WATER QUALITY AND ITS CONTROL

Edited by:

MIKIO HINO

Department of Civil Engineering, Tokyo Institute of Technology, O-okayama, Meguro-ku, Tokyo, Japan
Presently: Faculty of Political Science, Chuo University, Hachiohji, Tokyo, Japan

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Improvement of water quality in rivers by aeration at hydraulic structures

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P. NOVAK
University of Newcastle upon Tyne, UK (retired)

6.1 INTRODUCTION

The potential contribution of hydraulic structures through air entrainment to the improvement of water quality in rivers has long been recognised, but methods for predicting the oxygen uptake have evolved only rather recently. Even though some results of measurements from the field or laboratory were available, predictive equations that would account for at least the most important parameters and some scale effects were difficult to develop.

This text attempts to summarise the present position in methods for estimating oxygen uptake in the case of simple and free overfalls at weirs, multiple jets weirs and cascades, for outflows under gates with hydraulic jumps and for bottom outlets and turbines. Although a brief introduction to the theory of oxygen transfer is given, the main emphasis is on development of methods suitable for hydraulic engineering design; no full analysis of the physics of the process is attempted. However, even if this process is sometimes – because of the complex nature of the phenomena involved – treated almost like a 'black box', an analysis of the hydraulic parameters permits one to formulate a predictive model that should give the designer and operator of the structure some idea of the oxygen transfer likely to occur.

Although air entrainment is nearly always beneficial to water quality, not all environmental effects are positive. Oxygen supersaturation at high-head hydraulic structures can cause fish to die, and design measures may have to be taken to alleviate this danger. A brief note on supersaturation and some of the preventive measures is, therefore, included in this text.

6.2 GENERAL

Air-water flows have three distinct regions: air entrainment, detrainment and air transport by flowing water. This text is concerned only with the first two regions, and with only those cases of local aeration with free surface flows. In these cases local air entrainment is always connected with some form of surface discontinuity and resultant shear planes in the flow with air entrainment in distinct vortices.

The parameters that affect local air entrainment are the geometry, turbulence of the upstream flow and the Froude, Reynolds and Weber numbers of the flow. Local air entrainment may be governed by certain limiting conditions, i.e., the inception limit (requirement of minimum velocity), entraining limit (minimum approach Froude number), air supply limit (e.g., in aeration devices) and transport limit governed by downstream conditions; in this treatment again only the first two conditions are important, and in the practical cases dealt with here they are always satisfied. The order of magnitude for the inception limit can be characterised by the velocity v_c

$$\frac{v_c^3 \rho}{g\mu} \cong (0.5 \text{ to } 1) \times 10^5 \qquad (6.1)$$

giving for water jets $v_c \cong 0.8$ to 1 m/s (Ervine et al., 1980).

For hydraulic jumps air entrainment commences at upstream Froude numbers exceeding about 1.7. The relative air entrainment $\beta = Q\text{air}/Q\text{water}$ for a given boundary condition, fully turbulent flow and given fluid properties, is independent of Reynolds number and thus can be shown to be directly proportional to Fr^2 (Kobus, 1985): generally

$$\beta = f(\mathrm{Fr}) \qquad (6.2)$$

The physics of air bubble formation and transport have been dealt with by Kobus (1985).

Because the flow geometry and model scales affect the air entrainment process, no generally applicable design criteria can be formulated; thus each major hydraulic configuration has to be dealt with separately. As many of the data available were obtained in the laboratory, the range of scales from which they were obtained is important, and it can be exceeded in application only with due consideration of the scale effects likely to be involved.

In determining the transfer of oxygen in flows with local aeration, one must consider not only the mechanics of air entrainment but also the mass transfer between air and water, i.e., the transfer of oxygen from the air bubbles formed downstream of local entrainment into the solution. The parameters affecting this transfer are mainly the oxygen concentration difference between air and water, contact area, contact time, salinity, temperature, water quality and pressure.

According to Fick's first law of diffusion, the rate of diffusion $\partial M/\partial t$ of a gas across a gas liquid interface of area A into a quiescent body of liquid is a function of the coefficient of molecular diffusion D_m of the gas and the gas concentration gradient $\partial C/\partial x$:

$$\frac{\partial M}{\partial t} = D_m \cdot A \cdot \frac{\partial C}{\partial x} \tag{6.3}$$

The process of distributing gas throughout the body of liquid by diffusion alone is extremely slow, but it can be speeded up by mixing, which physically distributes the solute and fully exploits the diffusion capabilities by maintaining the maximum concentration gradient across the interface.

Various theories have been proposed to describe the process of gas transfer (Danckwerts, 1951; Higbie, 1935; Lewis & Whitman, 1924).

Equation 6.3, which forms the basis of equations developed to describe the oxygen transfer from air to water, is more generally expressed in the form:

$$\frac{dM}{dt} = AK_L(C_s - C) \tag{6.4}$$

where K_L is the liquid film coefficient and C_s the dissolved oxygen concentration at saturation. K_L is clearly a function of the gas diffusivity (a function of temperature T, the size of diffusing particles and the liquid viscosity ν) and the turbulence or degree of mixing in the liquid medium (time of exposure t_c of the fluid elements). In the case of air entrainment, K_L is further dependent on bubble size. The presence of surface active agents in the liquid may alter the physical properties of the liquid and thus the size d_b of entrained bubbles. Furthermore, films of surface active agents formed at the air/liquid interface can inhibit the diffusion process; however, some evidence indicates that in highly turbulent situations this film and any associated effects on aeration performance are effectively destroyed (Mancy & Okun, 1968).

Thus for a particular diffusion gas such as oxygen in a situation of turbulent air entrainment (mean bubble diameter d_b):

$$K_L = f(D_m, t_c, d_b) = f(T, \nu, t_c, d_b) \tag{6.5}$$

Equation 6.4 may be expressed as an instantaneous rate of change of concentration:

$$\frac{dC}{dt} = \frac{1}{V} \cdot \frac{dM}{dt} = K_L \cdot \frac{A}{V} \cdot (C_s - C) \tag{6.6}$$

(V is the volume of liquid into which mass dM of gas diffuses in time dt).

Integrating between the limits of 0 and t for time and C_o and C_t for gas concentration, one arrives at the most widely used equation of gas transfer:

$$\frac{C_s - C_t}{C_s - C_o} = \frac{1}{r} = e^{-K_L \cdot \frac{A}{V} \cdot t} \tag{6.7}$$

$$r \left(= \frac{\text{upstream deficit}}{\text{downstream deficit}} \right)$$

is commonly referred to as the deficit ratio.

Equation 6.7 expanded into a series becomes:

$$r-1=\sum_{1}^{n}\frac{\left(K_L\cdot\frac{A}{V}\cdot t\right)^n}{n!} \tag{6.8}$$

in which

$$(r-1)=\frac{C_{\text{downstream}}-C_{\text{upstream}}}{\text{downstream deficit}}$$

For oxygen transfer from bubbles only (neglecting the effect of the free surface) with n the number of bubbles:

$$\frac{A}{V}=\frac{6\pi\, d_b^2\, n}{Qt}=\frac{6\pi\, d_b^2\, Q_a\, 6t}{\pi\, d_b^3\, tQ}=\frac{36Q_a}{d_b\, Q}=\frac{36\beta}{d_b}$$

Thus (from Eq. 6.2)

$$\frac{A}{V}=f(\mathrm{Fr}, d_b) \tag{6.9}$$

For oxygen diffusion into water at a particular temperature (constant ν and T), Equations 6.5, 6.7 and 6.9 result in

$$r=f(\mathrm{Fr}, t_c, d_b, t) \tag{6.10}$$

The temperature dependence of Equation 6.7 is generally described by a temperature function of the form:

$$\left(K_L\cdot\frac{A}{V}\right)_{T_2}=\left(K_L\cdot\frac{A}{V}\right)_{T_1}\cdot\theta^{T_2-T_1} \tag{6.11}$$

Numerous values of θ have been published, the differences among them being attributable to variations in mixing conditions or turbulence. The greater the turbulence, the smaller is the value of θ; thus extrapolation to higher degrees of turbulence will probably result in θ tending to unity. The effect of increasing temperature is to reduce the gas solubility and thus the concentration difference (the driving force), and also to increase the diffusion rate.

Two phases of gas transfer can be visualised:

1. On exposure to an air interface, the water molecular boundary layer is instantaneously saturated with oxygen irrespective of the initial oxygen deficit.

2. The distribution of oxygen into the body of liquid is effected either by diffusion or by physical mixing or by a combination of these two processes.

In Phase 1 temperature is important in that it determines the saturation concentration. In Phase 2 the diffusion processes are accelerated by temperature increases whereas the mixing process is not affected by temperature (Imhoff & Albrecht, 1972). Thus the relative temperature influence in Phase 2 will be determined by the degree of mixing. The greater the mixing, the lower will be the dependence on diffusion for the distribution of oxygen throughout the fluid and thus the lower the effect of temperature.

Clearly the application of Equations 6.6 to 6.11 to practical situations (overfall, outflow with jump etc.) is difficult, and recourse has to be usually made to experimentation to establish aeration possibilities in a given situation. Thus, the analysis of scale effects or the observance of the limits of experimental work – as already emphasised – is important. From the above dicussion it is clear that one of the main difficulties in the modelling of flow with entrained air and of oxygen transfer stems from the fact that the size of entrained air bubbles remains – for a given water quality – constant irrespective of model scale, and furthermore a limit exists below which the scale of the model should not be decreased in order to preserve the same mode of oxygen transfer as in the prototype for a particular aeration system.

6.3 OXYGEN TRANSFER AT OUTFLOWS UNDER GATES WITH A HYDRAULIC JUMP

Most of the experiments used as a basis for predictive equations in this section and Section 4 were carried out in the hydraulics laboratory of the Department of Civil Engineering at the University of Newcastle upon Tyne (Avery & Novak, 1978; Avery 1976).

As a result of constant recirculation the water was aerated to saturation; it was necessary, therefore, to deoxygenate the water used in the aeration experiments to the extent that even after aeration, the oxygen level was still significantly below saturation. This condition was effected by controlled dosing of a sodium sulphite solution in the presence of a cobalt catalyst into the water upstream of the aeration apparatus. Precautions were taken to ensure sufficient reaction time so that residues of the sulphite were not carried forward to the experimental apparatus.

Oxygen levels were measured by means of calibrated dissolved oxygen meters each incorporating a temperature compensated electrode and a resistance thermometer.

Oxygen levels were expressed as percentage saturation, and oxygen transfer was expressed as the deficit ratio corrected to 15°C using a previously published temperature correction (Gameson et al., 1958) ($\theta = 1.018$ in Equation 6.11). Use of this correction was justified since the energy expenditures in the currently reported tests were comparable to those for which the above value of θ was derived and the temperature of the laboratory water did not vary substantially from 15°C.

As the laboratory water was treated with sodium nitrite ($N_a\ NO_2$) to prevent corrosion, a variation of the concentration of $N_a\ NO_2$ provided a welcome means of varying the bubble size d_b. Three $N_a\ NO_2$ concentrations were used – 0, 0.3%

and 0.6% with a mean bubble size of 2.53, 1.72 and 1.57 mm (Avery, 1976).

The hydraulic jump formed immediately downstream of an underflow sluice gate in a horizontal rectangular flume 100 mm wide and 3 m in length. The experimental range for the supercritical Froude number $Fr_1 = v_1/\sqrt{(gy_1)}$ was $2.8 < Fr_1 < Fr_1$ 8.5 and that for the specific discharge was $0.0145 < q < 0.0808$ m^2/s; the results could best be correlated by the equation (Avery & Novak, 1978)

$$r_{15} - 1 = k_1 \left(\frac{\Delta E}{y_1}\right)^{0.8} \left(\frac{q}{q_o}\right)^{0.75} \tag{6.12}$$

where k_1 is a function of salinity and $q_o = 0.0345$ m^2/s. All the data are compared with a plot of Equation 6.12 in Figure 6.1.

The results can also be described by the equations:

$$r_{15} - 1 = k_2 \, Fr_1^{2.1} \left(\frac{q}{q_o}\right)^{0.75} \tag{6.13}$$

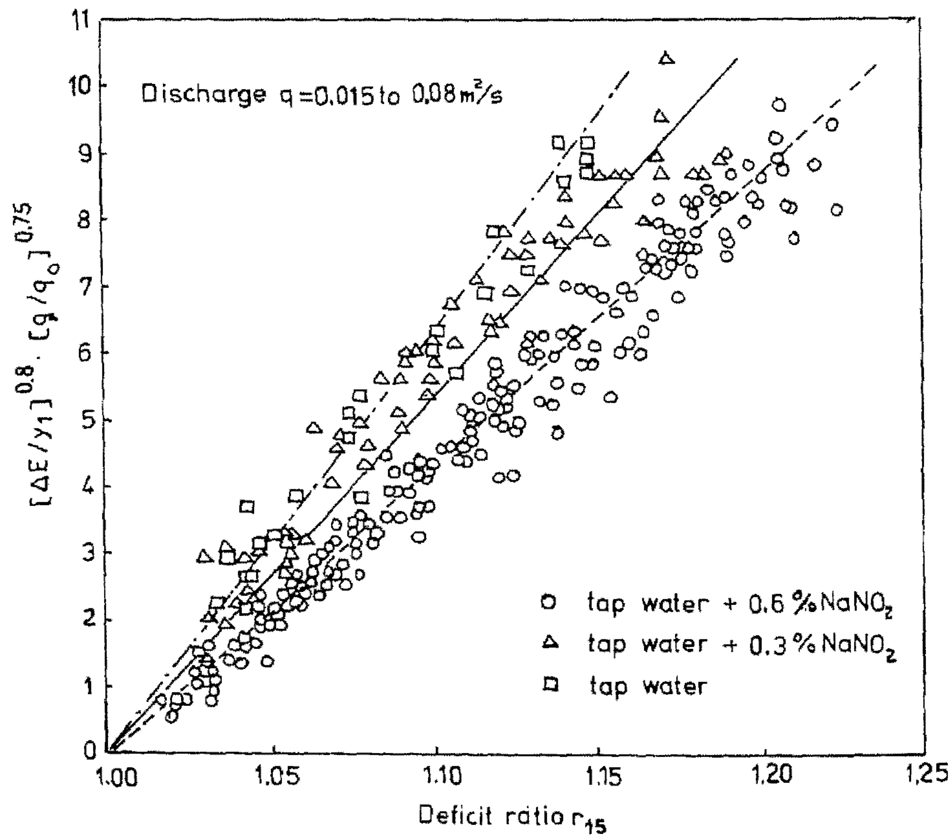

Figure 6.1. Oxygenation in hydraulic jumps: Correlation of measurements with Equation 6.12 (Novak, 1980)

and

$$r_{15} - 1 = k_3 \, \mathrm{Fr}_1^{2.1} \, \mathrm{Re}^{0.75} \tag{6.14}$$

where $\mathrm{Re} = q/\nu$ is the Reynolds number, and k_2 and k_3 are again functions of $N_a NO_2$ concentration. The values of k_1, k_2 and k_3 for the range of salinity tested and for $\nu = 1.143 \times 10^{-6}$ m^2/s are as follows:

Sodium nitrite concentration %	k_1 (Eq. 12)	k_2 (Eq. 13)	k_3 (Eq. 14)
0	0.0158	0.00230	1.0043×10^{-6}
0.3	0.0186	0.00285	1.2445×10^{-6}
0.6	0.0230	0.00355	1.5502×10^{-6}

Use of the Reynolds number in Equation 6.14 is not necessarily indicative of viscous effects; Re is used here merely to get a dimensionless expression using discharge q. Other expressions have been published (Apted & Novak, 1973; Holler, 1971) and the more reliable of these results have, after checking, been incorporated in the more generally valid Equations 6.12 to 6.14.

In accordance with Equations 6.7 to 6.9, a reduction in bubble size results in an increase in the oxygen uptake, and $(r - 1)$ is approximately inversely proportional to bubble size.

An attempt was made to compute K_L from Equation 6.7 from the experimental data; Equation 6.9 was used for the value A/V, Equation 6.14 for r, the Kalinske & Robertson (1942) equation for Q_a/Q and tracer experiments gave the time of contact t in the zone of the jump The range of K_L established in this way was (Avery, 1976) $1.6 \times 10^{-4} < K_L < 4.5 \times 10^{-4}$ and the corresponding range of Sherwood numbers $\mathrm{Sh} = K_{L} d_b / D_m$ was $120 < \mathrm{Sh} < 620$. The following relationship fits the experimental results (Avery & Novak, 1977)

$$\mathrm{Sh} = 0.266 \left(\frac{Q}{A} \frac{\Delta E}{\nu} \right)^{1.2} \tag{6.15}$$

Smutek (1974), using prototype measurements, obtained a value of $K_L \cong 11 \times 10^{-4}$ for a hydraulic jump; hence, some scale effect may occur. Nevertheless all these values are substantially larger than the values of K_L estimated for the free surface of normal open channel flow.

Field verification of Equations 6.12 to 6.14 is very limited (Burley, 1978). Nevertheless, from measurements at a treatment plant with discharges up to 3 times bigger than those used in the laboratory but at rather low Froude numbers ($\mathrm{Fr} < 3.6$) results agreeing reasonably well with the predictive equations were obtained.

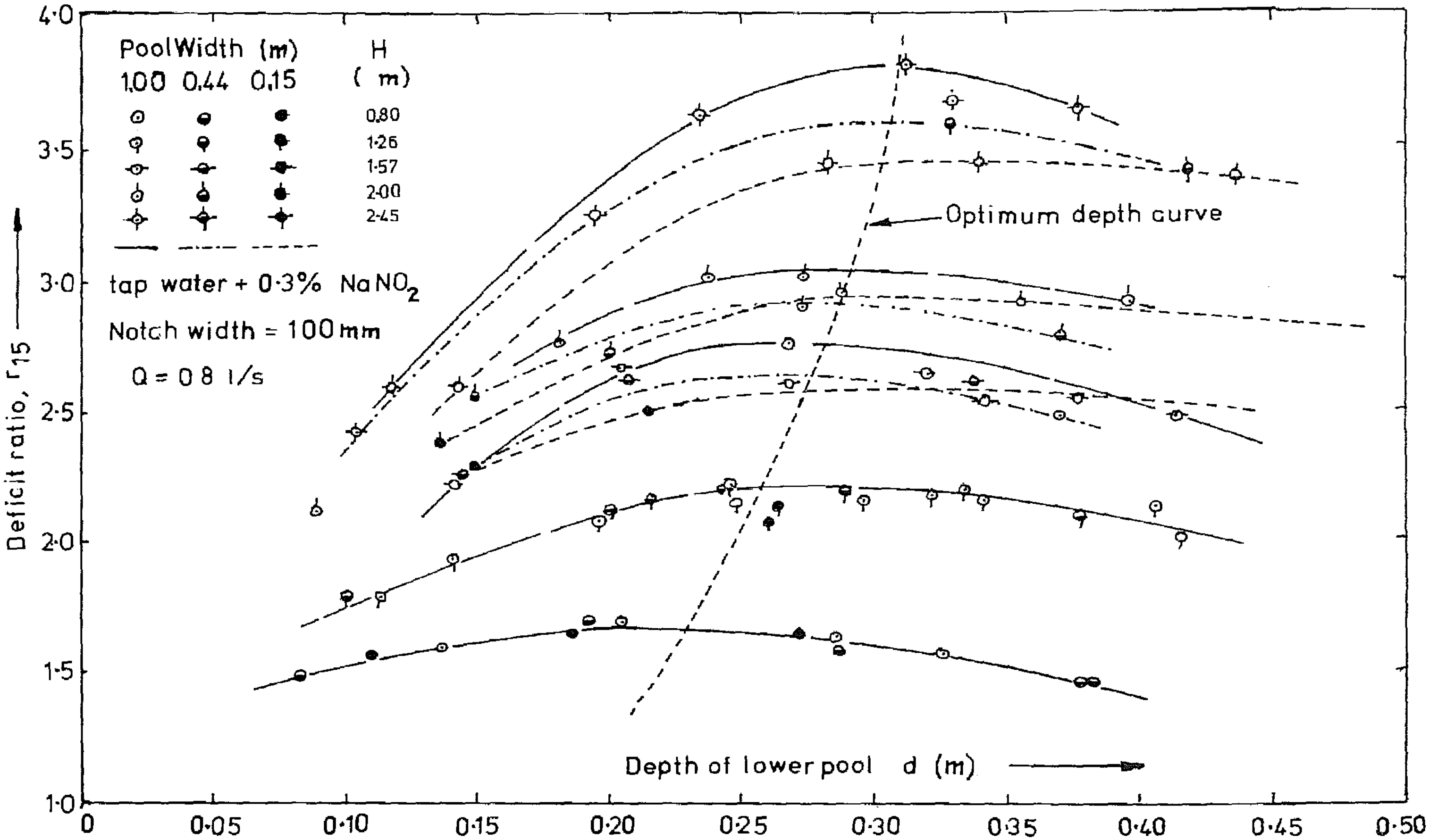

Figure 6.2. The effect on oxygen transfer of the lower pool geometry for a fixed discharge and various elevations of this pool

6.4 OXYGEN TRANSFER AT OVERFALLS

Although rates of oxygen transfer at overfalls have been measured fairly frequently both in the laboratory and in prototype, the diversity of conditions has impeded the process of reaching generally valid conclusions. The differences included structure and jet geometry, downstream conditions and water quality. Also many authors attempted to correlate the results in terms of height of fall only and ignored other parameters, particularly the discharge. A comprehensive summary of published approaches to this problem (Gameson, 1957; Gameson et al., 1958; Holler, 1971; Jarvis, 1970; Nakasone, 1975; Notes on Water pollution; 1973) was carried out by Avery & Novak (1978). They also clearly demonstrated that the plunge pool depth d and to a lesser degree its shape and width affect the oxygen uptake. For a given difference in elevation between the weir crest and pool bed H, an 'optimum' depth corresponding to the maximum penetration depth of air bubbles into the pool occurs (Fig. 6.2). For a depth d smaller than the optimum the contact time of the air drawn into the pool is smaller than for the maximum penetration depth; for a bigger depth this contact time is not increased and the height of fall (and thus the amount of entrained air) is decreased.

The contact time of air bubbles in the downstream pool is thus only one factor affecting oxygen uptake. Equally important is the amount of air drawn into the pool at impact, a quantity that depends on the shape, size and state of the jet and on its velocity at impact.

Because of the complicated and sometimes conflicting effects of bubble contact time and quantities of air entrained, Avery & Novak used a jet Froude number at *impact into the pool* in the form

$$\mathrm{Fr}_J = \frac{v}{\sqrt{gD}} = \left[\frac{\pi\sqrt{(2gh^5)}}{Q}\right]^{1/4} \tag{6.16}$$

where D is the jet diameter, h the height of fall (distance between the upstream and downstream water level) and Q the discharge. For jets with a non-circular shape at impact they used the jet discharge q_J per unit jet perimeter P ($q_J = Q/P$):

$$q_J = R\sqrt{(2gh)} \tag{6.17}$$

resulting in

$$\mathrm{Fr}_J = \left(\frac{gh^3}{2q_J^2}\right)^{0.25} \tag{6.16a}$$

These equations apply only if the jets at impact are solid, not disintegrated. Hoření (1956) established experimentally for $0.013 < q < 0.08$ m^2/s the distance L_o from a rectangular notch along the jet centreline to the point of total disintegration as a function of the specific discharge at the crest q:

$$L_o = 31.19\, q^{0.319}\ \text{(cm)} \tag{6.18}$$

or

$$L_o = 5.89\, q^{0.319} \cong 6\, q^{1/3}\ (\mathrm{m}) \tag{6.18a}$$

(In Equation 6.18 q is in cm^2/s, in Equation 6.18a in m^2/s).

As the minimum velocity for inception of air entrainment is approximately 1 m/s, corresponding to a height of fall 0.05 m, the limits of applicability of Equations 6.16 and 6.17 (and of the subsequent equations for the deficit ratio) are

$$0.05 < h < 6\, q^{1/3}\ (\mathrm{m})$$

Solid jets from a rectangular notch either converge rapidly to a circular cross-section, Equation 6.16, or diverge to flat rectangular shapes, Equation 6.17. The criterion for a jet to diverge was established as (Burley, 1978; Novak, 1980)

$$\frac{H'}{b} > 1.288 \tag{6.19}$$

where b is notch width, H' the head on the notch crest and the height of fall at which convergent jets ($H'/b < 1.288$) become circular as

$$h = 430\, b^{0.36}\, Q^{0.8}\ (\mathrm{m}) \tag{6.20}$$

in which b is in metres and Q in m^3/s.

The limits of experiments on jets from rectangular notches carried out in the laboratory at Newcastle upon Tyne that were used to derive the equations for r were:

$$0.1 < b < 0.3\ (\mathrm{m})$$

$$0.6 < Q < 5.8\ (\mathrm{l/s})$$

$$0.25 < h < 2.1\ (\mathrm{m})$$

Under these conditions the following equations were established for the optimum depth (d_{op} in cm) of the downstream pool (h in cm, q_J in cm^2/s in Equation 6.21a:

$$d_{op} = 0.41\, \mathrm{Re}^{0.39}\, \mathrm{Fr}_J^{0.24} \tag{6.21}$$

or

$$d_{op} = 3.3\, h^{0.18}\, q_J^{0.27} \tag{6.21a}$$

and for r_{15} at $d \geq d_{op}$ and for solid jets:

$$r_{15} - 1 = k_4\, h^{1.34}\, q_J^{-0.36} \tag{6.22}$$

(h in cm, q_J in cm^2/s). Or

$$r_{15} - 1 = k_5\, \mathrm{Fr}_J^{0.72}\, h^{0.8} \tag{6.23}$$

(h in cm). Or

$$r_{15} - 1 = k_6 \,\mathrm{Fr}_J^{1.78}\, \mathrm{Re}^{0.53} \tag{6.24}$$

The coefficients k_4, k_5 and k_6 are given as functions of salinity by the following values (Novak, 1980).

$N_a NO_2$ %	k_4 (Eq.22)	k_5 (Eq.23)	k_6 (Eq.24)
0	0.0108	0.0036	0.627×10^{-4}
0.3	0.0151	0.0050	0.869×10^{-4}
0.6	0.0214	0.0071	1.243×10^{-4}

Note: k_4 has dimensions $cm^{-0.62}/s^{0.36}$ and k_5 $cm^{-0.8}$

Figure 6.3 shows some of the experimental results leading to Equation 6.23. In the use of Equations 6.16a and 6.21 to 6.24 for weirs (as opposed to free falling jets from notches) two cases arise: if there is no access of air under the nappe (solid weir) $q = Q/b = q_J$, if the nappe springs clear of the weir with access of air underneath it $q = 2q_J$.

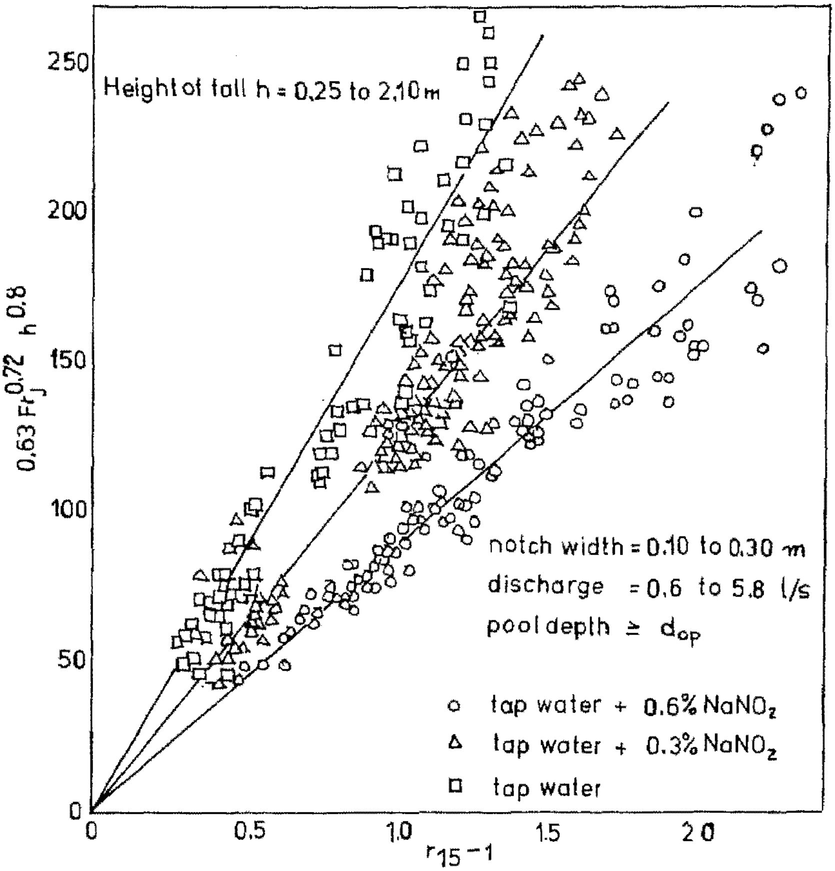

Figure 6.3. Weir oxygenation: Correlation of measurements with Equation 6.23 (Novak, 1980)

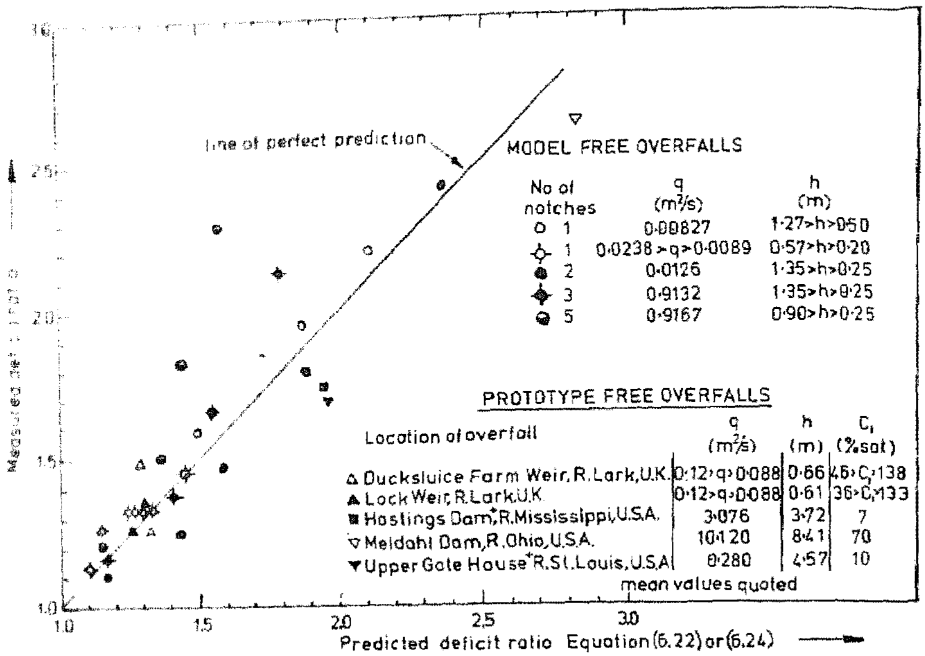

Figure 6.4. Correlation of Equation 6.22 to 6.24 with previously published oxygen transfer measurements for various model and prototype structures

Equations 6.22 to 6.24 were successfully correlated with a number of prototype structures and all published laboratory measurements for which sufficient data were available ((Holler, 1971; Notes on Water Pollution; 1973; Van Der Kroon, 1969) (Fig. 6.4). As can be seen from the figure, a large range of discharges and heights of fall as well as a variety of pollution loads (as evidenced by the initial dissolved oxygen levels) are encompassed in this correlation.

For all the data in Figure 6.4 zero salinity levels were assumed, in the absence of information to the contrary. Pool depths beneath the prototype free overfalls were sufficient to justify the assumption of optimum depth conditions, and jet widths were taken to be equal to the crest widths. Deficit ratios were corrected to 15°C by means of the temperature correction discussed earlier (Gameson, 1958). Once again, the energy expanded in the tailwater was found to lie within the same range as in the laboratory tests from which the above temperature correction was developed (Avery, 1976).

The good correlation obtained with the laboratory tap water experiments tends to confirm that the effect of pollutants on the liquid film coefficient is indeed negligible in highly turbulent flows; nevertheless, some contaminants may increase the air/water interface created (as observed earlier).

Nakasone (1975) incorporated the effect of h, q and d into his equations which are in the form

$$\ln r = K\, h^a\, q^b\, d^c \tag{6.25}$$

in which c is a constant 0.31, and K, a and b vary according to whether $h \lesseqgtr 1.2$ m and $q \lesseqgtr 0.065$ m^2/s as follows

	h	q	K	a	b
1	< 1.2	< 0.065	0.0785	1.31	0.428
2	< 1.2	> 0.065	5.39	1.31	− 0.363
3	> 1.2	< 0.065	0.0861	0.816	0.428
4	> 1.2	> 0.065	5.92	0.816	− 0.363

The correlation with (the simpler and physically more justified) Equations 6.22 to 6.24 (if the conditions for $d \geq$ optimum and $h < 6\, q^{1/3}$ are satisfied) is acceptable. The values of a and b in Case 2 correspond closely to the powers in Equation 6.22.

Markofsky & Kobus (1978) developed a diagram for sharp-crested aerated rectangular notches discharging into a deep pool that correlates ($r_{15} - 1$), Re and Fr (Fig. 6.5). The agreement with Equations 6.22 to 6.24 is fairly good even though the authors state that the graph is not intended for prediction of weir reoxygenation and should be used with caution (Kobus, 1985).

6.5 OXYGEN TRANSFER AT MULTIPLE JET OVERFALLS AND CASCADE WEIRS

Equations 6.22 to 6.24 are also applicable to multiple jet weirs and cascades with pools.

If one circular jet of Froude number Fr_j is split into N jets of equal diameters, the resultant Froude number will be $N^{1/4}\,\mathrm{Fr}_j$. The advantage afforded is obvious, a higher Froude number can be attained for the same discharge and height of fall by splitting the jet and therefore greater aeration is achieved. This result has been experimentally confirmed (Avery, 1976; Tekle, 1983). The application of the equations to multiple jet weirs is valid subject to two conditions:

1. That the jets do not interact (and disintegrate) during their free fall;
2. That the air entrainment and mixing process due to any jet is not unduly affected by that of any neighbouring jets.

Successful use of Equations 6.22 to 6.24 for multiple jets is also demonstrated in Figure 6.4 with the exception of the 5-notch weir for which jet interaction obviously resulted in a greater oxygen transfer than predicted by the equations.

If a cascade weir consists of a series of free overfalls with pools strung together, the oxygen transfer can be computed from Equations 6.22, 6.23 or 6.24, if applied consecutively to each step of the cascade. The process is simplified if the cascade consists of a series of n equal steps; the deficit ratio then would be r^n where r is the deficit ratio for one step.

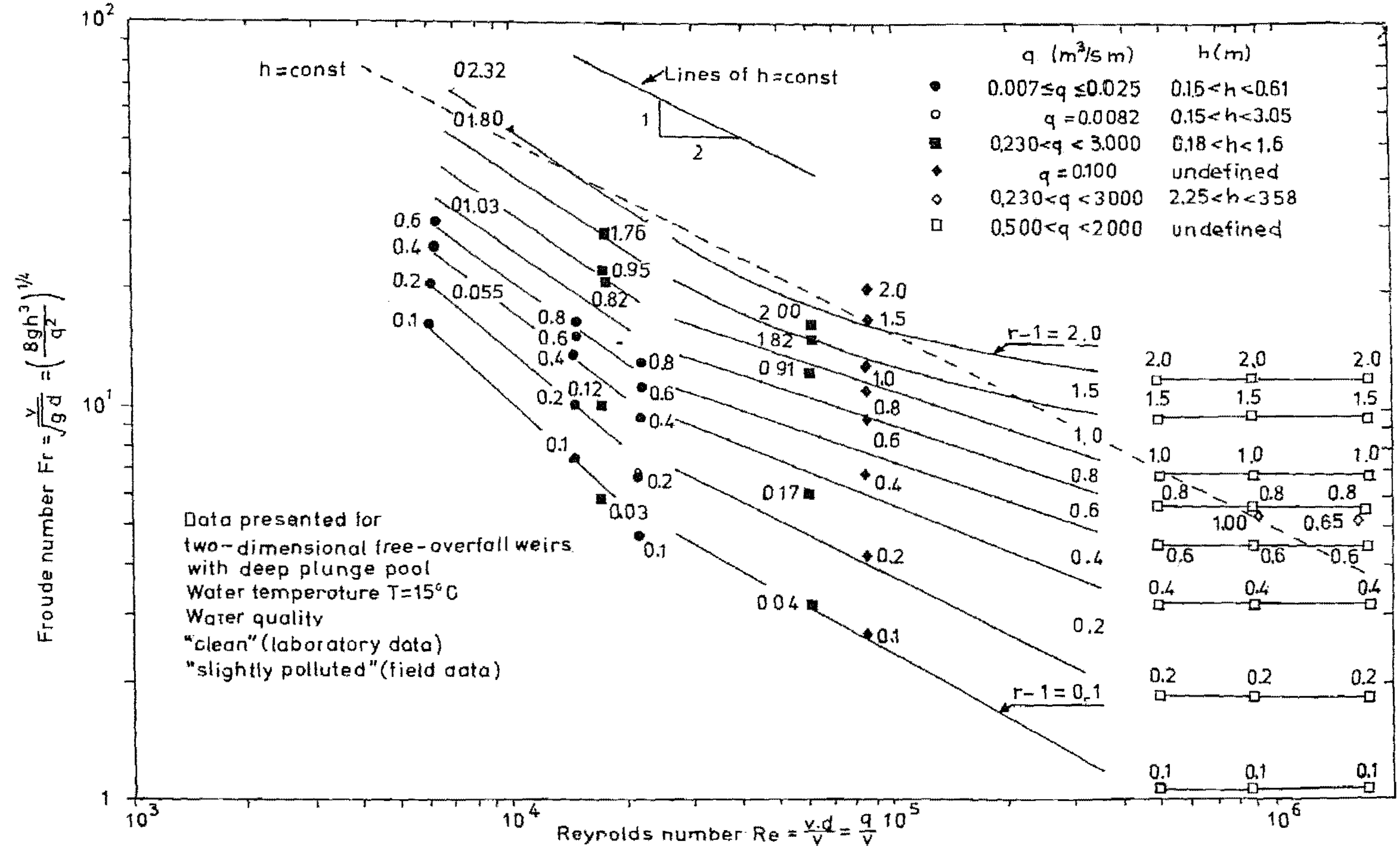

Figure 6.5. Unified presentation of weir aeration data (rectangular weirs with deep plunge pools); Markofsky and Kobus, 1978

The deficit ratio computed in this way can be compared with the ratio for a cascade without pools using the graph shown in Figure 6.6 from Tebutt et al. (1977) summarising the results of work carried out at the University of Birmingham. Experiments were carried out on cascades with three slopes (1:1, 1:2.5, 1:5) individual step heights of $0.05 < H < 0.50$ m and discharges of $0.0116 < q < 0.1447$ m^2/s. The graph relates the cascade slope, number of steps, specific discharge, height of steps and the deficit ratio. Thus, e.g., a cascade 1.00 m high, with $q = 150$ cm^2/s and with 5 steps 20 cm high would achieve a deficit ratio r\F\ 2.23 (practically irrespective of slope). The application of Equation 6.24 for the same discharge and a single step 1.00 m high with a pool of optimum depth yields r\F\ 2.2 i.e., a value identical to that for a cascade without pools. A cascade of 5 steps 20 cm high with a 20 cm pool at each step (Fig. 6.7) would, on the other hand, give an increased value of $r = 1.23^5 = 2.815$ ($r = 1.23$ is the deficit ratio for a single 20 cm high step).

The efficiency η of aerators is generally gauged as the quantity of oxygen transferred relative to the power expended (kg O_2/kwh):

$$\eta = \frac{3.6\,(C_s - C_1)\left(\dfrac{r-1}{r}\right)}{gh} \tag{6.26}$$

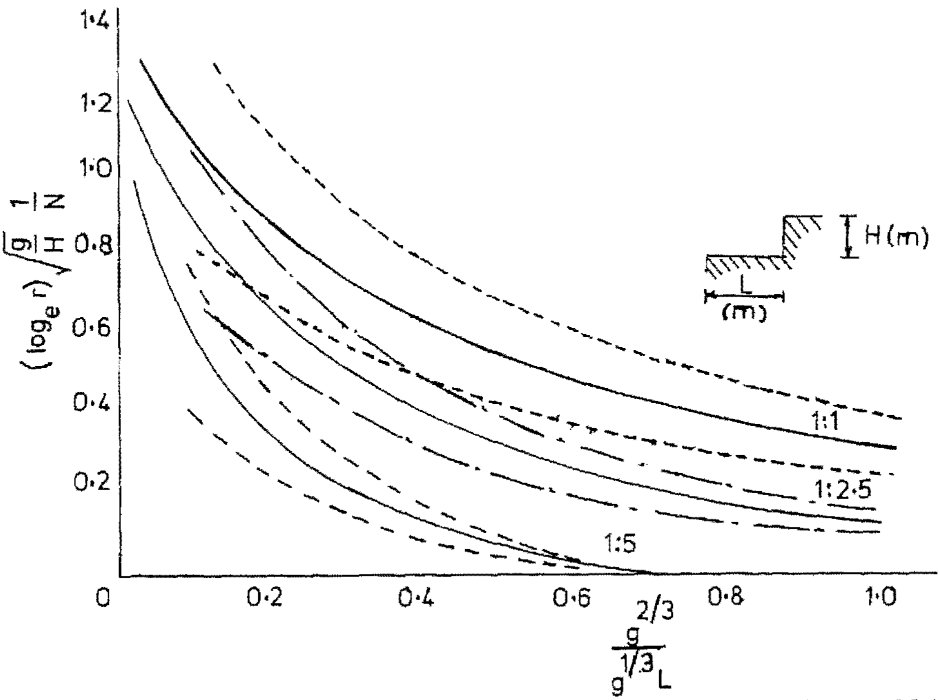

Figure 6.6. Relation between reaeration factor and flow number; dotted lines indicate 95% confidence limits (Tebbutt et al., 1977)

For a weir the efficiency is a function of the Froude and Reynolds numbers as well as of the initial oxygen deficit, which should be specified if efficiencies are compared. Figure 6.7 compares the efficiency of the hydraulic jump, free overfall and the illustrated cascade (with pools with optimum depth conditions) for varying heights of fall and a particular specific discharge.

The high efficiency of the free overfall at small heights of fall illustrates why a cascade of overfalls is more efficient than a single free overfall. The efficiences of

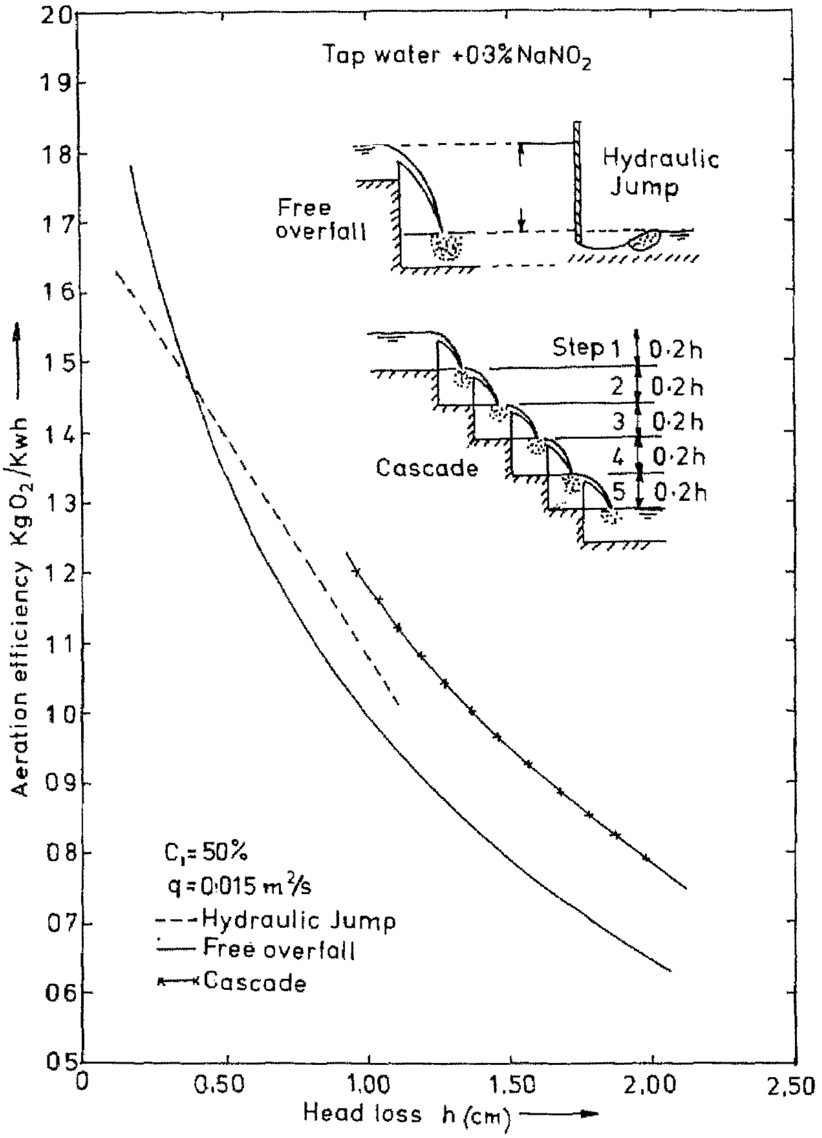

Figure 6.7. A comparison of aeration efficiency for a hydraulic jump, free overfall weir and a cascade weir

the free overfall and hydraulic jump are comparable, whilst the latter has the advantage of requiring the smaller tailwater depth.

The efficiencies for the prototypes in Figure 6.4 varied from 0.2 to 0.9 kg/kwh for a 50% initial deficit (Avery, 1976). The above efficiencies are not uncompetitive when compared with commercial aerators; for the latter the following are examples:

- Turbine aerators 0.45- 1.41 kg/kwh (Imhoff & Albrecht, 1972);
- Surface aerator 0.79-2.00 kg/kwh (Kaplovsky & Sosewitz, 1964);
- U-tube aerator 0.61-2.13 kg/kwh (Speece et al., 1969).

The efficiency of the single free overfall could, of course, be improved substantially by conversion to a cascade, or by serrating the crest to induce the formation of multiple jets.

6.6 OXYGEN TRANSFER AT BOTTOM OUTLETS AND TURBINES

The previous sections have amply demonstrated that the mass transfer and oxygenation of water are greatly affected by the degree of turbulence during mixing and length of contact time of air bubbles with water in highly turbulent flow. Thus even higher values of the deficit ratio and better efficiencies of aeration can be achieved at bottom outlets using valves (with air supply), which create highly turbulent aerated flow in the downstream region with long contact times. Typically this can be done by producing a hydraulic jump inside a conduit or better still be an internal ring hydraulic jump (e.g., downstream of a cone dispersing valve) discharging in the outlet itself (rather than into atmosphere). Haindl (1984) shows some outlets operating in this fashion and quotes oxygenation (C_2) in the region of 10-13 g/m^3, i.e., about double the values associated with normal dam bottom outlet valves (e.g., hollow jet valves discharging into the atmosphere and a downstream pool).

For turbines discharging oxygen deficient water, it is possible to introduce air into the negative pressure zones (in the draft tube) and thus to improve water quality; however this usually decreases the turbine output. It is, therefore, preferable to provide air supply into the outlet from draft tubes of turbines and to use this artificial air supply seasonally as required.

6.7 SUPERSATURATION AND ITS PREVENTION

So far attention has focussed on oxygen transfer at low head structures for which reductions in the oxygen deficiency benefit the environment. If, however, the upstream water is saturated with oxygen, or nearly so, then further oxygen enrichment can lead to oxygen supersaturation that may have detrimental effects on the environment. This situation is more likely to occur at high head structures with their associated high velocities of flow. Most of the research on the subject of supersaturation at hydraulic structures and its environmental effect and on prevention measures has been carried out in the USA (Johnson, 1984; Nebeker & Brett, 1976; Weitkamp & Katz, 1973) and Norway (Tekle, 1983; Tekle et al., 1984).

A liquid is supersaturated if it contains more dissolved gas than the amount that it can normally dissolve at the ambient pressure and temperature. Thus water below dam spillways and power inlets, where a substantial amount of air may be entrained, can be supersaturated; the degassing then takes place in the stilling basin, tailrace and downstream river stretch. This de-gassing, which begins rapidly, becomes slower with time without substantial mixing and turbulence. Dilution and mixing with water at natural saturation can also reduce supersaturation.

Supersaturation leads to de-gassing, or bubble formation, inside the blood vessels in fish, crabs, etc. and results in the gas bubble disease. The lethal supersaturation for fish is some 5-15% above saturation level depending on the time the fish is exposed to the supersaturated water (Nebeker & Brett, 1976; Weitkamp & Katz, 1973).

Supersaturation cannot readily be prevented below long spillways where self-aeration occurs, but it can be alleviated by the intensive turbulence in the stilling basin and by deflecting the flow at entry into the basin for low flows to the free downstream water surface (Johnson, 1984) (for large flows this deflection would be detrimental to the stilling basin operation). Power station intakes with drop shafts are particularly susceptible to supersaturation; therefore, a good hydraulic design should minimise air entrainment. Tunnels at mild slopes, intakes with level controls that keep the shaft filled and the intake submerged (Tekle et al., 1984) and degassing chambers are some of the possible design measures. At least some understanding of air transport under the roof of lined and unlined mild slope rock tunnels is also essential in the design of the power tunnels in case the prevention of air entrainment from a dropshaft intake is uneconomical or impossible (Tekle, 1983).

Normal Pelton turbines provide almost complete degassing of supersaturated water whereas Francis turbines do not provide any effective de-airing; in the latter case the best measure, if feasible, is to use a shallow free-surface tailrace tunnel. At outlets into lakes, reservoirs and fjords, a jet outlet in deep water and an air curtain are measures that can provide sufficient dilution and mixing. The power station can also be operated so as to control supersaturation within acceptable limits. Fish farms can be protected by an air curtain or boom (Tekle et al., 1984).

6.8 SIMILARITY AND FINAL CONSIDERATIONS

Kobus (1985) suggests a functional relationship between air entrainment β, and the Froude and Reynolds numbers that emphasises the importance of the inception limit (plunging jets) and entrainment limit (hydraulic jumps). When modelling the mass transfer process, water quality and surface tension also affect the results. Even, however, if the Reynolds numbers are high enough (Re > 10^5) and the models large enough (to overcome the effects of viscosity and surface tension) the problem that the bubble sizes (for given water salinity) are the same in model and prototype remains.

Thus some scale effects are likely to affect the predictions of oxygen uptake that are based on model experiments alone. After all, a simple analysis of the scales (M) in section 6.3 shows (Avery & Novak, 1977) that for a jump from Equations 6.12 to 6.14

$$M_{r-1} = M_l^{9/8} \tag{6.27}$$

where M_l is the length scale; similarly in section 6.4 from Equations 6.22 to 6.24

$$M_{r-1} = M_l^{0.8} \tag{6.28}$$

Equations 6.27 and 6.28 show that to obtain a prototype value of $(r - 1)$ from results of physical model experiments these results have to be scaled up approximately according to the geometrical scale of the model.

Care must evidently be exercised in using the developed predictive equations outside their original experimental limits, which for the case of the hydraulic jump were $2.8 < Fr_1 < 8.5$ and $0.0145 < q < 0.0808$ m^2/s and in the case of free overflow $0.1 < b < 0.3$ m, $0.6 < Q < 5.8$ l/s and $0.25 < h < 2.1$ m. Furthermore it must be emphasised that Equations 6.22 to 6.24 can be used only if $0.05 < h < 6\ q^{1/3}$ (to overcome the limit of inception of air entrainment and not to exceed the limit for total jet disintegration) and for plunge pool depths $d \geq d_{op}$ where d_{op} is given by Equation 6.21, or 6.21a. Nonetheless, the application of Equations 6.12 to 6.14 to available limited field data for the hydraulic jump and Equations 6.22 to 6.24 to a substantial amount of field data in the case of overfalls, resulted in good agreement. Thus, although the experimental limits of the derived predictive equations have been stressed, they probably give acceptable estimates of the likely oxygen deficit ratio even for conditions well outside these limits. In view of the substantial effect of structural details of overfalls in the case of weirs (possible jet interference etc.) and particularly of any pollutant influencing air bubble size, engineering judgement will always be necessary. In many cases of treatment plant outfalls and smaller structures predictions of oxygen uptake based on the discussed equations may be, however, used with reasonable confidence.

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NOTATION

A	area of air/water interface (L^2)
b	width of rectangular notch, length of weir (L)
C	gas or oxygen concentration in liquid (ML^{-3})
C_1, C_s, C_t	dissolved oxygen concentration prior to aeration, at saturation, at time t (ML^{-3})
d	tailwater depth below weir (L), thickness of jet at pool level (Fig. 5) (L)
d_{op}	optimum tailwater depth for oxygen transfer (L)
d_b	air bubble diameter (L)
D	jet diameter at pool level (L)

D_m	coefficient of molecular diffusion or diffusivity (L^2T^{-1})
ΔE	energy loss across hydraulic jump (L)
$Fr_1 =$	$\frac{v_1}{\sqrt{gy_1}}$ supercritical Froude number (hydraulic jump)
Fr_j	jet Froude number, Eq. (21)
h	difference between water levels above and below a weir (L)
H	difference between notch crest and downstream bed (L)
H'	head on a rectangular notch crest (L)
$k_1 - k_6$	functions of dissolved sodium nitrite concentration, Eq. (12)-(14), (22)-(24)
K_L	liquid film coefficient (LT^{-1})
L_a	distance measured along jet centreline to the point of total disintegration (L)
M	mass of gas transferred (M), scale (prototype/model)
n	number of steps in cascade
N	number of jets formed by weir crest
q	water discharge per unit channel or crest width (L^2T^{-1})
q_o	a reference water discharge per unit channel width (L^2T^{-1})
q_j	jet water discharge per unit jet perimeter at impact (L^2T^{-1})
r, r_{15}	oxygen deficit ratio at T°C, 15°C
R	hydraulic radius (L)
t_c	time of exposure of fluid elements (T)
T	temperature (°C)
v_c	inception limit velocity (LT^{-1})
V	volume of body of liquid (L^3)
y_1	initial (supercritical) hydraulic jump conjugate depth (L)
η	efficiency of oxygen transfer (M/ML^2T^{-2})
θ	temperature coefficient
β	relative air entrainment $\frac{Q_a}{Q}$