	5.44	14	1.360			54.3	45.8	40.5	5.21		4.48		
880427	98	CS(B)	C/A	3	40		15.96	5.6	11.75	1.460			47.9	52.1	34.4	17.62		10.93		
880427	98	CS(A)	A/C	2	30		8.89	5.83	12	1.380			51.4	48.6	45.6	3.04		6.44		
880427	98	BS(B)	A/C	2	80		5.85	5.69	13	1.100			86.9	13.1	11.4	1.72		5.32		
880427	98	CS(D)	C/A	3	60		10.64	5.94	12.5	1.460			45.5	54.5	50.5	4.03		7.29		
880427	98	BS(A)			70		13.83	5.44	13	1.410			47.9	52.1	46.3	5.82		9.81		
880427	99	S(C)	A/C	2	20		8.10	5.89	14	1.630			35.1	64.9	61.3	3.61		4.97		
880427	99	S(A)	A/C	2	-3.5		2.19	5.56	13.5	1.060			83.4	16.6	6.6	9.99		2.07		
880427	99	S(B)	C/A	3	10		15.65	5.88	14	1.450			45.1	54.9	51.5	3.39		10.79		
880503	98	BS(A)	C/A	3	70	15480	46.09	6.51	13	1.290	7385	7.38525	36.5	63.5	58.9	4.63	12000	35.73	5725	
880503	98	CS(D)	C/A	3	60	10725	22.04	6.04	13	1.430	6456	6.45645	38.5	61.4	59.9	1.54	7500	15.41	4515	
880503	98	CS(A)	C/A	3	30	6423	19.13	6.46	13.5	1.750	3544	3.54375	26.1	73.9	73.4	0.51	3670	10.93	2025	
880503	98	CS(C)	C/A	3	50	29400	55.61	5.89	13	1.470	14685	14.6853	42.0	58.0	50.5	7.54	20000	37.83	9990	
880503	98	CS(B)	A	1	40	5122	17.00	6.41	13.75	1.810	3430	3.42995	23.5	76.5	75.3	1.19	2830	9.39	1895	
880503	98	BS(B)	C/A	3	80	23700	24.35	5.87	13.25	1.580	13730	13.7302	35.5	64.5	60.3	4.18	15000	15.41	8690	
880503	99	S(B)	A/C	2	10	9099	27.12	6.46	13	1.760	4796	4.796	25.7	74.3	73.3	0.99	5170	15.41	2725	
880503	99	S(A)	A	1	-3.5	7969	9.70	6.27	13	1.650	4183	4.18275	20.8	79.2	78.0	1.17	4830	5.88	2535	
880503	99	S(C)	A/C	2	20	17400	36.07	6.04	12.75	1.740	9439	9.4395	32.2	67.8	65.5	2.22	10000	20.73	5425	
880511	98	CS(C)	C/A	3	50	10354	19.44	5.99	13.75	1.350	33075	33.075	43.5	56.5	50.0	6.53	7670	14.4	24500	
880511	98	BS(B)	A/C	2	80	21947	21.95	6.32	13.75	1.780	29014	29.014	27.8	72.2	68.1	4.15	12330	12.33	16300	
880511	98	BS(A)	C/A	3	70	12162	32.32	6.14	13.5	1.460	34894	34.894	38.9	61.4	58.5	2.83	8330	22.14	23900	
880511	98	CS(D)	C/A	3	60	33488	30.13	6.26	13.75	1.570	25591	25.591	28.8	71.2	65.4	5.77	21330	19.19	16300	
880511	98	CS(B)	C/A	3	40	4486	24.24	6.49	14	1.680	26712	26.712	29.6	70.4	68.4	2	2670	14.43	15900	
880511	98	CS(A)	C/A	3	30	5927	17.21	6.56	13.75	1.780	25810	25.81	26.3	73.7	72.4	1.329	3330	9.67	14500	
880511	99	S(B)	A/C	2	10	2789	14.51	6.89	13.75	1.670	24549	24.549	19.8	80.1	79.6	0.58	1670	8.69	14700	
880511	99	S(C)	A/C	2	20	7188	13.38	6.45	13	1.660	22244	22.244	29.0	71.0	69.1	1.87	4330	8.06	13400	
880511	99	S(A)	A	1	-3.5	28960	16.24	6.96	13.5	1.810	31675	31.675	19.9	80.1	78.7	1.44	16000	8.97	17500	
881006	98	C(v)	C	4	45	24900	129.86	5.406												
881006	98	C(ii)	C	4	45		123.72	5.966												
881006	98	C(i)	C	4	45		129.90	5.548												
881006	98	C(iv)	C	4	45	23800	125.62	5.192												
881006	98	C(iii)	C	4	45	18000	138.70	5.538												
881129			A/C	2	70	1495		6.82												
881129			A/C	2	60	1420		6.61												

Table F6

ALL MURRAYGATE/INTERCEPTOR SEDIMENT DATA 12/5/87 - 28/9/89

SEDIMENT DATA MAY 1987 - September 1989

																	wet solids			dry solids
SAMPLE		NUMBER	PERCEIVE	PERCEIVE	CHAINAGE	COD	AMM	pH	TEMP	DENSITY	BOD	BOD	LOD	TS	NVS	VS	COD	AMM	BOD	COD
<———	———	———>	CLASS	CLASS	(M)	(MG/L)	(MG/L)		(DEG C)	(KG/L)	(MG/L)	(g/l)	(%)	(%)	(%)	(%)	(MG/KG)	(MG/KG)	(MG/KG)	(G/KG)
890202	99		A/C	2	0	1080	74.5	5.53												
890227	Samuels		A	1	-250	230	48.4	6.84			65.9	0.0659								
890312	Samuels		A	1	-250	245	33.1	7.21			60.2	0.0602								
890316	Samuels		A	1	-250	340	37.9	7.03			53.9	0.0539								
890404	Samuels		A	1	-250	225	60.5	6.85												
890427	Samuels		A	1	-250	340	149	7.25			28.6	0.0286								
890427	99		A/C	2	0	520	50	6.09			222.3	0.2223								
890502	99		A/C	2	0	1200	85.6	5.5			190.3	0.1903								
890518	99		A/C	2	0	1380	37.7	6.17			219	0.219								
890809			A/C	2	40	20000	174.8	6.59		1.405	5755	5.755					14234.88	124.4128	4096.085	
890809			A/C	2	85	19000	63.14	6.91		1.589	5315	5.315					11957.21	39.73568	3344.871	
890809			A/C	2	120	4450	48.41	8.21		1.931	405	0.405					2304.505	25.06991	209.7359	
890809	98		A/C	2	174	22000	153.07	6.57		1.574	7365	7.365					13977.13	97.24905	4679.161	
890815			A/C	2	40	5900	50.09	7.26		1.571	1633	1.633					3755.57	31.88415	1039.465	
890815			A/C	2	85	13100	41.9	7.19		1.574	2910	2.91					8322.745	26.62008	1848.793	
890815			A/C	2	120	20000	71.64	7.98		1.798	4600	4.6					11123.47	39.84427	2558.398	
890815	98		A/C	2	174	29000	374.64	6.26		1.444	7450	7.45					20083.1	259.446	5159.28	
890907			A/C	2	40	7000	151.34	6.22		1.601	1502	1.502					4372.267	94.52842	938.1636	
890907	99		A/C	2	0	3750	49.97	7.05		1.659	2148	2.148					2260.398	30.12055	1294.756	
890907			A/C	2	120	5700	88.[illegible]3	6.85		1.797	3668	3.668					3171.953	48.9872	2041.18	
890907			A/C	2	85	8800	87.17	7.95		1.647	1533	1.533					5343.048	52.92653	930.7832	
890907	98		A/C	2	174	11200	198.43	6.7		1.528	3888	3.888					7329.843	129.8626	2544.503	
890914			A/C	2	40	15500	90.13	6.8		1.613	2770	2.77					9609.423	55.87725	1717.297	
890914			A/C	2	85	40000	42.96	5.89		1.436	5310	5.31					27855.15	29.91643	3697.772	
890914			A/C	2	120	3900	29.66	7.68		2.15	546	0.546					1813.953	13.79535	253.9535	
890914	98		A/C	2	0	12800	24	7.13		1.806	880	0.88					7087.486	13.28904	487.2647	
890914	99		A/C	2	174	27200	59.01	6.11		1.509	5940	5.94					18025.18	39.10537	3936.382	
890928			A/C	2	40	8000	70.58	6.54		1.611							4965.86	43.8113		
890928			A/C	2	85	20000	73.81	5.78		1.468							13623.98	50.27929		
890928			A/C	2	120	6000	59.02	6.83		1.882							3188.098	31.36026		
890928	98		A/C	2	174	25000	56.44	6.91		1.854							13484.36	30.44229		

Table F7

Particle size – ashed

SAMPLE		NUMBER	PERCEIVED CLASS	PERCEIVED CLASS	CHAINAGE (M)	D10 mm	D50 mm	D60 mm	D60/D10	D60-D10	D60-D10 /D60	D60-D10 /D10
<------	------	------>										
870512	99	S	A/C	2	0							
870512	98	S	A/C	2	173							
870520	98	S	C/A	3	0							
870527	98	S	C/A	3	173							
870527	99	S	C/A	3	0							
870527	98	S	D	6	173							
870603	98	S	A	1	0							
870609			E	5	125							
870609			C	3	48							
870616	99	S	C	3	0							
870616			C	3	85							
870624			A	1	250							
870624	Samuels		A	1	-250							
870702	99	S	C/A	3	0	0.078	0.18	0.205	2.62820513	0.127	0.6195122	1.62820513
870708	98	S	A	1	173	0.2	1.5	2.4	12	2.2	0.91666667	11
870708	99	S	A	1	0	0.14	0.78	1.4	10	1.26	0.9	9
870714	98	S	A	1	173	0.23	1.4	2.2	9.56521739	1.97	0.89545455	8.56521739
870714	99	S	C/A	3	0							
870721	98	S	A/C	2	173	0.2	1.25	2	10	1.8	0.9	9
870721	99	S	C/A	3	0	0.055	0.18	0.24	4.36363636	0.185	0.77083333	3.36363636
870819	98	S	A/C	2	0							
870908	98	S	A/C	2	173	0.06	0.45	0.58	9.66666667	0.52	0.89655172	8.66666667
870909	99	S	C/A	3	0							
870909		LAGGAN G	C/A	3								
871007	99	S	C	4	0	0.15	0.7	0.8	5.33333333	0.65	0.8125	4.33333333
871129	98	CS(A)	C/A	3	40							
871129	98	S	A	1	185							
871129	98	BS	C/A	3								
871129	98	CS(B)	C/A	3	40							
871129	99	AS	D	6								
871208	98	AS(INV)	A/C	2	173	0.3	0.85	0.9	3	0.6	0.66666667	2
871208	98	AS(CHAMB)	E	5	173							
871208	98	BS	A/C	2								
871208	99	S	A/C	2	0	0.3	0.7	0.75	2.5	0.45	0.6	1.5

SAMPLE	<---- PERCENT AGE PASSING Sieve size 63E-3 (mm)	150E-3 (mm)	212E-3 (mm)	300E-3 (mm)	425E-3 (mm)	600E-3 (mm)	1.18 (mm)	2.0 (mm)	3.35 (mm)	5.0 (mm)	6.3 (mm)	20.0 (mm)	28.0 (mm)	37.5 (mm) ---->
870512														
870512														
870520														
870527														
870527														
870527														
870603														
870609														
870609														
870616														
870616														
870624														
870624														
870702	2.86	31.62	63.1	80.57	85.98	91.64	97.57	99.07	99.71	100	100	100	100	100
870708	1.15	6.85	10.66	16.09	23.18	30.94	45.58	57.03	68.12	76.87	81.85	90.48	95.74	100
870708	0.83	11.91	21.94	30.32	38.98	45.88	57.33	68.04	78.63	85.26	88.19	100	100	100
870714	1.45	5.34	8.42	14.27	22.22	31.2	46.68	56.38	70.52	79.45	85.36	92.25	97.31	100
870714														
870721	3.16	7.1	10.6	16.85	25.42	34.24	48.81	58.68	70.89	79.11	84.91	93.78	98.46	100
870721	8.97	41.55	56.59	66.69	71.16	74.9	82.44	86.86	88.6	89.96	91.51	97.16	100	100
870819	1.11	11.36	29.63	46.86	61.76	75.85	92.43	97.08	98.63	99.29	100	100	100	100
870908	15.25	15.36	24.79	32.73	40.19	63.95	87.57	93.73	96.72	98.68	100	100	100	100
870909														
870909	0.3	0.3	2.6	7.3	11	19.8	37	48.4	60.3	70.6	83.2	100		
871007	0.87	1.03	7.82	21.72	28.33	45.75	73.48	92.6	98.12	99.96	100			
871129														
871129	0.5	0.6	10.5	32.1	41	66.9	87.5	95	97.9	98.7	100			
871129	0.8	0.9	11.6	41.9	51.6	74.7	91	96.9	99.3	99.9	100			
871129	0.5	0.7	9.3	23.4	34.1	80	90.9	96.4	98.9	99.9	100			
871129														
871208		0.43	3.98	12.43	17.19	26.72	73.96	84.89	89.9	93.22	100			
871208														
871208	6.5	6.7	25.6	63.6	67.5	80.9	91.5	96	98.6	99.6	100			
871208	0.42	0.48	3.01	10.68	17.73	41.16	94.74	94.74	98.17	98.98	100			

Table F8

Particle size - ashed

													<— PERCENT AGE PASSING Sieve size —>													
SAMPLE		NUMBER	PERCEIVED	PERCEIVED	CHAINAGE	D10	D50	D60	D60/D10	D60-D10	D60-D10	D60-D10	63E-3	150E-3	212E-3	300E-3	425E-3	600E-3	1.18	2.0	3.35	5.0	6.3	20.0	28.0	37.5
<——	——	——>	CLASS	CLASS	(M)	mm	mm	mm			/D60	/D10	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
880223	98	S	A/C	2																						
880223	98	S(B)	A	1																						
880223	98	S(A)	A/C	2		0.25	0.6	0.7	2.8	0.45	0.64285714	1.8														
880322	98	S(B)	C/A	3	0	0.2	0.45	0.52	2.6	0.32	0.61538462	1.6	0.2	4.1	15.8	28.3	48.6	68.2	80.3	96.8	99.2	99.9	99.9			
880322	98	S(A)	· C/A	3									0.6	3.3	8.6	18.6	33.9	51.6	79.9	91.8	97.2	99.1	99.8			
880329	98	S(A)	A	1	172	0.15	0.85	0.95	6.33333333	0.8	0.84210526	5.33333333	3.5	7.9	16.9	27.1	38	47.3	63.3	74	83.1	90.5	94.7			
880329	98	S(B)	A	1	174	0.4	2.1	3	7.5	2.6	0.86666667	6.5	1.4	3.9	6.3	9.2	14.5	21.5	38.2	48.2	63	71.5	78.3	98.7		
880329	99	S(A)	C/A	3	0	0.25	0.65	0.75	3	0.5	0.66666667	2	1.8	3.8	8.6	17	29.2	47.3	81.6	94.5	98.2	99.8	99.9			
880329	98	S(B)	A/C	2	-2								1.3	2.8	5.8	10.6	20.2	32.9	64.5	83.1	90.1	94	95.4			
880419	98	S	C	4	-4																					
880427	98	CS(C)	C/A	3	50																					
880427	98	CS(B)	C/A	3	40																					
880427	98	CS(A)	A/C	2	30																					
880427	98	BS(B)	A/C	2	80	0.1	0.23	0.295	2.95	0.195	0.66101695	1.95	0.92	18.93	47.77	60.85	74.41	83.75	93.59	97.98	99.88	99.88	99.94	100		
880427	98	CS(D)	C/A	3	60																					
880427	98	BS(A)			70																					
880427	98	S(C)	A/C	2	20																					
880427	98	S(A)	A/C	2	-3.5																					
880427	98	S(B)	C/A	3	10																					
880503	98	BS(A)	C/A	3	70																					
880503	98	CS(D)	C/A	3	60																					
880503	98	CS(A)	C/A	3	30	0.205	0.31	0.49	2.3902439	0.285	0.58163265	1.3902439	0.48	3.34	10.81	23.37	53.09	73.26	92.23	96.95	99.82	100				
880503	98	CS(C)	C/A	3	50																					
880503	98	CS(B)	A	1	40	0.14	0.47	0.53	3.78571429	0.39	0.73584906	2.78571429	0.21	1.9	6.31	16.44	41.9	63.73	89.6	96.28	99.91	99.98	100			
880503	98	BS(B)	C/A	3	80																					
880503	98	S(B)	A/C	2	10																					
880503	98	S(A)	A	1	-3.5	0.21	0.9	1.05	5	0.84	0.8	4	0.07	2.67	5.34	9.09	16.03	27.1	67.53	83.36	90.06	93.9	95.74	100		
880503	98	S(C)	A/C	2	20																					
880511	98	CS(C)	C/A	3	50																					
880511	98	BS(B)	A/C	2	80								0.64	6.98	19.85	35.29	51.8	61.96	75.44	83.04	90.8	93.69	96.42	100		
880511	98	BS(A)	C/A	3	70	0.17	0.4	0.57	3.35294118	0.4	0.70175439	2.35294118														
880511	98	C[illegible]	C/A	3	60	0.16469565	0.67045455																			
880511	98	CS(B)	C/A	3	40																					
880511	98	CS(A)	C/A	3	30																					
880511	98	S(B)	A/C	2	10																					
880511	98	S(C)	A/C	2	20																					
880511	98	S(A)	A	1	-3.5																					
881006	98	C(v)	C	4	45																					
881006	98	C(ii)	C	4	45								—	—	—	—	—	—	—	—	—	—	—	—	—	—
881006	98	C(i)	C	4	45								15.25	41.55	63.1	80.57	85.98	91.84	97.57	99.07	99.91	100	100	100	100	100
881006	98	C(iv)	C	4	45								0	0	0	0	0	0	0	0	0	0	0	0	0	0
881006	98	C(iii)	C	4	45								1.853	6.66166667	15.074	25.8513333	35.3323333	48.3043333	67.5683333	75.761	80.8083333	83.685	85.4110345	79.523125	71.9554545	72.7272727
881129			A/C	2	70																					
881129			A/C	2	60																					

Table F9

Particle size – ashed

													<— PERCENT AGE PASSING Sieve size —>														
SAMPLE		NUMBER	PERCEIVED	PERCEIVED	CHAINAGE	D10	D30	D60	D60/D10	D60-D10	D60-D10	D60-D10	63E-3	150E-3	212E-3	300E-3	425E-3	600E-3	1.18	2.0	3.35	5.0	6.3	20.0	28.0	37.5
<——	——	——>	CLASS	CLASS	(M)	mm	mm	mm			/D60	/D10	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
890202	99		A/C	2	0																					
890227	Samuels		A	1	-250																					
890312	Samuels		A	1	-250																					
890316	Samuels		A	1	-250																					
890404	Samuels		A	1	-250																					
890427	Samuels		A	1	-250																					
890427	99		A/C	2	0																					
890502	99		A/C	2	0																					
890518	99		A/C	2	0																					
890809			A/C	2	40																					
890809			A/C	2	85																					
890809			A/C	2	120				NOT TESTE																	
890809	98		A/C	2	174																					
890815			A/C	2	40																					
890815			A/C	2	85																					
890815			A/C	2	120																					
890815	98		A/C	2	174																					
890907			A/C	2	40																					
890907	99		A/C	2	0																					
890907			A/C	2	120																					
890907			A/C	2	85																					
890907	98		A/C	2	174																					
890914			A/C	2	40																					
890914			A/C	7	85																					
890914			A/C	2	120																					
890914	98		A/C	2	0																					
890914	99		A/C	2	174																					
890928			A/C	2	40																					
890928			A/C	2	85																					
890928			A/C	2	120																					
890928	98		A/C	2	174																					

Appendix G

Sediment Bed Surveys

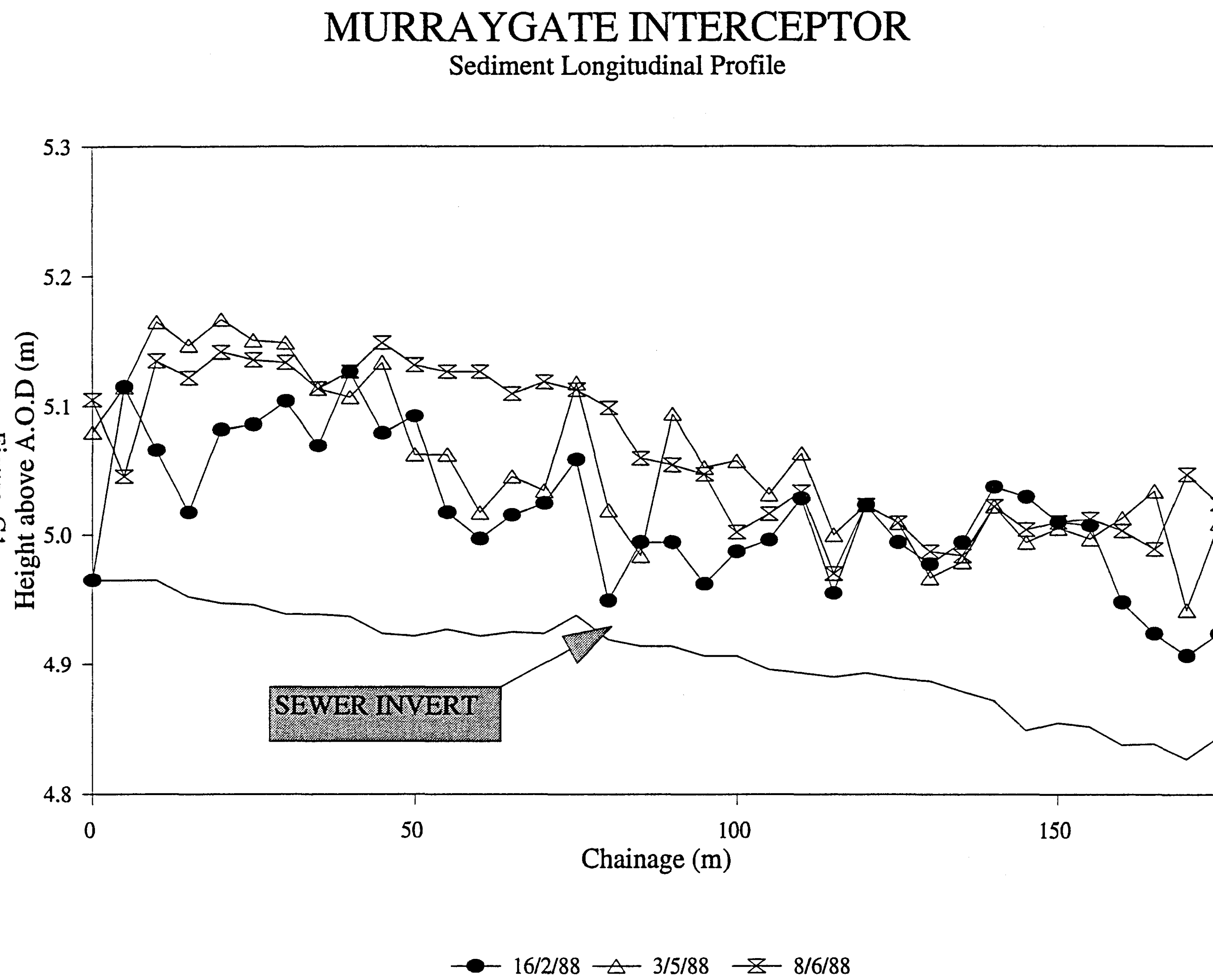

MURRAYGATE INTERCEPTOR
Sediment Longitudinal Profile
Height above A.O.D (m)
5.3
5.2
5.1
5.0
4.9
4.8
SEWER INVERT
0
50
100
150
Chainage (m)
16/2/88
3/5/88
8/6/88

Figure G1

G1

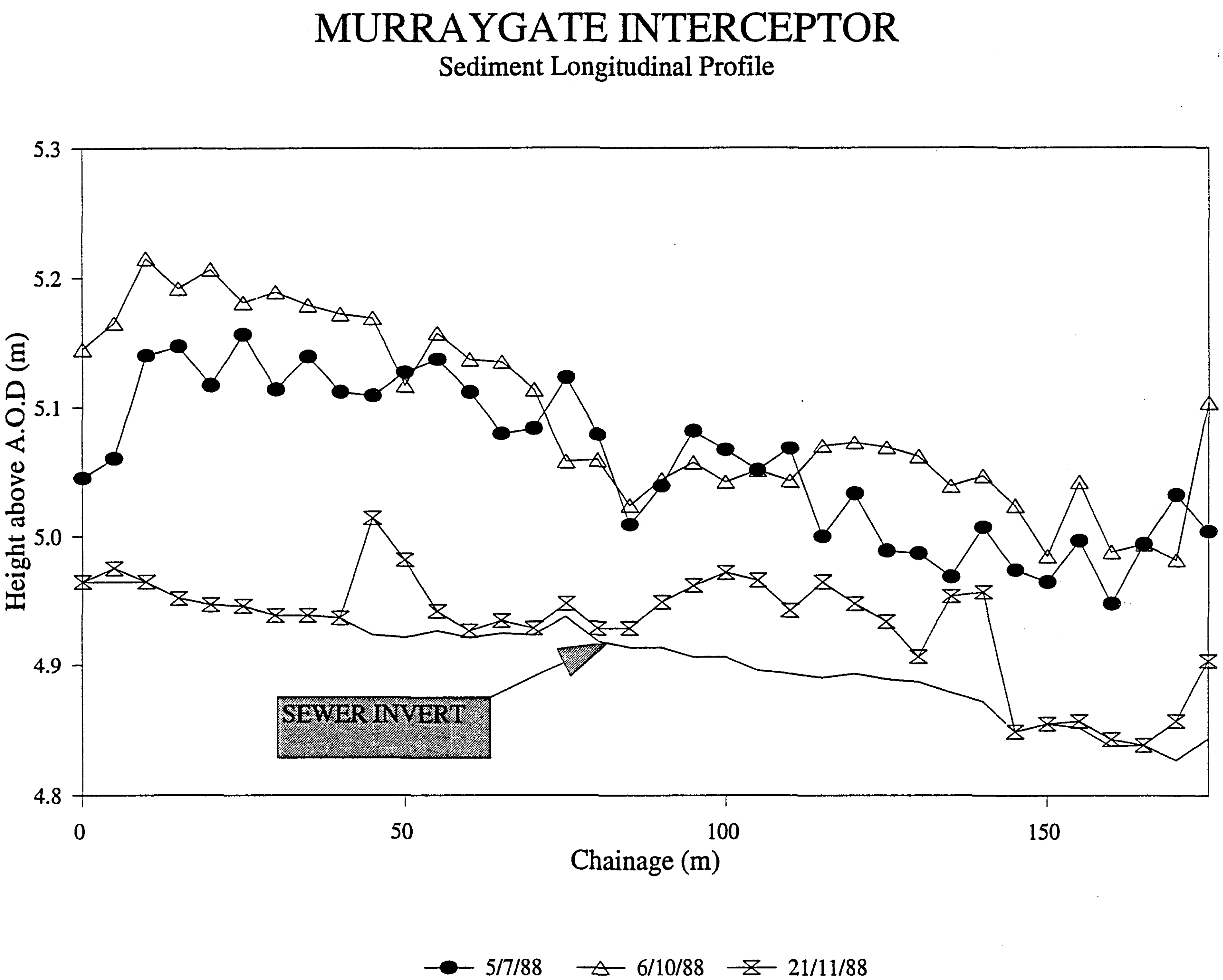

MURRAYGATE INTERCEPTOR
Sediment Longitudinal Profile
5.3
5.2
5.1
5.0
4.9
4.8
Height above A.O.D (m)
SEWER INVERT
0
50
100
150
Chainage (m)
5/7/88
6/10/88
21/11/88

Figure G2

G2

MURRAYGATE INTERCEPTOR

Sediment Longitudinal Profile

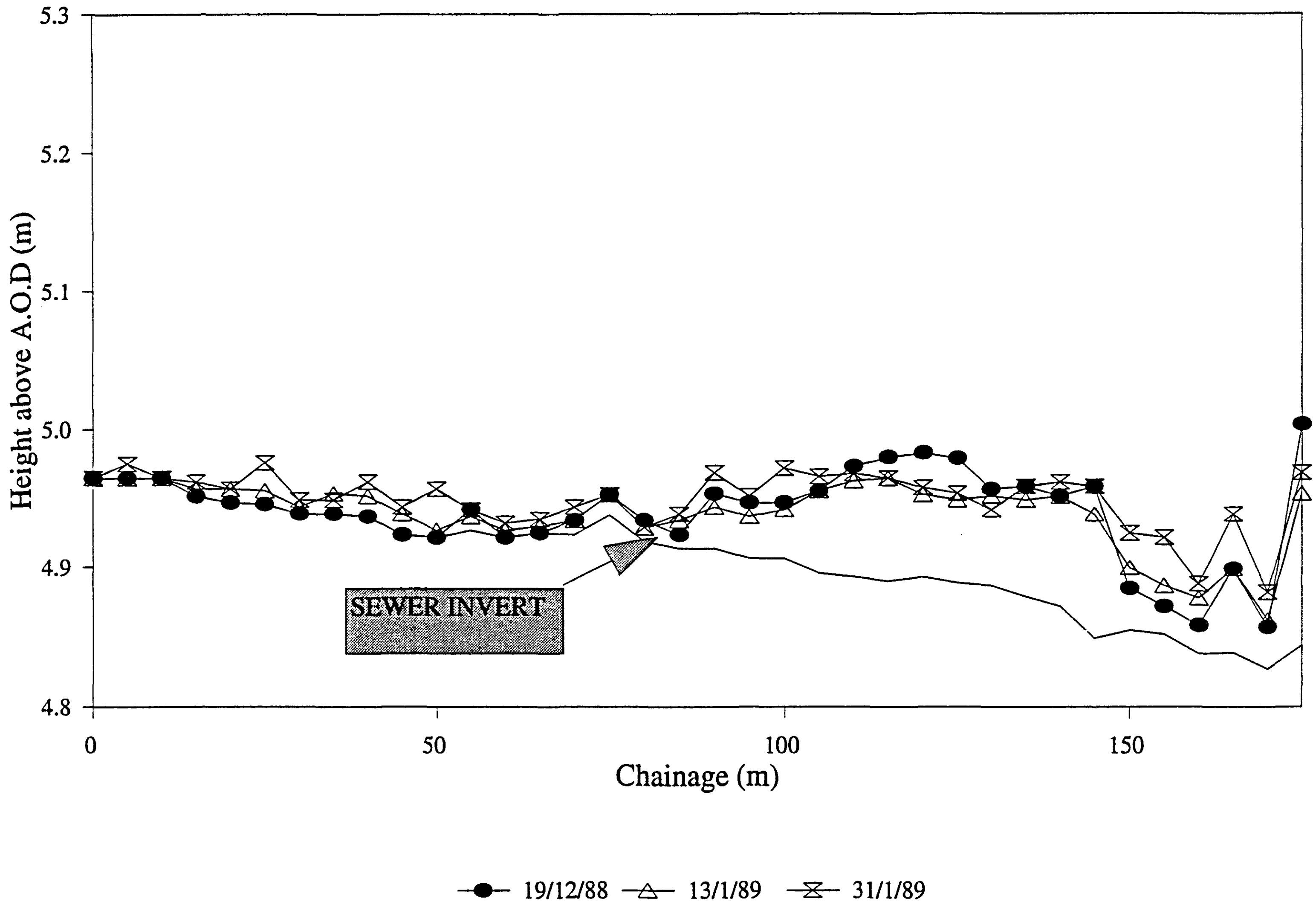


Figure G3

G3

MURRAYGATE INTERCEPTOR

Sediment Longitudinal Profile

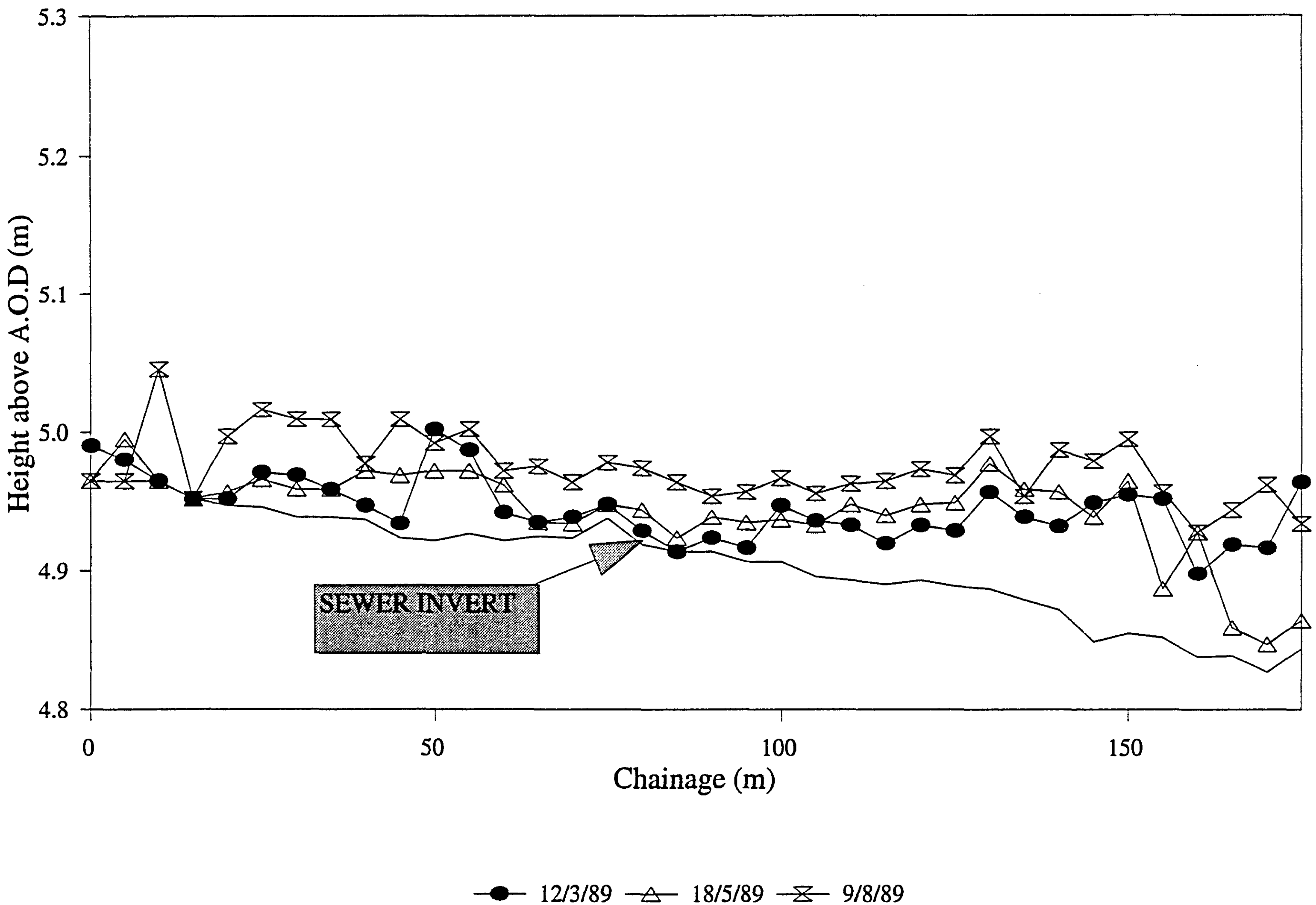


Figure G4

G4

MURRAYGATE INTERCEPTOR

Sediment Longitudinal Profile

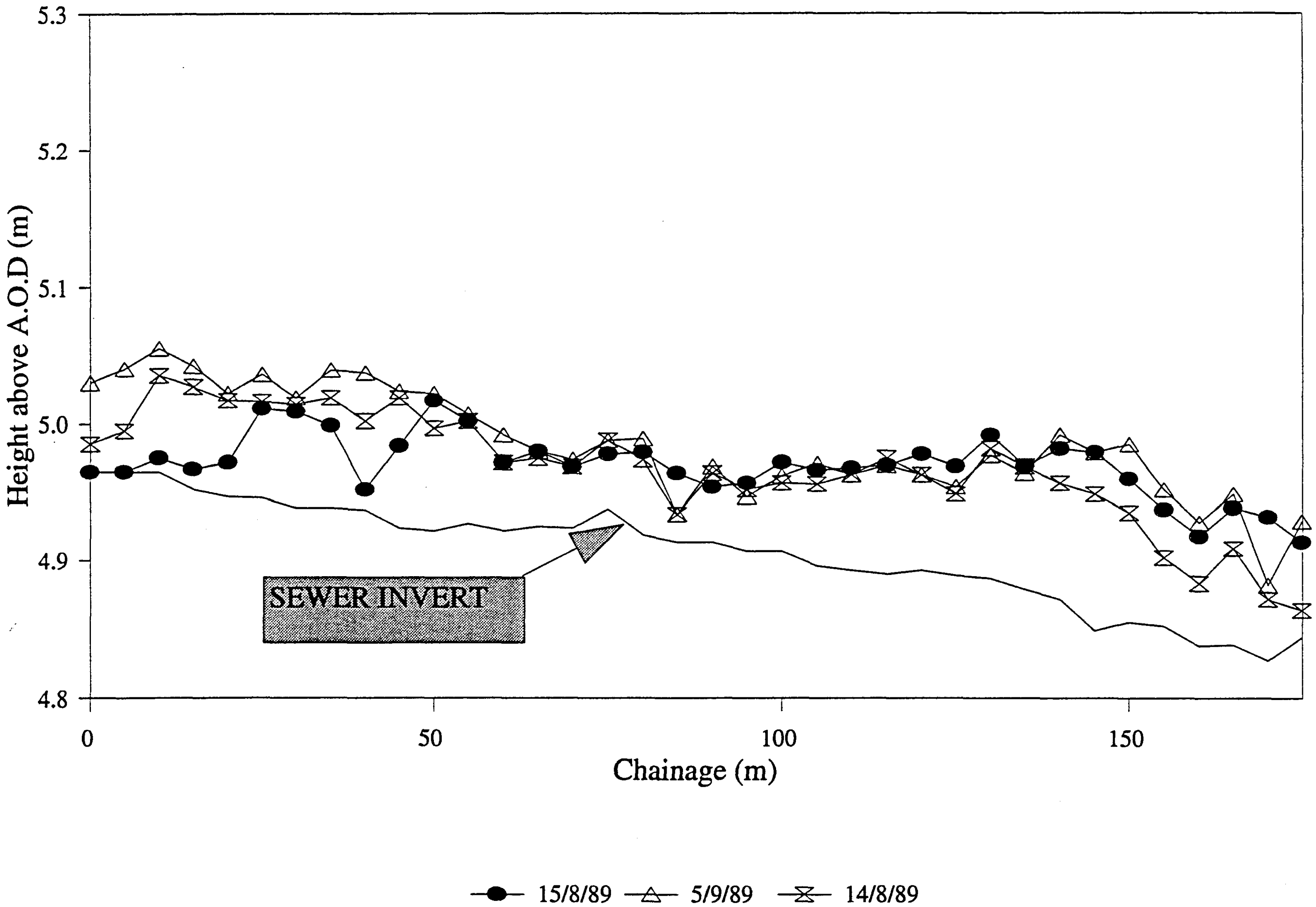


Figure G5

G5

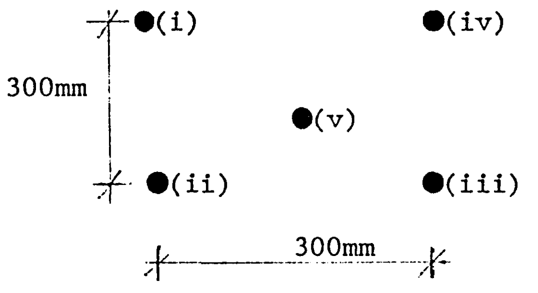
Appendix H

Comparative Grid Result, Interceptor Sewer

Tests on Sediment Samples Taken on 300 mm Grid Pattern

Samples obtained on 6/10/88 from invert of inlet joining main interceptor at CH.45 m approx. Sewer was drained down at time of sampling.

Samples taken on a grid pattern with following numbering system :-



Each of these 5 samples were blended, then split into 5 subsamples numbered e.g. (i) a (i) e.

Visual Description :- All 5 samples had similar appearance.
Highly organic greenish-grey sediment with coarse to fine sand, some gross solids.

Perceived Classification :- Type C

Sample No	pH	NH(mg/1)	COD(mg/1)
(i) a	5.56	128.9	-
(i) b	5.57	129.1	-
(i) c	5.57	130.0	-
(i) d	5.53	129.2	-
(i) e	5.51	132.3	-
(ii) a	6.01	125.4	-
(ii) b	5.93	121.4	-
(ii) c	5.92	124.8	-
(ii) d	5.99	123.7	-
(ii) e	5.98	123.3	-
(iii)a	5.64	140.9	18000
(iii)b	5.53	139.8	18500
(iii)c	5.50	140.3	17500
(iii)d	5.52	141.7	23000
(iii)e	5.50	130.3	13000
(iv) a	5.20	124.0	31500
(iv) b	5.22	121.4	20500
(iv) c	5.19	129.2	22500
(iv) d	5.16	125.8	22500
(iv) e	5.19	127.7	22000
(v) a	5.35	128.6	24500
(v) b	5.41	130.4	23500
(v) c	5.41	126.9	31500
(v) d	5.45	130.7	22000
(v) e	5.41	132.7	23000

Table 1 - Results

pH Results

Sample	Mean	Med	SD
(i)	5.5480	5.5600	0.0268
(ii)	5.9660	5.9800	0.0391
(iii)	5.5380	5.5200	0.0585
(iv)	5.1920	5.1900	0.0217
(v)	5.4060	5.4100	0.0358
	NH Results		
(i)	129.90	129.20	1.41
(ii)	123.72	123.70	1.54
(ii)	138.70	140.30	4.47
(iv)	125.62	125.80	3.07
(v)	129.86	134.40	2.20
	COD Results		
(i)	-	-	-
(ii)	-	-	-
(iii)	18000	18000	3553
(iv)	23800	22500	4382
(v)	24900	23500	3798

Table 2 - Statistical Analysis

The above figures for NH and particularly for pH imply that there is consistency in the results of tests on the subsamples at each point in the grid, and that the variation in values between points on the grid is much greater.

The results for COD are less conclusive. This may be explained by the fact that the procedure used in the HACH miniaturised COD test is inherently less accurate than the other tests owing to the small quantities of simple used. To illustrate the difference, the above results were obtained using the following quantities of sediment for each of the 3 tests :-

pH	100 ml	(undiluted, blended sample)
NH	10 ml	(100 ml of 10% blended solution)
COD	0.02 ml	(2 ml of 1% blended solution)

Appendix I

Sample Spreadsheet Layout

A	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P
1	SEWAGE			TOTAL RAIN FOR EVENT = 1.6 MM												
2	DATA,			ADWP = 16 HOURS												
3	21/9/99	TIME	TIME	FLOW	FLOW	VEL	DEPTH	DEPTH	TSS	FLOW	FLOW	VEL	DEPTH	DEPTH	TSS	TSS
4		(MINS)	DECIMAL	(M3/S)	(L/S)	(M/S)	(MM)	(M)	(MG/L)	(M3/S)	(L/S)	(M/S)	(MM)	(M)	(MG/L)	(MG/L)
5	TIME								(TUBE 1)						(TUBE 1)	(TUBE 2)
6	(HOURS)	(MH99)	(MH99)	(MH99)	(MH99)	(MH99)	(MH99)	(MH99)	(MH99)	(MH98)	(MH98)	(MH98)	(MH98)	(MH98)	(MH98)	(MH98)
7																
8	(MH99)	20	0.33	0.07	67.50	0.30	360.00	0.36	246.00							
9		20	0.33	0.06	57.20	0.28	346.00	0.35	165.00							
10	9	20	9.33	0.21	206.20	0.41	592.00	0.59	276.00							
11	10	20	10.33	0.11	112.20	0.29	497.00	0.50	128.00							
12	11	20	11.33	0.09	86.40	0.35	372.00	0.37	146.00							
13	12	20	12.33	0.08	75.30	0.33	363.00	0.36	152.00							
14	13	20	13.33	0.06	64.80	0.32	327.00	0.33	187.00							
15	14	20	14.33	0.06	57.20	0.28	329.00	0.33	102.00							
16	15	20	15.33	0.05	47.20	0.23	330.00	0.33	116.00							
17	16	20	16.33	0.06	55.20	0.27	329.00	0.33	126.00							
18	17	20	17.33	0.05	54.20	0.26	333.00	0.33	147.00							
19	18	20	18.33	0.03	31.00	0.16	318.00	0.32	130.00							
20	19	20	19.33	0.04	38.20	0.19	315.00	0.32	107.00							
21	20	20	20.33	0.04	37.50	0.20	312.00	0.31	96.00							
22	21	20	21.33	0.03	34.70	0.19	307.00	0.31	104.00							
23	22	20	22.33	0.03	27.80	0.17	287.00	0.29	70.00							
24	23	20	23.33	0.01	12.00	0.08	270.00	0.27	59.00							
25	24	20	24.33	0.01	9.70	0.07	256.00	0.26	41.00							
26	25	20	25.33	0.01	10.60	0.08	250.00	0.25	32.00							
27	26	20	26.33	0.01	10.70	0.08	249.00	0.25	30.00							
28	27	20	27.33	0.01	6.60	0.06	246.00	0.25	49.00							
29	28	20	28.33	0.02	23.30	0.17	253.00	0.25	20.00							
30	29	20	29.33	0.04	41.50	0.26	282.00	0.28	58.00							
31	30	20	30.33	0.06	63.40	0.30	336.00	0.34	180.00							
32	31	23	31.38												243.00	
33	32	23	32.38												177.00	
34	9	23	9.38												271.00	127.00
35	10	23	10.38												135.00	161.00
36	11	23	11.38												154.00	
37	12	23	12.38												150.00	
38	13	23	13.38												122.00	
39	14	23	14.38												125.00	
40	15	23	15.38												116.00	
41	16	23	16.38												150.00	
42	17	23	17.38												152.00	
43	18	23	18.38												125.00	
44	19	23	19.38												131.00	
45	20	23	20.38												105.00	
46	21	23	21.38												96.00	
47	22	23	22.38												88.00	
48	23	23	23.38												54.00	
49	24	23	24.38												29.00	
50	25	23	25.38												55.00	
51	26	23	26.38												38.00	
52	27	23	27.38												32.00	
53	28	23	28.38												34.00	
54	29	23	29.38												146.00	
55	30	23	30.38												263.00	
56	31	22	31.37							0.08	82.80	0.21	446.00	0.45		
57	32	22	32.37							0.07	86.40	0.17	442.00	0.44		
58	9	22	9.37							0.18	183.20	0.26	684.00	0.68		
59	10	22	10.37							0.12	123.00	0.22	575.00	0.58		
60	11	22	11.37							0.08	82.70	0.20	461.00	0.46		
61	12	20	12.33							0.08	78.60	0.20	444.00	0.44		
62	13	20	13.33							0.08	77.10	0.21	422.00	0.42		
63	14	20	14.33							0.07	66.30	0.18	423.00	0.42		
64	15	20	15.33							0.06	63.20	0.17	426.00	0.43		
65	16	20	16.33							0.06	55.60	0.15	425.00	0.43		
66	17	20	17.33							0.07	74.60	0.20	427.00	0.43		
67	18	20	18.33							0.06	64.60	0.18	415.00	0.42		
68	19	20	19.33							0.06	56.80	0.16	412.00	0.41		
69	20	20	20.33							0.06	56.30	0.16	409.00	0.41		
70	21	20	21.33							0.05	51.70	0.15	403.00	0.40		
71	22	20	22.33							0.06	61.90	0.19	387.00	0.39		
72	23	20	23.33							0.04	39.70	0.19	387.00	0.39		
73	24	20	24.33							0.04	40.50	0.14	356.00	0.36		
74	25	20	25.33							0.03	34.10	0.12	352.00	0.35		
75	26	20	26.33							0.03	28.40	0.10	352.00	0.35		
76	27	20	27.33							0.03	30.90	0.11	349.00	0.35		
77	28	20	28.33							0.03	31.70	0.11	355.00	0.36		
78	29	20	29.33							0.04	44.80	0.14	382.00	0.38		
79	30	20	30.33							0.07	68.20	0.18	432.00	0.43		
80	31															
81	32															
82																

Figure 11

Sample Spreadsheet "As Seen"

A	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P
1	SEWAGE			TOTAL RAIN FOR EVENT = 1.6 MM												
2	DATA,			ADWP = 18 HOURS												
3	21/9/99	TIME	TIME	FLOW	FLOW	VEL	DEPTH	DEPTH	TSS	FLOW	FLOW	VEL	DEPTH	DEPTH	TSS	TSS
4		(MINS)	DECIMAL	(M3/S)	(L/S)	(M/S)	(MM)	(M)	(MG/L)	(M3/S)	(L/S)	(M/S)	(MM)	(M)	(MG/L)	(MG/L)
5	TIME								(TUBE 1)						(TUBE 1)	(TUBE 2)
6	(HOURS)	(MH99)	(MH99)	(MH99)	(MH99)	(MH99)	(MH99)	(MH99)	(MH99)	(MH98)	(MH98)	(MH98)	(MH98)	(MH98)	(MH98)	(MH98)
7																
8	(MH99)	20	+A8+B8/60	6.750E-02	+D8*1000	3.000E-01	360	+G8/1000	245							
9		20	+A9+B9/60	5.720E-02	+D9*1000	2.800E-01	346	+G9/1000	165							
10	9	20	+A10+B10/60	2.082E-01	+D10*1000	4.100E-01	692	+G10/1000	276							
11	10	20	+A11+B11/60	1.122E-01	+D11*1000	2.900E-01	497	+G11/1000	126							
12	11	20	+A12+B12/60	8.640E-02	+D12*1000	3.500E-01	372	+G12/1000	146							
13	12	20	+A13+B13/60	7.530E-02	+D13*1000	3.300E-01	359	+G13/1000	182							
14	13	20	+A14+B14/60	6.480E-02	+D14*1000	3.200E-01	327	+G14/1000	187							
15	14	20	+A15+B15/60	5.720E-02	+D15*1000	2.800E-01	329	+G15/1000	102							
16	15	20	+A16+B16/60	4.720E-02	+D16*1000	2.300E-01	330	+G16/1000	116							
17	16	20	+A17+B17/60	5.620E-02	+D17*1000	2.7[illegible]E-01	329	+G17/1000	128							
18	17	20	+A18+B18/60	5.420E-02	+D18*1000	2.600E-01	333	+G18/1000	147							
19	18	20	+A19+B19/60	3.100E-02	+D19*1000	1.600E-01	318	+G19/1000	130							
20	19	20	+A20+B20/60	3.620E-02	+D20*1000	1.900E-01	315	+G20/1000	107							
21	20	20	+A21+B21/60	3.750E-02	+D21*1000	2.000E-01	312	+G21/1000	98							
22	21	20	+A22+B22/60	3.470E-02	+D22*1000	1.900E-01	307	+G22/1000	104							
23	22	20	+A23+B23/60	2.780E-02	+D23*1000	1.700E-01	287	+G23/1000	70							
24	23	20	+A24+B24/60	1.200E-02	+D24*1000	8.000E-02	270	+G24/1000	59							
25	24	20	+A25+B25/60	9.700E-03	+D25*1000	7.000E-02	255	+G25/1000	41							
26	25	20	+A26+B26/60	1.080E-02	+D26*1000	8.000E-02	250	+G26/1000	32							
27	26	20	+A27+B27/60	1.070E-02	+D27*1000	8.000E-02	249	+G27/1000	30							
28	27	20	+A28+B28/60	6.600E-03	+D28*1000	5.000E-02	246	+G28/1000	49							
29	28	20	+A29+B29/60	2.330E-02	+D29*1000	1.700E-01	253	+G29/1000	20							
30	29	20	+A30+B30/60	4.150E-02	+D30*1000	2.600E-01	282	+G30/1000	58							
31	30	20	+A31+B31/60	6.340E-02	+D31*1000	3.000E-01	336	+G31/1000	180							
32	31	23	+A32+B32/60												243	
33	32	23	+A33+B33/60												177	
34	9	23	+A34+B34/60												271	127
35	10	23	+A35+B35/60												136	1[illegible]
36	11	23	+A36+B36/60												164	
37	12	23	+A37+B37/60												150	
38	13	23	+A38+B38/60												122	
39	14	23	+A39+B39/60												125	
40	15	23	+A40+B40/60												116	
41	16	23	+A41+B41/60												150	
42	17	23	+A42+B42/60												162	
43	18	23	+A43+B43/60												128	
44	19	23	+A44+B44/60												131	
45	20	23	+A45+B45/60												105	
46	21	23	+A46+B46/60												96	
47	22	23	+A47+B47/60												88	
48	23	23	+A48+B48/60												64	
49	24	23	+A49+B49/60												29	
50	25	23	+A50+B50/60												65	
51	26	23	+A51+B51/60												36	
52	27	23	+A52+B52/60												32	
53	28	23	+A53+B53/60												34	
54	29	23	+A54+B54/60												146	
55	30	23	+A55+B55/60												263	
56	31	22	+A56+B56/60							8.280E-02	+J56*1000	2.100E-01	446	+M56/1000		
57	32	22	+A57+B57/60							8.640E-02	+J57*1000	1.700E-01	442	+M57/1000		
58	9	22	+A58+B58/60							1.832E-01	+J58*1000	2.600E-01	684	+M58/1000		
59	10	22	+A59+B59/60							1.230E-01	+J59*1000	2.200E-01	576	+M59/1000		
60	11	22	+A60+B60/60							8.270E-02	+J60*1000	2.000E-01	461	+M60/1000		
61	12	20	+A61+B61/60							7.860E-02	+J61*1000	2.000E-01	444	+M61/1000		
62	13	20	+A62+B62/60							7.710E-02	+J62*1000	2.100E-01	422	+M62/1000		
63	14	20	+A63+B63/60							6.630E-02	+J63*1000	1.800E-01	423	+M63/1000		
64	15	20	+A64+B64/60							6.320E-02	+J64*1000	1.700E-01	426	+M64/1000		
65	16	20	+A65+B65/60							5.560E-02	+J65*1000	1.500E-01	425	+M65/1000		
66	17	20	+A66+B66/60							7.460E-02	+J66*1000	2.000E-01	427	+M66/1000		
67	18	20	+A67+B67/60							6.460E-02	+J67*1000	1.800E-01	415	+M67/1000		
68	19	20	+A68+B68/60							5.680E-02	+J68*1000	1.600E-01	412	+M68/1000		
69	20	20	+A69+B69/60							5.630E-02	+J69*1000	1.600E-01	409	+M69/1000		
70	21	20	+A70+B70/60							5.170E-02	+J70*1000	1.500E-01	403	+M70/1000		
71	22	20	+A71+B71/60							6.190E-02	+J71*1000	1.900E-01	387	+M71/1000		
72	23	20	+A72+B72/60							3.970E-02	+J72*1000	1.900E-01	387	+M72/1000		
73	24	20	+A73+B73/60							4.050E-02	+J73*1000	1.400E-01	356	+M73/1000		
74	25	20	+A74+B74/60							3.410E-02	+J74*1000	1.200E-01	352	+M74/1000		
75	26	20	+A75+B75/60							2.840E-02	+J75*1000	1.000E-01	352	+M75/1000		
76	27	20	+A76+B76/60							3.090E-02	+J76*1000	1.100E-01	349	+M76/1000		
77	28	20	+A77+B77/60							3.170E-02	+J77*1000	1.100E-01	356	+M77/1000		
78	29	20	+A78+B78/60							4.460E-02	+J78*1000	1.400E-01	382	+M78/1000		
79	30	20	+A79+B79/60							6.820E-02	+J79*1000	1.800E-01	432	+M79/1000		
80	31															
81	32															

Figure 12

Sample Spreadsheet Showing Cell Operators

Appendix J

Graphs of DWF Sewage Sample and Flow Data

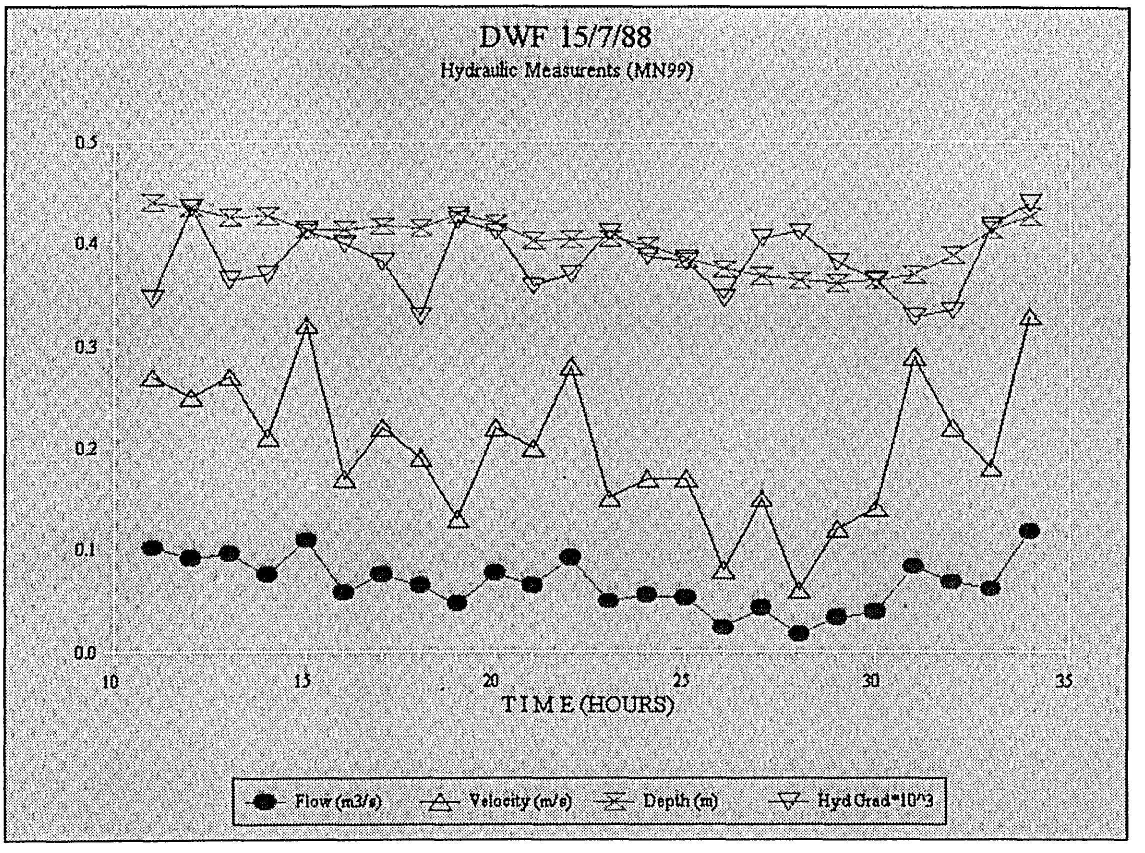


Figure J1

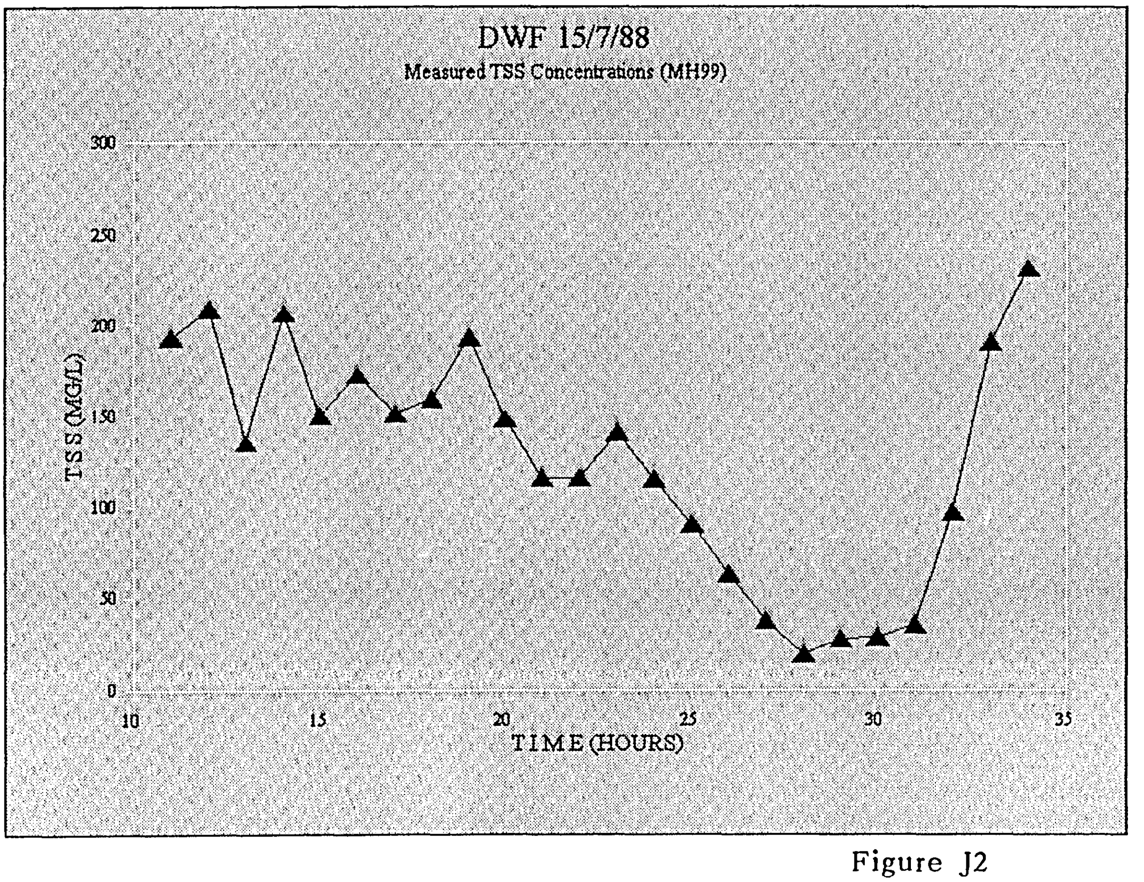


Figure J2

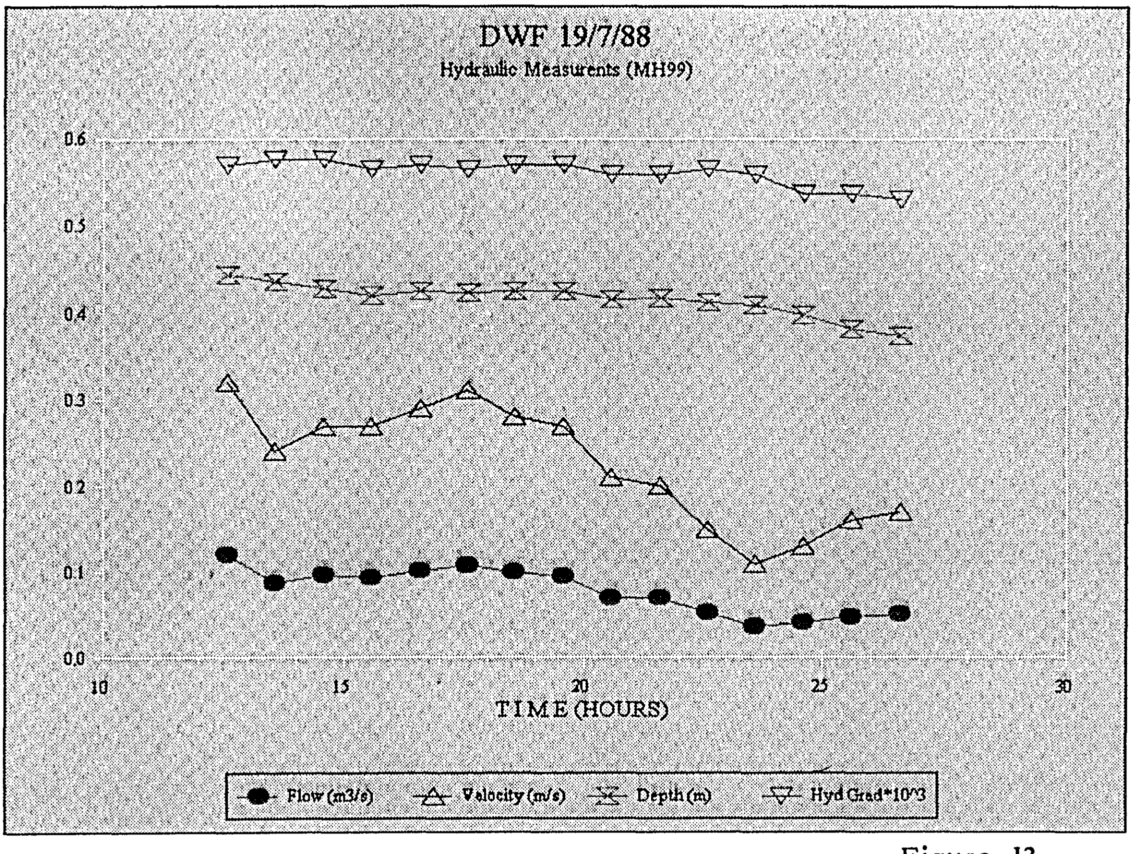


Figure J3

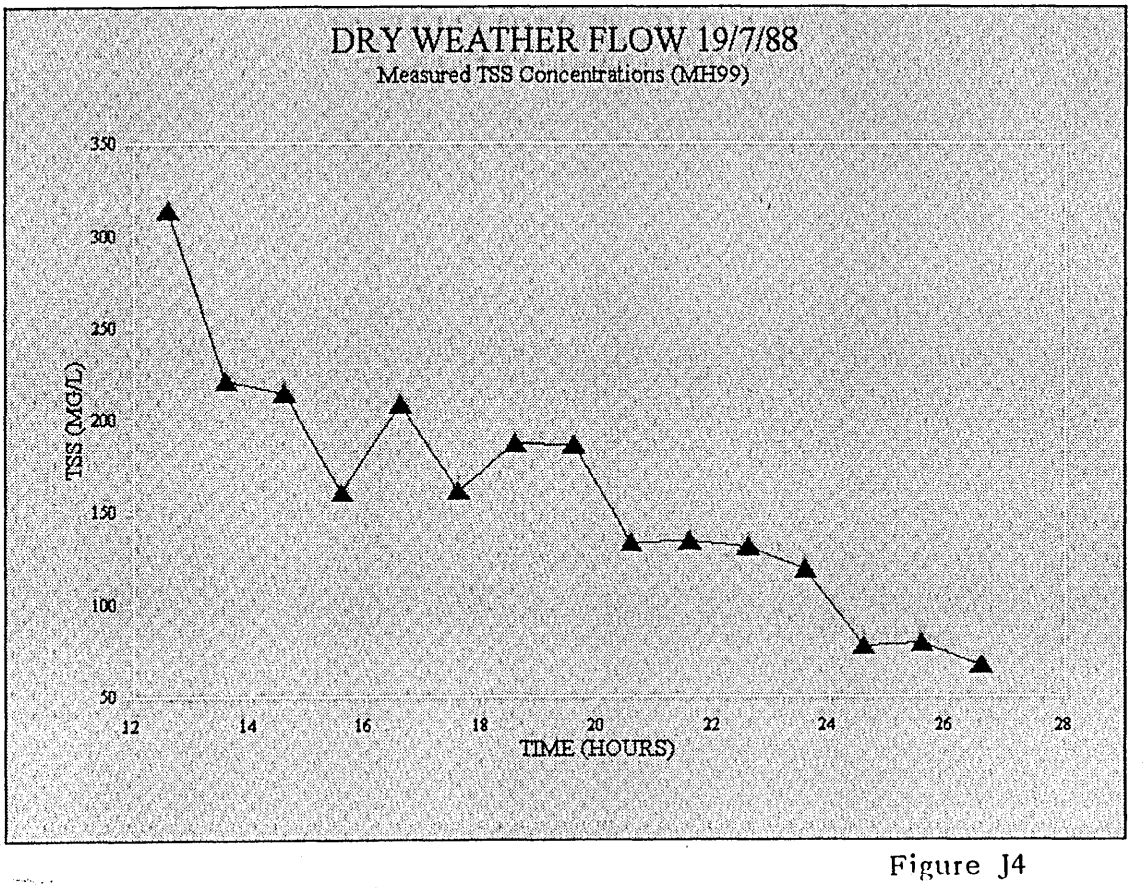


Figure J4

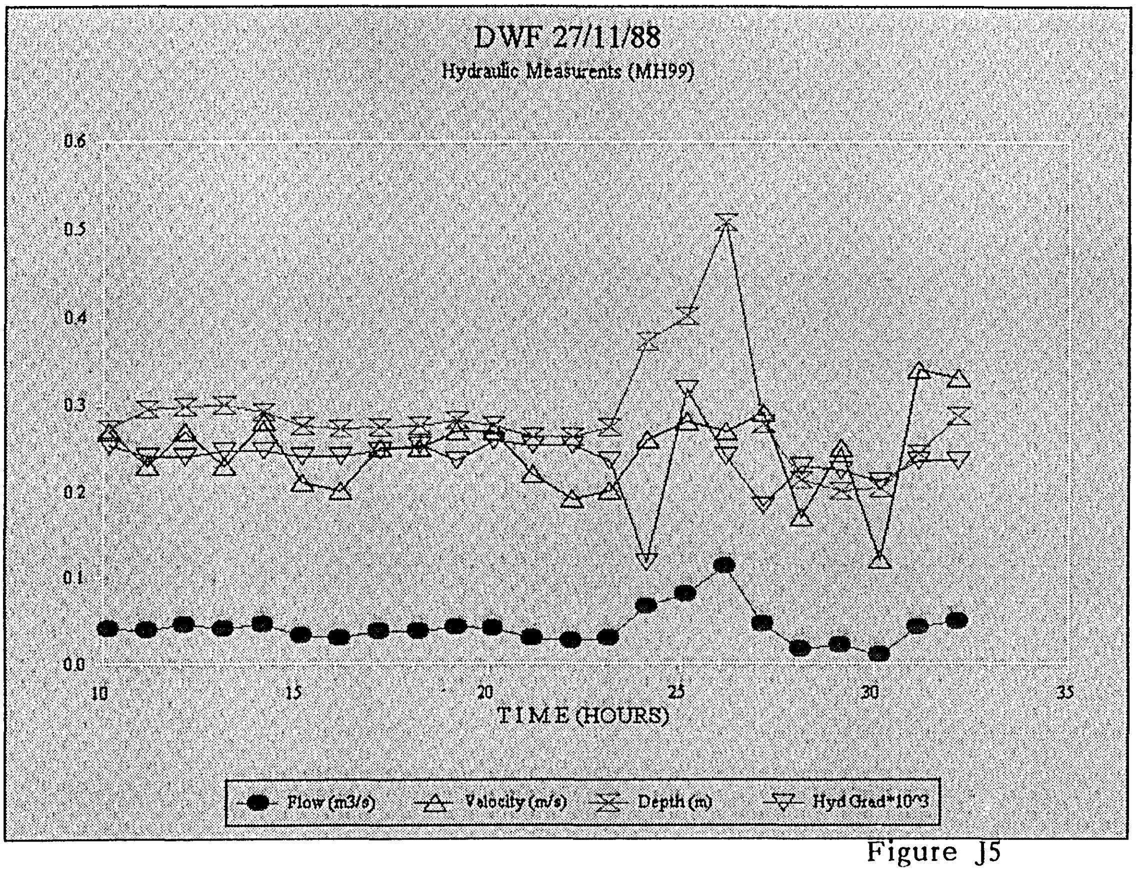


Figure J5

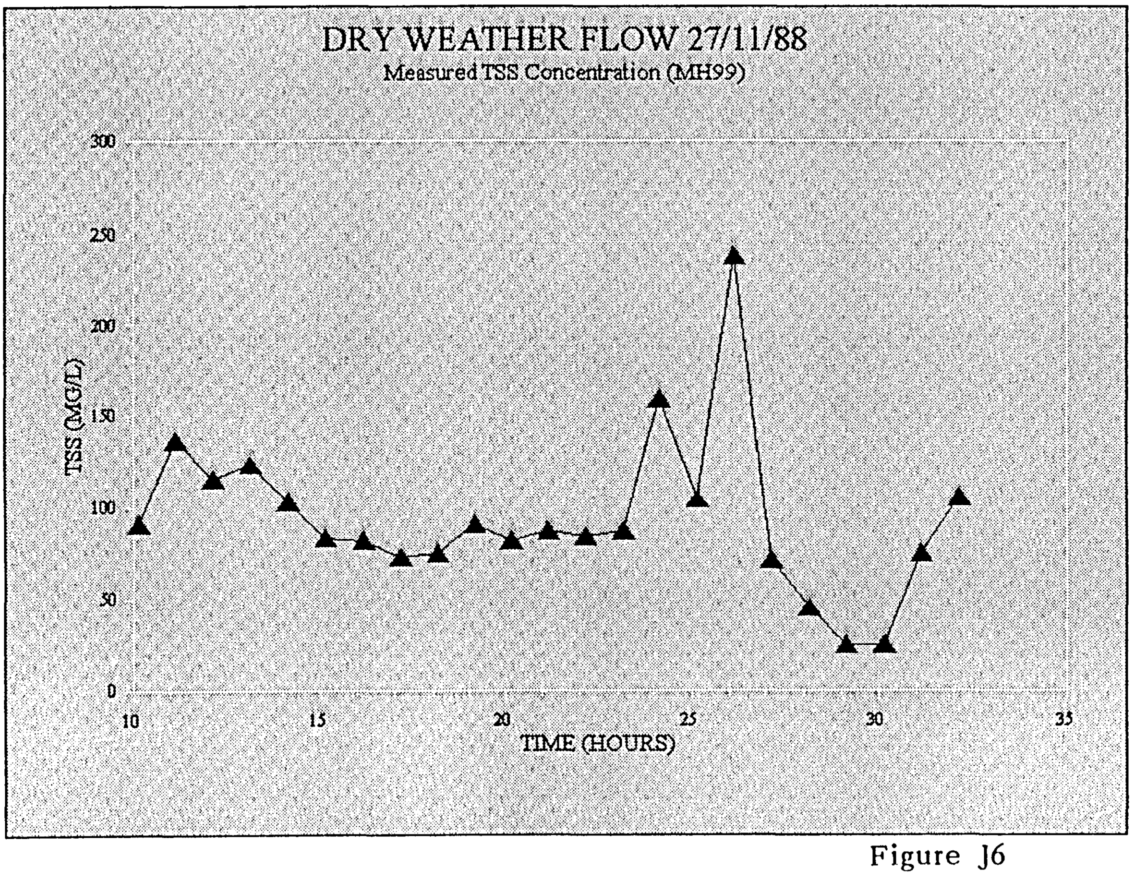


Figure J6

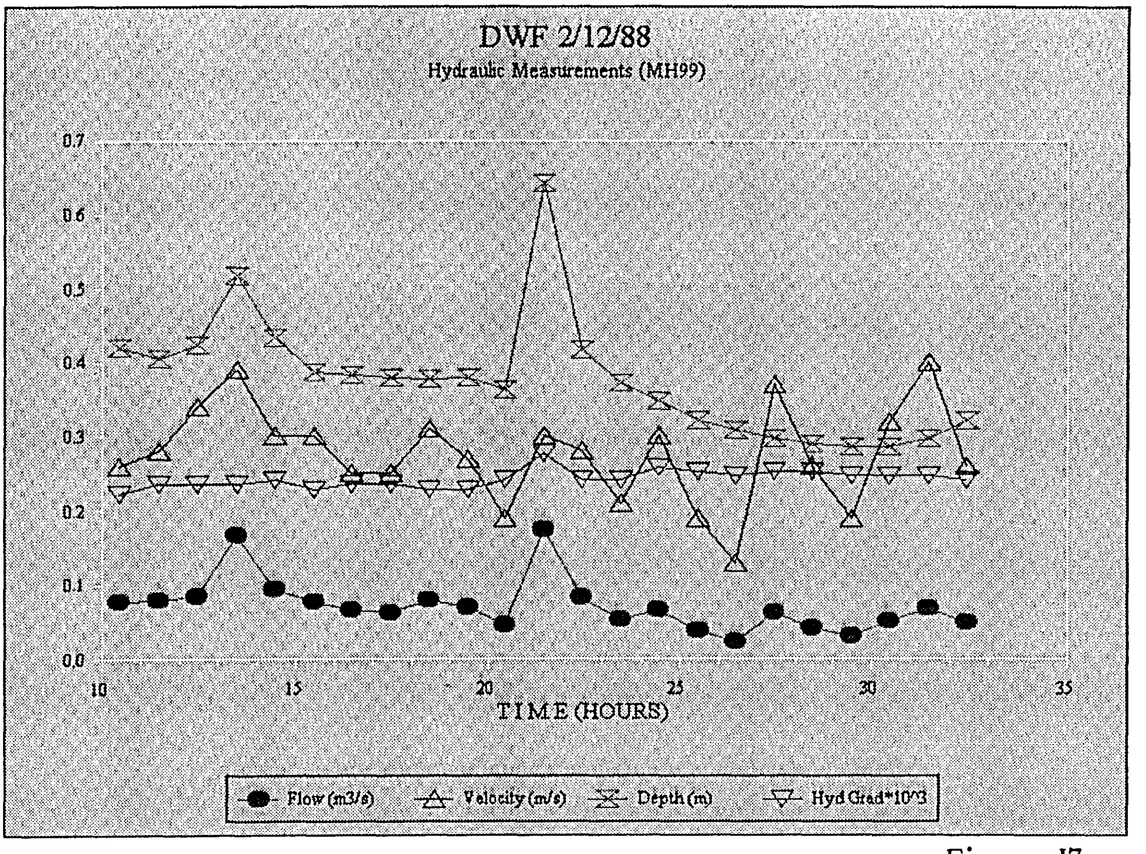


Figure J7

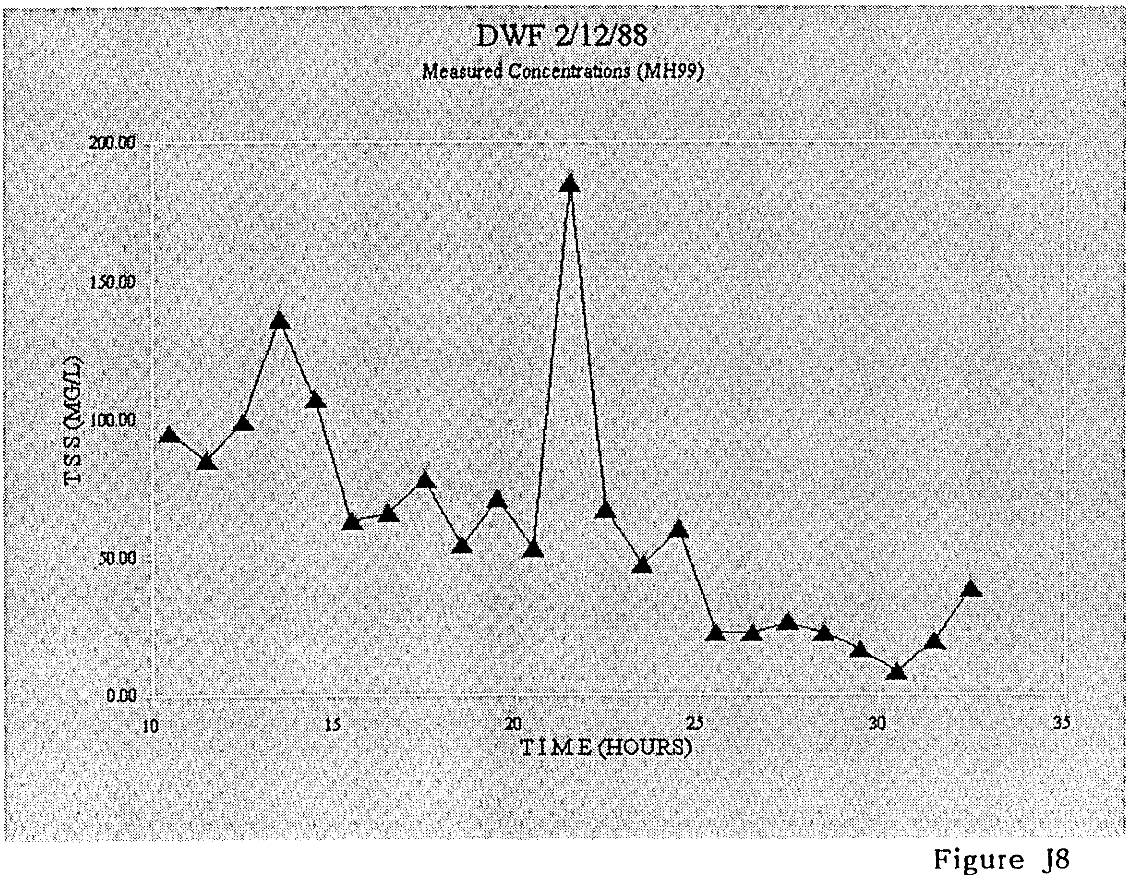


Figure J8

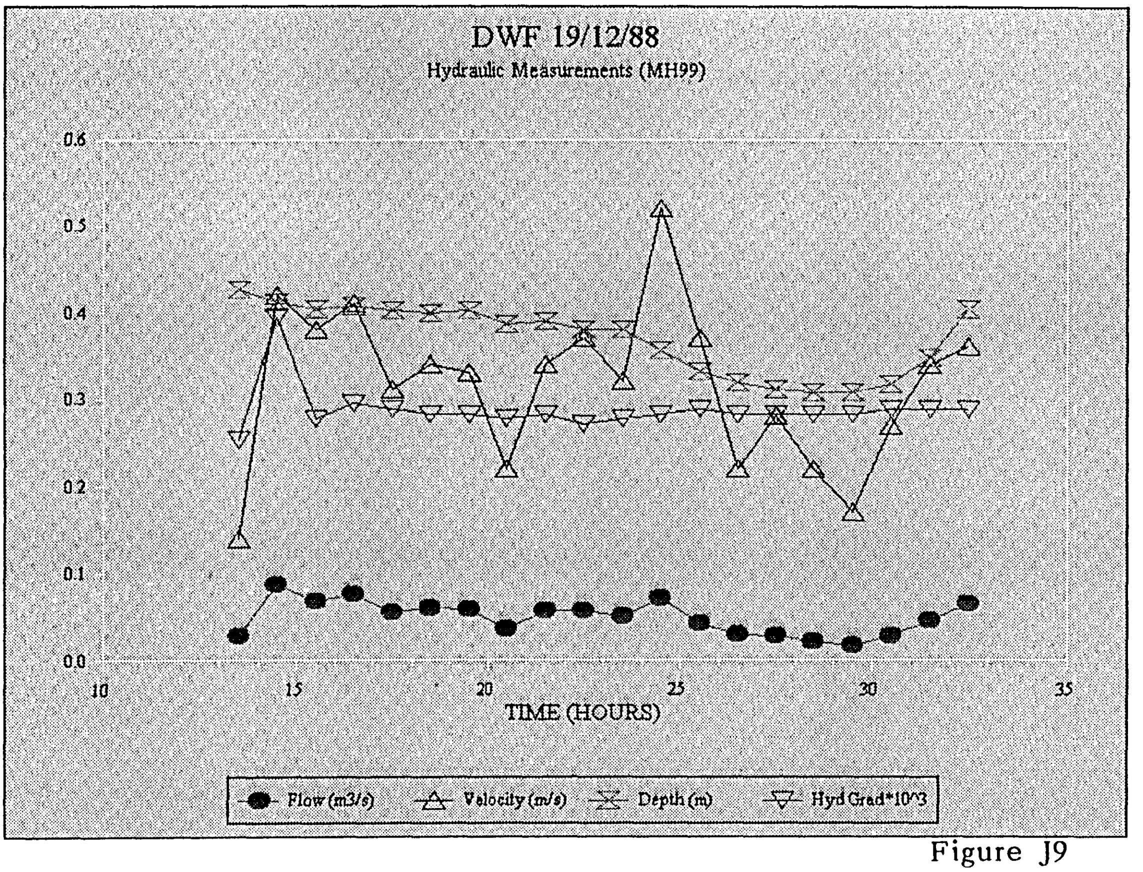


Figure J9

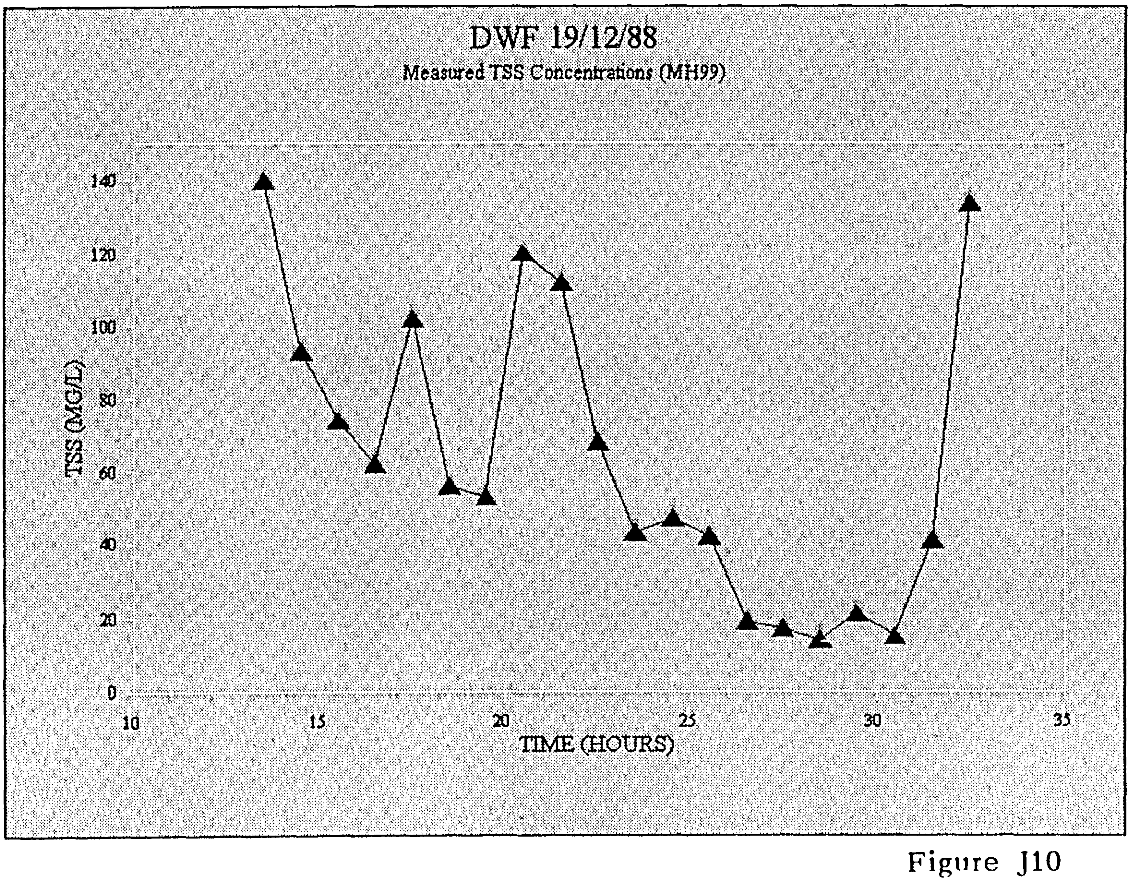


Figure J10

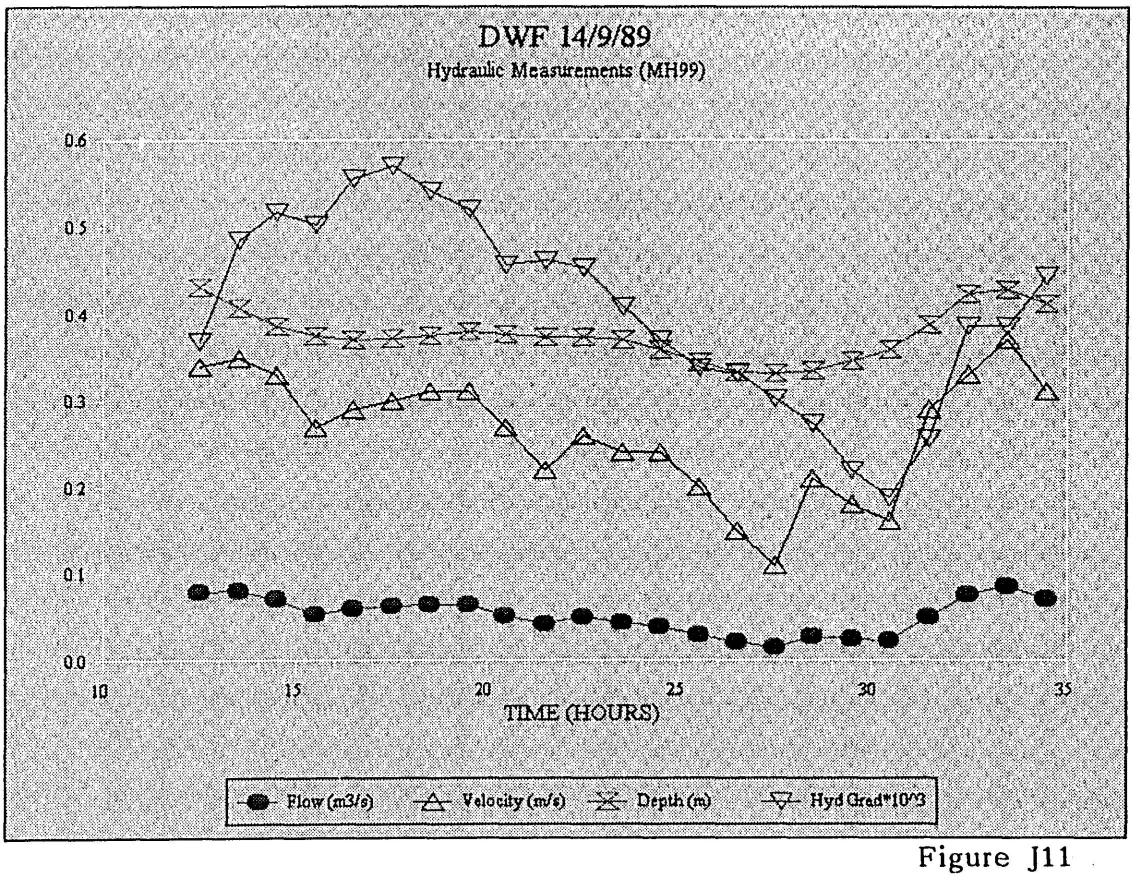


Figure J11

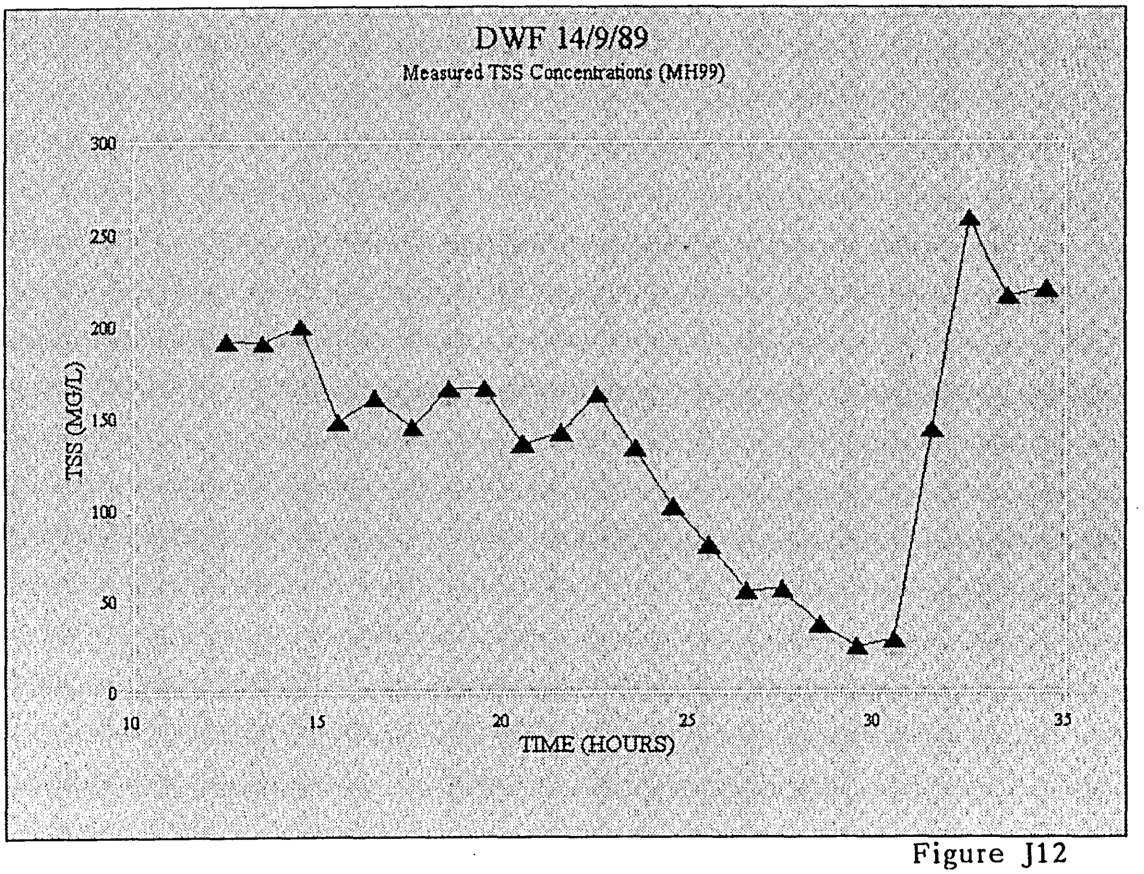


Figure J12

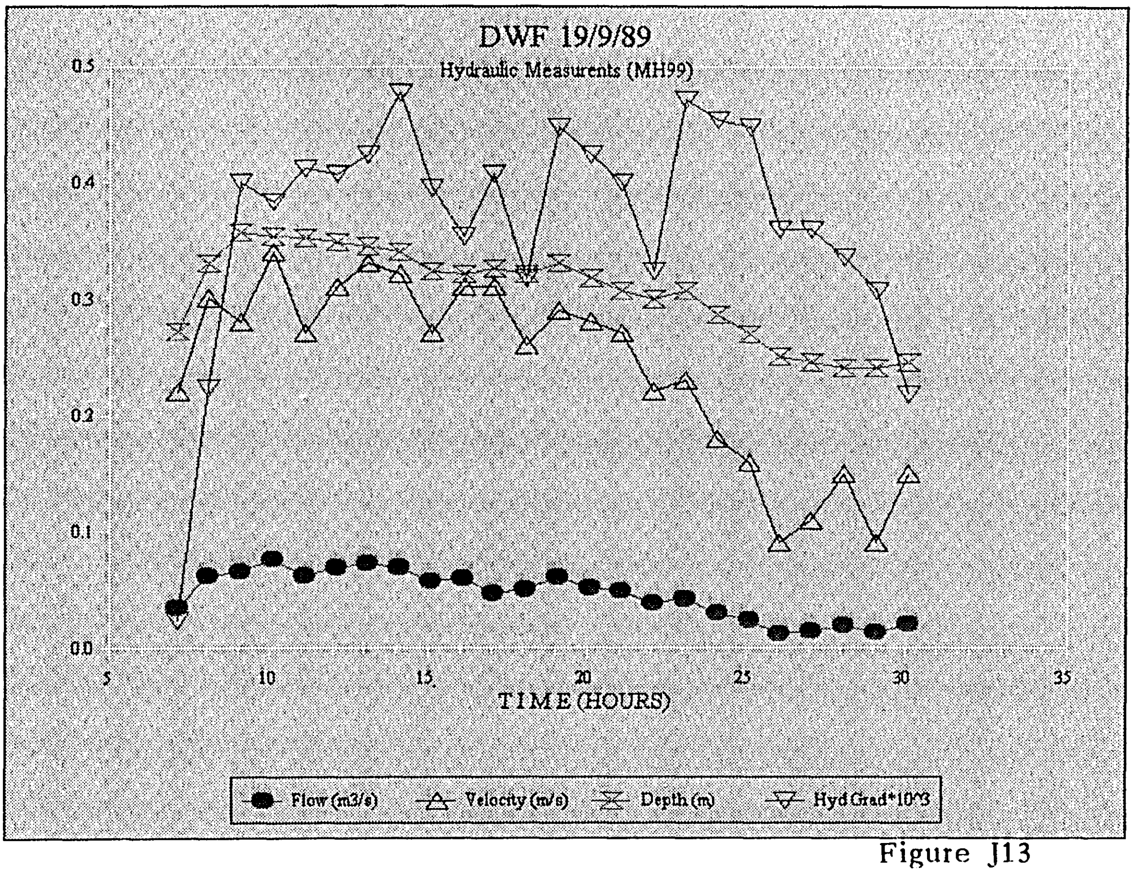


Figure J13

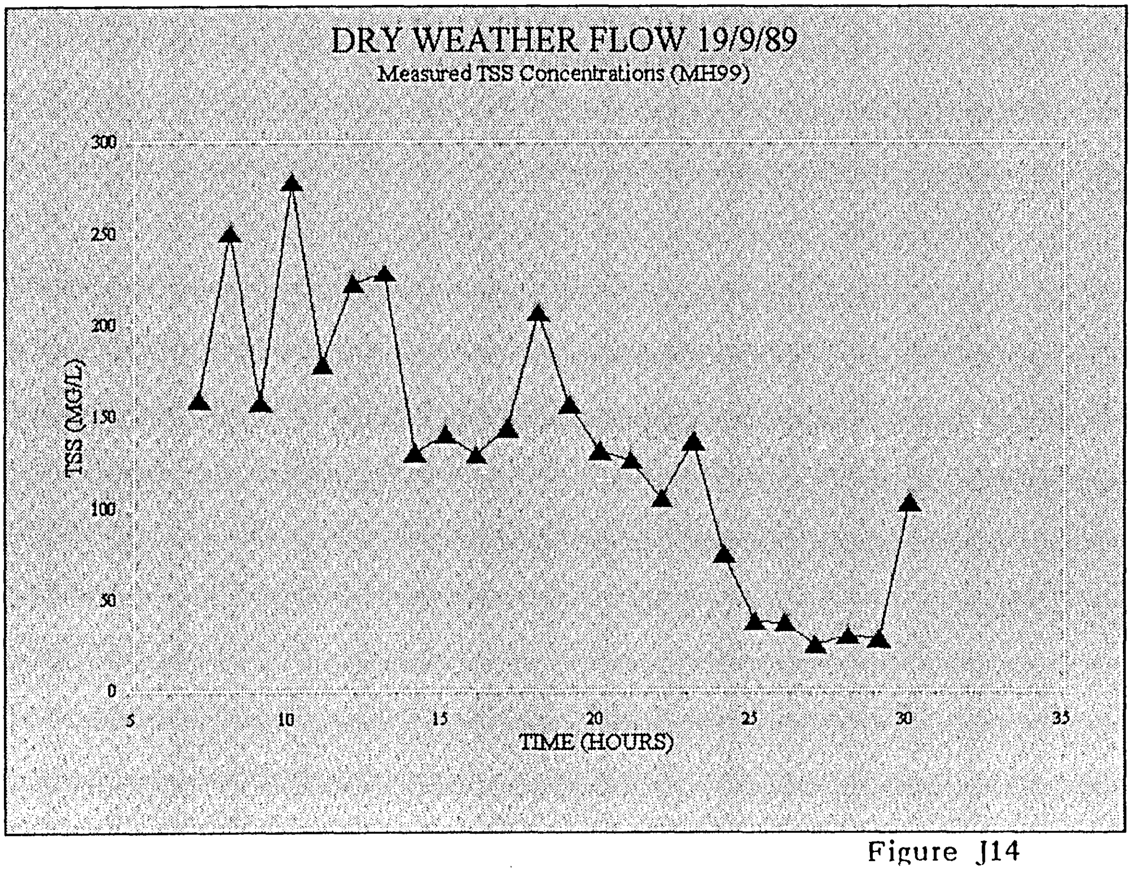


Figure J14

Appendix K

Graphs of Storm Sewage Sample and Flow Data

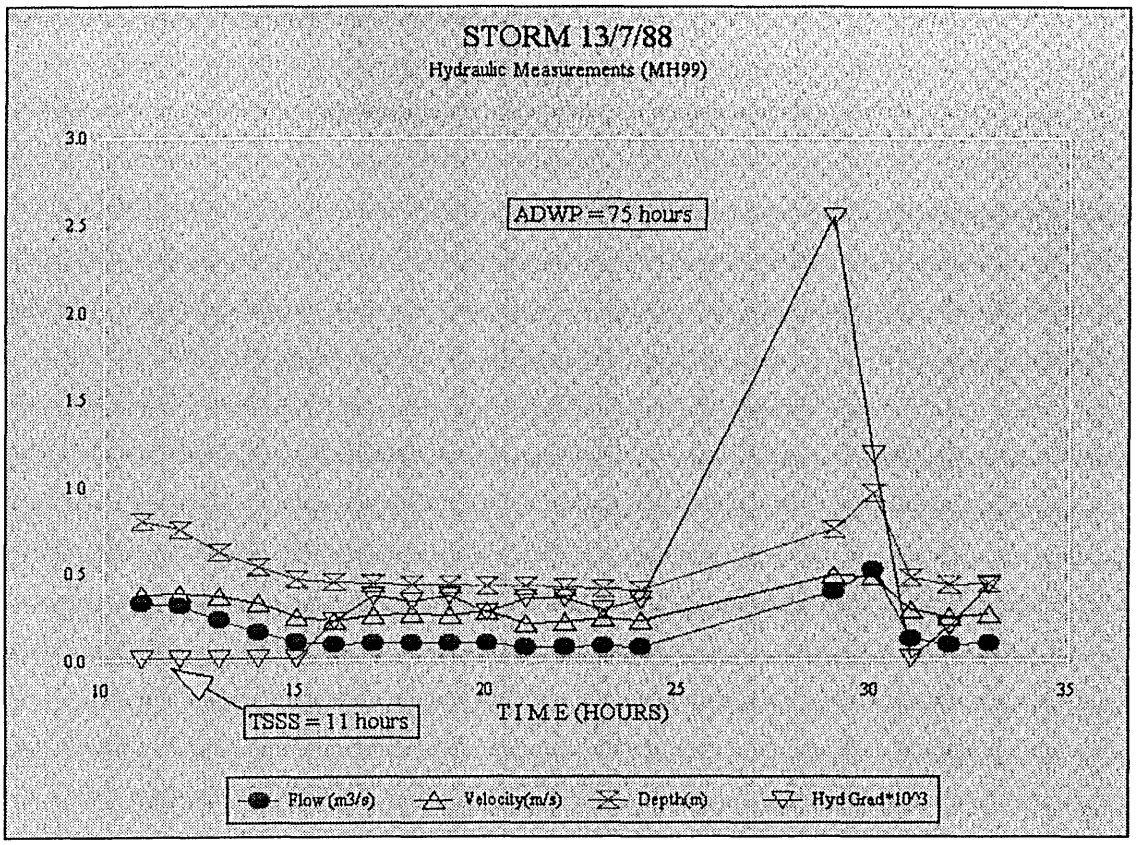


Figure K1

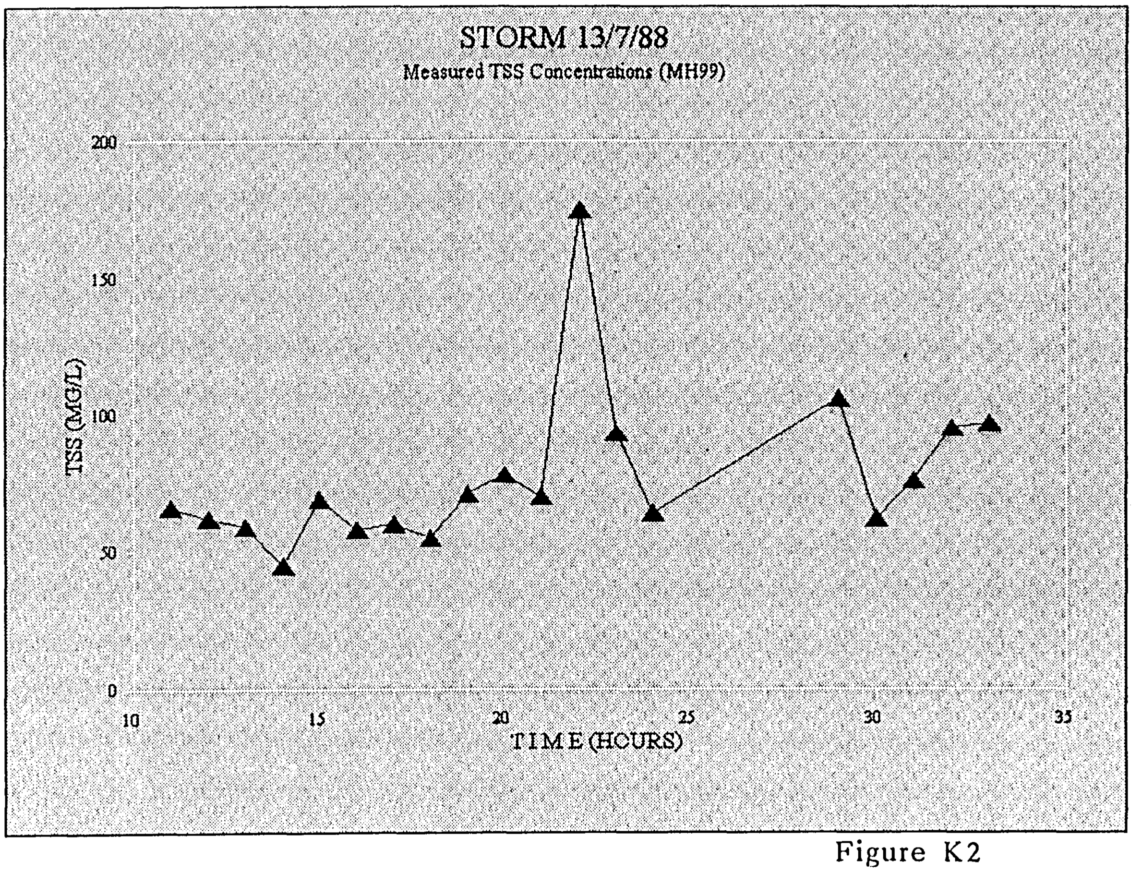


Figure K2

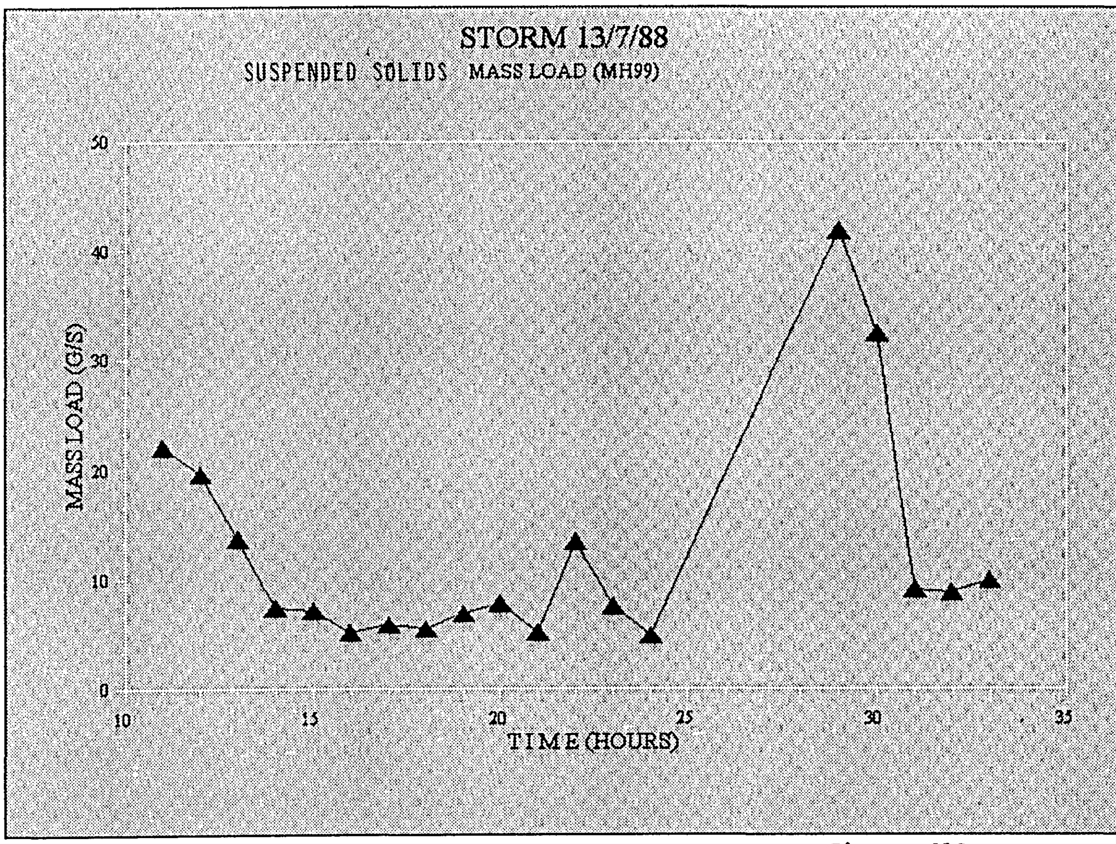

STORM 13/7/88
SUSPENDED SOLIDS MASS LOAD (MH99)
MASS LOAD (G/S)
TIME (HOURS)
0
10
20
30
40
50
10
15
20
25
30
35

Figure K3

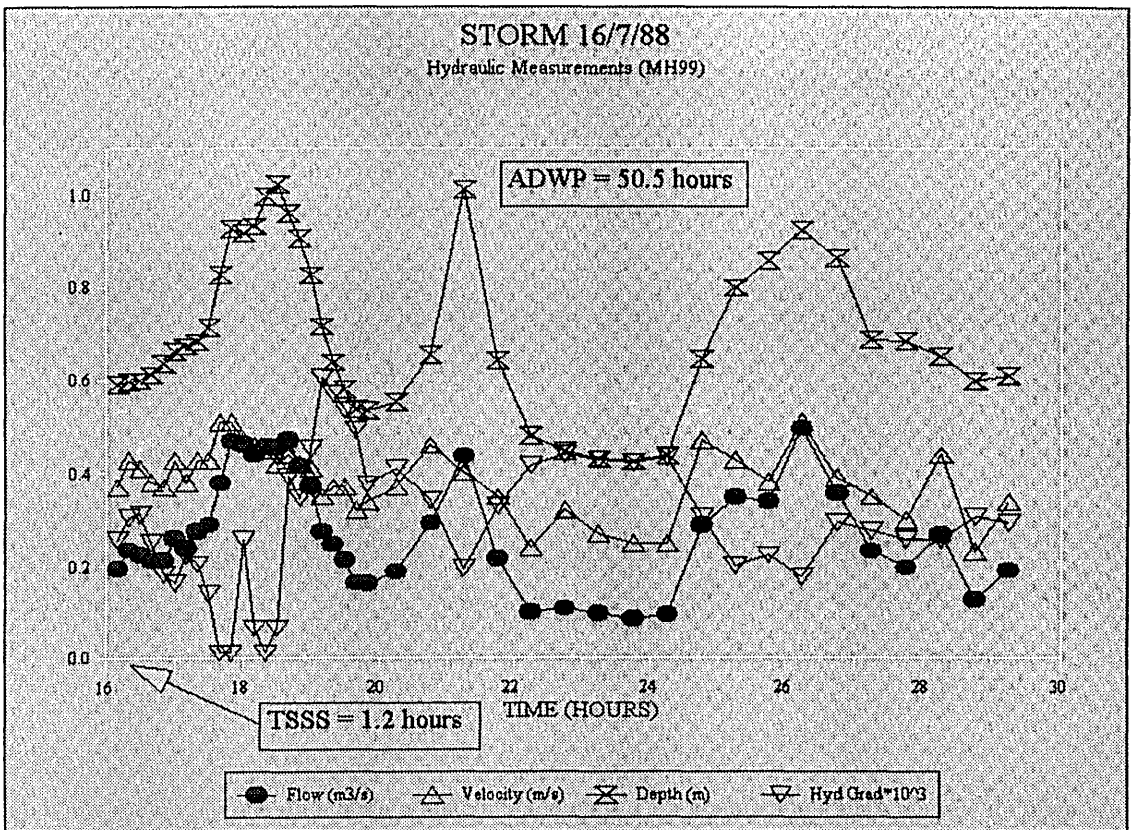


Figure K4

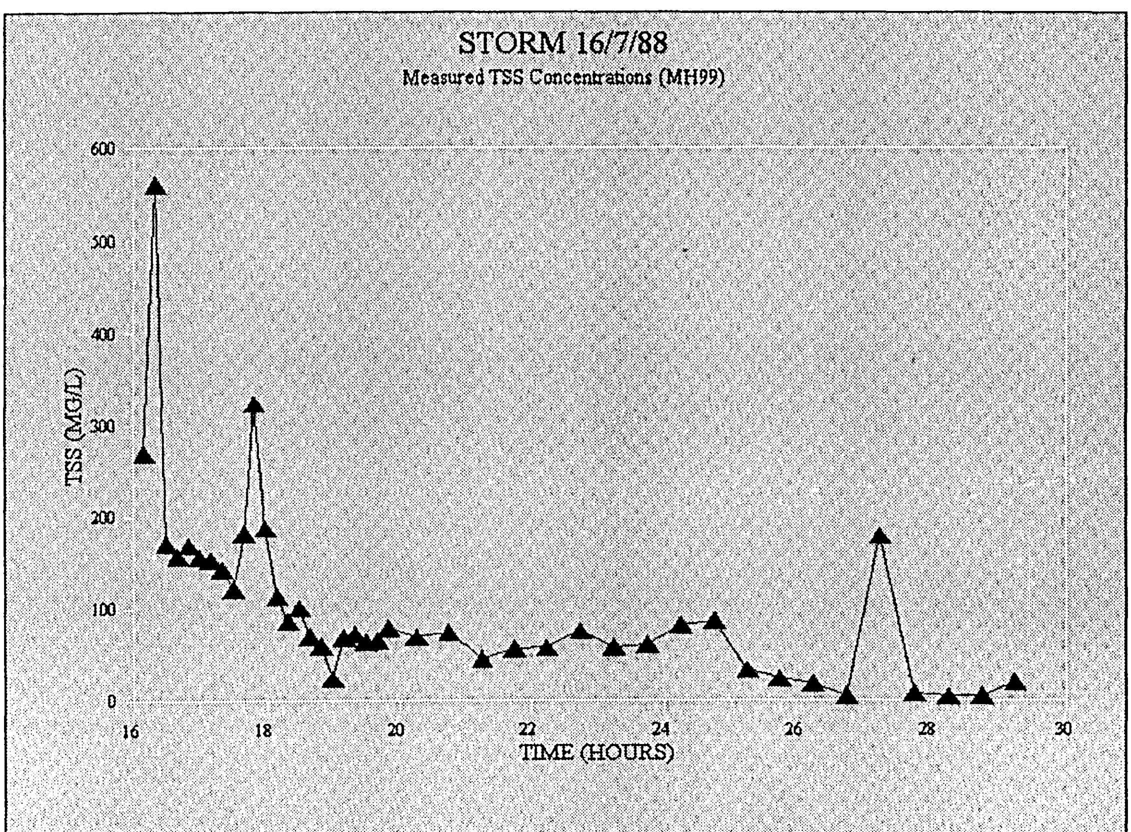
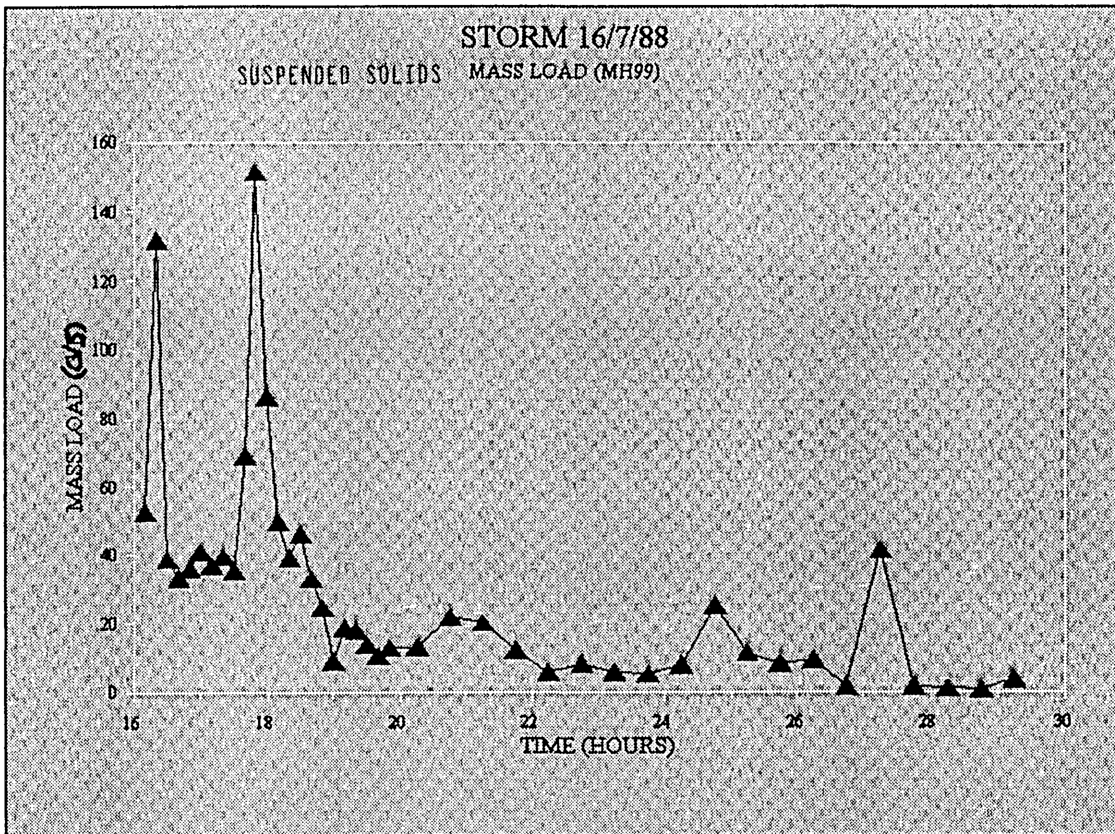


Figure K5



STORM 16/7/88
SUSPENDED SOLIDS MASS LOAD (MH99)
MASS LOAD (g/s)
160
140
120
100
80
60
40
20
0
16
18
20
22
24
26
28
30
TIME (HOURS)

Figure K6

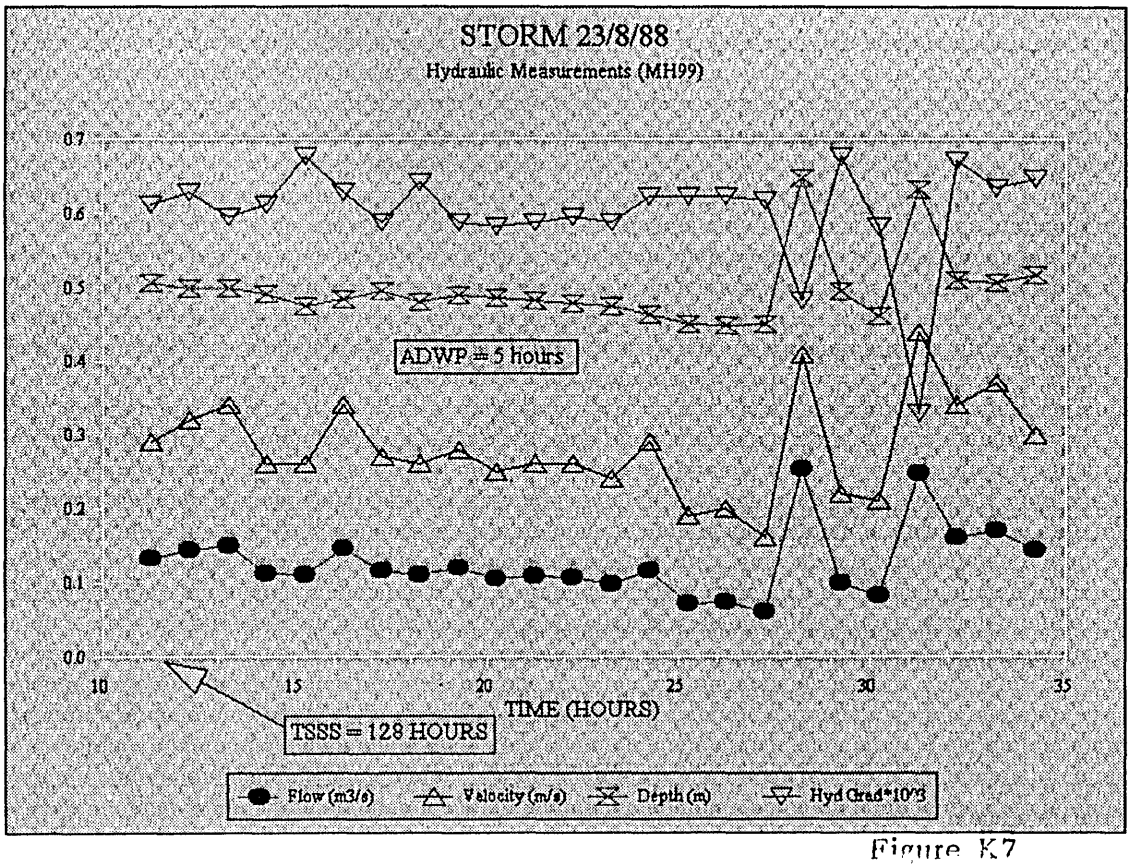


Figure K7

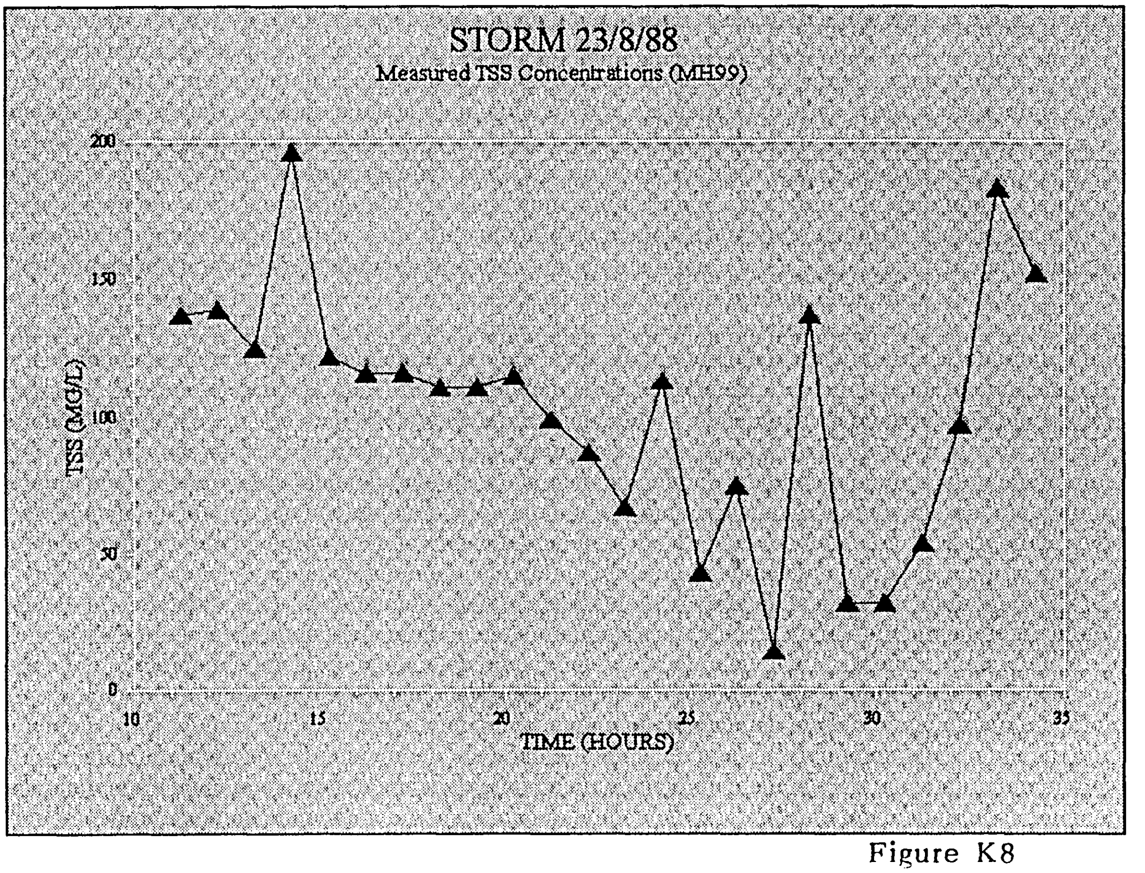


Figure K8

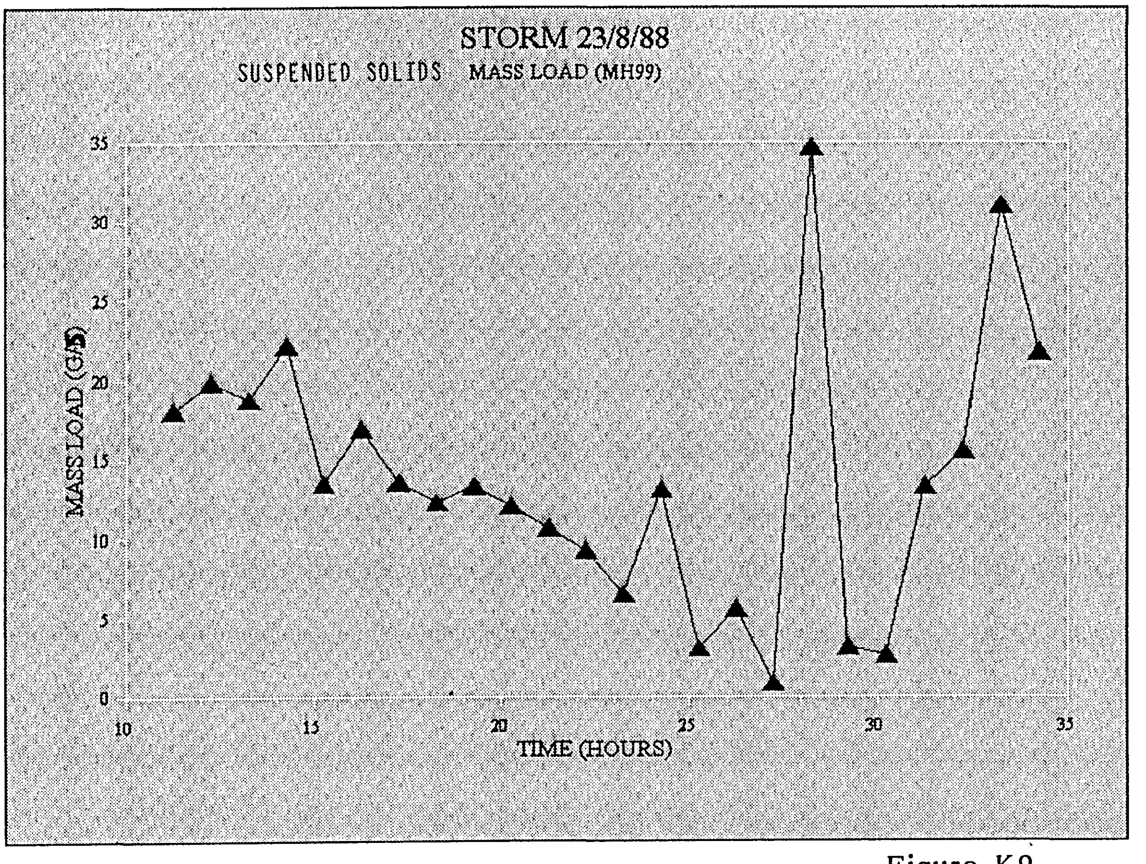

STORM 23/8/88
SUSPENDED SOLIDS MASS LOAD (MH99)
MASS LOAD (G/S)
TIME (HOURS)
0
5
10
15
20
25
30
35
10
15
20
25
30
35

Figure K9

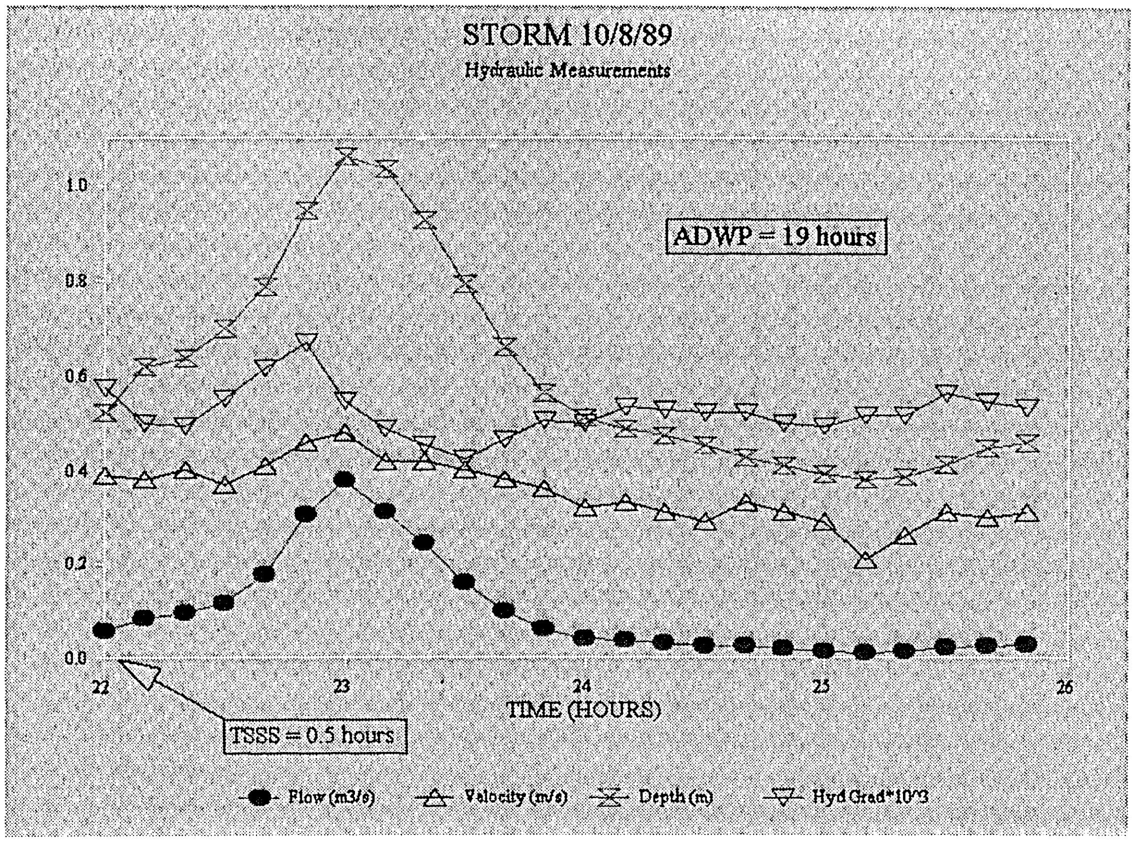


Figure K10

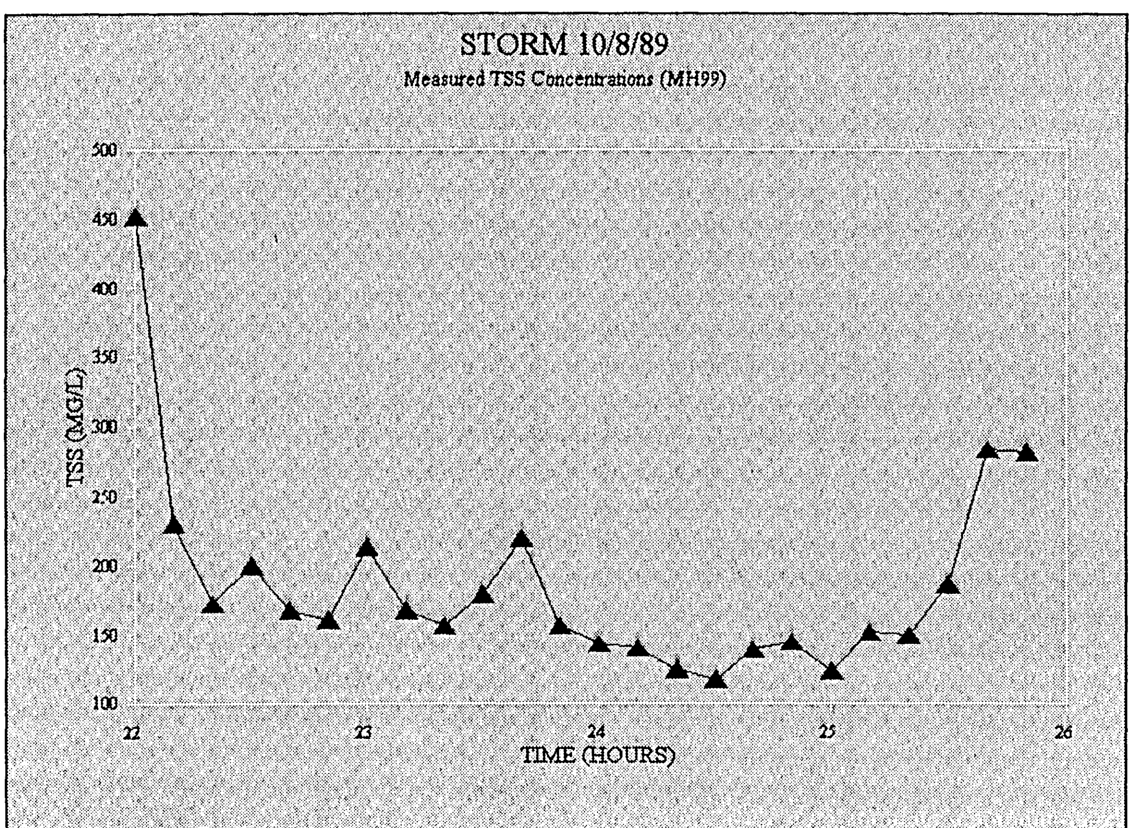


Figure K11

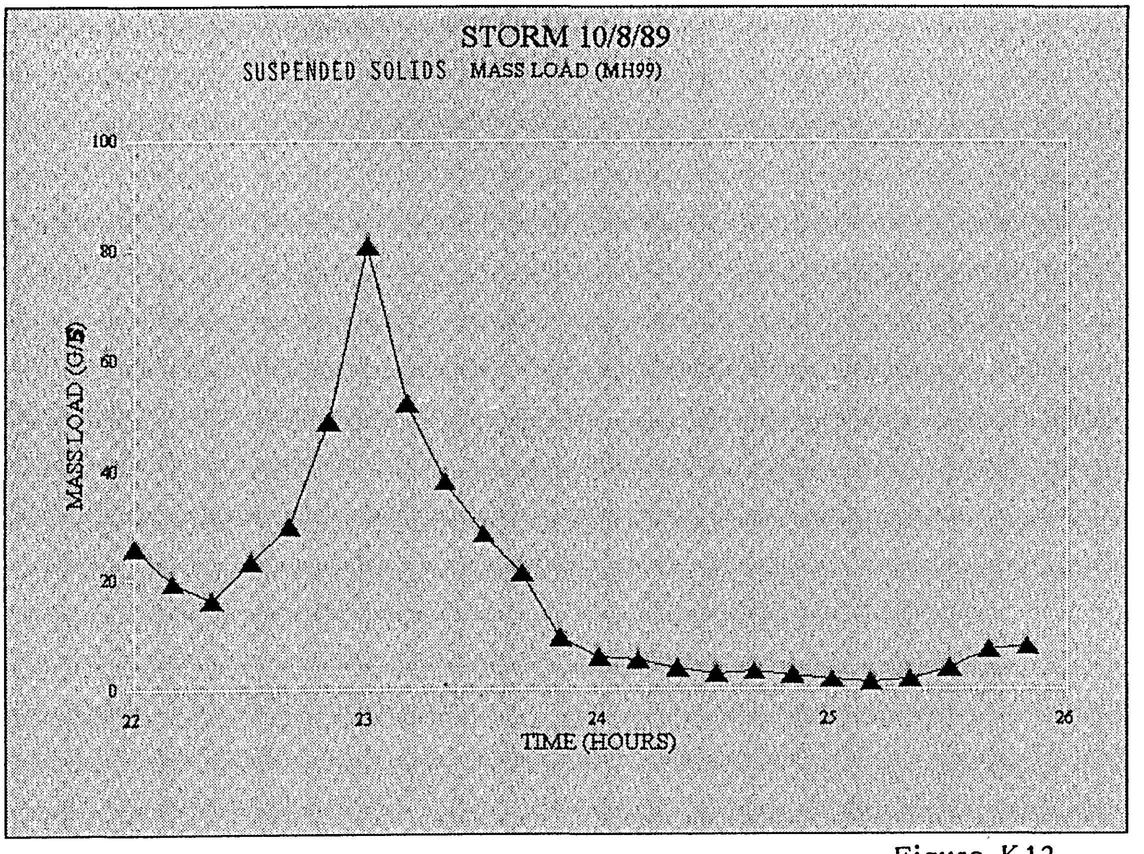

STORM 10/8/89
SUSPENDED SOLIDS MASS LOAD (MH99)
MASS LOAD (G/S)
100
80
60
40
20
0
22
23
24
25
26
TIME (HOURS)

Figure K12

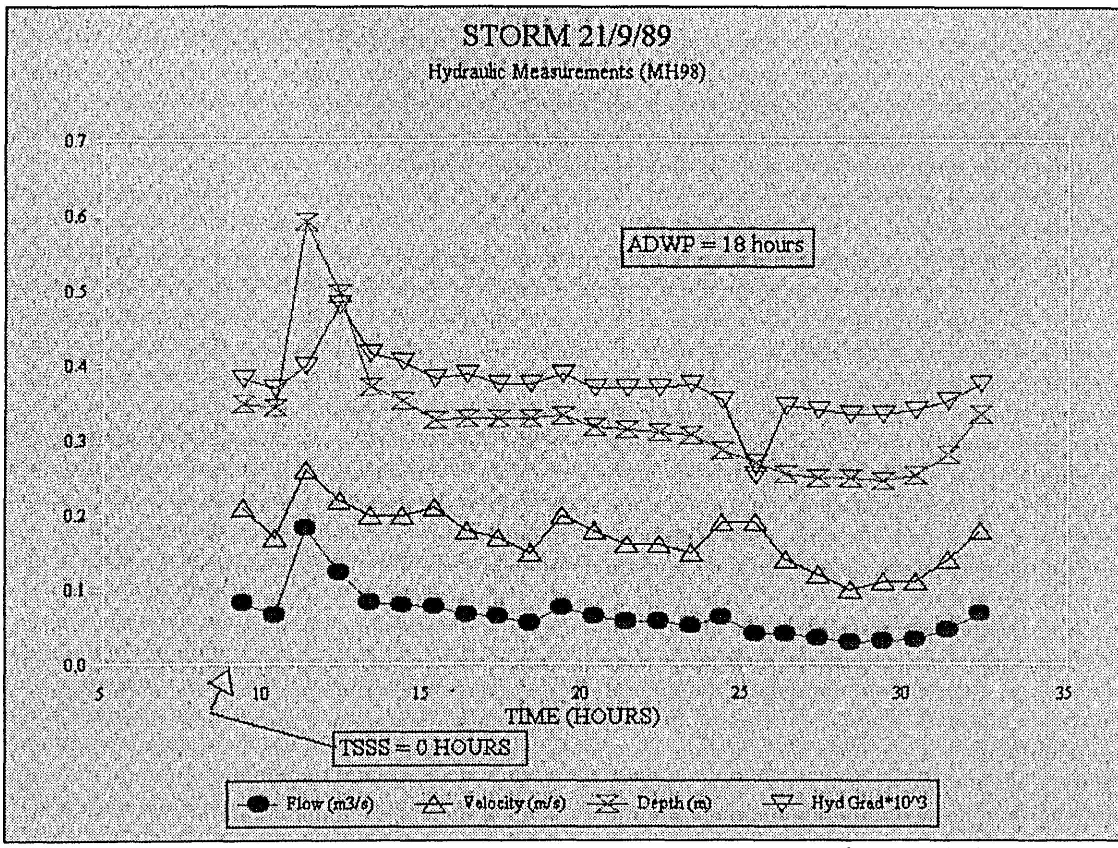


Figure K13

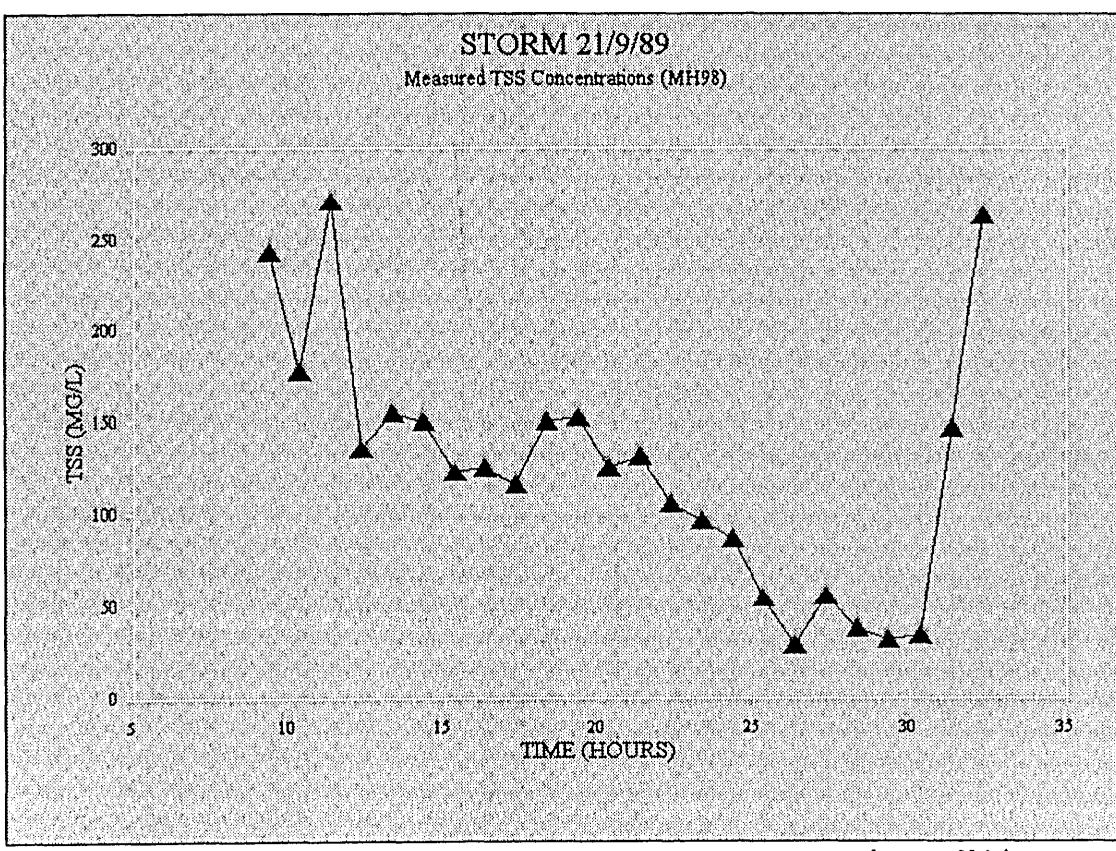


Figure K14

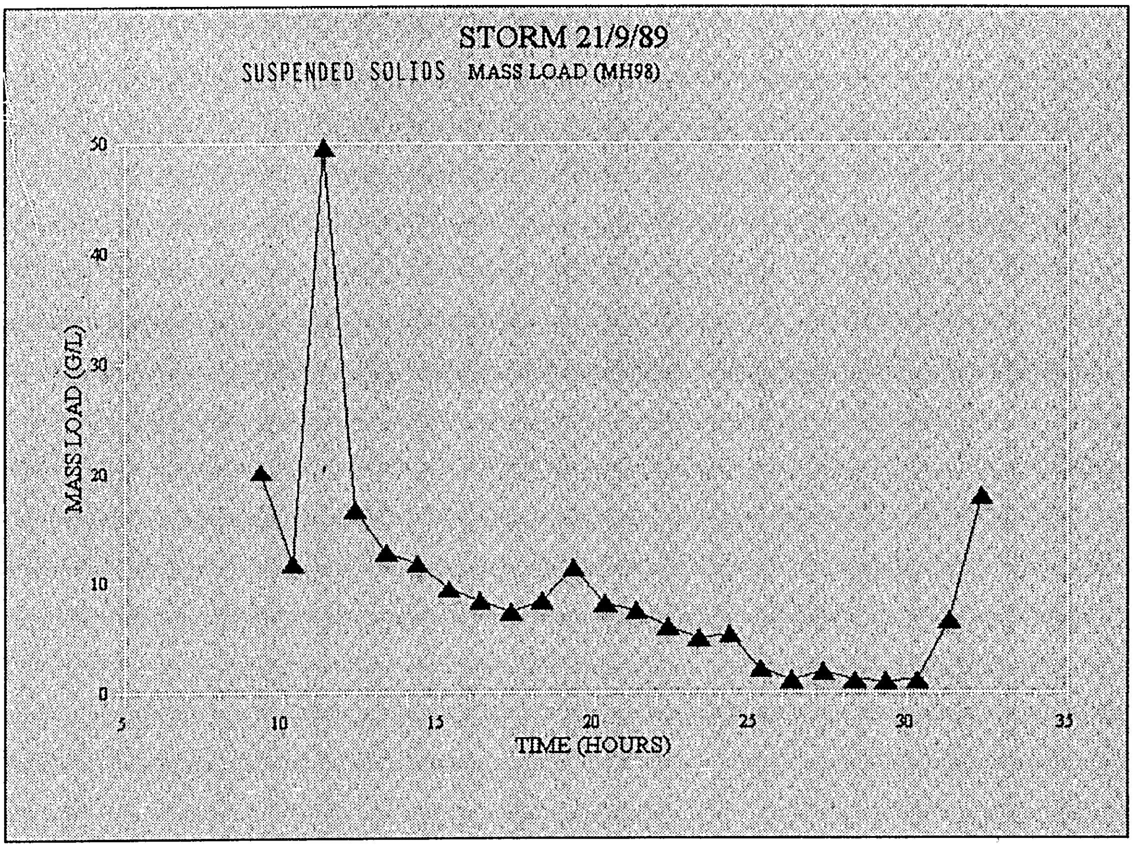

STORM 21/9/89
SUSPENDED SOLIDS MASS LOAD (MH98)
MASS LOAD (G/L)
50
40
30
20
10
0
5
10
15
20
25
30
35
TIME (HOURS)

Figure K15

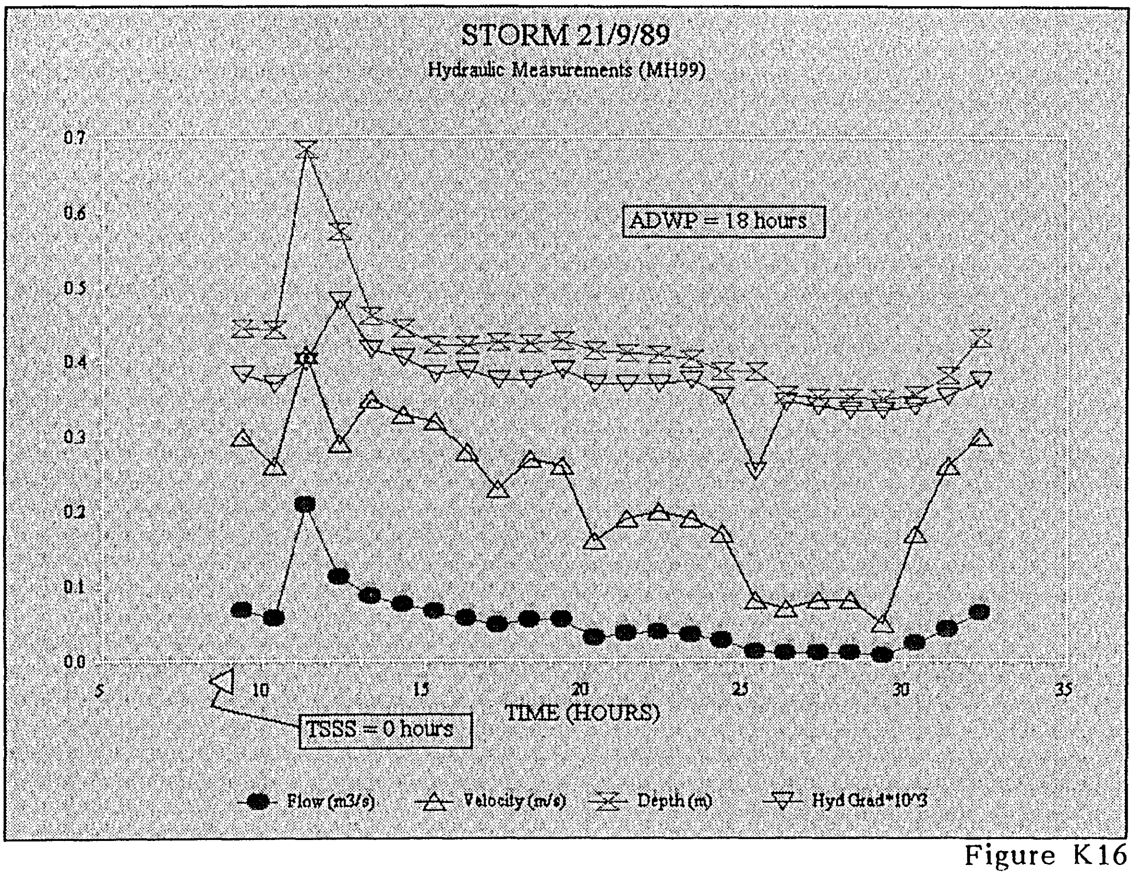


Figure K16

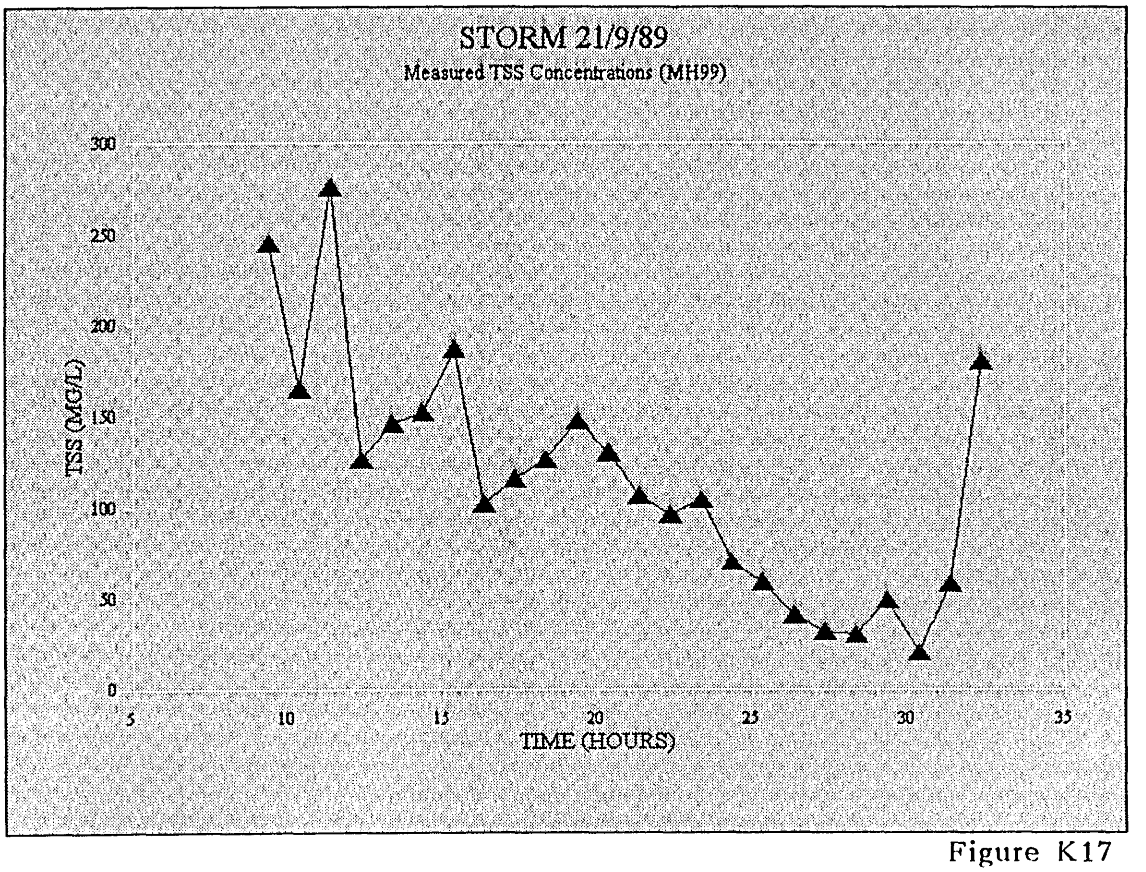


Figure K17

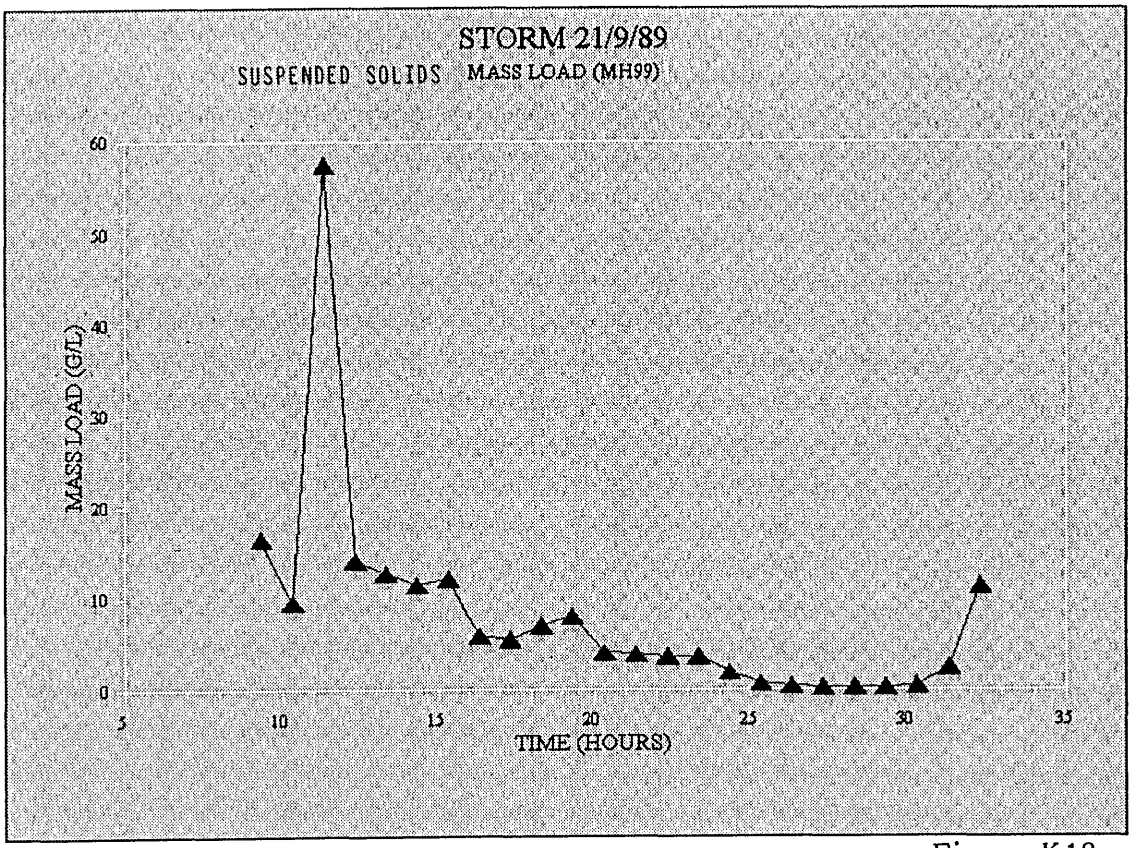

STORM 21/9/89
SUSPENDED SOLIDS MASS LOAD (MH99)
MASS LOAD (G/L)
0
10
20
30
40
50
60
5
10
15
20
25
30
35
TIME (HOURS)

Figure K18

Appendix L

Multiple Depth Storm Sewage Sample and Flow Data

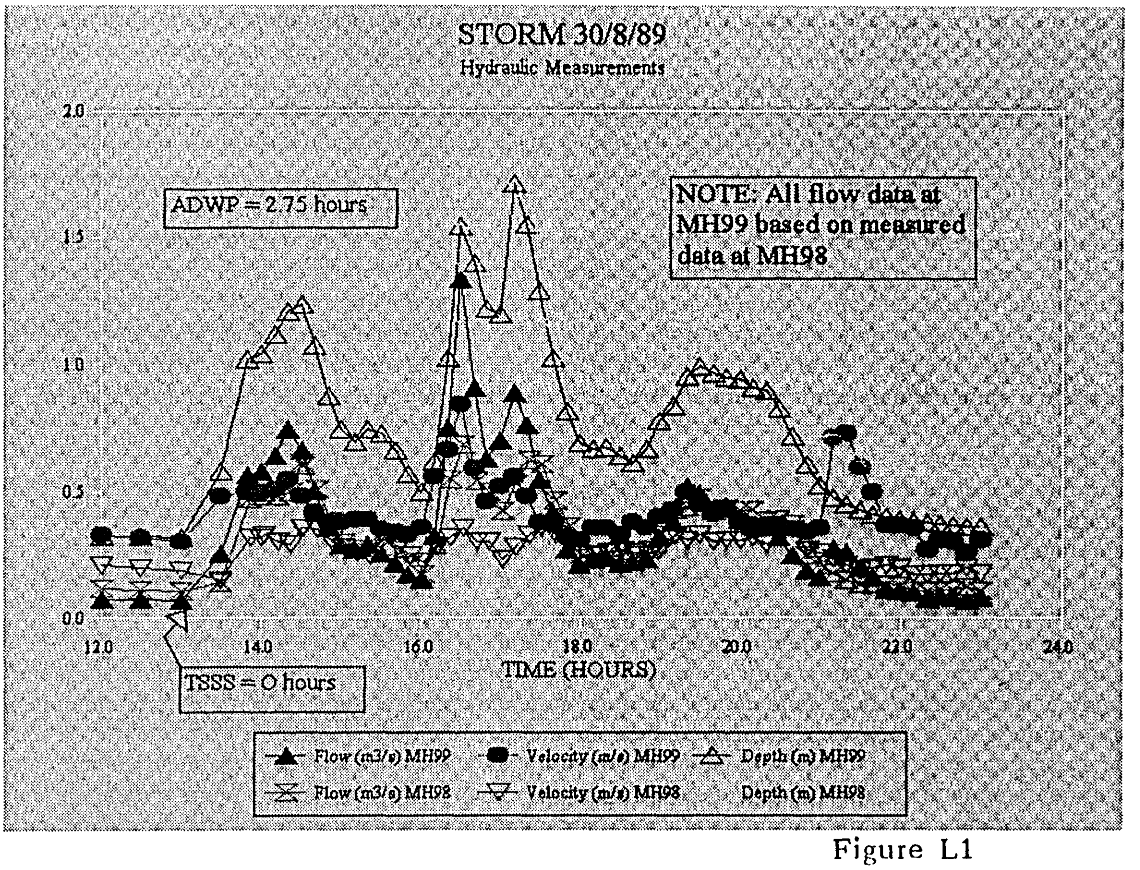


Figure L1

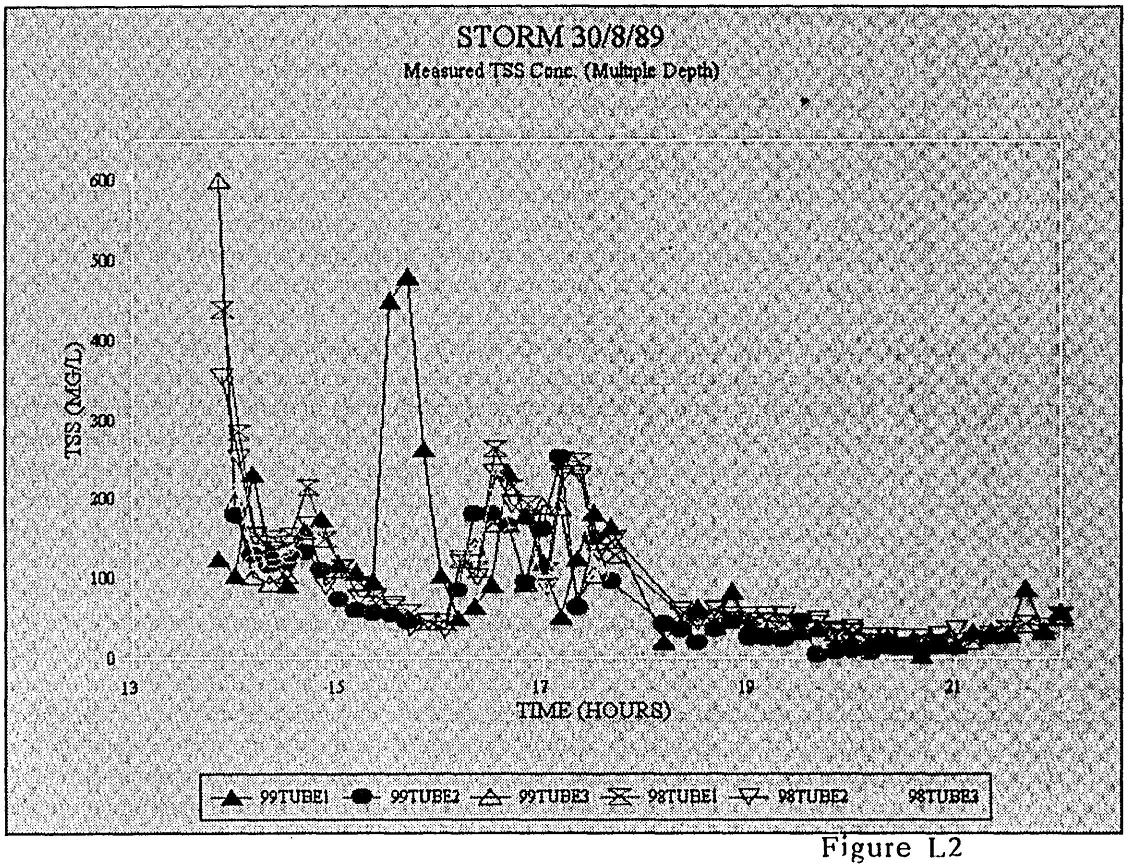


Figure L2

Appendix M

Ackers Model Spreadsheet

A	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Z	AA	AB	AC	AD
1	87/11/5	TIME	TIME	TIME	FLOW	VEL	DEPTH	TSS	DEPTH	S	d35	s	A	R	v*	Ws	Dgr	n	m	Agr	C	Fgr	Ggr	X					
2	MH98	(HOURS)	(MINS)	(DECIMAL)	(M3/S)	(M/S)	(MM)	(MG/L)	(MM)	(hyd grad)	(m)	(spec grav)				(sed width)								(MG/L)					
3								TUBE 1					(area)		(shear vel)									(mh99)					
4														(hyd rad)															
5																													
6	88/07/15	11	1.4694E-39	11	0.102	0.27	441	193	462	0.00034706	0.001729	1.006	0.53788	0.1634394	0.02358927	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.86570446	3.25477325	60.8151407			Regression Output:		
7	MH99	12	1.4694E-39	12	0.092	0.25	435	209	441	0.00043529	0.001729	1.006	0.53658	0.1612165	0.026236	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	2.01087081	3.57248594	60.9980594		Constant			0
8		13	1.4694E-39	13	0.096	0.27	426	135	444	0.00036471	0.001729	1.006	0.51958	0.15788214	0.02378689	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.90823418	3.32547877	64.051293		Std Err of Y Est			96.1290248
9		14	1.4694E-39	14	0.075	0.21	427	208	444	0.00037059	0.001729	1.006	0.5208	0.15825283	0.02398589	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.77388443	2.33811801	38.7873175		R Squared			0.44321158
10		15	1.4694E-39	15	0.109	0.32	414	150	424	0.00041176	0.001729	1.006	0.50494	0.15343634	0.02489563	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	2.17903136	4.67199099	99.3010951		No. of Observations			180
11		16	1.4694E-39	16	0.058	0.17	414	172	428	0.0004	0.001729	1.006	0.50494	0.15343634	0.0245374	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.64283679	1.79555572	28.895016		Degrees of Freedom			179
12		17	1.4694E-39	17	0.078	0.22	417	151	432	0.00038235	0.001729	1.006	0.5088	0.15454779	0.02407878	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.81859201	2.53558537	44.1040889					
13		18	1.4694E-39	18	0.065	0.19	415	159	439	0.00032941	0.001729	1.006	0.50616	0.15380682	0.0222942	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.63250211	1.75753743	29.5295717		X Coefficient(s)		1.01103401	
14		19	1.4694E-39	19	0.047	0.13	428	193	436	0.00042353	0.001729	1.006	0.52202	0.15862311	0.02587199	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.49700708	1.29936598	15.7844198		Std Err of Coef.		0.0472469	
15		20	1.4694E-39	20	0.077	0.22	420	148	430	0.00041176	0.001729	1.006	0.51228	0.15565924	0.02507532	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.8580465	2.73754571	48.2014732					
16		21	1.4694E-39	21	0.065	0.2	403	116	422	0.00035882	0.001729	1.006	0.49152	0.14936101	0.02292945	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.69910289	2.01738462	35.3861479		################################			
17		22	1.4694E-39	22	0.092	0.28	404	116	421	0.00037059	0.001729	1.006	0.49274	0.1497315	0.02333119	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.98522841	3.4229769	71.7128971		########	d35=	0.001729	########
18		23	1.4694E-39	23	0.049	0.15	405	141	415	0.00041176	0.001729	1.006	0.49398	0.15010198	0.02462364	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.58108193	1.50456804	21.4184332		########	s=	1.006	########
19		1.4694E-39	1.4694E-39	24	0.055	0.17	398	115	412	0.00038824	0.001729	1.006	0.48542	0.14750659	0.02370231	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.61470971	1.69208975	28.8660461		########	Ws=	0.5	########
20		1	1.4694E-39	25	0.052	0.17	385	90	400	0.00038235	0.001729	1.006	0.46958	0.1426923	0.02313487	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.59802636	1.62512488	27.0626351		################################			
21		2	1.4694E-39	26	0.023	0.08	375	63	396	0.00034706	0.001729	1.006	0.45736	0.13898746	0.02175324	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.11331446	0.44422376	5.13211834					
22		3	1.4694E-39	27	0.043	0.15	369	37	380	0.00040588	0.001729	1.006	0.45004	0.13676456	0.02333573	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.52317344	1.38078694	22.2401678					
23		4	1.4694E-39	28	0.017	0.06	364	19	374	0.00041176	0.001729	1.006	0.44394	0.13491214	0.0233445	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.02471104	0.32424364	3.15058037					
24		5	1.4694E-39	29	0.033	0.12	362	27	377	0.00038235	0.001729	1.006	0.4415	0.13417117	0.02243346	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.36378284	0.9094465	13.4564062					
25		6	1.4694E-39	30	0.039	0.14	364	28	382	0.00036471	0.001729	1.006	0.44394	0.13491214	0.02197007	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.42987397	1.10516246	17.9566533					
26		7	1.4694E-39	31	0.083	0.29	370	35	394	0.00032941	0.001729	1.006	0.45128	0.13713504	0.02105127	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.91219543	3.01700244	74.6210405					
27		8	1.4694E-39	32	0.068	0.22	389	97	412	0.00033529	0.001729	1.006	0.47444	0.14417424	0.02177066	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.72365854	2.11925525	41.8272048					
28		9	1.4694E-39	33	0.061	0.18	414	190	423	0.00041765	0.001729	1.006	0.50494	0.15343634	0.02507283	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.704[illegible]066	2.0398218	31.1754494					
29		10	1.4694E-39	34	0.118	0.33	427	231	432	0.00044118	0.001729	1.006	0.5208	0.15825283	0.02617075	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	2.26730594	5.32880617	108.627419					
30	88/07/19	12	35	12.5833333	0.121	0.32	443	314	428	0.00057059	0.001729	1.006	0.54032	0.16418037	0.03031493	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	2.[illegible]90927	6.65738006	118.283217					
31	MH99	13	35	13.5833333	0.087	0.24	435	221	417	0.00057647	0.001729	1.006	0.53058	0.1612165	0.0301945	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	2.13669248	4.39200189	87.6754603					
32		14	35	14.5833333	0.097	0.27	428	214	410	0.00057647	0.001729	1.006	0.52202	0.15862311	0.02996065	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	2.24294812	5.14183486	86.4748201					
33		15	35	15.5833333	0.093	0.27	420	160	404	0.00058471	0.001729	1.006	0.51228	0.15565924	0.02936521	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	2.22081291	4.97425631	86.2080601					
34		16	35	16.5833333	0.101	0.29	424	208	407	0.00057059	0.001729	1.006	0.51714	0.15714118	0.02965794	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	2.30200974	5.60337453	99.6062036					
35		17	35	17.5833333	0.108	0.31	423	161	407	0.00058471	0.001729	1.006	0.51592	0.15677089	0.02946986	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	2.36146405	6.09387711	113.167117					
36		18	35	18.5833333	0.1	0.28	424	187	407	0.00057059	0.001729	1.006	0.51714	0.15714118	0.02965794	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	2.26728378	5.32871173	92.8802132					
37		19	35	19.5833333	0.095	0.27	424	186	407	0.00057059	0.001729	1.006	0.51714	0.15714118	0.02965794	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	2.23180722	5.05778608	86.3425279					
38		20	35	20.5833333	0.07	0.21	414	133	399	0.00055882	0.001729	1.006	0.50494	0.15340634	0.02900253	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.97907059	3.36743869	52.0229863					
39		21	35	21.5833333	0.07	0.2	416	134	401	0.00055882	0.001729	1.006	0.50738	0.1541773	0.02907248	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.93968915	3.16586065	47.0037208					
40		22	35	22.5833333	0.051	0.15	412	131	398	0.00058471	0.001729	1.006	0.5025	0.15266537	0.0288843	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.71355526	2.07060844	26.4494185					
41		23	35	23.5833333	0.036	0.11	408	119	393	0.00055882	0.001729	1.006	0.49762	0.15121343	0.02879168	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.49020797	1.27874333	13.8748777					

Figure 1 - Ackers Model Spreadsheet

A	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y
1	87/11/5	TIME	TIME	TIME	FLOW	VEL	DEPTH	TSS	DEPTH	S	d35	s	A	R	V*	Ws	Dgr	n	m	Agr	C	Fgr	Ggr	X	
2	MH98	(HOURS)	(MINS)	(DECIMAL)	(M3/S)	(M/S)	(MM)	(MG/L)	(MM)	(hyd grad)	(m)	(spec grav)				(sed width)								(MG/L)	
3								TUBE 1					(area)		(shear vel)									(mh99)	
4														(hyd rad)											
5																									
42		1.4694E-39	35	24.5833333	0.041	0.13	397	77	366	0.00053529	0.001729	1.008	0.4842	0.14713811	0.02779671	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.57329584	1.54606037	19.3315463	
43		1	35	25.5833333	0.047	0.16	381	79	370	0.00053529	0.001729	1.008	0.46466	0.14121036	0.02723103	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.70605033	2.04586965	30.3320177	
44		2	35	26.5833333	0.05	0.17	373	67	363	0.00052941	0.001729	1.008	0.45492	0.13824649	0.02679528	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.73790844	2.17990518	34.4787237	
45	88/11/27	10	9	10.15	0.04	0.27	273	91	375	0.00025294	0.001729	1.008	0.33292	0.10119609	0.01584639	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.60977541	1.67422993	63.2987931	
46	MH99	11	9	11.15	0.039	0.23	295	136	399	0.00024118	0.001729	1.008	0.35976	0.10934874	0.01608455	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.50665586	1.32900651	42.1070598	
47		12	9	12.15	0.045	0.27	298	115	402	0.00024118	0.001729	1.008	0.36342	0.11046019	0.01616609	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.61873617	1.70676414	58.4511798	
48		13	9	13.15	0.04	0.23	300	123	403	0.00024706	0.001729	1.008	0.36586	0.11120116	0.01641665	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.52253001	1.37674537	42.4604398	
49		14	9	14.15	0.045	0.28	291	103	394	0.00024706	0.001729	1.008	0.35486	0.1078668	0.01616663	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.64719313	1.81282398	64.893147	
50		15	9	15.15	0.032	0.21	276	83	380	0.00024118	0.001729	1.008	0.33668	0.10230954	0.01555823	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.42758101	1.09863901	36.0146026	
51		16	9	16.15	0.03	0.2	272	82	376	0.00024118	0.001729	1.008	0.3317	0.10082761	0.01544513	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.39324813	1.00782715	32.7392265	
52		17	9	17.15	0.037	0.25	274	73	377	0.00024706	0.001729	1.008	0.33414	0.10156858	0.01568969	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.54781184	1.46047797	52.967083	
53		18	9	18.15	0.036	0.25	276	75	378	0.00025294	0.001729	1.008	0.33658	0.10230954	0.01593318	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.56061702	1.50307351	53.847099	
54		19	9	19.15	0.043	0.27	282	91	387	0.00023529	0.001729	1.008	0.3439	0.10453245	0.01553337	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.58628921	1.59791553	59.150136	
55		20	9	20.15	0.04	0.27	275	82	378	0.00025882	0.001729	1.008	0.33536	0.10193906	0.01606617	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.62265377	1.72184274	64.0736065	
56		21	9	21.15	0.03	0.22	262	87	364	0.00025294	0.001729	1.008	0.3195	0.09712277	0.01552404	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.45967699	1.18938494	42.2136339	
57		22	9	22.15	0.026	0.19	263	84	365	0.00025294	0.001729	1.008	0.32072	0.09749325	0.01555362	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.37105963	0.95171998	30.936274	
58		23	9	23.15	0.029	0.2	274	87	379	0.00023529	0.001729	1.008	0.33414	0.10156858	0.01531157	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.36574149	0.9666087	32.0376365	
59		1.4694E-39	9	24.15	0.067	0.28	371	159	496	0.00011765	0.001729	1.008	0.45248	0.13750552	0.01259752	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.3633954	0.93263125	28.9286028	
60		1	9	25.15	0.081	0.28	400	104	491	0.00031765	0.001729	1.008	0.48786	0.14824958	0.02149333	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.89628434	2.93301308	65.0136914	
61		2	9	26.15	0.112	0.27	507	236	611	0.00024118	0.001729	1.008	0.6184	0.18789135	0.02108411	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.81968799	2.55031476	44.1701317	
62		3	9	27.15	0.045	0.29	277	71	391	0.00018235	0.001729	1.008	0.3378	0.10268003	0.01355295	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.51832528	1.36545314	57.8639491	
63		4	9	28.15	0.017	0.17	213	45	319	0.00022941	0.001729	1.008	0.25972	0.07896905	0.01333125	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.21445129	0.61341144	25.2253139	
64		5	9	29.15	0.022	0.25	201	25	308	0.00022353	0.001729	1.008	0.24508	0.07452324	0.01278344	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.40751701	1.04504149	58.0168187	
65		6	9	30.15	0.011	0.12	203	25	312	0.00021176	0.001729	1.008	0.24752	0.07526421	0.01250419	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.01031166	0.30705194	11.2790985	
66		7	9	31.15	0.042	0.34	244	75	349	0.00023529	0.001729	1.008	0.29754	0.09045406	0.01444954	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.70119457	2.02593283	102.884922	
67		8	9	32.15	0.049	0.33	286	105	391	0.00023529	0.001729	1.008	0.34878	0.10601438	0.01584309	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.73804743	2.18050233	66.8092701	
68	88/12/02	10	28	10.4666667	0.078	0.28	420	95	527	0.00022353	0.001729	1.008	0.51228	0.15565924	0.01847521	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.66040272	1.94202062	42.8294663	
69	MH99	11	28	11.4666667	0.08	0.28	406	85	511	0.00023529	0.001729	1.008	0.49518	0.15047248	0.01863666	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.74755293	2.22159936	52.5969347	
70		12	28	12.4666667	0.065	0.34	425	99	530	0.00023529	0.001729	1.008	0.51836	0.15751166	0.01906762	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.92040508	3.06095467	76.2805451	
71		13	28	13.4666667	0.106	0.39	518	136	623	0.00023529	0.001729	1.008	0.63182	0.19196667	0.02105004	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	2.13966042	4.32921716	90.458181	
72		14	28	14.4666667	0.094	0.3	433	107	537	0.00024118	0.001729	1.008	0.52812	0.16047553	0.01946527	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.83925066	2.6446967	59.5286927	
73		15	28	15.4666667	0.078	0.3	387	63	493	0.00022941	0.001729	1.008	0.472	0.1434327	0.01796666	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.76699562	2.31618423	61.0732059	
74		16	28	16.4666667	0.065	0.25	383	66	488	0.00023529	0.001729	1.008	0.46712	0.14195133	0.0181013	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.64247659	1.79495813	42.9509659	
75		17	28	17.4666667	0.063	0.25	380	78	485	0.00023529	0.001729	1.008	0.46346	0.14083466	0.0180303	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.63963875	1.784253	43.1277306	
76		18	28	18.4666667	0.079	0.31	378	54	484	0.00022941	0.001729	1.008	0.46102	0.14009891	0.0177566	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.78506815	2.36660365	66.1350505	
77		19	28	19.4666667	0.069	0.27	379	71	485	0.00022941	0.001729	1.008	0.46224	0.1404694	0.01778006	0.5	5.95105366	0.56622743	2.96324196	0.23428246	0.00882909	1.882216	1.94924546	49.7362728	

Figure 2 - Ackers Model Spreadsheet (Continued)

A	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y
1	87/11/5	TIME	TIME	TIME	FLOW	VEL	DEPTH	TSS	DEPTH	S	d35	s	A	R	V*	Ws	Dgr	n	m	Agr	C	Fgr	Ggr	X	
2	MH98	(HOURS)	(MINS)	(DECIMAL)	(M3/S)	(M/S)	(MM)	(MG/L)	(MM)	(hyd grad)	(m)	(spec grav)				(sed width)								(MG/L)	
3								TUBE 1					(area)		(shear vel)									(mh99)	
4														(hyd rad)											
5																									
78		20	28	20.4666667	0.045	0.19	362	53	466	0.00024118	0.001729	1.008	0.4415	0.13417117	0.01781686	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.45028112	1.1620017	25.4109819	
79		21	28	21.4666667	0.175	0.3	642	185	740	0.00027647	0.001729	1.008	0.7831	0.23799809	0.02540167	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	2.08717329	4.04793297	52.8865221	
80		22	28	22.4666667	0.062	0.28	417	67	521	0.00024118	0.001729	1.008	0.5086	0.15454779	0.019122	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.77022845	2.32170195	52.7436673	
81		23	28	23.4666667	0.053	0.21	371	47	475	0.00024118	0.001729	1.008	0.45246	0.13750652	0.01803691	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.5228201	1.37967508	30.8400073	
82		1.4694E-39	28	24.4666667	0.066	0.3	347	60	448	0.00025882	0.001729	1.008	0.4232	0.12861391	0.01897992	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.78710273	2.39810273	70.2909335	
83		1	28	25.4666667	0.037	0.19	321	22	423	0.00025294	0.001729	1.008	0.39148	0.11898132	0.01718239	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.43177762	1.11038492	27.9513766	
84		2	28	26.4666667	0.023	0.13	308	22	411	0.00024706	0.001729	1.008	0.37582	0.11416503	0.01663417	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.19556762	0.57905056	12.4810804	
85		3	28	27.4666667	0.063	0.37	297	28	399	0.00025294	0.001729	1.008	0.3622	0.11008971	0.0165279	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.87962617	2.84675561	115.469688	
86		4	28	28.4666667	0.042	0.28	290	22	392	0.00025294	0.001729	1.008	0.35368	0.10749632	0.01633206	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.60454463	1.6554338	56.6984436	
87		5	28	29.4666667	0.03	0.19	286	16	369	0.00024706	0.001729	1.008	0.34876	0.10601436	0.01602939	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.38694471	0.991671	29.1414727	
88		6	28	30.4666667	0.052	0.32	287	8	390	0.00024706	0.001729	1.008	0.35	0.10638467	0.01605736	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.74017565	2.18965953	86.0533346	
89		7	28	31.4666667	0.066	0.4	298	19	401	0.00024706	0.001729	1.008	0.36342	0.11046019	0.01636205	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.93278193	3.12801582	132.916437	
90		8	28	32.4666667	0.05	0.28	321	38	425	0.00024118	0.001729	1.008	0.39146	0.11898132	0.01677805	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.61846197	1.70582838	51.9819333	
91	88/09/19	7	5	7.08333333	0.0341	0.22	272	159	428	2.3529E-05	0.001729	1.008	0.3317	0.10082761	0.00482425	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	0.75133512	0.09218572	8.10846485	
92	MH99	8	5	8.08333333	0.0619	0.3	330	250	452	0.00022353	0.001729	1.008	0.40246	0.12231568	0.01637732	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.89569509	2.00350697	65.2862703	
93		9	5	9.08333333	0.0648	0.28	357	157	449	0.0004	0.001729	1.008	0.4354	0.13231875	0.02278037	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.97416156	3.35927574	80.7156058	
94		10	5	10.0833333	0.0762	0.34	353	278	448	0.00038235	0.001729	1.008	0.43052	0.13083881	0.02215296	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	2.11513176	4.23162068	116.624288	
95		11	5	11.0833333	0.0613	0.27	352	178	442	0.00041176	0.001729	1.008	0.4293	0.13046833	0.02295664	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.96322346	3.24089247	77.0420096	
96		12	5	12.0833333	0.069	0.31	348	222	439	0.00040588	0.001729	1.008	0.42442	0.12898439	0.02268228	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	2.08025022	3.8781162	101.524048	
97		13	5	13.0833333	0.0722	0.33	344	228	432	0.00042353	0.001729	1.008	0.41954	0.12750246	0.0230163	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	2.13712776	4.37995419	119.185732	
98		14	5	14.0833333	0.069	0.32	340	130	419	0.00047647	0.001729	1.008	0.41466	0.12602052	0.02427019	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	2.174718	4.64135194	121.861528	
99		15	5	15.0833333	0.0567	0.27	323	140	416	0.00039412	0.001729	1.008	0.39392	0.11972229	0.02151468	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.89318238	2.91682147	78.3901554	
100		16	5	16.0833333	0.06	0.31	321	129	421	0.00035294	0.001729	1.008	0.39146	0.11898132	0.02029668	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.94563249	3.19866814	96.6766091	
101		17	5	17.0833333	0.0463	0.31	326	143	417	0.00040588	0.001729	1.008	0.39758	0.12083374	0.02193455	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	2.03100927	3.69505881	105.22933	
102		18	5	18.0833333	0.0499	0.26	320	206	426	0.00031765	0.001729	1.008	0.39026	0.11861084	0.01922512	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.74653614	2.22586603	62.9934745	
103		19	5	19.0833333	0.0612	0.29	330	156	414	0.00044706	0.001729	1.008	0.40246	0.12231568	0.02316103	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	2.03323201	3.70862058	97.430256	
104		20	5	20.0833333	0.0512	0.26	317	131	405	0.00042353	0.001729	1.008	0.3866	0.11749939	0.022095	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.95465525	3.25001765	89.4906775	
105		21	5	21.0833333	0.0481	0.27	308	126	398	0.0004	0.001729	1.008	0.37316	0.11342406	0.02106683	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.87864054	2.84272962	81.5433791	
106		22	5	22.0833333	0.0365	0.22	299	105	404	0.00032353	0.001729	1.008	0.36464	0.11083068	0.01875519	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.61076609	1.67780586	46.8829332	
107		23	5	23.0833333	0.0418	0.23	306	136	386	0.00047059	0.001729	1.008	0.37316	0.11342406	0.02286272	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.83512748	2.6246164	65.6608669	
108		24	5	24.0833333	0.0293	0.18	286	75	369	0.00045294	0.001729	1.008	0.34876	0.10601436	0.0217039	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.60842552	1.66906579	40.074904	
109		25	5	25.0833333	0.0232	0.16	269	38	353	0.00044706	0.001729	1.008	0.32804	0.09971618	0.02091219	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.50255577	1.31635627	32.0981014	
110		26	5	26.0833333	0.0115	0.09	250	37	349	0.00035882	0.001729	1.008	0.30486	0.09267686	0.01806179	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.08274769	0.36999557	8.23217062	
111		27	5	27.0833333	0.0137	0.11	245	25	344	0.00035882	0.001729	1.008	0.29876	0.09082454	0.01786037	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.17606122	0.54495551	12.8950398	
112		28	5	28.0833333	0.0186	0.15	241	30	344	0.00033529	0.001729	1.008	0.29366	0.0893426	0.01714128	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.3151806	0.8196754	24.0704055	
113		29	5	29.0833333	0.0122	0.09	241	26	349	0.00030588	0.001729	1.008	0.29366	0.0893426	0.01637348	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.02875144	0.32873035	7.37398609	
114		30	5	30.0833333	0.0197	0.15	246	103	369	0.00021765	0.001729	1.008	0.29986	0.09119802	0.01350920	0.5	5.95105366	0.58622743	2.96324198	0.23428246	0.00882909	1.16889504	0.53272593	17.2201969	

Figure 3 - Ackers Model Spreadsheet (Continued)

Appendix N

Sample of Grid Pattern of R^2 Values

d35=.0028	s=1.00045	d35=.003	s=1.00045	d35=.0031	s=1.00045
W=1.1E-08		W=1.2E-07		W=3.3E-07	
Intercept	-11.79242	Intercept	-6.493365	Intercept	-3.876929
Slope	0.9998484	Slope	1.0016855	Slope	0.9996056
R^2	0.4407866	R^2	0.4412077	R^2	0.4411119
Sum X^2	4139639	Sum X^2	4139639	Sum X^2	4139639
Sum Y^2	5169979.4	Sum Y^2	5413177.8	Sum Y^2	5505766.4
Sum X*Y	3868670.3	Sum X*Y	3997756	Sum X*Y	4049127.5

d35=.0028	s=1.00048	d35=.003	s=1.00048	d35=.0031	s=1.00048
W=3.5E-08		W=3.1E-07		W=8E-07	
Intercept	-10.50467	Intercept	-4.879793	Intercept	-2.24275
Slope	1.0232917	Slope	0.9973906	Slope	0.9918989
R^2	0.4410784	R^2	0.4412535	R^2	0.4410383
Sum X^2	4139639	Sum X^2	4139639	Sum X^2	4139639
Sum Y^2	5480378.7	Sum Y^2	5435849.6	Sum Y^2	5492854.1
Sum X*Y	3995238.5	Sum X*Y	4016967.8	Sum X*Y	4054688.3

d35=.0028	s=1.0005	d35=.003	s=1.0005	d35=.0031	s=1.0005
W=6.5E-08		W=5.5E-07		W=1.4E-06	
Intercept	-9.362586	Intercept	-3.853098	Intercept	-1.225306
Slope	1.0076675	Slope	0.9929289	Slope	0.9944627
R^2	0.4412262	R^2	0.4412448	R^2	0.4409539
Sum X^2	4139639	Sum X^2	4139639	Sum X^2	4139639
Sum Y^2	5355234.9	Sum Y^2	5431550.1	Sum Y^2	5567825.1
Sum X*Y	3956742.5	Sum X*Y	4022034.7	Sum X*Y	4088626.2

APPENDIX O

EQUATIONS FOR THE SONNEN AND FIELD MODEL

Suspended Load Calculation

Rouse's Equation for Suspended Load:-

$$C = C_{ae}\left[\frac{(Y-y)}{y}\frac{a_e}{(Y-a_e)}\right]^Z \tag{O.4}$$

where C = concentration of sediment with fall velocity ω at level y (lb/ft^3)
C_{ae} = average concentration of the bed layer (lb/ft^3)
y = elevation above datum (ft)
a_e = the lower limit of y where suspended load begins (ft) = $2d_{60}$
$Z = V_s/Ku_*$
V_s = settlement velocity in quiescent conditions (ft/sec)
K = von Karman constant (assumed = 0.385)

To calculate C_{ae}:

$$C_{ae} = \frac{1}{11.6}\frac{g_{sb}}{u_*' a'} \tag{O5.}$$

where

u_*' = shear velocity with respect to solids (ft/sec)
$= \sqrt{g R_h' S}$
R_h' = hydraulic radius with respect to solids (ft)
a' = reference depth (ft) ($\approx a_e = 2d_{60}$)
$g_{sb} = g_{sb}$ from bed load calculation

Also

$$\frac{V_p}{u_*} = 5.75\log\left(12.27\frac{R_h' X}{k_s}\right) \tag{O6.}$$

where V_p = pipe velocity (ft/sec)

X= correction factor for wall effects
$\approx$ 1.0 for rough wall
k_s= sand roughness height (ft) ($\approx d_{60}$)

Assuming that

$$V_p = \overline{V_p} \ = \text{ average pipe velocity (ft/sec)}$$

and $$u_* = u_*' \text{ (ft/sec)}$$

then $$\frac{V_p}{u_*'} = 5.75 \log\left(2.27 \frac{R_h'}{k_s}\right) \qquad (O7.)$$

Procedure for Calculation of Suspended Solids

(1) Solve $F = V - 5.75\sqrt{gR_h'S}\ \log\left(12.27\frac{R_h'}{d_{60}}\right)$ for $F = 0$

(2) Hence, knowing R_h', find u_*'

(3) Calculate C_{ae}

(4) Calculate Z

(5) Calculate C for, say, 5 depth intervals from $y = 0$ to Y

(6) Calculate corresponding values of velocity (u_y) for each interval using Vanoni's equation:

$$u_y = u_*\left[5.5 + 5.75 \log\frac{u_* y}{\upsilon}\right] \quad \text{if} \quad \frac{u_* d_{60}}{\upsilon} \leq 10 \qquad (O8.)$$

or $$u_y = u_*\left[8.5 + 5.75 \log\frac{y}{d_{60}}\right] \quad \text{if} \quad \frac{u_* d_{60}}{\upsilon} > 10 \qquad (O9.)$$

(υ = kinematic viscosity of water
$= 1.0877 \times 10^{-5}$ ft/sec at 20°C
$= 1.0105 \times 10^{-2}$ cm^2/s))

(7) Calculate $$g_{ss} = \sum\left(C u_y \tfrac{Y}{5}\right) \qquad (O10.)$$

(8) Hence calculate $$G_{ss} = g_{ss} \times \frac{Q}{VY} \qquad (O11.)$$

Appendix P

Related Published Papers

FULL LIST - PAPERS PRESENTED/PUBLISHED August 1995

1. **The quality of sewage flows and sediment in Dundee** (1989)
   R.M.Ashley, B.P.Coghlan & C.Jefferies
   Wat Sci. Tech Vol 22, No 10/11, pp39-46, 1990

2. **Sewer sediments - their occurence, movement and polluting potential** (1990)
   R.M.Ashley, B.P.Coghlan & R.W.Crabtree
   Presented to Scottish branch of the Institution of Water and Environmental Management. 23rd January 1990.

3. **Erosion and movement of sediments in combined sewers** (1991)
   R M Ashley, D J J Wotherspoon & B P Coghlan
   Presented to IAHR/IAWPRC Joint Committee Workshop on Real Sewer Sediments, Universitie Libre de Bruxelles, 4-6 Sept 1991

4. **The deposition and erosion of sediments in sewers** (1992)
   R M Ashley, D J J Wotherspoon, M J Goodison, I McGregor, & B P Coghlan
   presented at IAWPRC Biennial Conference, Washington, May 1992, and
   Wat.Sci.Tech. Vol 26, No.5-6, pp1283-1293, 1992

5. **The erosion and movement of sediments and associated pollutants in combined sewers** (1992)
   R M Ashley, D J J Wotherspoon, B P Coghlan, and I McGregor
   Wat. Sci. Tech. Vol 25, No.8 pp101-114, 1992

6. **An appraisal of suspended sediment transport modelling methods for an interceptor sewer** .(1992)
   B P Coghlan, R M Ashley, C Jefferies
   Proc Int. Conf. 'Sewage into 2000', Amsterdam, Sept 1992 IAWQ/EWPCA/NVA

7. **Cohesive sediment erosion in combined sewers** (1993)
   R M Ashley, D J J Wotherspoon, B P Coghlan, E Ristenpart
   6th ICUSD, Niagara Falls, Sept 1993

8. **Fluid sediment movement and first flush in combined sewers** (1993)
   R M Ashley, S Arthur, B P Coghlan, I McGregor
   6th ICUSD, Niagara Falls, Sept 1993

9. **An appraisal of suspended sediment transport modelling methods for an interceptor sewer** (1993)
   B P Coghlan, R M Ashley, C Jefferies
   Wat. Sci. Tech. Vol 27 No. 5-6, pp 81-93, 1993

10. **Fluid sediment in combined sewers** (1994)
    R M Ashley, S Arthur, B P Coghlan, I McGregor
    Wat.Sci.Tech. Vol 29, No.1-2, pp113-123, 1994

11. **Solids Flux in Sewer Systems - an Empirical Approach** (1995)
    B P Coghlan
    Water Pollution '95 Conf., Wessex Inst. Tech., Porto Carras, Greece, April 1995

APPENDIX Q

TABLES OF REGRESSION OUTPUT, MURRAYGATE INTERCEPTOR

Results of regression analysis of correlations between measured TSS and other recorded parameters.

DRY WEATHER FLOW

Correlation

Linear correlation:

Variables	r^2	Std Error of Est.	
Q	0.346758	53.54727	← **Best option**
V	0.122568	62.0593	
Y	0.27313	56.4844	
S	0.176255	60.130	
R	0.27313	56.4844	
V*	0.269385	56.62972	
Q & Y	0.349797	53.67397	
Q & R	0.349797	53.67397	

Equation:

$$TSS = 35.39479 + 1295.345\ Q \qquad (Q1.)$$

% between ½ and 2 = 77.98 %

Log/Log correlation:

Variables (Log)	r^2	Std Error of Est.	
Q	0.353126	0.260925	← **Best option**
V	0.139192	0.300995	
Y	0.268953	0.277382	
S	0.067533	0.313273	
R	0.268952	0.277382	
V*	0.16581	0.269304	
Q & Y	0.35638	0.261493	

Equation:

$$TSS = 10^{2.98} * Q^{0.798434} \qquad (Q2.)$$

% between ½ and 2 = 82.57 %

Linear/ln correlation:

Variables (ln)	r^2	Std Error of Est.	
Q	0.365633	52.76797	
V	0.140853	61.40927	
Y	0.274344	56.43723	
S	0.090752	63.17444	
R	0.274346	56.43715	
V*	0.194904	59.4462	
Q, R, Y & V*	0.401412	51.99233	← **Best option**
Q, R, Y, V*, & V	0.404387	52.11415	
Q, R, Y, V*, V & S	0.404387	52.36899	

Equation:

$$TSS = 3185926 + 71.217 \ln Q + 403304 \ln R - 403161 \ln Y + 60.099 \ln V^* - 53.94 \ln V \qquad (Q3.)$$

% between ½ and 2 = 80.73 %

Hence **adopt equation** **(Q2.).**

Validation

Using Equation (Q2.),

% between ½ and 2 = 81.4 %

STORM FLOW

Correlation

Linear correlation:

Variables	r^2	Std Error of Est.	
TSSS	0.029877	80.75586	
ADWP	0.01003	81.57774	
Flow	0.000115	81.98525	
Velocity	0.08047	78.62189	
Depth	0.000368	81.97486	
Hyd. Grad.	0.004374	81.81042	
Hyd. Rad.	0.000366	81.97495	
Shear Vel.	0.00158	81.92517	
V & TSSS	0.099015	78.11275	
V, TSSS, & ADWP	0.304271	68.89659	← **Best option**
V, TSSS, ADWP & S	0.304414	68.89659	

Equation:

$$TSS = 104.44 + 416.41\ V - 0.79674\ TSSS - 3.1238\ ADWP \qquad (Q4.)$$

% between ½ and 2 = 78.26 %

Log/Log correlation:

Variables (Log)	r^2	Std Error of Est.	
TSSS	0.029907	0.359343	
ADWP	0.021248	0.360943	
Q	0.000224	0.364799	
V	0.058532	0.354002	
Y	0.001439	0.364578	
S	0.001141	0.364632	
R	0.000224	0.364799	
V*	0.000215	0.364801	
V & ADWP	0.111281	0.345213	
V, ADWP & TSSS	0.144669	0.339928	
V, ADWP, TSSS & Y	0.203543	0.329251	
V, ADWP, TSSS, Y & S	0.210856	0.328975	
V, ADWP, TSSS, Y, S & Q	0.249913	0.321953	
V, ADWP, TSSS, Y, S, Q & R	0.263653	0.320215	← **Best option**
V, ADWP, TSSS, Y, S, Q, R & V*	0.263653	0.321454	

Equation:

$$TSS = 10^{5.95} * TSSS^{-0.0093} * ADWP^{-0.28526} * V^{1.685142} * Y^{-1.14914} * S^{-0.20662} * Q^{-0.63195} * R^{1.194024} \qquad (Q5.)$$

% between ½ and 2 = 76.09 %

Linear/ln correlation:

Variables (ln)	**r^2**	**Std Error of Est.**	
Q	**0.00364**	**81.8406**	
V	**0.093882**	**78.04643**	
Y	**0.002756**	**81.87689**	
S	**0.001031**	**92.29312**	
R	**0.002656**	**81.88099**	
V*	**3.76E-05**	**81.98841**	
TSSS	**0.053127**	**79.7823**	
ADWP	**0.000297**	**81.97778**	
V & TSSS	**0.144187**	**76.12946**	
V, TSSS & Q	**0.212425**	**73.30334**	**← Best option**
V, TSSS, Q & Y	**0.213045**	**73.54941**	
V, TSSS, Q, Y & R	**0.213155**	**73.82232**	
V, TSSS, Q, Y, R & S	**0.215273**	**74.00376**	
V, TSSS, Q, Y, R, S & ADWP	**0.232517**	**73.4671**	
V, TSSS, Q, Y, R, S, ADWP & V*	**0.232517**	**73.75131**	

Equation:

$$TSS = 190.84 + 111.663 \ln V - 1.1401 \ln TSSS - 33.518 \ln Q \qquad (Q6.)$$

% between ½ and 2 = 76.09 %

Since the best fit for storm flows was obtained by linear correlation, try linear correlation separately for data associated with first foul flush (FFF), and for other data ie non- first foul flush (NFFF). In order to achieve this some method of selection of FFF data is required. The simplest definition FFF is data at the start of a storm event during the time period when there is a continuous increase in velocity and depth. Any data whether during a rising or falling limb of a storm which is subsequent to a drop in depth or velocity is deemed to be NFFF data.

Linear correlation (FFF only):

Variables	r^2	Std Error of Est.	
V	0.128360	44.07291	
TSSS	0.007636	47.02602	
ADWP	0.047735	46.06615	
Y	0.050767	45.99276	
S	0.105264	44.65299	
V*	0.004425	47.10208	
R	0.015316	46.84374	
Q	0.003608	47.12140	
V & S	0.221449	43.90615	← **Best option**
V, S & Y	0.258533	45.44687	

Equation:

$$\text{TSS} = 376.31 + 224.64\ \text{V} - 503017\ \text{S} \qquad \text{(Q7.)}$$

Linear correlation (non-FFF only):

Variables	r^2	Std Error of Est.	
V	0.0793750	81.37617	
S	0.0058963	84.56148	
Y	0.005707	84.56937	
ADWP	0.009712	84.39887	
R	0.000234	84.80179	
TSSS	0.042620	82.98470	
V*	0.001709	84.73920	
Q	0.000158	84.80500	
V & TSSS	0.108820	80.38906	
V, TSSS & ADWP	0.320081	70.50428	← **Best option**
V, TSSS, ADWP & S	0.321421	70.72522	

Equation:

$$\text{TSS} = 104.9 + 434.42\ \text{V} - 0.9135\ \text{TSSS} - 3.245\ \text{ADWP} \qquad \text{(Q8.)}$$

For combined use of equations (Q7) and (Q8),

% between ½ and 2 = 79.0 %

Validation

Using equation (Q4.) (as discussed in main text, Section 4),

% between ½ and 2 = 79.0 %

Appendix R

Perth Road Sewage Sample and Flow Data

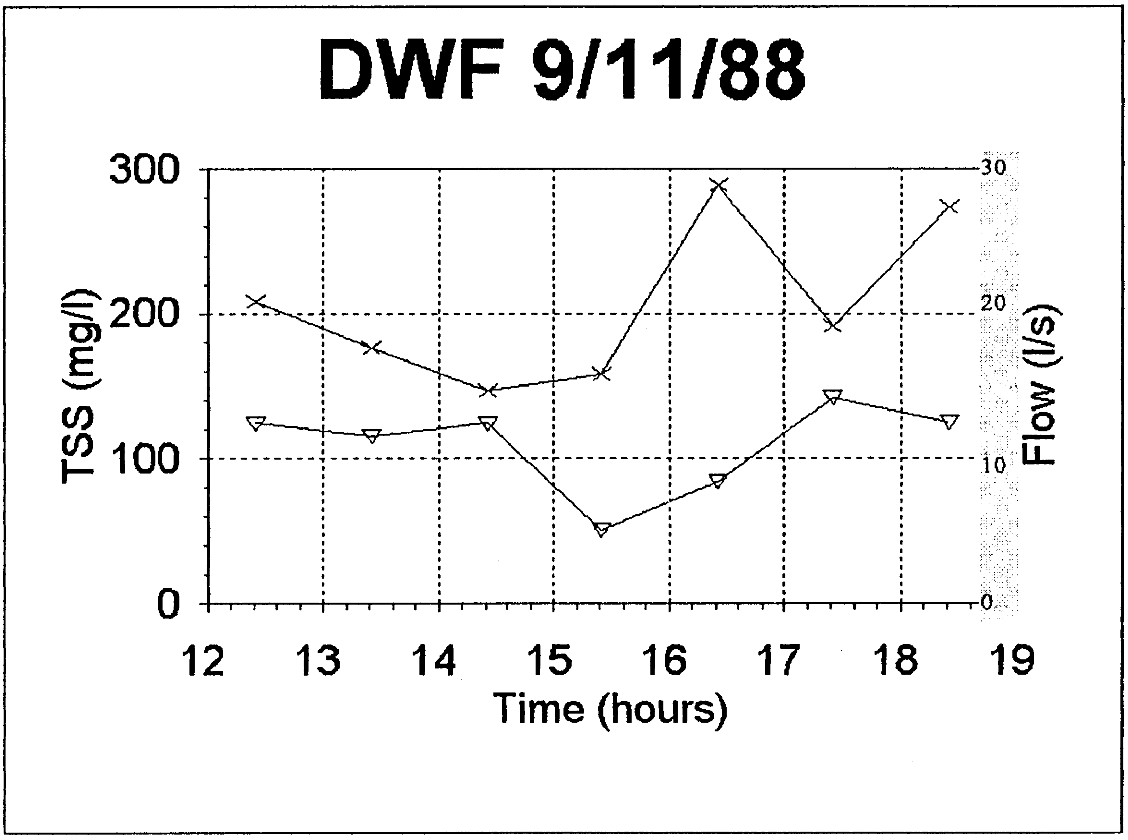


Figure 1

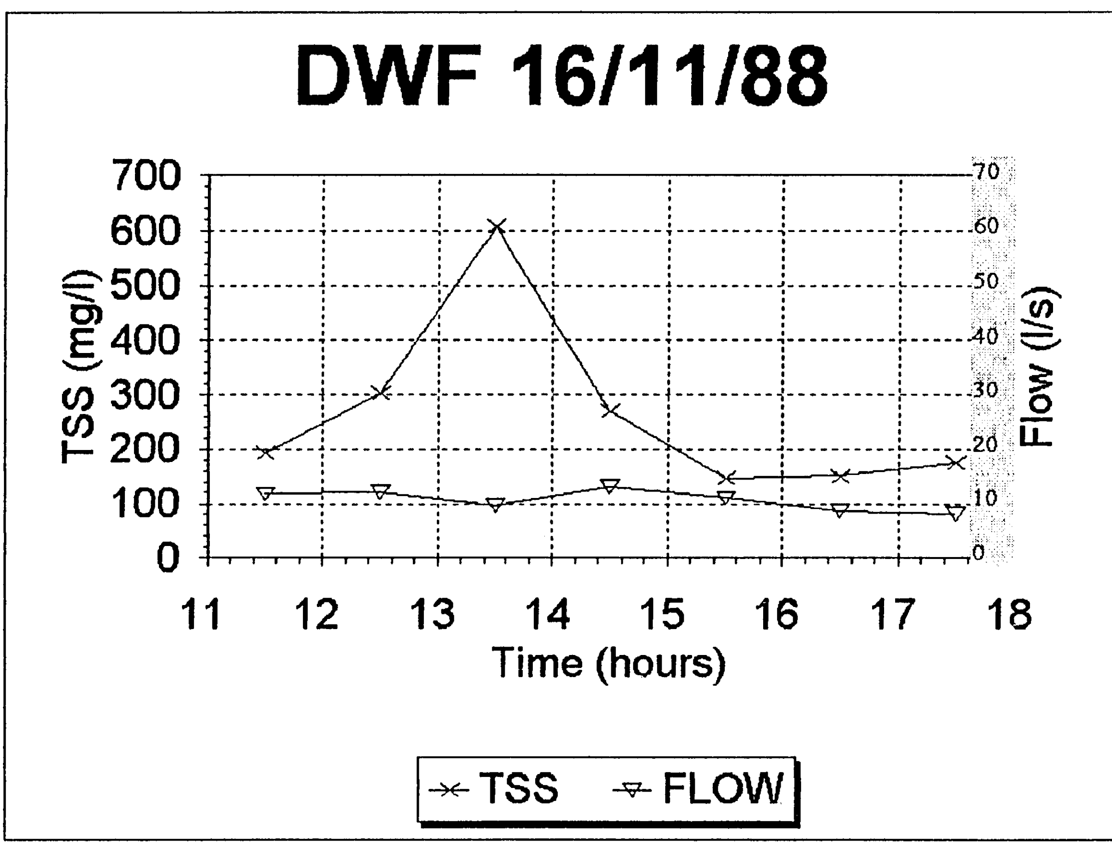


Figure 2

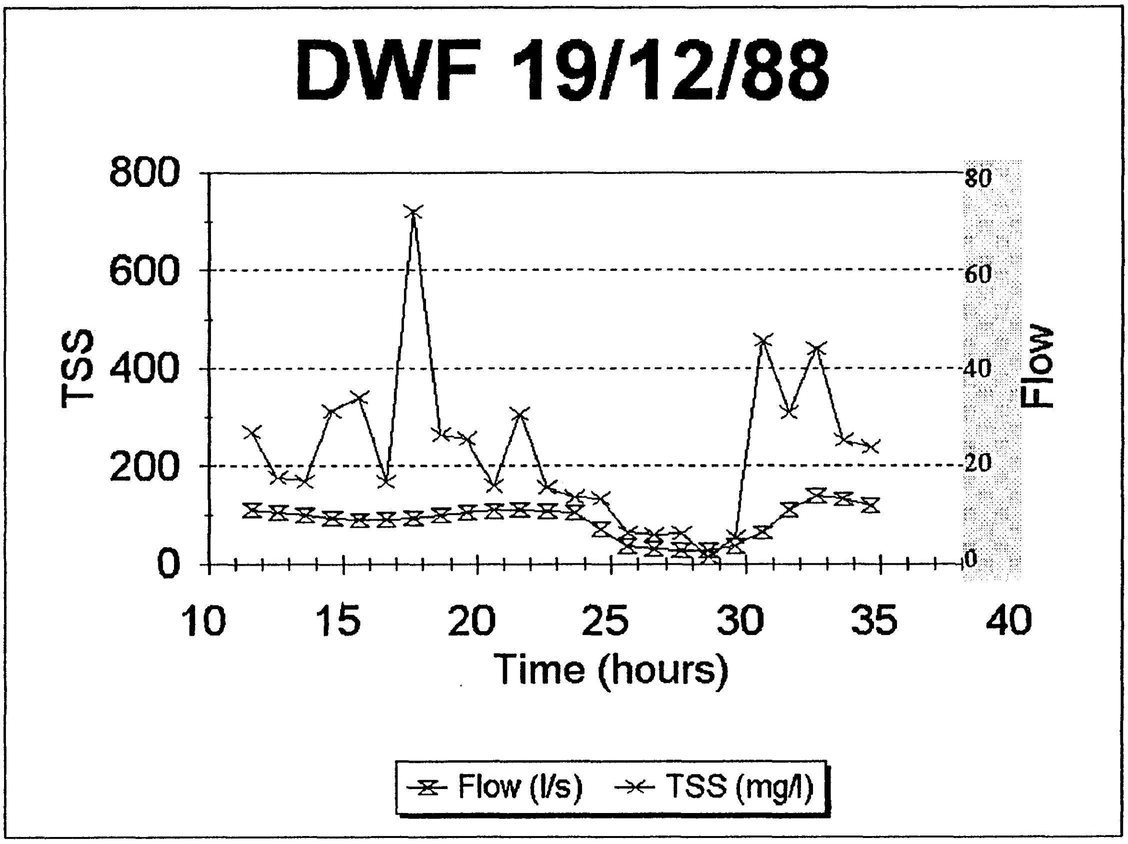


Figure 3

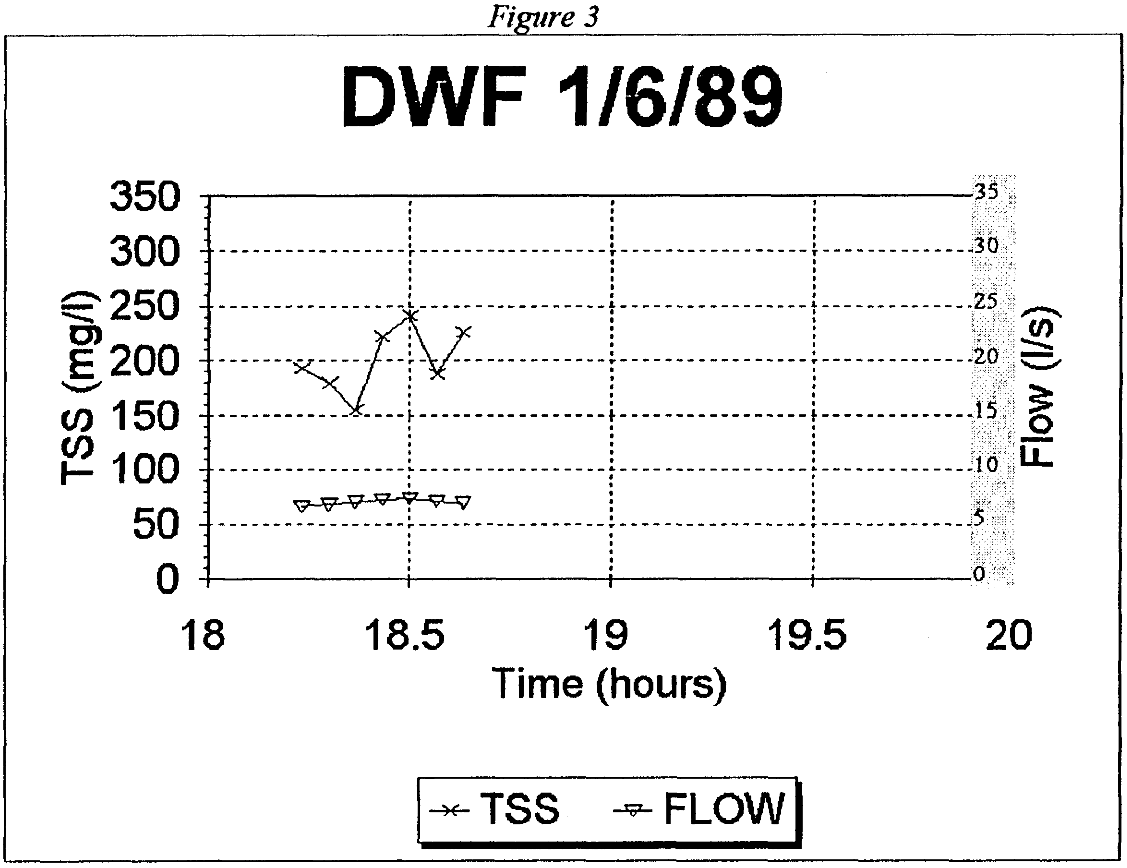


Figure 4

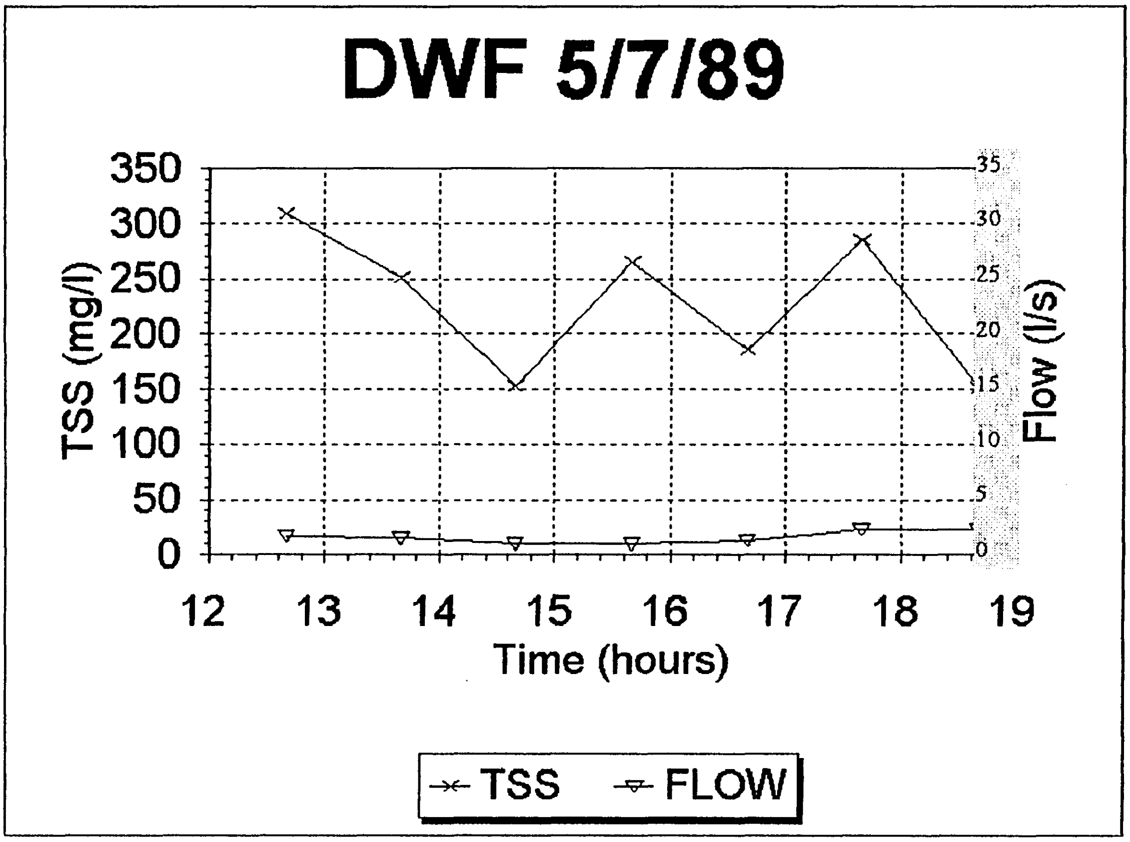


Figure 5

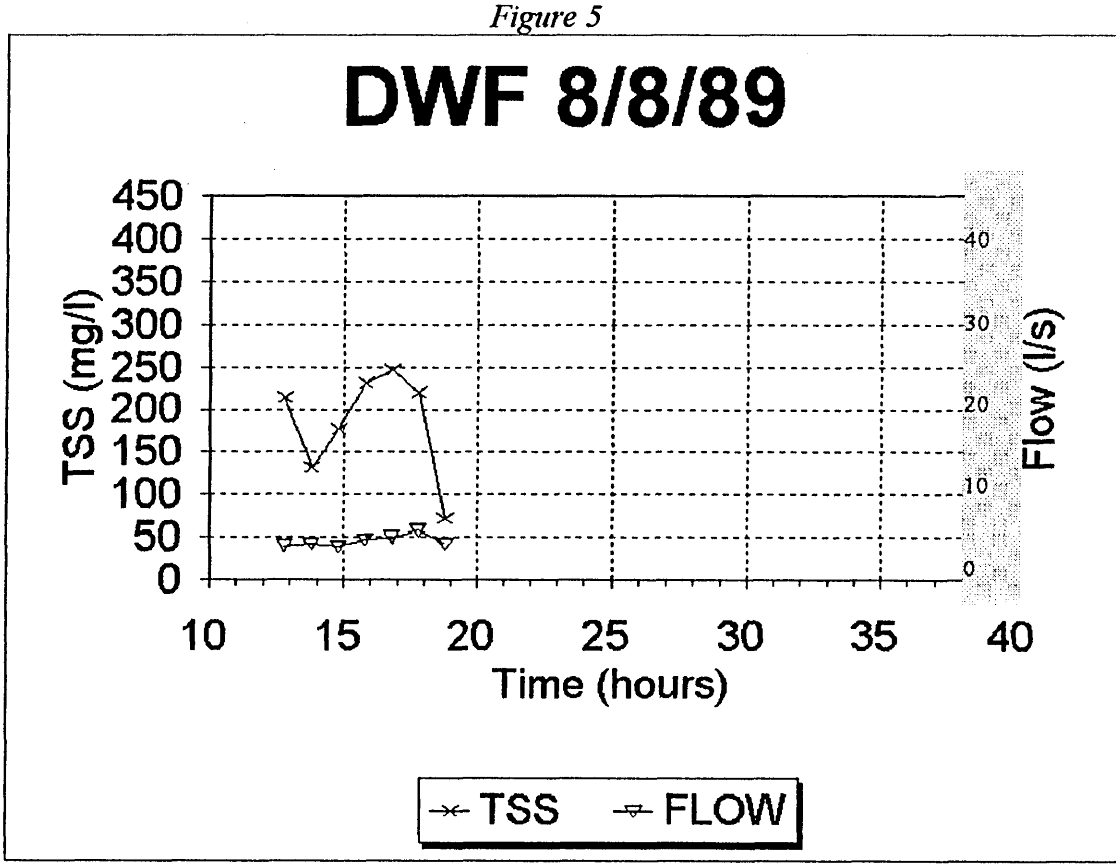


Figure 6

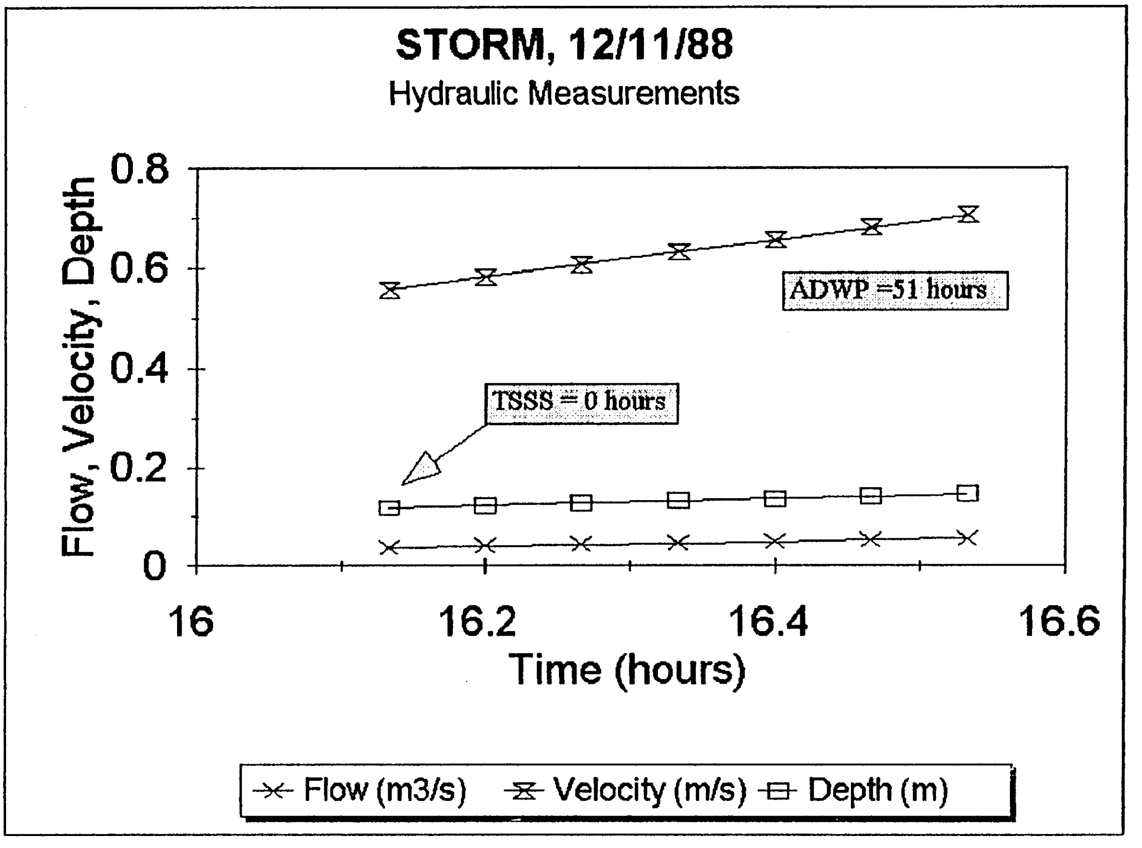


Figure 7

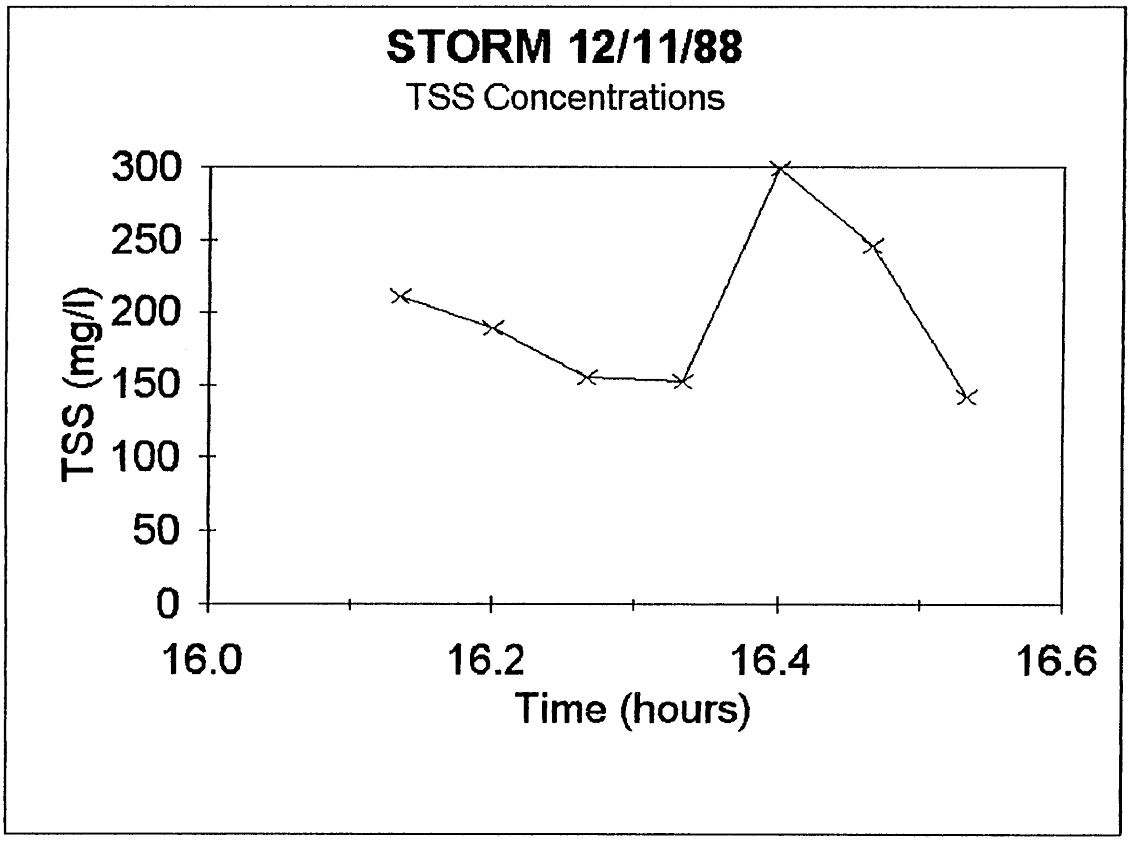


Figure 8

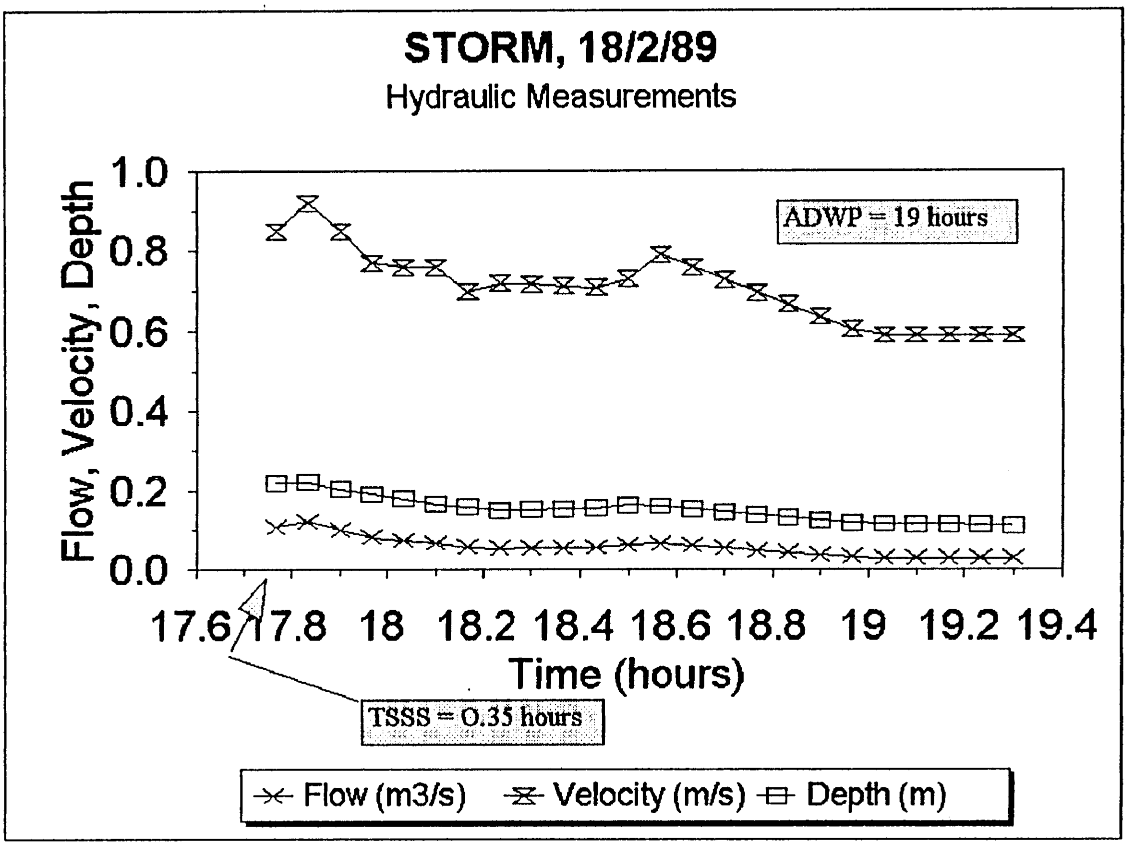


Figure 9

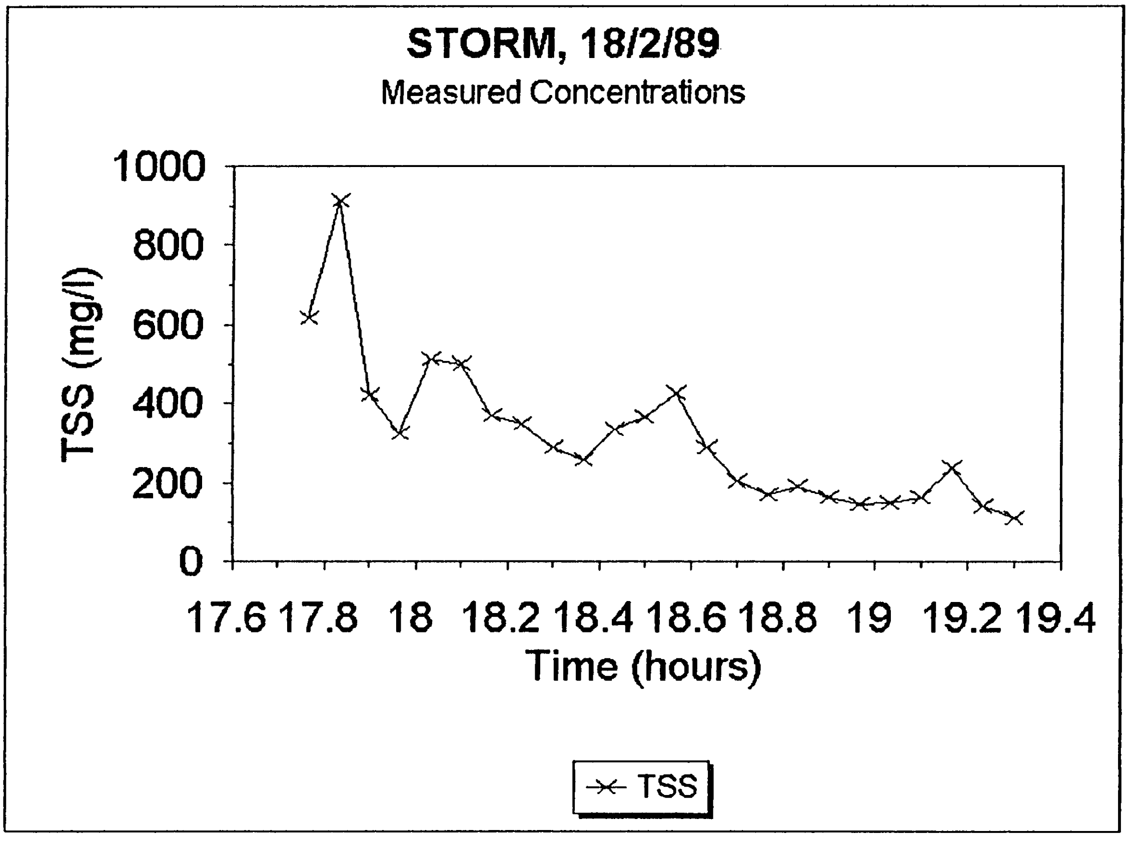


Figure 10

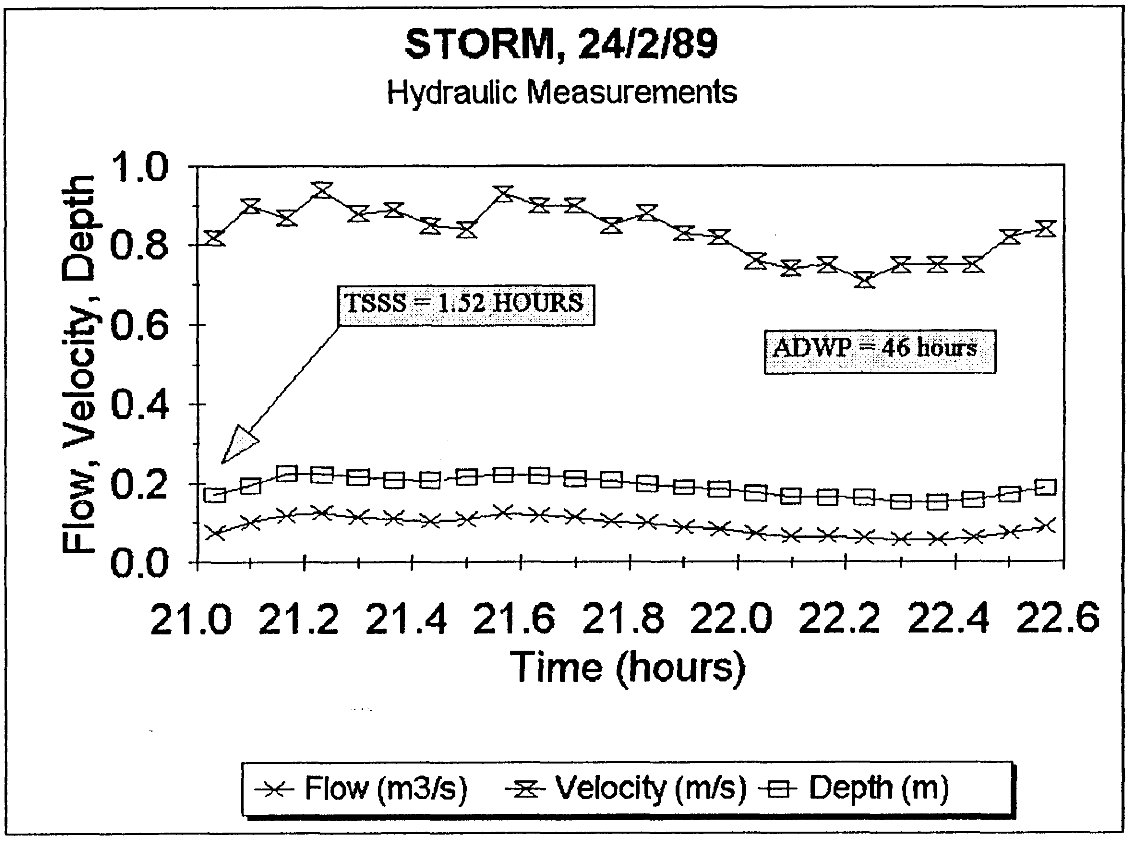


Figure 11

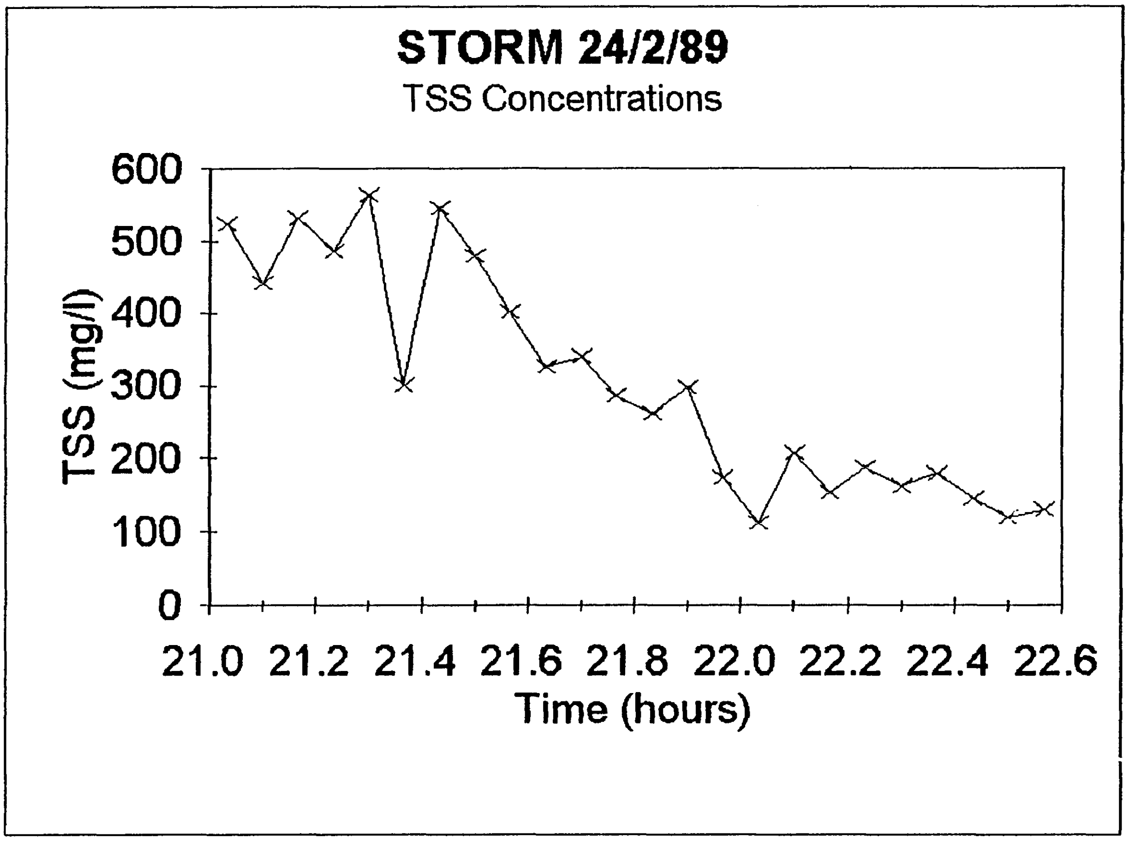


Figure 12

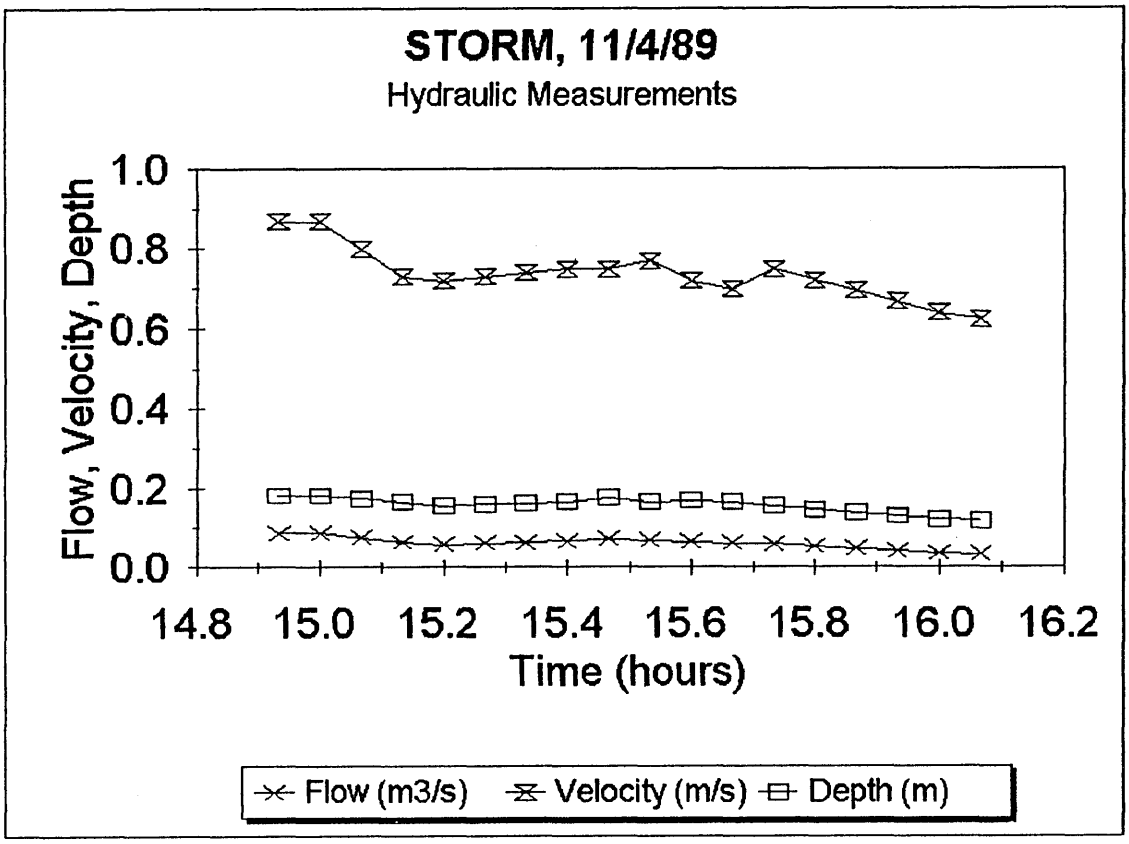


Figure 13

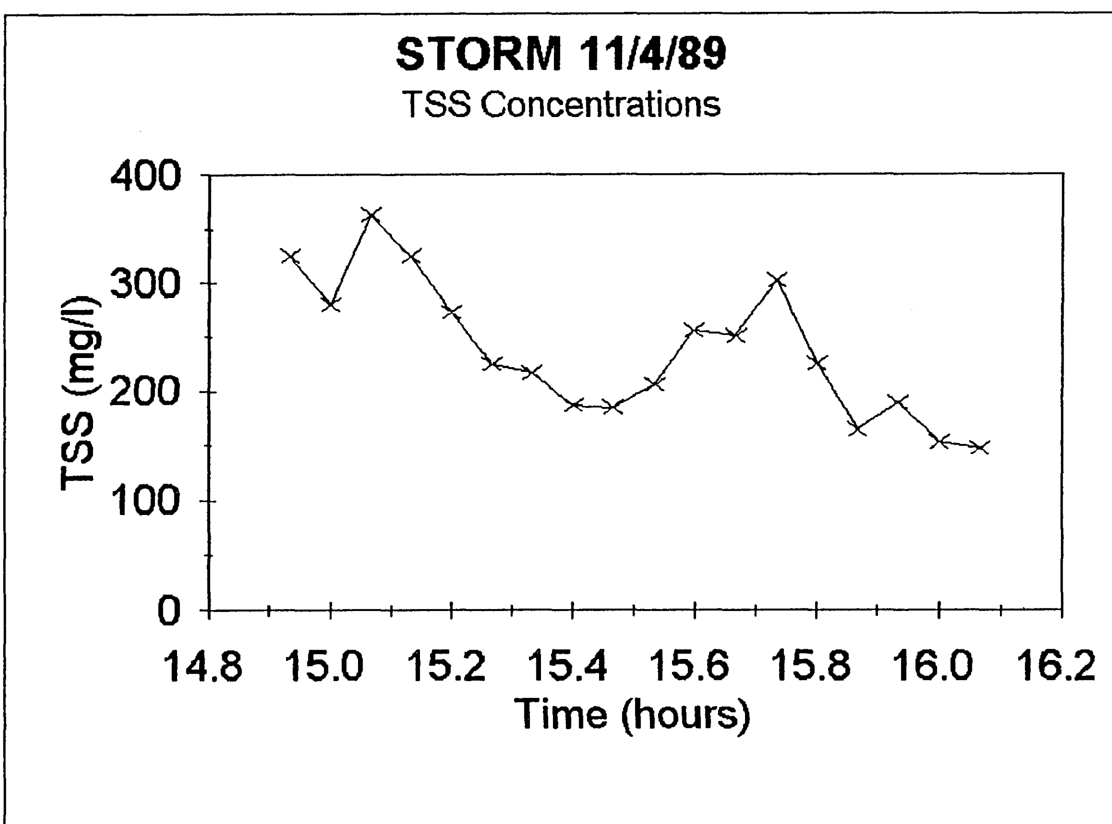


Figure 14

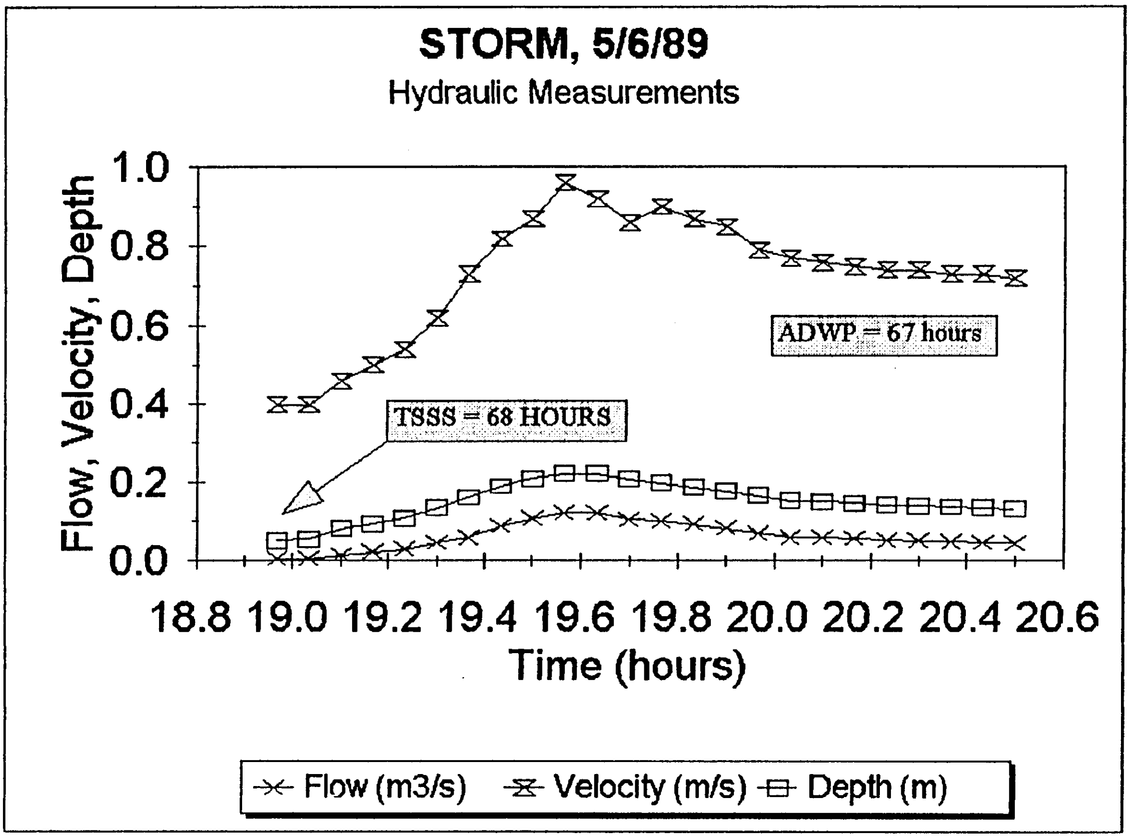


Figure 15

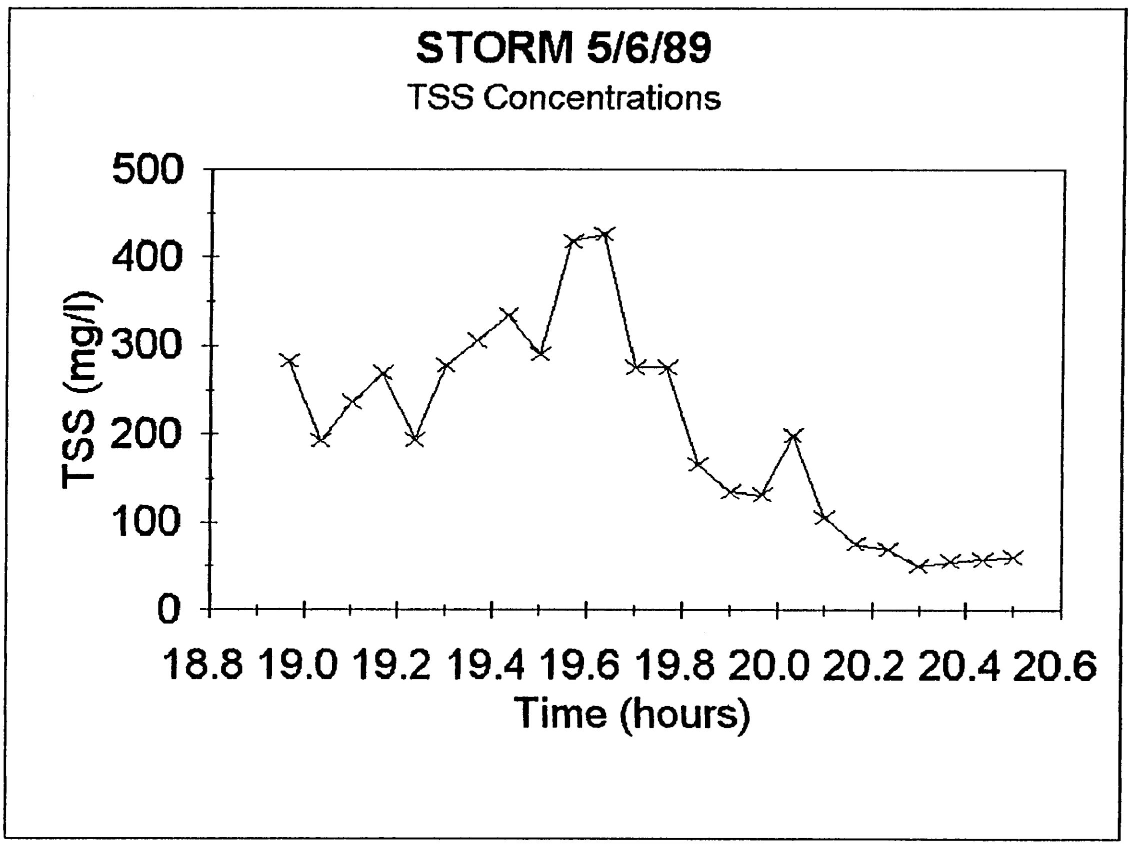


Figure 16

APPENDIX S

TABLES OF REGRESSION OUTPUT, PERTH ROAD TRUNK SEWER

Results of regression analysis of correlations between measured TSS and other recorded parameters.

Dry Weather Flow:

NOTE: Regression of TSS with flow data only, since no other data available for DWF at this site.

Linear correlation:

Variables	**r^2**	**Std Error of Est.**
Q	**0.175527**	**0.17553**

Equation:

$$TSS = 113.74 + 9559.8\,Q \qquad (S1.)$$

% between ½ and 2 = 77.8%

Log/Log correlation:

Variables (Log)	**r^2**	**Std Error of Est.**
Q	**0.456261**	**0.23097**

Equation:

$$TSS = 10^{3.28552} * Q^{0.47552} \qquad (S2.)$$

% between ½ and 2 = 80.6 %

Linear/ln correlation:

Variables (ln)	**r^2**	**Std Error of Est.**
Q	**0.263416**	**77.01171**

Equation:

$$TSS = 417.11 + 44.951 \ln Q \qquad (S3.)$$

% between ½ and 2 = 79.6 %

Hence select **equation (S2.)** for dry weather flow.

Storm Flow:

Linear correlation:

Variables	r^2	Std Error of Est.	
Q	0.364279	98.50644	
V	0.170709	112.5083	
Y	0.302135	103.2088	
TSSS	0.058983	119.8478	
ADWP	0.019476	122.3377	
Q & Y	0.412527	95.36858	
Q, Y & V	0.56453	82.70184	
Q, Y, V & TSSS	0.584805	81.34525	
Q, Y, V, TSSS & ADWP	0.610187	79.40564	← **Best option**

Equation:

$$TSS = 950.14 + 10529.2\ Q - 2344.2\ Y - 1259.2\ V + 0.36205\ TSSS - 2.434\ ADWP \qquad \text{(S4.)}$$

% between ½ and 2 = 94.5 %

Log/Log correlation:

Variables (Log)	r^2	Std Error of Est.	
Q	0.108264	0.227673	
V	0.064566	0.233185	
Y	0.134972	0.224237	
TSSS	0.052829	0.234643	
ADWP	0.041696	0.236018	
Y & Q	0.264801	0.208198	
Y, Q & V	0.338166	0.198963	
Y, Q, V & TSSS	0.352622	0.19822	← **Best option**
Y, Q, V, TSSS & ADWP	0.356624	0.199075	

Equation:

$$TSS = 10^{5.10879} * Y^{8.3114} * Q^{-2.9661} * V^{-2.4135} * TSSS^{-0.0291} \qquad \text{(S5.)}$$

% between ½ and 2 = 90.4 %

Linear/ln correlation:

Variables (ln)	r^2	**Std Error of Est.**	
Q	**0.161427**	**113.1363**	
V	**0.124283**	**115.6147**	
Y	**0.189212**	**111.2461**	
TSSS	**0.019749**	**122.3207**	
ADWP	**0.009534**	**122.9564**	
Y & Q	**0.282873**	**105.3682**	
Y, Q & V	**0.308357**	**104.2262**	← **Best option**
Y, Q, V & TSSS	**0.312555**	**104.6707**	

Equation:

$$TSS = 1506.9 + 1578.6 \ln Y - 548.79 \ln Q - 413.15 \ln V \qquad (S6.)$$

% between ½ and 2 = 86.3 %

Hence best equation for storms is equation (S4.) This equation has 5 variables. Try next most complex version with 4 variables:

$$TSS = 769.1 + 9134.4\ Q - 1661.7\ Y - 1162.5\ V - 0.62466\ TSSS \qquad (S7.)$$

% between ½ and 2 = 95.9 %

Try next most complex version with 3 variables:

$$TSS = 707.55 + 8025.6\ Q - 310.43\ Y - 1290.8\ V \qquad (S8.)$$

% between ½ and 2 = 93.1 %

ie **equation (S7.)** is less complex with highest accuracy of prediction.

APPENDIX T

TABLES OF REGRESSION OUTPUT, GENERAL APPLICATION

Results of regression analysis of correlations between measured TSS and other recorded parameters.

DRY WEATHER FLOW

Linear correlation:

Variables	r^2	Std Error of Est.	
Q	0.001414	83.4	
DAS	0.116645	78.44855	
Q & DAS	0.251881	72.36273	← **Best option**

Equation:

$$TSS = 167.76 + 1432.6\,Q - 0.01801\,DAS \qquad \text{(T1.)}$$

% between ½ and 2 = 79.3 %

Log/Log correlation:

Variables (Log)	r^2	Std Error of Est.	
Q	0.005107	0.330903	
DAS	0.084557	0.317414	
Q & DAS	0.431969	0.250618	← **Best option**

Equation:

$$TSS = 10^{4.393024} * Q^{0.549509} * DAS^{-0.44683} \qquad \text{(T2.)}$$

% between ½ and 2 = 80.6 %

Linear/ln correlation:

Variables (ln)	r^2	Std Error of Est.
Q	0.00033	83.45369
DAS	0.11665	78.44855
Q & DAS	0.36888	66.46389

← **Best option** (Q & DAS)

Equation:

$$TSS = 68.6 + 51.162 \ln Q - 45.366 \ln DAS \qquad (T3.)$$

% between ½ and 2 = 80.2 %

Validation

Using Equation (T2.),

% between ½ and 2 = 73.1 % for all validation data,
% between ½ and 2 = 75.0 % for site 160 data,
% between ½ and 2 = 73.1 % for site 98/99 data.

STORM FLOW

Linear correlation:

Variables	r^2	Std Error of Est.
Q	0.012455	113.1674
V	0.347806	91.9668
Y	0.100896	107.9811
TSSS	0.036072	111.8060
ADWP	0.037770	111.7075
DAS	0.262848	97.7735
V & DAS	0.348431	92.1434

← **Best option** (V)

Equation:

$$TSS = 41.942 + 272.31\, V \qquad (T4.)$$

% between ½ and 2 = 77.7 %

Log/Log correlation:

Variables (Log)	r^2	Std Error of Est.	
Y	0.125283	0.343365	
Q	0.004362	0.366331	
V	0.242864	0.319455	
TSSS	0.106470	0.347038	
ADWP	0.013363	0.364671	
DAS	0.210877	0.326133	
V & DAS	0.256424	0.317342	
V, DAS & Y	0.280194	0.312981	
V, DAS, Y & TSSS	0.335542	0.301464	
V, DAS, Y, TSSS & ADWP	0.387285	0.290167	
V, DAS, Y, TSSS, ADWP & Q	0.398728	0.288148	← **Best option**

Equation:

$$TSS = 10^{2.302515} * V^{1.078575} * DAS^{0.120798} * Y^{-0.27531} * TSSS^{-0.09427} * ADWP^{-0.29764} * Q^{-0.20079} \qquad (T5.)$$

% between ½ and 2 = 78.2 %

Linear/ln correlation:

Variables (ln)	r^2	Std Error of Est.	
Y	0.116936	107.0136	
Q	0.000154	113.8700	
V	0.299310	95.3248	
TSSS	0.100167	108.0249	
ADWP	0.054898	110.7088	
DAS	0.262848	97.7735	
V & DAS	0.317253	94.3223	← **Best option**
V, DAS & Y	0.317398	94.5398	

Equation:

$$TSS = 325.0 + 70.108 \ln V - 12.458 \ln DAS \qquad (T6.)$$

% between ½ and 2 = 76.3 %

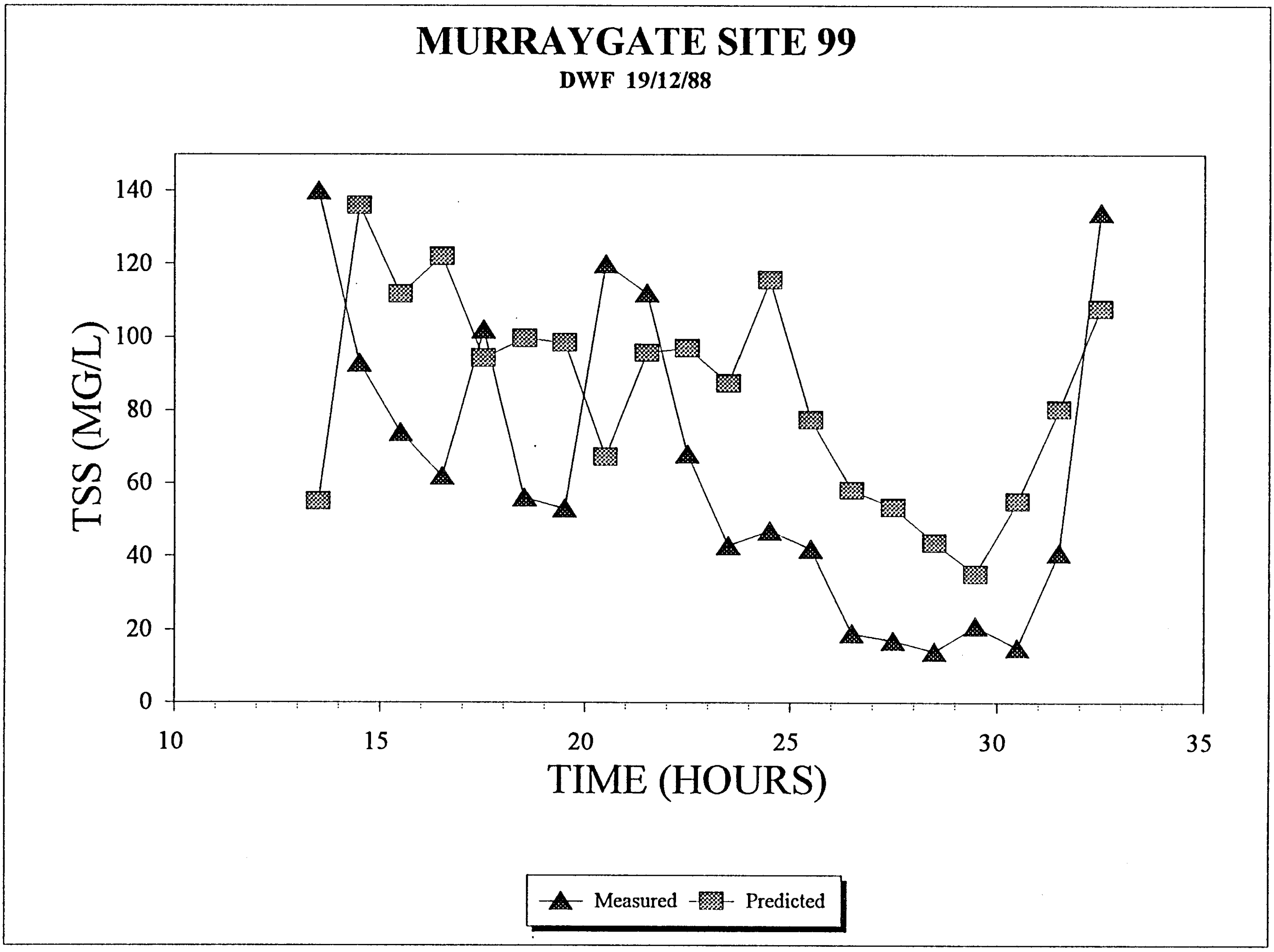
Validation

Using Equation (T4.),

% between ½ and 2 = 76.7 % for all validation data,
% between ½ and 2 = 79.2 % for site 160 data,
% between ½ and 2 = 73.7 % for site 98/99 data.

Appendix U

Graphs of Measured Concentrations versus Concentrations Predicted by the Validated Models



MURRAYGATE SITE 99
DWF 19/12/88
TSS (MG/L)
0
20
40
60
80
100
120
140
10
15
20
25
30
35
TIME (HOURS)
Measured
Predicted

Figure U1

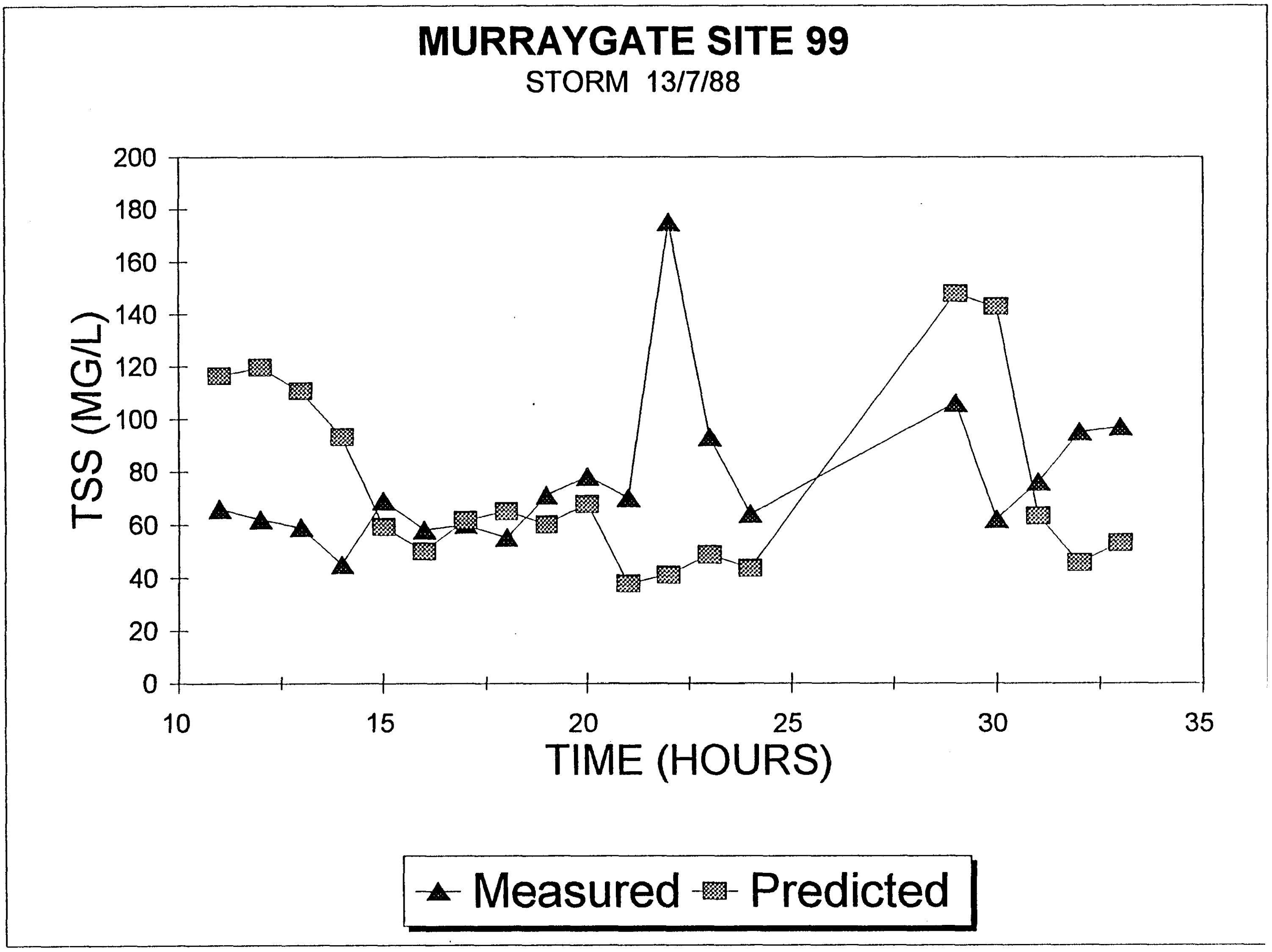

MURRAYGATE SITE 99
STORM 13/7/88
200
180
160
140
120
100
80
60
40
20
0
TSS (MG/L)
10
15
20
25
30
35
TIME (HOURS)
Measured
Predicted

Figure U2

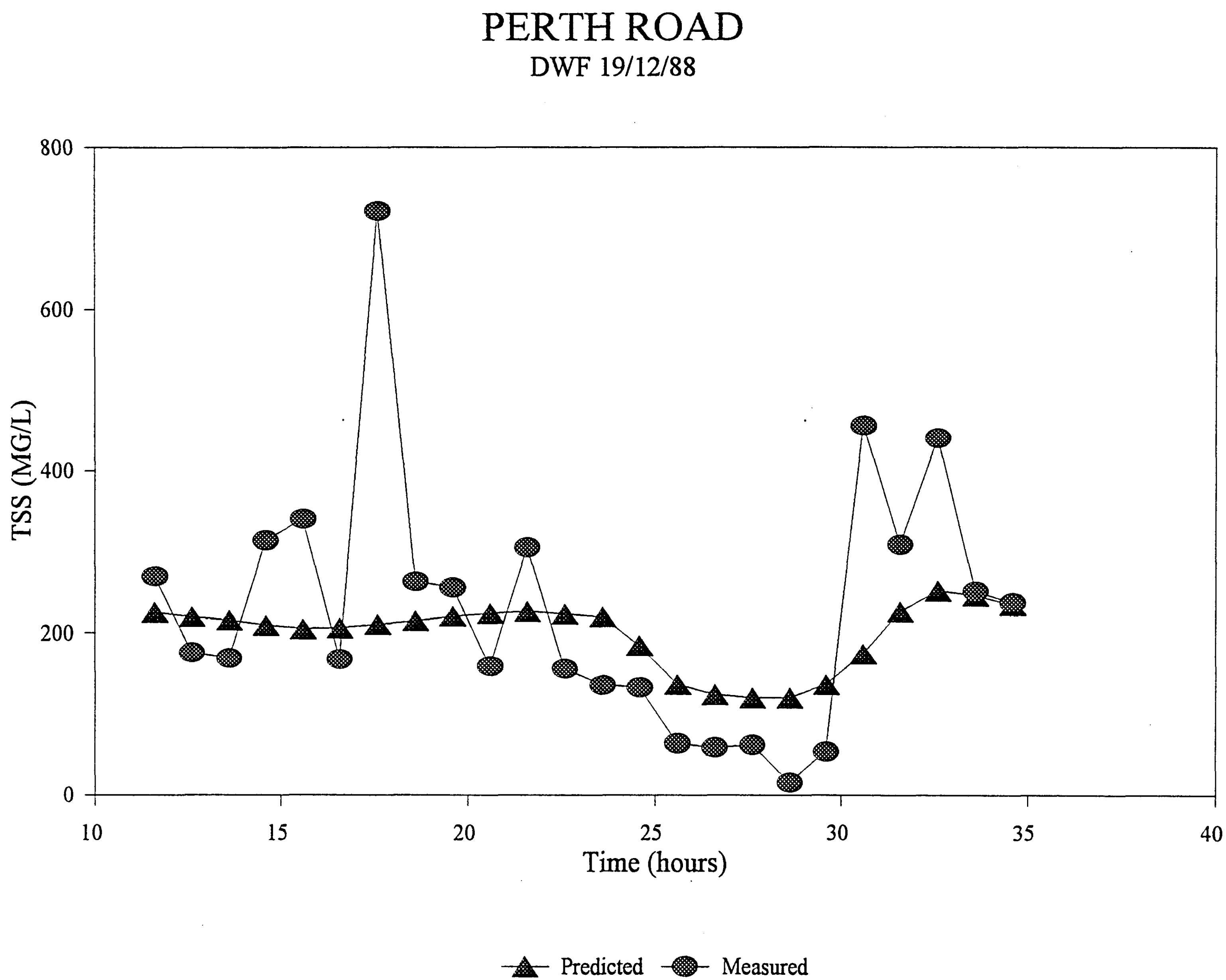

PERTH ROAD
DWF 19/12/88
TSS (MG/L)
800
600
400
200
0
10
15
20
25
30
35
40
Time (hours)
Predicted
Measured

Figure U3

U3

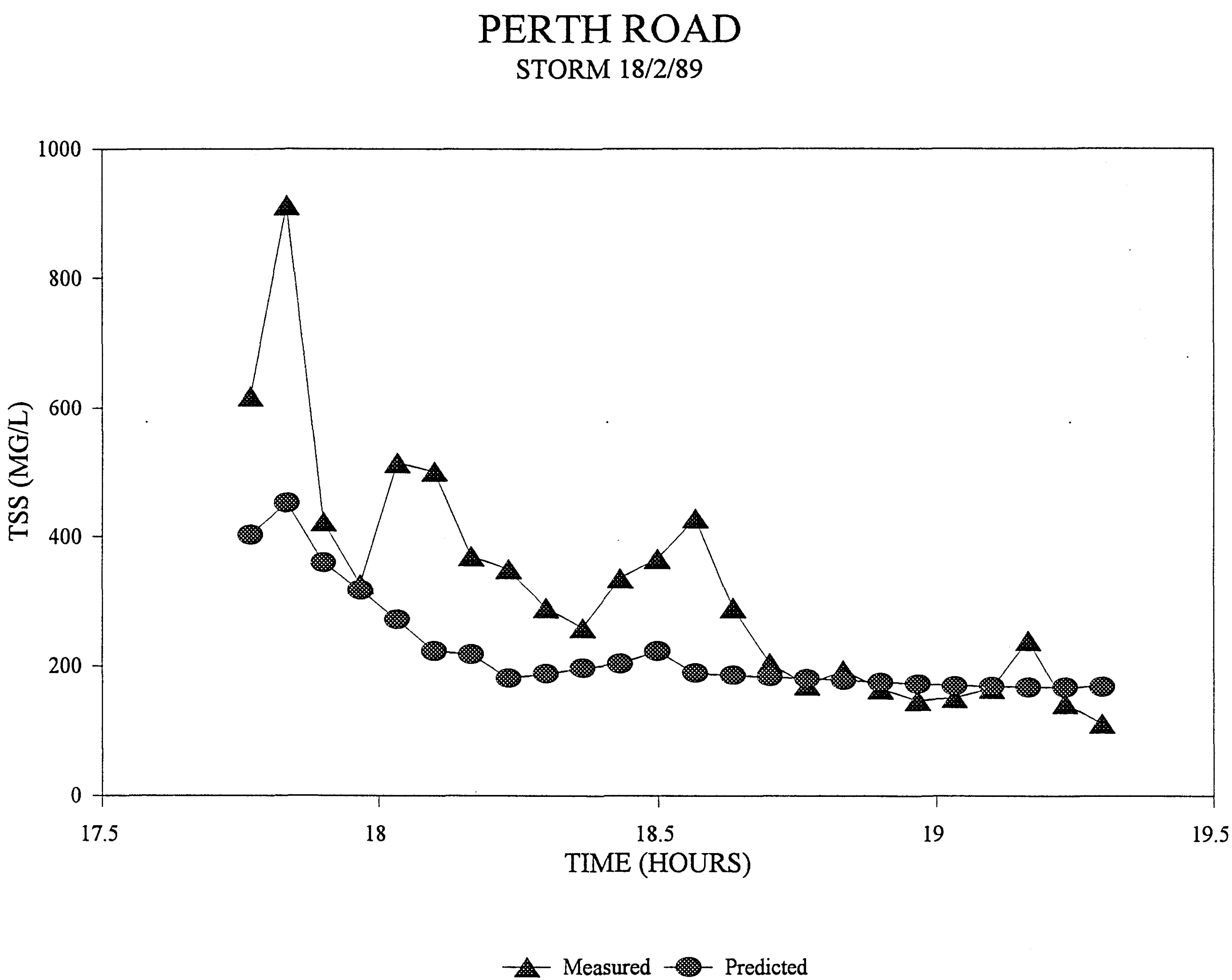

PERTH ROAD
STORM 18/2/89
TSS (MG/L)
1000
800
600
400
200
0
17.5
18
18.5
19
19.5
TIME (HOURS)
Measured
Predicted

Figure U4

COMBINED MODEL (PERTH ROAD)

DWF 19/12/88

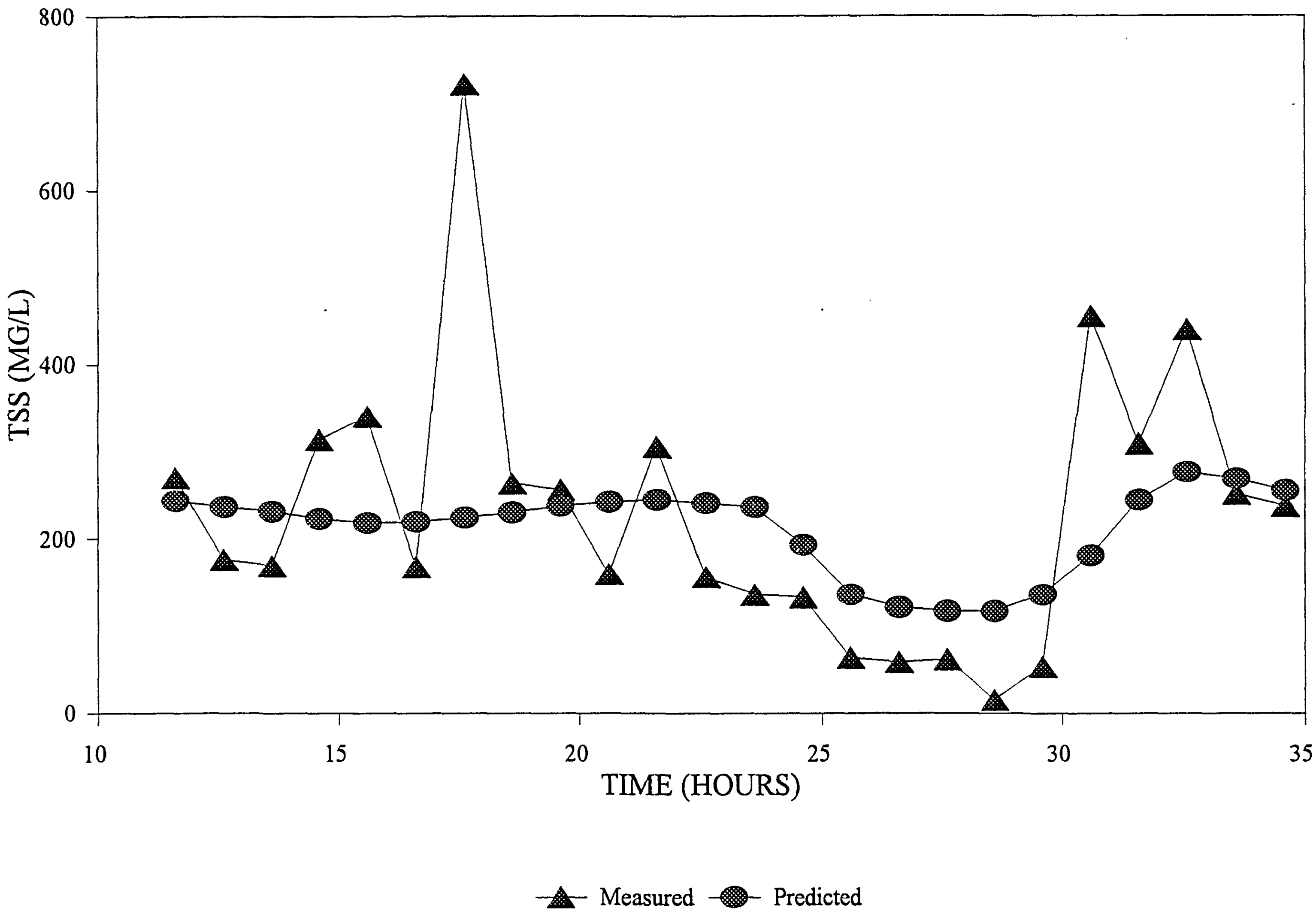
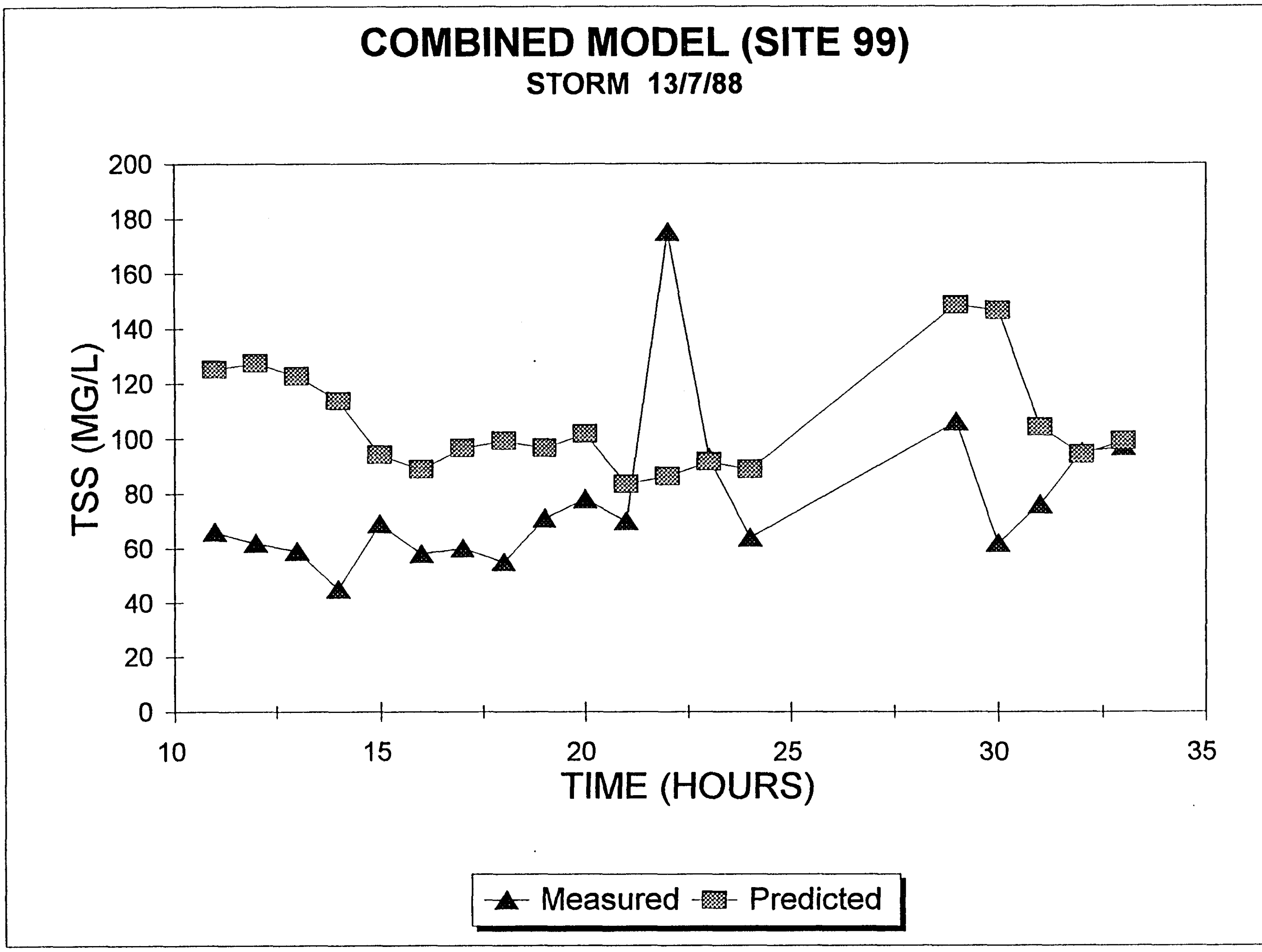


Figure U5

U5



COMBINED MODEL (SITE 99)
STORM 13/7/88
TSS (MG/L)
200
180
160
140
120
100
80
60
40
20
0
10
15
20
25
30
35
TIME (HOURS)
Measured
Predicted

Figure U6

APPENDIX V

Plans of Central Area, Dundee

V1

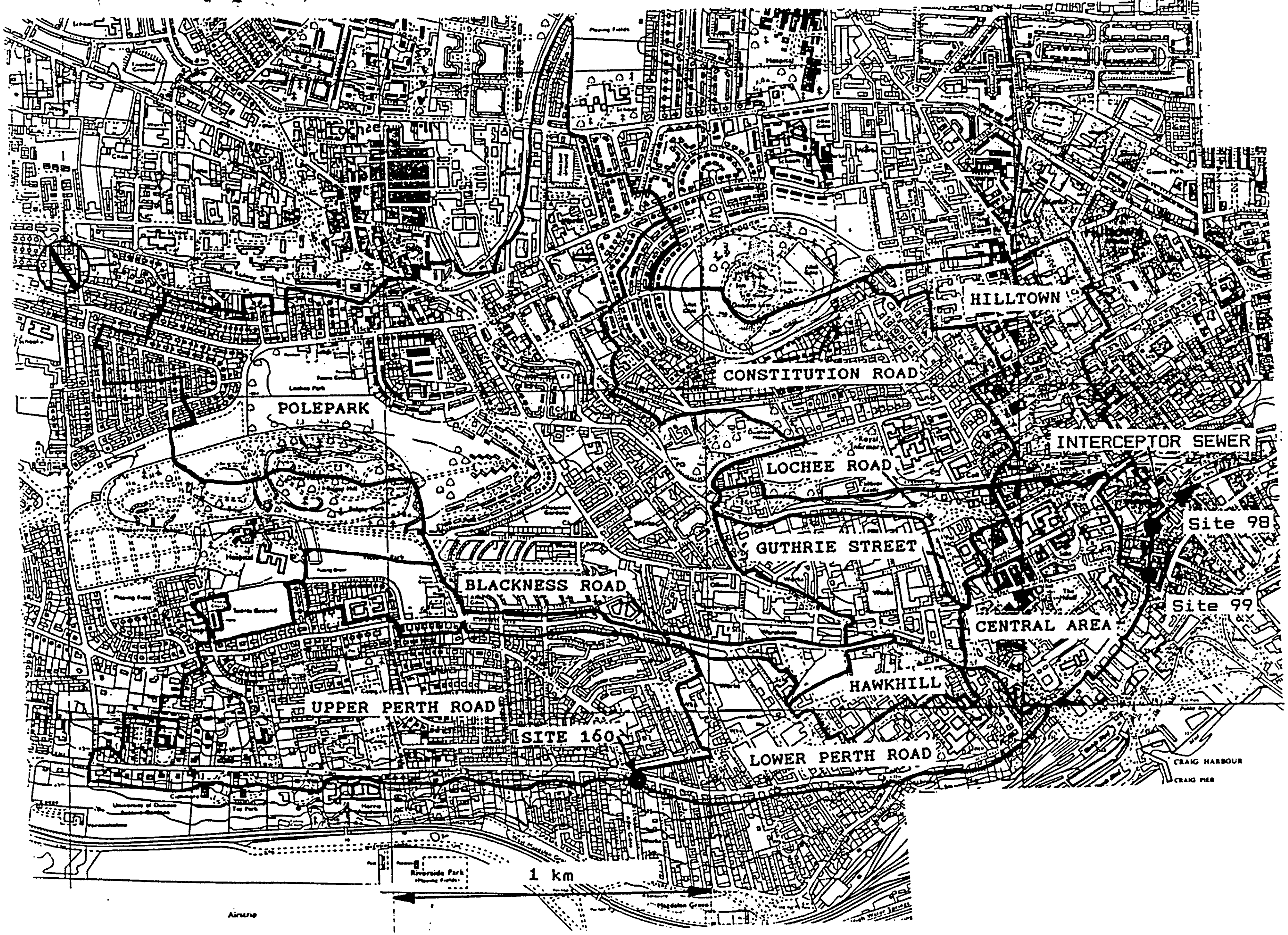


Figure V1 - Plan of Dundee Central Area (City Centre)

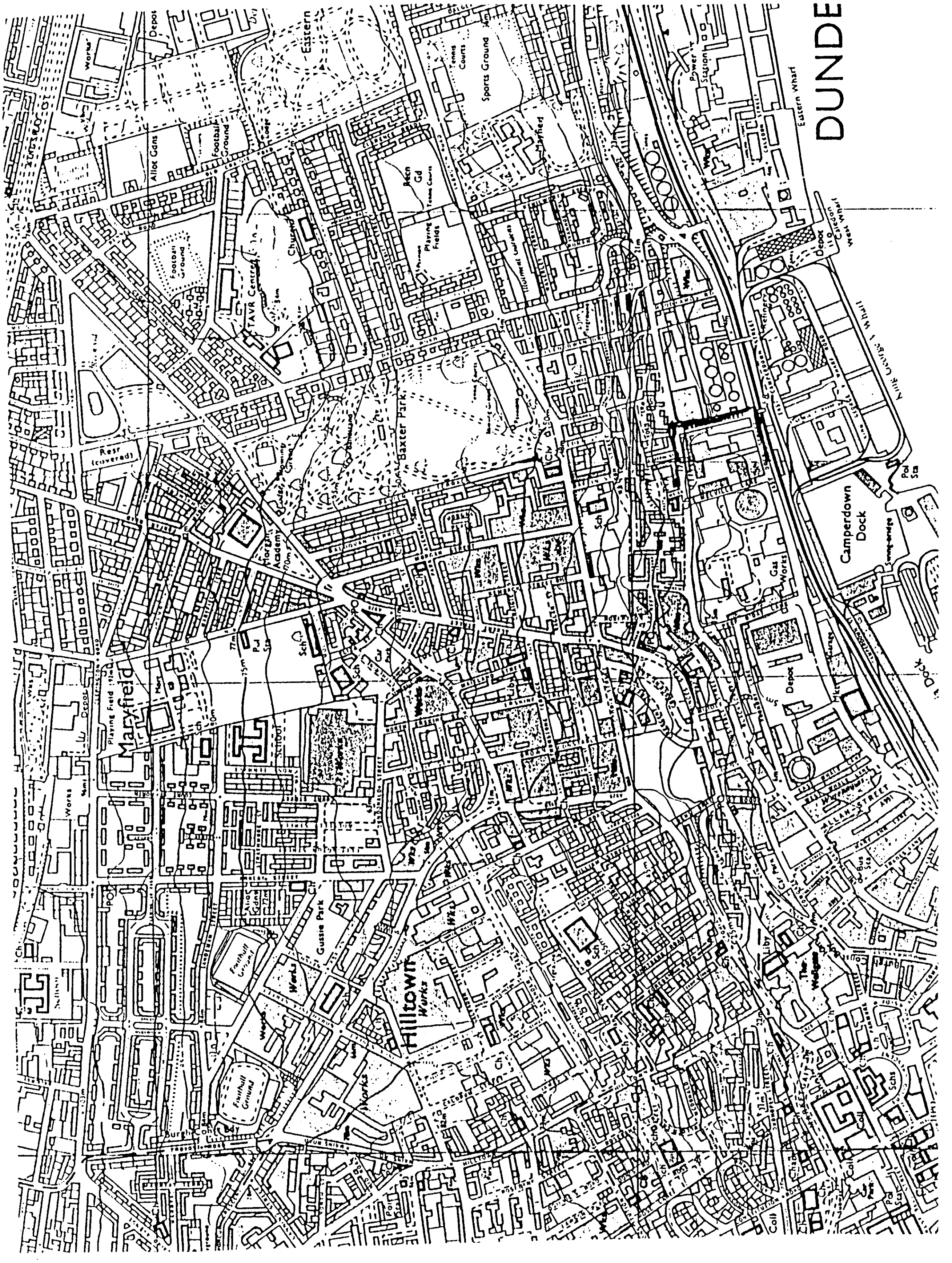


Figure V2 - Plan of Dundee Central Area (West of City Centre)