CHAPTER 15

WATER QUALITY

POLLUTION PARAMETERS

Stormwater runoff from urban catchments in particular contains a surprisingly high pollulant concentration even if no sewage or waste water is discharged into the system. Setting aside combined sewage-stormwater systems, the source of contaminants ranges from material precipitated from the atmosphere (dust or in rain) to seepage from waste tips or industrial zones. There may be discharge of wastes from factories and commericial concerns which, intentionally or not, finds it way to stormwater drains. This type of pollution is referred to as point-source, whereas natural spread inflow is referred to as non point-source pollution. Animal faeces, garden fertilizer, soil erosion, motor vehicles (oils and rubber from tyres) and decaying vegetable matter are some of the known sources. The concentration of sulphates, nitrates and suspended solids in rain is not inconsiderable. Increasing attention is being focussed on the acidity of rain - caused by fumes from traffic and industry. Runoff from rural catchments is also frequently contaminated. Fertilizers and decaying vegetable matter are frequently the source in agricultural areas, and salts leached from the ground may contaminate water in mineral-rich areas,

The rate and amount of pollution of streams due to incoming stormwater can vary widely. The intensity of rain will affect the rate of transport. The 'first flush' is known to bring down most of the pollutants. In fact many pollution models assume an exponential decay rate in the pollution washoff through a storm. The Environmental Protection Agency (EPA)(1971) in the USA indicated a storm depth of 12.5 mm would remove 90% of road surface particles.

Table 15.1 indicates a reasonable range of measured parameters as obtained from various sources. The values cannot be taken as representative for any particular catchment. They merely indicate that pollution does occur and the degree of pollution can vary widely.

The parameter by which pollution is measured is in terms of concentration. Thus dissolved salts e.g. chlorides and sulphates are measured per litre (mg/ℓ) as are suspended solids such as silt. A specific nutrient such as nitrate is measured in terms of the mg/ℓ of nitrogen. The total nitrogen content may comprise organic nitrogen, ammonia nitrogen, nitrite and nitrate. The oxidation of ammonia to nitrite,

TABLE 15.1 Urban runoff quality characteristics

Characteristics		Low	Average	High
BOD	(mg/ℓ)	10	30	500
Suspended Solids	(mg/ℓ)	20	200	10 000
Coliform	(No./100mℓ)	50	10 000	100×10^6
Total chlorides	(mg/ℓ)	10	200	10 000
Total dissolved solids	(mg/ℓ)	300	1 000	10 000
pH	(mg/ℓ)	5.3	7	8.7
Nitrogen	(mg/ℓ)	1	3	100
Phosphate	(mg/ℓ)	0.1	1	50
Phenols	(mg/ℓ)	0		0.2
Oils	(mg/ℓ)	0		110
Lead	(mg/ℓ)	0		2

TABLE 15.2 Accepted limits for selected water quality parameters in streams

Parameter	Designation	Limit	Units	Reason
Dissolved oxygen	DO	5 minimum	mg/ℓ	Aquatic life
Temperature	T	30 max	°C	Life
Free hydrogen	pH	6-9		Acid-Alkali
Coliform	MPN	10000max	No/100mℓ	Disease
Total dissolved solids	TDS	1000max	mg/ℓ	Agriculture
Cloride	Cl	250 max	mg/ℓ	Agriculture
Pesticide	DDT	0.04 max	mg/ℓ	Health
Phenols		0.001 max	mg/ℓ	Taste
Suspended solids	SS	100		Colour
Nitrate as N	N	0.9		Eutrophication

then nitrate, and subsequent biological reduction to free nitrogen is termed the denitrification process and occurs in nature but is also forced at wastewater treatment works. Phosphate is also a nutrient, and in the correct proportions in the presence of nitrate can support life such as aquatic weeds and algae. Formation of algae in warm climates in particular is objectionable, and is termed eutrophication.

Biological matter in waters requires oxidation in order to render it innocuous. This includes decaying vegetable matter, faeces, and some industrial wastes, e.g. from paper factories or abattoirs. The oxygen required is measured in mg/ℓ and termed the biochemical oxygen demand (BOD). It is a slow test to determine BOD and frequently the 5-day or 20-day values are taken as indicators of the ultimate BOD. Due to the difficulty in measuring BOD many researchers prefer to use chemical oxygen demand (COD) or total organic carbon (TOC) as an indicator of oxygen demand.

Free oxygen is measured as dissolved oxygen (DO). Other parameters of interest are the conductivety which is often related to the dissolved salts content, turbidity, (related to suspended solids), colour and temperature. pH is a measure of acidity, with 7.0 being neutral and lower values indicating acidity. Bacteria are measured in terms of the most probably number (MPN) per 100mℓ sample. Faecal coliform and faecal streptacocci are the significant bacteria.

The water quality has an effect on many uses of water. Thus agriculture can accommodate nutrients but not high salt contents. For domestic water supplies, coliform count, colour and taste are important as well as most other parameters. For recreational purposes, similar criteria are often applied. The standards required vary from country to country and Table 15.2 indicates some of the acceptable limits.

OXYGEN BALANCE IN STREAMS

Polluted water, especially if it contains organic pollutants, may require oxygenation to aid purification. The pollution may be measured in terms of the oxygen shortage. The BOD (biochemical oxygen demand) is a commonly used indicator of the oxygen deficit. Oxygen may be absorbed at the water surface or in some cases produced by plant photosynthesis. Dissolved oxygen (DO) in the water will reduce the BOD at a rate dependent on the relative concentration.

A first attempt to describe the relationships between DO, atmospheric reaeration and bacterial respiration was by Streeter and Phelps (1925). They postulated a linear decay rate for BOD as follows:

$$\frac{dB}{dt} = - K_1 B \qquad (15.1)$$

similarly for DO deficit

$$\frac{dD}{dt} = K_1 B - K_2 D \qquad (15.2)$$

where D is the oxygen deficit, C_s-C, C is the saturation DO which is dependent on temperature and other parameters, t is time, B is the BOD, K_1 is a decay constant and K_2 a reaeration coefficient.

The equations maybe refined by introducing terms for diffusion, convection, sources and sinks, (Chevereau, 1973)

$$\frac{\partial B}{\partial t} = E \frac{\partial^2 B}{\partial x^2} - U\frac{\partial B}{\partial x} - K_1 B + S \quad \text{and} \qquad (15.3)$$

$$\frac{\partial D}{\partial t} = E\frac{\partial^2 D}{\partial x^2} - U\frac{\partial D}{\partial x} + K_1 B - K_2 D - P \qquad (15.4)$$

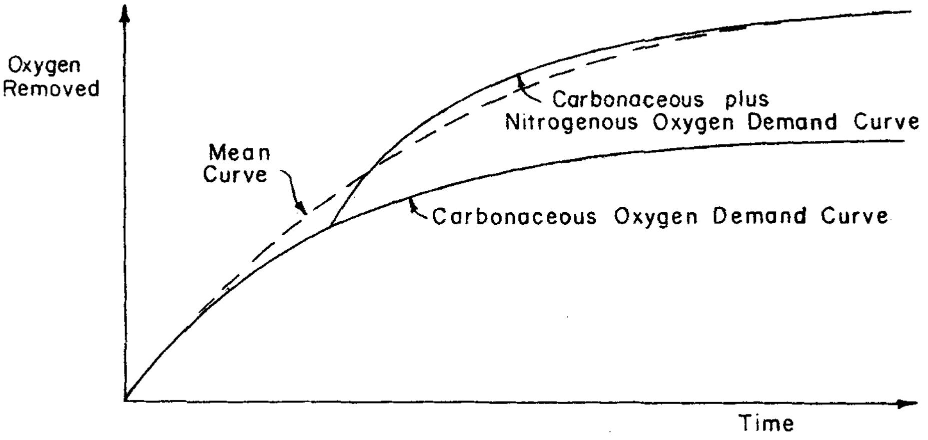

Fig. 15.1 Carbonaceous and nitrogenous oxygen demand curves

where E is the longitudinal dispersion coefficient, U is the flow velocity in the x direction, S is a net source of BOD and P is a net source of oxygen, both per unit volume and time.

The dispersion or diffusion term is primarily due to turbulence and the molecular diffusion is negligible. In real systems the longitudinal term is usually negligible, implying plug flow. The latter two equations are termed the coupled BOD-DO equations. If the river is depleted of oxygen the first equation (15.3) is no longer valid and must be replaced by

$$K_1B = K_3D - P \tag{15.5}$$

which states that the rate of oxygen consumption equals the rate of oxygen introduction.

The reaeration coefficient K_2 is a function of temperature and may be approximated by the formula

$$K_2(T) = K_2(20°C)1.024^{(T-20)} \tag{15.6}$$

where K_2 (20°C) = $3.9U^{0.5}/H^{1.5}$ per day (15.7)

and T is in degrees Celsius, U is the water velocity in metres per second and H is the water depth in metres.

The saturation concentration of dissolved oxygen, C_s, may be obtained from the following empirical relationship for water at 760 mm Hg:

$$C_s=14.652-0.41022T+0.007991T^2-0.00077774T^3 \tag{15.8}$$

The variation in the decay coefficient K_1 with temperature may be obtained from the following relationship (Thomann, 1972).

$$K_1(T)=K_1(20°C)1.047^{(T-20)} \tag{15.9}$$

where K_1 (20°C) must be determined on site. Laboratory values of 0.1 per day are typical whereas values obtained from stream tests have been as high as 20 per day although 1.0 is more representative.

The linear decay assumption is a gross simplification of the process which occurs in a river. There are many reactions, but laboratory BOD tests indicate two predominant oxygen depletion reactions. Initially oxygen is taken up by carbonaceous matter, and oxygen removed increases asymptotically as indicated in Fig. 15.1. At a later stage nitrogenous matter takes a more important part in absorbing oxygen.

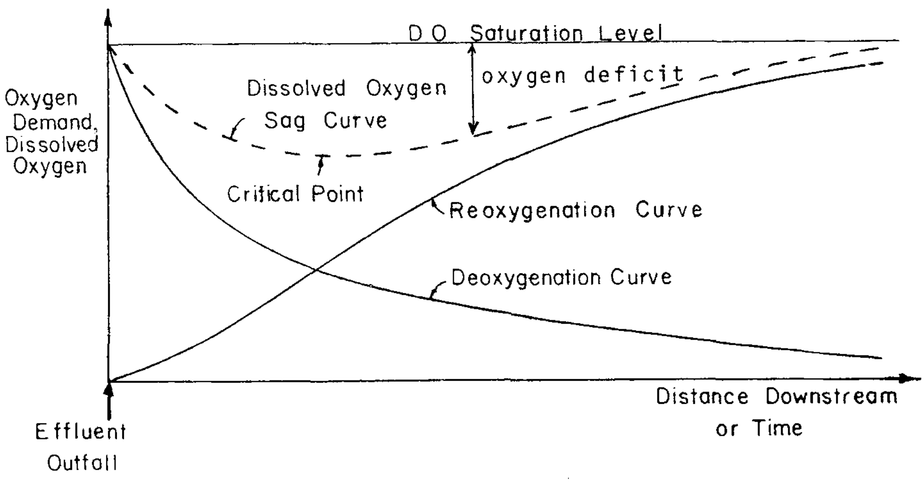

Fig. 15.2 Dissolved oxygen sag curve

The inter-relationships between the oxygen demand and dissolved oxygen in a stream may be indicated graphically as in Fig. 15.2. Assume a BOD is introduced into a stream initially saturated with oxygen. Immediately beyond (downstream of) the point of injection, the dissolved oxygen diminishes. This will cause reoxygenation due to the deficit, so further downstream the DO again increases as the BOD is depleted and the input rate of oxygen again exceeds the depletion rate.

EUTROPHICATION OF RECEIVING WATERS

Algal growth in water bodies is a nuisance from the health and appearance points of view. Algae may be present as a result of high nutrient loading, i.e. nitrogen and phosphorus. Problems are likely to be experienced if the phosphorus concentration is somewhere above about 0.1 mg/ℓ and nitrogen level above 10 mg/ℓ. Residence time, temperature, carbonaceous matter and cell availability also appear to have a bearing on the formation of algae.

Chlorophyll is frequently used as an indicator of eutrophic level. Thus a chlorophyll level of about 100 mg/ℓ is usually eutrophic while a level less than 10 mg/ℓ indicates an oligotrophic level (underenriched) The intermediate stage is referred to as mesotrophic.

Reservoirs are often stratified as indicated in Fig. 15.3. Thermal strata form the epilimnion overlying a hypolimnion. Following a cooling of the upper layers, one may get temperature inversion of the water body resulting in mixing. Wind action can also contribute to the mixing. One would expect oxygen concentration decreasing from the surface to the bed but this may be upset by mixing. In fact Henderson-Sellers (1979) indicates ways of causing mixing in order to improve water quality. The balance between life and inputs to a water system is delicate and complicated as depicted by Roesner (1979) (Fig. 15.4).

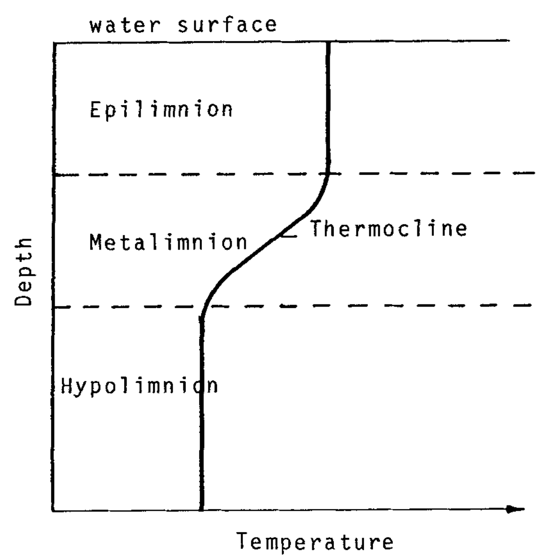

Fig. 15.3 Temperature profile in a reservoir

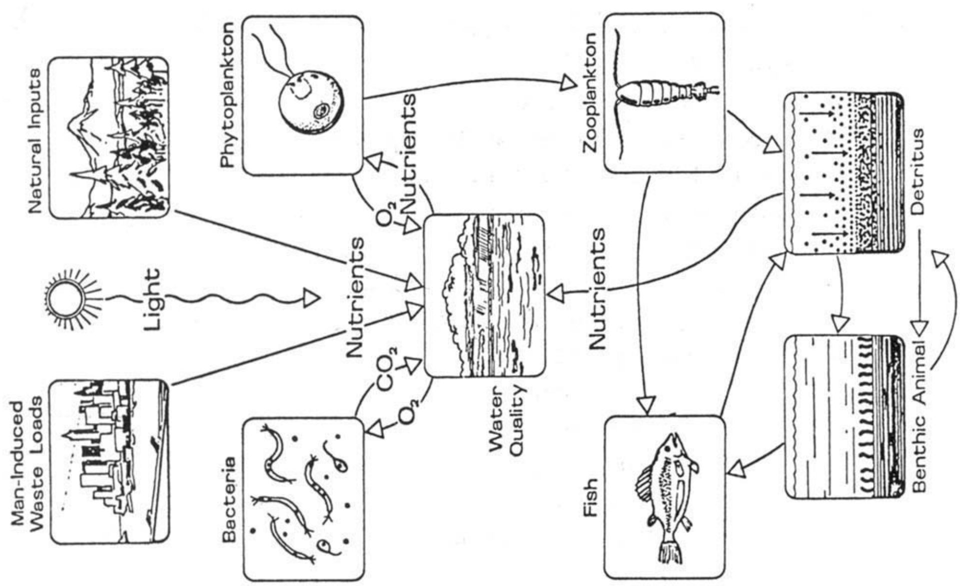

Fig. 15.4 Depiction of a healthy ecosystem (Roesner, 1979)

SEDIMENT EROSION

The erosion of soil in catchments, in the form of sheet erosion or rill and gully erosion is a complex phenomenon. Research into the process is manifesting itself in the form of mathematical models. Some of the relationships thus produced are outlined below.

The amount of sediment detached by rainfall is based on kinetic energy of the rain and may be approximated by an equation of the form

$$D_r = S_{dr} A_s I^2 \tag{15.10}$$

where A_s is the land surface area, I is the rainfall intensity and S_{dr} is a constant dependent on soil type and land surface conditions.

Sediment detachment by overland flow is assumed proportional to the square of the flow velocity which in turn is proportional to the cube root of the slope and flow rate:

$$D_f = S_{df} A_s S^{2/3} Q^{2/3} \tag{15.11}$$

where S is the land surface slope, Q is the flow rate and S_{df} is a constant. To account for imperviousness (or ground cover) the above equations may be multiplied by (1-IMP) where IMP is the proportion of impervious area.

The sediment transport capacity due to rainfall is

$$T_r = S_{tr} SI \tag{15.12}$$

The sediment transport capacity due to overland flow is

$$T_f = S_{tf} S^{5/3} Q^{5/3} \tag{15.13}$$

Thus provided all the constants could be evaluated, the computational procedure is to estimate the erosion rate due to rainfall and overland flow. The transport rate will increase as indicated by (15.10) plus (15.11) until the limit indicated by either (15.12) or (15.13) is reached. (Meyer and Wischmeir, 1969).

The Universal Soil Loss Equation

An empirical equation for the prediction of soil erosion rates was developed at Purdue University in the 1950's (Wischmeir and Smith, 1965) Although it was originally developed for croplands it has been adapted to other erosion loss problems. The equation, termed the Universal Soil Loss Equation (USLE) is

$$A = RKLSCP \tag{15.14}$$

where A = soil loss in tons per acre per time period
R = rainfall factor per unit time period
K = soil erodibility factor
L = slope length factor
S = slope gradient factor
C = cropping management factor
P = erosion control practice factor

The rainfall factor R is a function of the kinetic energy of a storm times its maximum 30-minute intensity, summed over the time period which is usually a year. Values of R were given by Wischmeir and Smith (1965) for the United States. These values range from 20 per year on the west coast and in the arid north west, to 350 in the wetter south east.

The soil erodibility factor K depends on soil size distribution, structure and organic content. It varies from 0.02 for sands to 0.2 for clay and it may be as high as 0.6 for silt. It decreases slightly for high organic content.

The slope length factor L and slope gradient factor S are usually combined to give a topographic factor, which may be estimated from the formula

$$LS = \left(\frac{X}{72.6}\right)^m \frac{(430Z^2 + 30Z + 0.43)}{6.57} \quad (15.15)$$

where X = field slope length in feet

Z = slope in feet per feet

m = 0.5 for slope equal to or greater than 5%

0.4 for slope of 4%

0.3 for slope less than or equal to 3%

It may be necessary to break the catchment into a number of planes to account for varying topographic factors.

The cropping management factor C is also called the cover factor. It varies widely, from 0.01 for urban land or pasture to 0.1 for cropland. It may be as high as 1.0 for fallow land.

The erosion control practice factor P allows for reduction in erosion due to practices such as contouring, terracing and strip cropping.

For land slopes less than 2% it may be as low as 0.3, increasing to 0.5 for slopes over 20%. The factor is halved by terracing but increases for contour strip cropping.

Considerable experience is obviously necessary in applying the USLE. It does, however, hold promise for urban systems subject to further research into the various factors.

SEDIMENT TRANSPORT IN DRAINS

Stormwater drains and sewers are usually designed hydraulically on the basis of clear water flowing through them. The depth of flow, friction gradients and other head losses are estimated without allowance for suspended matter in the water.

It frequently occurs that silt, sand and organic matter is picked up by overland flow and transported into and down drains. Although suspended matter is seldom more than one percent by weight in stormwater drains from built up areas, in rural areas silt concentrations can be as high as 5 percent or more if severe erosion is possible. Particle sizes are typically less than 0.1 mm but in the case of runoff from gravel roads, sand and grit may be considerably larger. A unique problem occurs in some developing countries such as parts of Africa where folk use sand for washing pots.

Fine particles (less than 0.04 mm nominal diameter) normally mix completely with the water and the mixture is effectively homogeneous with a relative density equal to $1+c(S-1)$ where c is the volumetric concentration of suspended matter and S is the relative density (or specific gravity) of the sediment.

In the case of larger particles, there is a tendency to settle out, and the mixture is a heterogeneous one. Energy is expended in maintaining the particles in suspension and friction gradients are larger than for clear water. Conversely in any pipe flowing part-full at a preselected grade, the mixture will flow slower and hence deeper. The capacity of the pipe is thereby reduced. The reduction in velocity may also result in excessive deposition and even blockage.

Transport Mechanics

There are many theories in use for predicting silt loads in channels (see eg. Vanoni, 1975). Most theories for the transport of silt in open channels are based on uniform flow where the silt in suspension is in equilibrium with the bed. The rate of settling out is equal to the rate of re-suspension from the bed due to turbulence. There has been little effort to establish the effect of the suspended load on the energy gradient. In many channels this effect is low on account of the low silt concentrations.

On the other hand, research into the transport of sediment in pipes has been concerned primarily with the effect of the sediment on the energy gradient. Sediment concentrations up to 60 percent by weight are used in pumping systems and at these concentrations there is a considerable energy requirement to maintain the sediment in suspension or drag it along the conduit.

The effect of bed load on the sediment transport process is difficult to assess. Many of the pipe transport formulae are said to apply whether or not there is bed load. It is accepted that coarse particles especially at low velocity will tend to settle, or roll or slide or hop along the bed, rather than remain in suspension. Open channel theories tend to consider the bed load separately from the suspended load.

There is in fact a complicated interaction between channel shape, sediment transport rate, flow velocity, depth and energy gradient. One of the degrees of freedom, namely, channel shape, is absent in pipe flow but there remains still the possibility of settlement which could reduce the cross sectional bore.

If the sediment characteristics are incompatible with the normal flow velocity i.e. the water velocity in the pipe without sediment would be insufficient to maintain the sediment in suspension then the following may occur; coarse particles would settle out initially thereby changing the cross-sectional shape. This has the effect of reducing the hydraulic radius and increasing friction drag so that the flow velocity could reduce even further. The final result may be a blocking of the bore. It is more likely to reduce the cross-sectional area until the velocity increases again for some flow conditions, provided a head buildup at the entrance was possible.

If the suspended sediment added significantly to the energy gradient, the dropping of sediment may permit an increase in velocity so that an equilibrium is reached in time. Shedding of sediment, however, implies a non-equilibrium or a gradual buildup in sediment within the pipe, initially at the top end and subsequently lower down the length of the pipe. Ultimately the entire pipe length will have the same bed load and the system will tend to adjust to a regime with minimum specific energy. That is, the flow depth and consequently the flow velocity adjust until the energy gradient is a minimum consistent with the sediment load.

Alternatively, for a given conduit gradient (which in turn must equal the energy gradient for uniform flow) the flow depth and consequently velocity adjust to cause the friction gradient to equal the bed gradient

Head Loss in Sediment-Laden Pipes

The relationships between the head loss and flow in a pipe conveying sediment have been investigated by many researchers. For particles less than 0.04 mm, the mixture is generally homogeneous and the Darcy equation would apply. Generally large particles (0.04 mm to 0.15 mm) form a heterogeneous mixture and particles larger than 0.15 mm proceed by saltation and in suspension. The equation for heterogeneous flow most accepted is that of Durand and Condolios, which may be written in the form

$$i_{mw}/i_w = 1 + 81c \left\{ \frac{gD(S-1)}{V^2 C_D{}^{1/2}} \right\}^{1.5} \qquad (15.16)$$

where i_{mw} is head loss gradient of the mixture, in metres of water per metre of pipe. i_w is the head loss gradient of water at the same velocity V, c is the volume concentration of sediment in the pipe as a fraction, g is gravitational acceleration, D is the pipe internal diameter, S the sediment relative density or specific gravity and C_D is the effective drag coefficient of the suspended particles. The term on the right hand side is the excess energy gradient relative to water, in metres of water per metre of pipe, due to the particles.

For particles of varying size, the expression may be replaced by

$$i_{mw}/i_w = 1 + 81c\, \{\frac{gD(S-1)}{V^2}\}^{1.5} \sum \frac{P_i}{C_{Di}^{\;0.75}} \tag{15.17}$$

The equation is really only applicable to particles of limited size such that they travel at about the same velocity as the water. Very coarse particles (greater than about 25 mm) obey a relationship of a different form.

Although the equation was not originally intended for use in other than full circular pipes, according to Bain and Bonnington (1970) it may be rewritten in a form applicable to part-full pipes by replacing D by 4R where R is the hydraulic radius, Q is the discharge rate and A is the cross sectional area of the flow. If one substitutes the term C_f for $81c\{gD(S-1)A_f^{\,2}/Q^2C_D^{\,1/2}\}^{1.5}$ the equation may be written in dimensionless form as follows:

$$i_{mw}/i_w = + C_fY^{1.5} \tag{15.18}$$

where Y is $RA^2/R_fA_f^{\,2}$ and is a function of the relative depth of flow y/D only, and R_f and A_f are the hydraulic radius and area of the full pipe i.e. D/4 and $\pi D^2/4$ respectively.

One thus has an expression relating the hydraulic gradient of the solids/water mixture to that in water. The equation applies for any given depth of flow. In view of the difficulty in solving for Y as a function of y/D, the relationship is plotted in Fig. 15.5. Thus for any specified depth of flow and sediment function C_f one may establish the energy gradient relative to that of clear water. Thus for a sediment concentration of one percent, a C_D of 1.0 and a pipe diameter of 500 mm it will be found that at 50% full flow the head loss increases 60 percent.

Invariably the pipe gradient cannot be altered to suit the sediment load, unless at design stage. What would happen in practice is that the depth would adjust to accommodate the additional energy requirement to transport the sediment. Thus if one writes the energy equation as

$$i_{mw}/i_{wf} = i_w/i_{wf}\ (1+C_fY^{1.5}) \tag{15.19}$$

where i_{wf} is the head loss gradient at flow Q of water in the pipe running full, then Fig. 15.6 may be plotted. Now if the Darcy equation

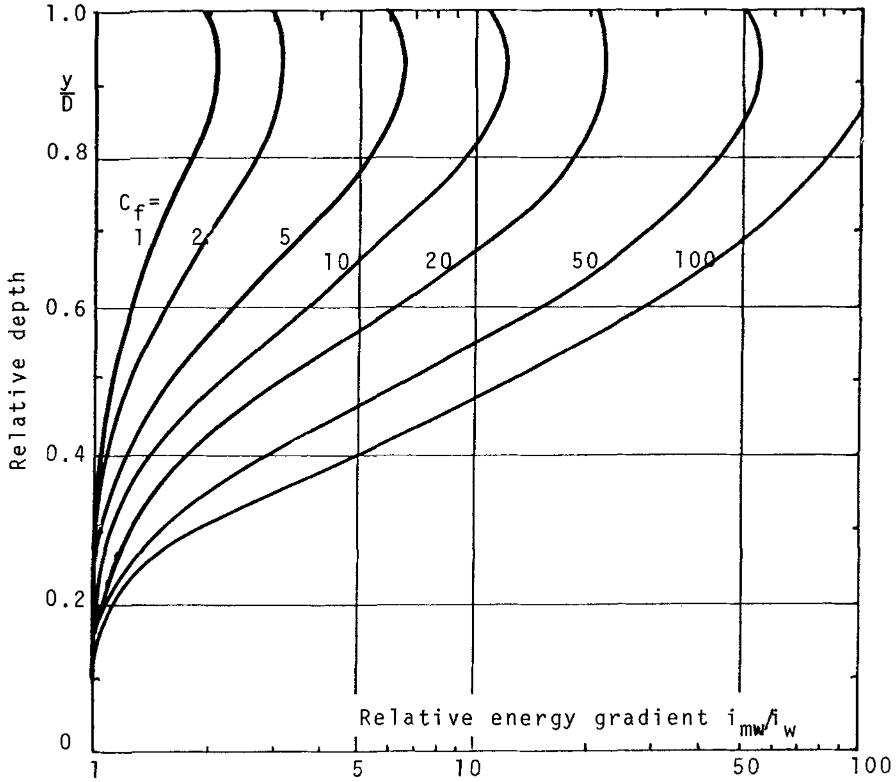

Fig. 15.5 Energy gradient for part full pipes transporting sediment

is used for friction losses

then $\quad i_w/i_{wf} = A_f{}^2R_f/A^2R = 1/Y \qquad (15.20)$

since $\quad i_{wf} = \lambda Q^2/2gDA_f{}^2 \qquad (15.21)$

Fig. 15.6 may thus be plotted to yield the depth of flow at different concentration functions for any given gradient. It should be noted that i_w and i_{mw} are the gradients in metres of water per metre length. For high concentrations the gradient should be corrected for the relative density of the silt laden mixture. Thus the relative density of mixture is $1 + c\ (S - 1)$

Hence the true gradient in metres of mixture per metre of pipe is

$i_m=i_{mw}/\{1+c(S-1)\} \qquad (15.22)$

For a gradient of $i = 0.0185$, $i_{mw} = 1+.05\ (2.6-1) = 0.02$, $Q=0.3\ m^3/s$, $\lambda = 0.012$, $D = 0.5m$, then $i_{wf} = 0.00286$ and $i_{mw}/i_{wf} = 7.0$. From Fig. 15.6 the depth of flow for clear water ($C_f=0$) is $0.42x0.5=0.21m$ and for c=0.05, S=2.6 and C_D=10.0 then C_f=4.43 and y=0.48x0.5=0.24m, i.e. the depth of flow increases. If C was much higher however, it may be found that at no depth would the pipe transport the load, so that blockage is possible.

Self Cleansing Velocities

It is commonly accepted that the minimum velocity in sanitary sewers to avoid settling is 0.6 metres per second (Yao, 1974). This criterion applies to the full-flow condition. It takes no account of sediment characteristics and is probably applicable to biological matter. During some periods the sewers may not run at capacity and the flow velocity will be less than that at full capacity. Under these conditions in the case of sanitary sewers it was recommended that the minimum velocity be attained at least once a day.

The criteria for storm sewers are different. There is normally little problem of putrefaction of deposits which is fortunate, as self cleansing velocities may not occur for months on end. The sediment transported by storm sewers is likely to be silt, sand, and even refuse such as bottles or cans.

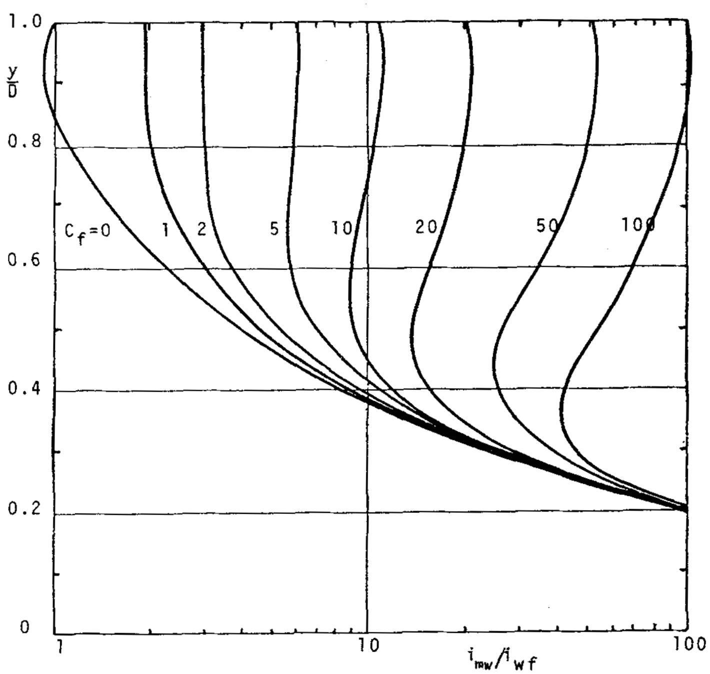

Fig. 15.6 Depth of flow in part-full pipes transporting sediment

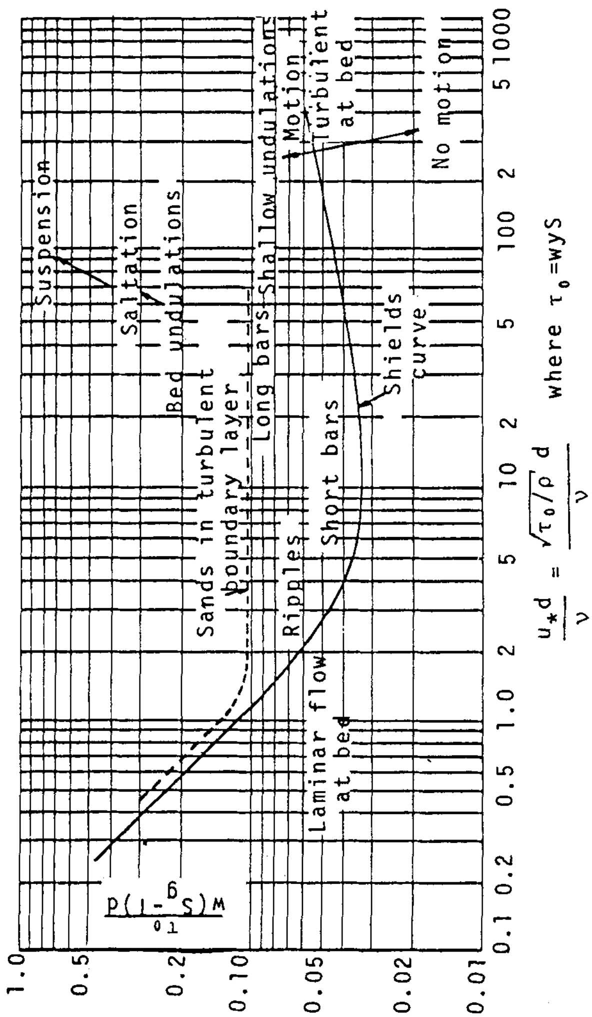

Fig. 15.7 Shields' diagram for scour of sediment

A more rational theory than the minimum velocity concept is the tractive shear stress approach. The theory was developed by Shields who obtained his results from tests in a horizontal flume. He showed that the limiting bed shear stress for incipient motion of non-cohesive particles was given by an equation of the form

$$\frac{\tau}{w(S-1)d} = F(R_d) \qquad (15.23)$$

where τ is the bed shear stress, w is the unit weight of water, S is the relative density of silt and d is the particle size. R_d is a Reynolds number in terms of particle size

$$R_d = U_* d/\nu \qquad (15.24)$$

and U_* is the shear velocity $\sqrt{(\tau/\rho)}$ (15.25)

The numerical value of F is 0.06 for coarse particles increasing for smaller particles (Fig. 15.7).

Employing the Darcy friction equation and solving for water velocity V, for a full pipe

$$v = \sqrt{8g\ d\ (S-1)F/\lambda} \qquad (15.26)$$

For part-full pipes the relationship between τ and friction factor λ is more complicated. The fact that the roughness of the deposits is different to that of the rest of the wetted perimeter further complicates the solution for self cleansing velocity V. Camp (1946) indicated self-cleansing velocities assuming the equation for permissible velocity as above and varying friction factor λ with relative flow depth.

The Equ. 15.26 is similar in form to the critical deposit velocity found in pipeline transport. For full pipes this is

$$V = F_c\ \sqrt{2gD(S-1)} \qquad (15.27)$$

where F_c lies between 0.8 and 1.0 for concentrations between 2% and 15% by volume and particle size d greater than about 0.2 mm.

PLANNING AND MANAGEMENT

Our water resources of the future may be limited more by pollution than by drought. Integrated water resource management such as initiated in the United Kingdom will become necessary. This means the control of surface water and ground water, wastewater treatment and possibly stormwater treatment. Methods of treating stormwater will be similar in principle to those for concentrated sewage and industrial wastes but will vary greatly in scale and effort. Nevertheless, locations of collection works, treatment facilities and discharge points will require integrated planning (e.g. Stephenson, 1978; Rinaldi et al 1978).

Management of catchments and drainage systems will require the attention of sanitary engineers as well as hydrologists. Preservation of resources is closely related to optimal management and research on these lines in the USA is now bearing fruit (e.g. Wanielista, 1978).

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