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Wallingford

CAVITATION IN HYDRAULIC STRUCTURES:

Occurrence and Prevention

by

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ABSTRACT

A review is made of literature on cavitation in large hydraulic structures in order to summarise the present state of knowledge, provide guidance to designers, and identify areas requiring further research. The topics covered include: (1) mechanisms of cavity formation and collapse; (2) cavitation at surface irregularities, gate slots, and energy dissipators; (3) cavitation resistance of engineering materials; (4) self-aeration and use of aerators for preventing cavitation damage; (5) modelling of cavitation and aeration; (6) research needs. The first part of the report provides summaries of the available information on each topic. The second part consists of a series of Appendices which describe in more detail the information contained in over 200 references.

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1 INTRODUCTION

The purpose of this literature review is firstly to describe the present state of knowledge about the occurrence and prevention of cavitation in large hydraulic structures, and secondly to identify areas where further research is needed. The study has been carried out as part of a research programme funded by the Construction Industry Directorate of the Department of the Environment.

Since the survey is concerned with cavitation produced by the flow of water in high-head structures, it does not cover other specialist areas such as pumps and ship propellers. Despite this restriction, there exists a very large amount of information spread across several disciplines, and therefore it is possible that some significant references may have been inadvertently omitted. Many useful studies have been carried out in the USSR and P R China, and for descriptions of these it has been necessary to rely mainly on papers presented at international conferences or on English-language summaries.

It is intended that the review should be of use to engineers as well as researchers, and it therefore covers a fairly broad field. Sections 2 and 3 of the report give a general description of the nature of cavitation and of the factors which govern its occurrence. Sections 4 to 9 briefly summarise the available information on individual topics, and are linked to Appendices B to G which give more detailed descriptions of the relevant information in the references. The first group of topics deals with the main sources of cavitation in hydraulic structures: surface irregularities in channels (Section 4 and Appendix B); tunnel inlets and high-head gates (Section 5 and Appendix C); and energy dissipators (Section 6 and Appendix D). The cavitation

resistances of engineering materials, such as concrete, steel, resins and plastics, are considered in Section 7 and Appendix E. Since the presence of air in water has the beneficial effect of reducing or preventing cavitation damage, Section 8 and Appendix F describe information on self-aeration and the design of aerators for spillways and tunnels. Most studies on cavitation and aeration have been carried out in the laboratory, so the problems of scale effects in modelling are dealt with in Section 9 and Appendix G. Finally, topics requiring further research are identified in Appendix H. Within the Appendices, references on a particular subject have normally been presented in chronological sequence; also Figures are numbered in the order in which they are referred to in the Appendices.

Comparing results and drawing conclusions from different, and sometimes conflicting, studies can be difficult because there are usually variations in the experimental conditions, the techniques of measurement, or the methods of analysis. The summaries in Sections 4 to 9 therefore concentrate on general areas of agreement, and for more detailed information readers should refer to the Appendices and the original references.

2 MECHANISM OF CAVITATION

2.1 Description

This brief description of the cavitation phenomenon is based on information contained in a comprehensive textbook by Knapp et al (1970) and in surveys produced by Eisenberg (1961), Johnson (1963), Kenn (1968) and Knapp (1952).

A suitable definition for the type of cavitation which will be considered in this report was given by Knapp (1952) as "the formation and collapse of cavities in a stream of flowing liquid which results from pressure changes within the stream caused by changes in the velocity of flow". This excludes cavitation associated with the vibration of bodies in stationary fluids. Throughout this report it will be assumed that the liquid in question is water and that the gas is either air or water vapour.

The negative pressure required to form a cavity within pure water is extremely high and can be of the order of several hundred atmospheres. The fact that normal samples of water form cavities at much smaller pressures indicates that the cavities grow from pre-existing nuclei containing either water vapour or water vapour and air. The sizes of these nuclei need to be in the range 0.1 to 10 μ m, and two theories have been proposed to explain their existence and persistence. The first is that the nuclei are stabilized within the interstices of microscopic dust particles; the second is that an organic film forms around a nucleus and thereby maintains the internal pressure and prevents diffusion of air.

When the ambient pressure in the liquid falls close to the vapour pressure, the nuclei grow rapidly and become visible as a cloud of tiny cavitation bubbles. The inception pressure which triggers this growth is usually slightly lower than the vapour pressure, but depends upon the initial size of the nuclei and upon the ratio of air pressure to vapour pressure within them. The ultimate size of the cavities is determined by the time that they are subject to pressures lower than the inception pressure.

The main types of cavitation encountered in civil engineering situations are:

1. "travelling cavitation" in which cavities form in areas of low pressure, travel with the flow and collapse in regions of higher pressure;
2. "fixed cavitation" in which flow separates from a body and forms a quasi-steady cavity attached to the boundary; when the cavity extends beyond the generating body it is referred to as "super-cavitation";
3. "vortex cavitation" in which cavities form in the cores of fast-rotating eddies created in regions of high shear.

When the ambient pressure in the fluid exceeds the vapour pressure, cavities collapse very rapidly and generate extremely high pressures in their immediate vicinity; pressures of up to 15,000 atmospheres (1500MPa approx) were measured by Lesleighter (1983). Sound is also generated when cavities collapse and provides a method of determining the onset of cavitation. In some situations collapsing cavities are observed to rebound and go through several cycles of expansion and contraction. However, when the air content in the cavity is low, the bubble collapses without rebounding.

Solid surfaces are damaged by pitting when cavities collapse close up against them. Measurements of rates of pitting indicate that only a very small proportion of the available cavities are large enough and collapse close enough to a boundary to cause damage. During most of their life travelling cavities appear to remain spherical, but experimental evidence

suggests that they may distort when collapsing close to boundaries. In these circumstances the wall of the cavity remote from the boundary may fold inwards to form a needle-like jet of fluid. The micro-jet passes through the cavity and emerges at very high velocity into the fluid adjacent to the boundary.

Damage to solid surfaces may be caused by the impact of micro-jets and also by shock waves generated during the rapid collapse of cavities. However, experimental work by Tomita & Shima (1986) indicated that there is a third and more damaging mechanism, that of ultra-jets. These jets are formed when shock waves from a larger cavity trigger the very sudden asymmetric collapse of smaller cavities. In the experiments it was found that cavitation pitting was caused by the ultra-jets, which produced impact velocities of up to 370m/s, compared with an average of 130m/s for the larger micro-jets.

Cavitation can damage nearly all materials including very strong ones such as stainless steel. High pressures generated by collapsing cavities cause mechanical damage to surfaces, and with chemically inert solids and liquids this is probably the only mechanism involved. However, in the case of metals the damage is accelerated by chemical and electrochemical effects, perhaps because protective oxide layers are continually being removed by the mechanical action of the cavitation. No single mechanical or chemical property (for instance ductility or hardness) has been found to correlate the relative resistances of different materials to cavitation attack.

This resistance is often measured in terms of the rate of loss of mass per unit area. For ductile materials

the loss rate tends to vary considerably with time. During an initial "incubation" period the mechanical attack produces work-hardening of the surface but little loss of weight; beyond the incubation period the loss rate increases considerably. By contrast, more brittle materials do not exhibit an incubation period, but lose mass at a steadier speed. In the case of concrete, cavitation attacks the weaker mortar until the aggregate is undermined and then removed. For these reasons it is necessary to take account of the duration of attack when considering the relative resistance of different materials.

The rate of damage for a given material clearly also depends upon the intensity of the cavitation. If, for example, the ambient pressure in a test is gradually decreased, a point of "incipient" cavitation will be reached at which tiny bubbles first become visible; alternatively this limit is sometimes defined by the start of cavitation noise or by a sudden change in the turbulence characteristics of the flow. Measurements show that the rate of material loss is negligible at the point of incipient cavitation, increases to a peak at a higher stage of cavitation, and then decreases again. Different materials may reach their peak erosion rates at different intensities of cavitation so that comparative tests may be misleading if they are not carried out under equivalent prototype conditions. The occurrence of cavitation also exhibits a hysteresis effect with varying ambient pressure (or velocity). With a decreasing pressure the cavitation begins at a lower pressure than the one at which it ceases when the pressure is increased. The term "incipient" is applied to the limit of cavitation if the cavitation is starting, and "desinent" if it is ending.

Injecting air into water cushions the pressures generated by collapsing cavities, and can significantly reduce or eliminate the amount of damage. Cathodic or anodic protection of metals in water is effective in reducing cavitation erosion; gas (hydrogen or oxygen) released at the surface cushions the collapse of the cavities in a similar way to injected air.

Techniques for measuring the cavitation resistance of materials include:

1. Venturi tubes - cavities are generated in the throat and a sample is placed downstream at the point where they collapse;
2. Water tunnels - samples are placed downstream of a cylindrical body which produces cavities in its wake;
3. Vibrating equipment - application of an oscillating electromagnetic field to a suitable metal or crystal produces small amplitude extensions and contractions; this magnetostrictive principle is used to produce cavitation on samples by vibrating them at high frequency (typically 5-20kHz) in a stationary liquid. An alternative technique uses ultrasonic vibrations of a liquid to cause cavitation on a stationary sample;
4. Drop-impact equipment - samples are attached to a disc which is rotated at high speed through a jet of liquid. Although the method does not produce cavitation, the resulting erosion is quite similar in nature; this lends support to the theory

that cavitation damage is caused by high-speed jets of liquid (see above).

Since techniques 1 and 2 use flowing water, they should reproduce cavitating conditions in hydraulic structures more closely than 3 and 4. However, results from 1 and 2 are susceptible to changes in water temperature, air content and dust content. Machines using techniques 3 or 4 are cheaper to build and simpler to operate, and method 4 is less sensitive to variations in the properties of the water. None of these techniques can be expected to predict the precise behaviour of a material in a prototype situation; however, they can be used to rank materials in terms of their relative resistance to cavitation. In general the four methods produce similar rankings, but some inconsistencies do arise, even between machines using the same technique. Knapp et al (1970, Tables 9.1 to 9.14) give comprehensive data for a wide range of metals and alloys.

2.2 Cavitation parameters

Consider the conditions required to produce cavitation at a particular point in a flow (eg at a step in the boundary or at an obstruction). Let p_0 be the time-averaged absolute static pressure and V_0 the time-averaged velocity at a reference point 0 in the undisturbed flow. The instantaneous static pressure p_1 at the point of interest is found from Bernoulli's equation to be

$$p_1 = p_0 - \rho g z - \frac{1}{2} \rho V_0^2 [(1+\delta)^2 (1+\epsilon)^2 - 1] \quad (1)$$

where ρ is the density of the fluid, g is the acceleration due to gravity and z is the elevation of point 1 above the reference point 0. (A full list of symbols is given in Appendix A). The factor δ is the

proportionate change in the time-averaged velocity caused by the obstruction or change in boundary shape. The factor ε describes the instantaneous fluctuation in velocity due to the general turbulence in the flow and any additional fluctuations produced by the change in boundary shape or by eddies. If the absolute pressure p_1 falls below a critical value p_c , nuclei already existing in the flow will expand rapidly to form cavities.

An important requirement for dynamic similarity between different tests is the cavitation index of the flow defined by

$$K = \frac{(p_o - p_v)}{\frac{1}{2} \rho V_o^2} \quad (2)$$

where p_v is the vapour pressure of the liquid at the test temperature. Incipient cavitation occurs when the local pressure p_1 drops to the critical pressure p_c . The corresponding value of the cavitation index, defined in terms of the mean flow conditions at the reference position, is

$$K_1 = \left[\frac{p_o - p_v}{\frac{1}{2} \rho V_o^2} \right]_1 \quad (3)$$

which shows that cavitation may be initiated by decreasing p_o or increasing V_o . From Equations 1 and 3 it follows that

$$K_1 = [(1+\delta)^2(1+\varepsilon)^2 - 1] + \frac{[(p_c - p_v) + \rho g z]}{\frac{1}{2} \rho V_o^2} \quad (4)$$

It can be seen that K_1 may not necessarily remain constant for a particular flow geometry. The critical pressure p_c is usually slightly lower than p_v but

varies according to the size and number of nuclei that the liquid contains (see 2.1). The factor δ is a function of the boundary geometry, and may also depend upon the Reynolds number of the flow. The factor ϵ varies with the turbulence level of the fluid and the intensity of eddies generated in shear zones. These differences serve to explain why measured values of K_1 do not always agree between model and prototype or between one model and another.

When comparing different test results it is necessary to ensure that the cavitation parameters have been defined in the same way. The cavitation index is more correctly defined with p_0 in Equation 2 replaced by $(p_0 - \rho g z)$, but this alternative definition is less common, partly because the point of cavity formation can vary or may not be known precisely. The reference position 0 might be chosen upstream of the point of interest, as in the case of an upward step in the floor of a channel. However, in the case of an orifice the reference point might be chosen downstream in the vena contracta. The reference velocity V_0 is sometimes taken to be the depth-averaged velocity and sometimes the undisturbed local velocity close to the point of interest.

The intensity of cavitation can be described in terms of the parameter I given by:

$$I = \frac{K_i - K}{K_i} \quad (5)$$

Cavitation damage does not occur if $I < 0$, and for a given material reaches a maximum rate at a value of I_m between 0 and 1.

In order to calculate values of the cavitation parameter K , it is necessary to take account of any

variation of atmospheric pressure with altitude and also the strong dependence of the vapour pressure of water, p_v , on temperature; values of p_v (from Batchelor, 1967) are given in Table 1.

3 OCCURRENCE IN HYDRAULIC STRUCTURES

In most hydraulic structures the ambient pressure is close to atmospheric, so cavitation is normally associated with flows of high velocity. Cavitation problems can arise when the velocity reaches about 15m/s, and above 25m/s serious damage can be expected if adequate precautions are not taken. Structures where damage has been reported include:

1. open-channel spillways
2. bottom outlets in dams
3. high-head gates and gate slots
4. energy dissipators including hydraulic-jump stilling basins.

Cavitation can also occur in pumps, valves and in pipelines under surge conditions, but these instances are outside the scope of this review.

If a flow remains attached to a bounding surface, cavitation-producing pressures are normally the result of turbulent velocity fluctuations in the boundary layer and/or of flow curvature. The point of minimum pressure on a surface can be measured or can sometimes be calculated theoretically from potential theory, with if necessary a suitable allowance for the displacement thickness of the boundary layer. However, turbulent fluctuations may cause cavitation to occur sooner than predicted, while the position at which it starts may be downstream of the point of minimum pressure (due for example to the formation of

a laminar separation bubble). If a pressure transducer, mounted at a suitable point on the boundary, indicates transient values close to vapour pressure, then cavitation is likely to occur. Damage will normally take place close to the spot at which the cavities are generated.

If a flow separates from a surface, cavities will form first in the fast-rotating eddies that are shed downstream. The pressure in the eddies will be lower than at the point of separation, so surface-mounted transducers will not provide a good indication of the likelihood of cavitation. The cavities will be swept downstream and will collapse when they enter a region of high pressure. Damage caused by shear flows can therefore occur a considerable distance downstream of the point of separation. This type of cavitation can be produced by local irregularities in the boundary (e.g. sharp steps at joints) or by the overall geometry of the structure. Examples of the latter include horizontal shear flows generated by high-velocity submerged jets, or vertical shear flows created by a sudden increase in channel width (e.g. two or more control gates discharging to a single tunnel).

4 SURFACE IRREGULARITIES

The principal method of predicting whether a surface irregularity will cause cavitation in a prototype structure is to calculate the cavitation number K of the flow from Equation 2, and compare it with previously determined values of the incipient cavitation index K_1 for that type of irregularity; cavitation will occur if $K < K_1$.

Values of K_1 have been obtained for many types of irregularity, some of which are shown in Figure 1.

The methods of determining K_1 include:

1. theoretical predictions of the minimum pressure on the surface of the irregularity;
2. laboratory measurements of the minimum pressure on the surface of the irregularity;
3. laboratory observations of cavity formation using cavitation tunnels (no free surface) or vacuum test rigs (with free surface);
4. field measurements of surface pressure or cavitation damage at irregularities.

Results based on field studies are the most appropriate, but very few are available because of the difficulties of carrying out controlled tests. If the flow separates at an irregularity, the lowest pressures will occur in eddies within the fluid; values of K_1 determined from measured or predicted surface pressures may thus be under-estimated. Data from cavitation tunnels and vacuum test rigs, backed up by field measurements, should therefore be used where possible.

In general, most of the experimental results for a given type of irregularity are in reasonable agreement. Discrepancies between tests do exist, but they are normally fairly small in comparison with the effects produced by minor changes in shape (e.g. rounded edges instead of sharp edges). Moreover, irregularities due to construction faults in spillways and tunnels have three-dimensional shapes which will seldom match precisely those tested in the laboratory.

Movement of concrete formwork is the most common cause of irregularities, and can give rise to abrupt offsets and chamfers (both into and away from the flow), sudden changes in slope, cusped joints, and undulations (see Types 1, 2, 3, 4, 5, 6 and 7D in Figure 1). Of these, abrupt offsets into the flow (Type 1A) have the greatest cavitation potential, and a suitable formula for calculating the K_1 value is that due to Liu (1983),

$$K_1 = 1.02 h^{0.326}, \quad h \leq 15\text{mm} \quad (6)$$

where h is the height of the step in mm. This equation gives values which are in reasonable agreement with the data of Ball (1963), and somewhat higher than those given by Falvey (1982) and Scheur (1985); see Section B.3 in Appendix B. If the edge of the offset is rounded to a radius of $r = 0.5h$, the value of K_1 is reduced to 86% of that given by Equation 6. When calculating the cavitation number K of the flow from Equation 2, the values of velocity V_o and absolute static pressure p_o should be those at the level of the top of the offset; for a fully-developed boundary layer V_o can be determined from Equation B.26. Surface irregularities just downstream of high-head gates are particularly liable to cause cavitation because the boundary layers are very thin, and do not protect the irregularities from the high free-stream velocities.

The cavitation potential of construction faults can be reduced by grinding them to form chamfers. For an into-flow chamfer (Type 3A), the slope needed to lower the value of K_1 below the cavitation number K of the flow can be estimated from the following empirical equations obtained by Novikova & Semenov (1985)

$$K_1 = 2.3, \quad \text{for } n \leq 1 \quad (7)$$

$$K_i = 2.3n^{-0.7}, \text{ for } n > 1 \quad (8)$$

where the slope is n units parallel to the flow to one unit normal to the flow. These equations give somewhat higher values of K_i than most of the other laboratory studies described in Section B.3 of Appendix B.

Data for chamfers angled away from the flow (Types 4A, B) are limited, and may not be comparable because of different definitions of the characteristic velocity (e.g. near the bed, or depth-averaged). Laboratory studies indicate that the values of K_i tend to be lower than for into-flow chamfers of equal slope.

As the flow velocity is increased, the standards of surface finish required to prevent cavitation eventually become impracticable, particularly in cases where a convex surface reduces the static pressure, or the boundary layers are not fully developed. Some references suggest that use of the parameter K_i for cavitation inception is not appropriate in design, because damage does not occur until the cavitation index K of the flow falls below K_i . Wang & Chou (1979) proposed that the design criterion should be $K \geq 0.8 K_i$. Field tests at Bratsk Dam (USSR) reported by Galperin et al (1977) and Oskolkov & Semenov (1979) provided values of the index K_{id} for incipient damage at chamfers angled into and away from the flow. The results are presented in Figure 3, and indicate that chamfers away from the flow have slightly higher values of K_{id} than chamfers projecting into the flow. Comparison with Equations 7 and 8 also shows that the field measurements of K_{id} are larger than the laboratory values of K_i for slopes of $n > 8$; this apparent discrepancy may be due to different definitions of the characteristic velocity used when calculating the cavitation index.

Information about the cavitation characteristics of other types of surface irregularity is provided in Appendix B.

Another factor to be considered in design is the likely duration of the cavitation attack; as the cavitation number K of the flow decreases, the safe operating time is reduced. Falvey (1983) used field data to produce Figure 2, which shows a relationship between the value of K , its duration and the amount of cavitation damage.

5 TUNNELS AND GATES

Cavitation can be a potentially serious problem in intermediate and low-level outlets in dams, and may occur at inlets to tunnels, at high-head gates, and in tunnels downstream of gates.

Convergence and curvature of the flow entering a tunnel can produce sub-atmospheric pressures, which together with the effect of turbulent fluctuations may be low enough to cause cavitation. Section C.1 in Appendix C describes some studies which give information on pressures along the boundaries of circular and elliptical entrances. However, if the flow separates in an inlet, such methods will under-estimate the likelihood of cavitation, because the lowest pressures will not occur at the boundaries but within the fluid. Separation may be caused by a poorly-designed transition, by a notch or slot, or by a secondary flow issuing from a connecting shaft.

The supports and lifting mechanisms for vertical leaf gates are normally located on the downstream side of the gate, and are accommodated in slots in the side walls so as to protect them from high velocity flow. Such slots have often been a cause of cavitation

damage. High velocity flow past a rectangular slot may produce cavitation in three ways:

1. flow separation at the upstream corner, with cavities being generated in the free shear layer and carried downstream by the flow;
2. flow separation at the downstream corner, with cavities collapsing where the flow re-attaches to the wall of the tunnel;
3. vortex formation within the slot, with possible damage to the sides and the gate supports.

The relative importance of these sources varies with the aspect ratio of the slot, and may be altered by the use of offsets and transitions.

Many studies have been made of two-dimensional flow past various shapes of slot, some of which are shown in Figure 5. The tests correspond approximately to the conditions which exist when a gate is fully open and the slot is not occupied by the lifting mechanism. Some studies have compared different shapes of slot on the basis of pressure measurements around the boundaries. However, studies carried out in cavitation tunnels are more useful and reliable, because the conditions for cavitation inception can be measured directly.

There is general agreement between studies about which types of gate slot have the lowest cavitation potential. A plain rectangular slot (Type 1A in Figure 5) is satisfactory for low heads, but Jin et al (1980) recommend that the length/depth ratio should be kept in the range $1.4 \leq L/h \leq 2.5$, and if possible between $1.6 \leq L/h \leq 1.8$ for the best performance.

Strong vortex action occurs if $L/h < 1.2$, and cavitation due to flow separation becomes serious if $L/h > 2.5$. Offsetting the wall downstream of the slot (as in Type 1B) is, by itself, not effective; the offset reduces the risk of cavitation at the downstream corner of the slot, but increases it at the upstream one. The designs which were found to have the lowest cavitation potential were slots with an offset ($t/h \approx 0.2$) and either a radiused transition (Type 4B, $100 < r/t < 250$) or an elliptical transition (Type 5A, $E/t = 5$).

Information on values of the incipient cavitation parameter K_i for gate slots of Type 1A and 1B are given by Galperin et al (1977). Separate values of K_i are calculated for the upstream and downstream corners of the slot, and take account of the width of the conduit, the aspect ratio of the slot, the amount of any downstream offset, and the relative thickness of the boundary layer. The method of determining K_i using Equation C.1 and Figure 6 is described in Section C.3 of Appendix C.

The results of Galperin et al are in reasonable agreement with the following empirical equation which Jin et al (1980) obtained for a plain rectangular slot (Type 1A):

$$K_{ir} = 0.38 (L/h) , \quad \text{for } 1.5 \leq L/h \leq 3.5 \quad (9)$$

The cavitation index is defined in terms of the average velocity and pressure just upstream of the slot, and its value relates to the slot as a whole (not to the upstream and downstream corners separately). The cavitation index K_i for a Type 3D slot was found to be related to K_{ir} for a rectangular slot of the same aspect ratio by the relation:

$$\frac{K_i}{K_{1r}} = 0.128 \left[(t/L) + \left(\frac{r/L}{30} \right)^{1/2} \right]^{-0.36} \quad (10)$$

This result was obtained for a transition slope of $n = 12$, and it was recommended that the radius should be approximately $r = 0.1h$, and the offset of the downstream corner should be in the range $0.05 \leq t/L \leq 0.08$. Equation 10 can also be used to estimate K_1 for slots of Type 3B (with $n = 12$) or 4A by putting either $r = 0$ or $t = 0$.

Although slots of Type 4B and 5A are recommended, information on their K_1 values is limited. Rosanov et al (1965) gave separate values of K_1 for the upstream and downstream corners of slots, and found that K_1 was less than 0.3 for an elliptic transition (Type 5A) of length $E = L$.

The results described above are for empty slots, but the presence of a gate rail can alter the flow conditions at the downstream corner. If a gate rail projects into the slot, the notch between the edge of the rail and the downstream face of the slot should be faired in order to prevent flow separation.

When a leaf gate is partially open, the flow past the slot becomes three-dimensional, and is influenced by the shape and proximity of the gate. The incipient cavitation number K_i of a gate is higher if it is submerged on the downstream side than if it discharges freely. Above the level of the gate lip, the lifting mechanism should, if possible, fully occupy the slot. If it does not, downward flow develops in the slot; this increases the value of K_1 , and can result in additional cavitation damage on the wall near the floor of the tunnel.

Gate lips should be designed to produce a clean flow separation without re-attachment. A lip with a smooth upstream profile produces less intense separation under submerged conditions, and reduces the risk of cavities forming in the horizontal shear layer between the high-velocity jet and the water above it. Cavitation in such shear layers can cause serious damage along walls downstream of partially-open gates.

Radial gates with attached seals have the advantage of not requiring slots. Under submerged conditions, cavitation occurs along the bottom edge of the gate, and is particularly intense at the side walls. Alternatively, radial gates may close against recessed seals mounted in offsets in the walls and floor of the tunnel. The values of K_1 for the offsets are similar to those for the upstream corners of gate slots.

High-velocity flow through small gaps and at gate seals can lead to cavitation damage. Seals should have smooth profiles in order to prevent flow separation. Gaps of more than 2mm can result in serious erosion, and the seals may themselves be damaged by vibrations induced by unstable cavity formation.

Information on the cavitation characteristics of gates tends to be specific, and model tests may be needed to investigate a particular arrangement. Galperin et al (1977) give results of several studies, details of which are summarised in Section C.3 of Appendix C.

6 ENERGY DISSIPATORS

Most types of energy dissipator produce large amounts of flow turbulence. Cavitation will occur if the velocity fluctuations are large enough to cause the static pressure to fall occasionally to the vapour pressure of the water.

Laboratory and prototype measurements of pressures beneath hydraulic jumps indicate that the maximum root mean-square (rms) values of the fluctuations are typically between 3% and 9% of the velocity head entering the jump. Using a sill to produce a forced jump shortens the distance over which the energy dissipation occurs, and tends, as might be expected, to increase the magnitude of the rms fluctuations on the floor of the basin. Flow separation behind baffle blocks and chute blocks can produce much larger variations in pressure; for example, Lopardo et al (1982) measured rms fluctuations on the rear face of a chute block equal to 27% of the upstream velocity head.

Near the toe of a jump, the positive pressure fluctuations tend to be larger than the negative ones, but further downstream the departures from the mean become more symmetrical and conform approximately to a Gaussian probability distribution. However, in zones of flow separation, the negative fluctuations may become bigger than the positive ones. Thus, for a given rms level of turbulence, cavitation is more likely behind a sill or baffle block than on a level floor.

Lopardo et al (1985) compared model and prototype data, and suggested that cavitation may occur if the pressure falls to vapour pressure for more than 0.1% of the time. This limit can be used to obtain a very approximate guide as to when cavitation might be

expected to develop on the floor of a stilling basin. Assuming an rms pressure fluctuation of 9% of the upstream velocity head, a Gaussian distribution, and a mean absolute pressure of 13m head of water, leads to a limiting velocity of about 30m/s. For sills and baffle blocks, a higher turbulence level of 27% would indicate that cavitation might occur at velocities above about 17m/s. As explained above, all these assumptions are affected by changes in the flow conditions and the configuration of the basin, so each case needs to be assessed individually.

Another factor to be considered is the favourable effect which entrained air has on reducing cavitation damage (see Section 8). Self-aeration on long spillways, the use of aerators, and entrainment at the jump itself may all contribute to reducing the danger of cavitation in stilling basins.

Chute blocks and baffle blocks are the features most vulnerable to cavitation damage in hydraulic jump basins, because they are subject to the highest velocities and produce the largest pressure fluctuations. Thus, although they allow the use of shorter basins, they are often omitted in high-head installations. To be effective, blocks need to have high drag coefficients (C_d), but this also results in high values of the cavitation inception parameter K_1 ; rounding the corners reduces K_1 but also C_d . Shapes of baffle blocks investigated by Oskolkov & Semenov (1979) and by Rozanova & Ariel (1983) are shown in Figure 7. Cavitation damage can be reduced or avoided by using a super-cavitating design which causes the flow to separate at the upstream face and form a large fixed cavity that encloses the block; damage is avoided by removing the solid surfaces from the region in which the individual cavity bubbles collapse. This can be achieved by sloping the sides of the block away

from the flow in the downstream direction and by introducing a step in the floor (see, for example, Type 1 in Figure 7).

Sudden expansions in high-head tunnels can be used to convert kinetic energy to turbulence. Cavities are liable to be formed around the perimeter of the high velocity jet, and can damage the walls of the chamber if they are too close. The performance of the expansion chamber can be affected by small changes in configuration, and model tests are normally necessary. Information from several studies is given in Appendix D, but direct comparisons of the results are difficult because the cavitation numbers were defined in a variety of ways.

7 MATERIALS

Cavitation tests carried out in the laboratory enable the relative resistances of different materials to be assessed. However, it is seldom possible to compare results from different laboratories on a quantitative basis because of variations in the types of equipment and experimental techniques used. Methods have been proposed for predicting from laboratory data the amount of erosion that will occur under prototype conditions, but they do not appear to be generally applicable. Therefore, for the present at least, it is necessary to rely on comparative tests and previous prototype experience when selecting appropriate materials for hydraulic structures.

The cavitation resistance of concrete is determined by the internal cohesion of the binder and by the adhesion between the binder and the aggregate; the strength of the aggregate itself is not usually a factor. Comprehensive laboratory tests carried out by Inozemtsev et al (1965) indicated that best results are obtained if the aggregate is porous, if the cement

and aggregate are as similar as possible, and if the aggregate reacts chemically with the cement.

Many studies have shown that cavitation resistance increases as the compressive strength M of the concrete increases; Jiang & Chen (1982), for example, found that for a given intensity of cavitation the rate of material loss was proportional to $M^{-4.84}$. Kudriashov et al (1983) presented data on allowable flow velocities over concrete; the results can be approximated by the relation:

$$V = 3.0 + 0.43 M, \text{ for } 20 \leq M \leq 50 \text{ MPa} \quad (11)$$

where V is the velocity in m/s above which cavitation damage will occur, and M is the compressive strength in MPa.

The resistance of ordinary concrete can be increased by grinding the cement to make the particles finer; this produces a denser mortar which adheres more strongly to the aggregate. A similar effect is achieved if very fine silica particles are added to sulphate-resisting portland cement. A different method of producing a dense surface finish is to cast concrete against absorptive formwork; Galperin et al (1977) mention the successful use of panels lined with timber-fibre sheets covered with dense coarse calico.

Adding steel fibres to concrete can increase its cavitation resistance by a factor of about three. Schrader & Munch (1976) describe the satisfactory use of concrete containing 1% of 25mm long steel fibres for replacing areas of ordinary concrete damaged by cavitation. The fibres help the concrete to absorb high-frequency fluid impacts without suffering fatigue failure, but the material may still be eroded by the grinding action of debris in the flow.

A similar improvement in cavitation resistance can be obtained by polymerizing concrete. The technique is described by Murray & Schultheis (1977) and Stebbins (1978), and consists of soaking an area of cured concrete with a monomer which is then polymerized by the application of heat. The method is effective in producing a good bond at joints and repairs, but considerable effort may be needed to ensure that the concrete is free of moisture before it is soaked with the monomer. Concrete containing steel fibres can also be polymerized, and this further enhances its cavitation resistance. Other examples of the use of fibrous and polymerized concretes are mentioned in Appendix E.

Practical aspects of constructing concrete structures which may be liable to cavitation are considered by Schrader (1983). Reinforcement should be designed so as to ease the placing of the concrete, because otherwise there may be a tendency to use too wet a mix. Attempts to obtain a smooth finish by overworking newly-placed concrete weaken the surface and can lead to crazing. Although it may be necessary to chamfer irregularities in order to reduce their cavitation potential (see Section 4), the grinding process can weaken the aggregate particles at the surface and allow them to be plucked out more easily by the flow; the consequent roughening of the surface may also promote cavitation downstream.

Epoxy and polyester resins have good properties of strength and adhesion, and can be applied either neat in the form of protective layers, or mixed with inert fillers to produce mortars. Epoxy mortars have been widely used for repairing or replacing areas of

concrete damaged by cavitation, but the references detailed in Section E.3 of Appendix E indicate that, in general, they have not performed well. It is possible, however, that the failures may have received more attention than the successes. Three types of problem have contributed to the failures:

1. inappropriate formulation of resin or mortar;
2. insufficient standards of control on site;
3. incompatibility of physical characteristics.

The design of a resin or mortar requires specialist knowledge, and should be tailored to the specific needs of each job; particular consideration should be given to the effect of moisture, either present naturally or generated during curing. To obtain satisfactory results on site, it is necessary to control quantities precisely, and to adopt higher standards of mixing and placing than are necessary when working with ordinary concrete. One of the main factors causing failures of repairs has been differential thermal expansion between the epoxy and the surrounding concrete, leading to failure of the concrete beneath the joint and subsequent loss of the epoxy patch. Other problems have been caused by epoxy and concrete having different surface textures, and by the tendency for an epoxy patch to project above the surrounding concrete as a result of the greater hardness of the epoxy. In the case of mortars, some of these problems can be reduced by suitable choices of filler.

The addition of a relatively small amount of polymer to concrete can increase its cavitation resistance

considerably. Test data given by Inozemtsev et al (1965) and Galperin et al (1977) showed that the resistance of plastic concretes was 10-100 times that of normal cement concrete; an epoxy-thiokol plastic concrete had a performance similar to that of steel.

Steel linings are often used downstream of gates in high-head tunnels, where the boundary layers have not developed sufficiently to protect the walls from high velocity flows. Information from several sources is presented by Knapp et al (1970) on the comparative resistances of different metals to cavitation damage; a representative selection of the data is given in Section E.2. The resistance of alloyed steels can vary widely, depending upon the chemical content and whether they are forged, cast or rolled. Cavitation can also accelerate the corrosive effects of water, perhaps by stripping the protective oxide layer away from the surface of the metal.

Information on the length of steel lining needed downstream of a gate or orifice is limited, but an ICOLD Committee (1986) recommended, for flow velocities exceeding 25m/s, the following distances:

floor - 50 R

full wetted height of side walls - 15 R

half wetted height of side walls - 30 R

where R is the hydraulic radius of the orifice or gate opening. The use of steel to armour chute blocks and baffle blocks in stilling basins has not, in general, proved successful because of the difficulty of fixing.

Several types of protective lining for concrete or steel have been tested, but most suffer from inadequate bond. Abelev et al (1971) found that a

layer of nyrite applied to carbon steel significantly reduced the amount of erosion by cavitation. Wagner & Jabara (1971) reported that, in US Bureau of Reclamation experience, a neoprene compound was found to be the only suitable coating material; however, it required careful application in a large number of thin coats.

8 AERATION

8.1 Self-aeration

Laboratory studies and prototype experience have shown that the presence of air in water can reduce or eliminate cavitation damage. The concentration of air needed to prevent damage was found by Peterka (1953) and other researchers (see Section F.1 of Appendix F) to be about 7-8%. As a result of these laboratory tests, it has generally been assumed that an air concentration of at least 7-8% is required adjacent to prototype structures in order to protect them against cavitation. However, experiments carried out by Clyde & Tullis (1983) on orifices in pipes indicate that the limiting air concentration necessary to prevent cavitation may be subject to significant scale effects; for a given orifice ratio, it was found that increasing the pipe size or decreasing the flow velocity both served to reduce the limiting air concentration (for details see Section G.2 in Appendix G). Such scale effects could have an important bearing on the design of aerators (see later), because their size and spacing are often determined by the requirement to produce a certain minimum air concentration.

Air can be entrained by turbulence at the surface of high-velocity flows. The buoyancy of the air bubbles tends to be counteracted by the fluid turbulence, and this can cause them to diffuse downwards as they are

carried along by the flow. The floor of the channel will be protected from possible cavitation damage if this self-aeration process produces a sufficient concentration of air at the bed.

There is general agreement that self-aeration begins on a spillway at a point where the boundary layer has grown sufficiently for its thickness to be nearly equal to the depth of flow. Theoretical and experimental results obtained by Wood et al (1983) and Wood (1985) can be combined to produce the following equation for the distance L_i to the point of inception of air entrainment:

$$L_i = 37.3 \left[\frac{g^{0.5} H_s^{0.39} k_s^{0.10}}{q} \right]^{-1/1.01} \quad (12)$$

The distance L_i is measured along the spillway from the crest; g is the acceleration due to gravity, q is the discharge per unit width, k_s is the Nikuradse sand roughness of the channel, and H_s is the vertical distance from the reservoir level to the water surface in the channel. Prototype measurements of the inception distance on high-head spillways are given by Galperin et al (1977); values of L_i varied from 30m at a unit discharge of $q \approx 4.2 \text{ m}^3/\text{s}/\text{m}$ to 100m at $q = 18.5 \text{ m}^3/\text{s}/\text{m}$.

The growth of the boundary layer is not the only factor governing the start of aeration, because the entrainment process requires the flow to have sufficient turbulent energy at the free surface to overcome the effects of surface tension. Several investigators have produced criteria for describing the conditions at the onset of aeration, and these are listed in Section G.2 of Appendix G. Three of the criteria are expressed in terms of the Froude number of the flow, and indicate that entrainment will begin

if the value is greater than about $F = 5-6$. The physical significance of the Froude number in determining the start of aeration is not clear, but its use appears justified because both model and prototype data indicated similar limiting values of F .

The concentration of air in the flow increases with distance downstream of the inception point, and eventually reaches an equilibrium value, provided the channel is long enough and is of constant slope. Various formulae have been developed for estimating the depth-averaged equilibrium air concentration $\bar{C}$, and details of these are given in Section F.2 of Appendix F. The equations have widely differing forms, and can therefore only properly be compared on the basis of independent prototype measurements, which were not available for this review. In the absence of such data, it is suggested that estimates of $\bar{C}$ for spillways be calculated from several of the formulae (e.g. Equations F.6, F.7, F.16, F.19, F.24, and the data of Wood (1983) tabulated in Section F.2), and compared to establish a "likely" value. For air entrainment in steep partially-filled pipes, the only equation for $\bar{C}$ appears to be that due to Volkart (1982), Equation F.21; this result was obtained using both model and prototype data. It should be noted that some researchers have defined concentration in terms of the volumes of air and water ($\bar{C}_1$), and others in terms of their rates of flow ($\bar{C}_2$), see Equations F.4 and F.5; in cases where the quantity was not precisely defined, the symbol $\bar{C}$ has been used in Appendix F.

An analysis by Wood (1983) of laboratory results obtained by Straub & Anderson (1958) indicated that the vertical distribution of air at a point along a channel is determined only by the local value of the

mean air concentration $\bar{C}$ at that point; this finding applies at all points and not just far downstream where the flow has reached an equilibrium state. The results show that in order to obtain an air concentration at the bed of 7% (so as to avoid possible cavitation damage), the mean air concentration needs to be about 30%; such a figure will not be achieved if the slope of the channel is less than about 22.5° .

Many spillways are not long enough for the aerated flow to reach an equilibrium state. Numerical models for determining the developing region of air entrainment have been developed by Wood (1985) and by Ackers & Priestley (1985), and have been calibrated against laboratory and prototype data (for unit discharges of up to $3.2\text{m}^3/\text{s}/\text{m}$). Details of the models are given in Section F.2 of Appendix F.

The research that has been carried out on self-aeration indicates that, in favourable circumstances, enough air can be entrained to prevent cavitation damage. However, the distance required for air to reach the bed of a channel increases rapidly with increasing discharge. The mechanism may therefore provide protection at low unit discharges (e.g. $< 5\text{m}^3/\text{s}/\text{m}$), but not the larger flows for which most spillways are designed. However, all cases should be investigated on an individual basis in order to estimate the likely effects of self-aeration.

8.2 Aerators on spillways

If the tolerances on the surface finish required to avoid cavitation are too severe to be practicable, and there is not enough self-aeration, possible damage to a channel may be prevented by using an aerator to

supply air around the perimeter. The air can be pumped under pressure, but nearly all aerators work by creating a suction which is used to draw the air naturally from the atmosphere. Such aerators consist of an offset or deflector which causes the flow to separate from the surface of the channel and form a large air cavity. The water passing over the cavity entrains air strongly, and thereby produces the necessary sub-atmospheric pressure.

Typical features of aerators are shown in Figure 8, and can comprise deflectors, offsets, notches or slots, either singly or in combination. Deflectors tend to produce strong aeration, but may disturb the flow considerably. An offset causes less disturbance, but needs to be larger than a deflector in order to provide the same air demand. If an existing structure requires modifications to prevent cavitation damage, it is usually easier to incorporate a deflector than an offset. Means of supplying air to an aerator include ducts discharging at the base of the side walls or at points across the floor of the channel. Alternatively, deflectors and offsets in side walls can be added so as to allow air to reach aerators located in the channel floors; similar use can also be made of piers and walls with blunt ends which create vertical separation pockets in the flow. Some examples of these types of arrangement are shown in Figure 9.

The requirements of an effective aeration system are that:

1. its air demand should be sufficient to give local air concentrations at the channel boundaries that are high enough to prevent cavitation damage (e.g. $C > 7\%$);

2. the air cavity produced by the device should remain stable over the full range of operating conditions and should not tend to fill with water;
3. the aerator should not produce too great a disturbance of the flow or an excessive amount of spray;
4. the spacing between successive aerators should be such that the local air concentration at the floor does not fall below the amount required to protect against cavitation damage.

Model and prototype data obtained in a series of studies by Pinto (1979), Pinto et al (1982) and Pinto & Neidert (1982, 1983a) have helped to identify the factors which determine the amount of air entrained by an aerator. The most important are the length L_c of the air cavity (measured from the aerator to the point where the flow re-attaches), and the velocity V of the water just upstream of the aerator. The studies showed that the rate of air demand (q_a) per unit width of channel can be described by the equation:

$$q_a = k VL_c \quad (13)$$

The value of the non-dimensional coefficient k depends upon the geometry of the aerator, and on several other flow parameters which are detailed in Section F.3 of Appendix F. One of the most important of these is the amount Δp by which the pressure in the air cavity is below that at the free surface. For a given air demand, the pressure difference Δp is determined by the head-loss characteristics of the air supply system. However, Δp itself helps to determine the air

demand because it affects the value of k in Equation 13 and also the length of the air cavity. Therefore, when considering the performance of an aerator, it is always necessary to take the particular characteristics of the air supply system into account.

Despite the interactions between these various factors, it appears that Equation 13 may still provide a useful basis for determining the performance of a given aeration system. Pinto et al (1982) obtained model and prototype data for aerators at Foz do Areia Dam (Brazil), and found that the values of k remained approximately constant over a six-fold range of water discharges. For air supplied laterally from both sides of the channel the value was $k = 0.033$, and for supply from one side only it was $k = 0.023$. Independent confirmation of the validity of Equation 13 was provided by Pan et al (1980), who obtained fairly similar values of k using theoretical and experimental results. However, each design of aerator needs to be considered on an individual basis, because the value of k may vary considerably according to the particular characteristics of the system.

Analytical or empirical methods of determining the length of air cavity formed by an aerator have been developed by several researchers (see Section F.3). The equations are valid only for two-dimensional flows in channels of constant slope. The analytical solutions contain various simplifying assumptions, but the one obtained by Schwarz & Nutt (1963) has an advantage in that it takes account of the pressure difference Δp between the upper and lower surfaces of the nappe. Numerical solutions of Laplace's equation have been used to determine trajectories at aerators (e.g. Wei & De Fazio (1982)), and such techniques are

capable of allowing for three-dimensional effects and channel curvature. Analytical and numerical methods do not take account of air resistance and turbulence, and may therefore tend to over-estimate the length of the air cavity.

Dimensions and characteristics of some aerators which have been used in prototype installations are given in Table 3. Prusza et al (1983) recommend that the mean air concentration produced by an aerator should be limited to $\bar{C} = 40\text{--}50\%$ in order to prevent atomisation of the flow; at this limit the length of the cavity will be about 3-5 times the water depth. Values of the pressure difference Δp for aerators supplied by air ducts are typically between 0.5m and 2.0m head of water. High air velocities in ducts supplying aerators should be avoided, because they can cause objectionable noise; Falvey (1980) recommends maximum velocities of 30m/s for continuous operation, and 90m/s for short durations. The required spacing between successive aerators is determined by the rate at which the local air concentration near the floor of the channel decreases with distance. Prototype data from several Russian dams (see Section F.3) suggest that, in a straight channel, the mean air concentration decreases at a rate of between 0.2% and 0.8% per metre; in channels with convex curvature, the loss rate can increase to 1.5% per metre due to the effects of centripetal pressure. Distances between aerators are typically in the range 30-100m.

Prototype data obtained by Pinto (1986) for the Foz do Areia spillway indicate that factors not highlighted by model tests may contribute to the effectiveness of aerators in preventing cavitation damage. Measurements of flow depths along the channel showed that considerable entrainment occurred at the aerators, but that only a small proportion of the air

(of the order of 25% or less) was supplied directly by the aerators. The remainder was entrained at the surface as a result of the strong turbulence created in the flow by the presence of the aerators. Results such as these suggest that a more efficient method of preventing cavitation damage might be to use smaller but more closely-spaced devices that cause less disturbance to the flow.

8.3 Tunnels

Aerators are often located immediately downstream of gates in high-head tunnels in order to protect the walls and floors from cavitation damage, and these operate in a similar way to aerators in spillways. Ducts may be used to supply air to an offset in the floor or, for example, to the seating of a radial gate with recessed seals. For tunnels flowing partly full, a more common arrangement is to form, just downstream of the gate, a vertical U-shaped slot in the walls and invert so as to allow air from above the water surface to reach the invert.

Recommendations on the design of aerators for tunnels are given by Beichley & King (1975) as follows:

1. Offsets in the wall and floor are normally preferable to deflectors and air slots;
2. Deflectors may be the only option when modifying an existing structure;
3. Offsets at the floor and at the side walls should be respectively $1/6$ and $1/12$ of the frame width of the gate (with a minimum of 100mm);

4. Wall deflectors need to be used in conjunction with air slots if the downstream sides of the tunnel are parallel;
5. Air slots should be square in cross-section, and a size of 300mm x 300mm should be adequate for gates measuring up to 1.2m x 2.3m with heads of up to 100m.

Further details are given in Section F.4 of Appendix F. A potential problem that can arise with aerators in tunnels is that, at the walls, they can produce fins of water which may be large enough to seal the conduit. To avoid this effect it may be necessary to limit the size of the offsets or deflectors.

High-velocity water flowing in a tunnel can draw large quantities of air along with it. If this "natural" air demand is not satisfied, the ambient pressure downstream of the gate may be reduced significantly below atmospheric (increasing the risk of cavitation), and undesirable surging may also occur. In large tunnels the necessary air is often supplied by a system of ducts or galleries connecting the downstream side of the gate to the atmosphere. Use of an aerator creates an additional "forced" demand which can normally be met by the same supply system.

It is important, when considering the "natural" air demand, to distinguish cases where a tunnel downstream of a gate flows part-full over its full length from those where the tunnel is sealed by a hydraulic jump; in the latter cases the air flow is determined by the amount of entrainment in the jump and by the capacity of the flow to transport the bubbles of air along the tunnel.

Many researchers have fitted data on the "natural" air demand in tunnels to an equation of the form:

$$\beta = \frac{Q_a}{Q_w} = a (F_c - 1)^m \quad (14)$$

where F_c is the value of the Froude number at the vena contracta downstream of the gate. Values of β given by some of the resulting equations are plotted in Figure 10, and it can be seen that the predictions vary considerably. In general, it is found that tunnels flowing freely produce higher air concentrations than tunnels sealed by hydraulic jumps. Also, it appears that prototype values of β are somewhat higher than those measured in equivalent models. Without a close study of the original data, it is difficult to identify the reasons for the discrepancies. In the interim, air concentrations for prototype tunnels with jumps might be estimated from the US Army Corps of Engineers (1964) equation (with $a = 0.03$ and $m = 1.06$ in Equation 14). However, it should be borne in mind that the results of a few studies would suggest somewhat higher values of β (for details, see Section F.4 of Appendix F). For tunnels flowing freely, Sharma's (1976) equation

$$\beta = 0.09 F_c \quad (15)$$

might be used.

At small gate openings, spray-type flow may occur, and this can give rise to large values of β . However, since the discharge of water is low under these conditions, the total air flow will generally be less than at larger gate openings.

If an aerator is used in a gated tunnel, the additional air demand that it creates should be

assessed separately. The air supply system should be sized to cater for the combined "natural" and "forced" air demands.

9 MODELLING

Studies of cavitation can be carried out at a reduced scale in three main ways. Firstly, a model may be operated at atmospheric pressure according to the Froudian scaling law. Pressures along the boundaries of the flow are measured and scaled to prototype conditions. Cavitation is predicted to occur if the scaled pressure at a point reaches the vapour pressure of water. The pressure tappings should be located so as to identify the points of minimum pressure, and account should be taken of both the mean and fluctuating pressure components. The method will under-estimate the likelihood of cavitation if flow separation occurs, because the lowest pressures will be located in the body of the fluid and not at the boundaries.

The second kind of test is carried out in a cavitation tunnel, in which the pressure in the working section is reduced below atmospheric so as to obtain equal values in model and prototype of the parameter K defined in Equation 2. This method enables the occurrence of cavitation in the model to be detected directly, and is suitable for both separating and non-separating flows. Since the working section flows full, the technique is not appropriate where free-surface effects are important (e.g. at baffle blocks in stilling basins). Having equal values of K in model and prototype does not necessarily ensure complete dynamical similarity, and model results may still be subject to some scale effects.

The third way of studying cavitation is to use a vacuum test rig in which the air pressure can be

reduced below atmospheric. This allows models with free-surface flows to be operated at prototype values of K . In general, vacuum rigs provide the best means of carrying out cavitation tests, but they can be expensive to construct and difficult to operate.

Results from model studies of cavitation can be affected by the pressure, velocity and scale at which the tests are carried out. Several investigators have found that values of the incipient cavitation index K_i tend to increase with increasing size of model, but there is conflicting evidence concerning the effects of changes in pressure and velocity (for details, see Section G.1 of Appendix G). Other factors which can be significant are the gas and dust contents of the water, and the number and size of the nuclei that it contains. These factors influence the value of the critical pressure p_c at which cavities begin to grow; as explained in Section 2.2, p_c is usually close to but not equal to the vapour pressure p_v of the water. Keller (1984) developed a laboratory technique for measuring p_c , and showed that water samples of different qualities gave consistent values of K_i if these were calculated using p_c instead of p_v . Use of this technique would allow data from different studies to be standardised, and would enable scale effects to be identified more precisely. However, in order to apply the laboratory results to prototype conditions, it will be necessary to determine values of the critical pressure for typical prototype flows.

The fact that water will not entrain air unless the velocity and turbulence of the flow are great enough demonstrates clearly that prototype air demands can be under-estimated by models which are too small. However, it is necessary to distinguish between air which is entrained into the flow by turbulence and air which is drawn along above the free surface. The

former is relevant to self-aeration and the performance of aerators; the latter can account for a significant proportion of the total air demand in a tunnel flowing part-full.

Complete models of spillways are not suitable for predicting self-aeration because it is not possible to scale the inception lengths correctly, and because the velocities are not usually high enough. Numerical models based on prototype data, such as those developed by Wood (1985) and Ackers & Priestley (1985), offer a better means of estimating the amount of self-aeration (see Section F.2 of Appendix F).

Large-scale sectional models of aerators in spillways have been used to determine their hydraulic performance and to estimate their air demands. Sectional models are necessary because of the limited pumping capacity available in most laboratories, but allowance may need to be made for the extra resistance and entrainment produced by the side walls. Tests of similar models at different scales, and comparisons between model and prototype data, indicate that reasonable estimates of air demand can be obtained from a model if its scale is 1:15 or larger (see Section G.2 of Appendix G for examples), and if the flow velocity in the model exceeds about 6-7m/s. However, for such a model to give reliable results, it must also reproduce correctly the head-loss characteristics of the air supply system in the prototype. If the sizes of the air ducts have not been determined at the time that the model study is carried out, the aerator should be tested for a range of possible head-loss characteristics.

Numerous model studies have been carried out to predict air demands in gated tunnels, and comparisons with prototype measurements suggest that scales of

1:25 or larger will give satisfactory results (see Section G.2 for examples). However, it is again important that all the air and water passages should be correctly reproduced in such models. Some laboratory studies of air entrainment in tunnels flowing freely have indicated that Froudian scaling is inappropriate (see Section F.4); nevertheless, several Froudian model studies have shown reasonable agreement with prototype air demands.

Measurements of two-phase flows are difficult, and most rely on indirect methods, e.g. the variation in electrical current caused by the passage of air bubbles or water droplets. In order to interpret such signals, it is usually necessary to make assumptions about the behaviour of two-phase flows that are difficult to verify. Apparent discrepancies between the results of different studies may thus be due to instruments having different operating characteristics. Examples of devices used to measure velocities and air concentrations in aerated flows are described in Section G.3 of Appendix G.

10 CONCLUSION

This review has indicated the very considerable amount of work that has been carried out on cavitation and aeration in hydraulic structures. The research has identified the principal factors involved in both problems, although the physical processes underlying them are still imperfectly understood. Due to the complexities, it has not been possible to plan many experimental studies within a firm theoretical framework. Inevitably, therefore, the results sometimes disagree, and lead to empirical equations which link the various factors in different ways. This tends to make it difficult to give designers hard-and-fast rules concerning the occurrence of cavitation and methods of preventing it. Nevertheless, there are areas of broad agreement, and

in several of the preceding sections it has been possible to draw general conclusions which may be of use in design.

Differences between results from studies of a particular problem can be viewed in several ways. Are they due to shortcomings in some of the experiments? Can they help to explain the physical processes involved? Are they significant in terms of practical application?

A good example is provided by the tests which have been carried out to determine the cavitation potential of surface irregularities. Detailed comparisons for a given shape of irregularity show that differences can be caused by scale effects, and by variations in turbulence, boundary layer thickness and water quality. If these factors can be quantified and explained, a better understanding of the fundamental processes will have been obtained. However, such differences may not be very large compared with the effects produced by small changes in shape.

Construction faults in hydraulic structures cannot be predicted accurately in advance, and their shapes will seldom conform precisely to those tested in the laboratory. Therefore, from the point-of-view of designers, present knowledge may be sufficient to enable them to assess the risks of cavitation with reasonable accuracy.

Aerators have proved an effective means of reducing or preventing cavitation damage in high-head spillways and gated tunnels. However, our understanding of air entrainment is less advanced than that of cavitation inception. As a result, it is at present difficult to predict the performance of a prototype aerator theoretically, or to scale results from a physical model reliably. Well-planned research on the

behaviour of aerators is therefore likely to lead to worthwhile improvements in the design of such structures. Detailed recommendations for research on each of the main topics covered in this review are given in Appendix H.

11 ACKNOWLEDGEMENTS

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T A B L E S

TABLE 1: Properties of Pure Water

Temperature °C	Density kg/m ³	Kinematic Viscosity m ² /s	Vapour Pressure		Surface Tension (water/air) N/m
			N/m ²	mm head of water	
0	999.9	1.787 x 10 ⁻⁶	610	62	0.0757
5	1000.0	1.514 x 10 ⁻⁶	870	89	0.0749
10	999.7	1.304 x 10 ⁻⁶	1230	125	0.0742
15	999.1	1.138 x 10 ⁻⁶	1700	174	0.0735
20	998.2	1.004 x 10 ⁻⁶	2330	238	0.0728
25	997.1	0.894 x 10 ⁻⁶	3160	323	0.0720
30	995.7	0.802 x 10 ⁻⁶	4230	433	0.0712
35	994.1	0.725 x 10 ⁻⁶	5600	574	0.0704
40	992.3	0.659 x 10 ⁻⁶	7400	760	0.0696

Source: Batchelor (1967)

TABLE 2: Values of K_1 for Surface Irregularities
From: Ball (1963)

Type	h (mm)	r/h	n	K_1
1A	0.8	-	-	0.36
	1.6			0.84
	3.2			1.37
	4.8			1.75
	6.4			2.00
	7.9			2.17
	9.5			2.34
	12.7			2.42
1B	6.4	0.50	-	1.59
	9.5	0.33		2.01
	9.5	0.67		1.44
1C	3.2	1.00	-	0.92
	6.4	1.00		1.02
	9.5	1.00		1.12
3A	-	-	1	1.83
			2	1.22
			4	0.72
			8	0.43
			12	0.33
			16	0.27
			20	0.23
			30	0.17
			50	0.12
			100	0.07

TABLE 3: Data on prototype aerators

A. SPILLWAYS

DAM	COUNTRY	REFERENCE	NET WIDTH OF CHANNEL B (m)	MAXIMUM DISCHARGE Q (m^3/s) (m^2/s)		NO OF AERATORS OF SAME TYPE	DEPTH BELOW RESERVOIR LEVEL H_s (m)	CHANNEL SLOPE θ (deg)	RAMP SLOPE ϕ (deg)	RAMP HEIGHT $h, (h_1)$ (m)	OFFSET DEPTH $u, (u_1)$ (m)	GROOVE DEPTH $c, (c_1)$ (m)	SLOT DEPTH $v, (v_1)$ (m)	HORIZ SPACING OF AERATORS L_h (m)	METHOD OF AIR SUPPLY
Alficura	Argentina	Volkart et al (1984a)	39	3000	77	4	48-114	19.3	*	0.150	-	1.20	-	63	Lateral ducts
Bratsk	USSR	Volkart et al (1984a)	60	6050	101	1	30	51.3	11.3	(0.450)	-	-	-	-	Separation d/s of pier
Emborcacao	Brazil	Pinto (1984)	*	7800	*	1 1	55 75	10.2 10.2	7.1 7.1	0.300 0.200	- -	* *	* *	- ~ 100	Slot in floor Slot in floor
Foz do Aria	Brazil	Pinto (1984) Volkart et al (1984a)	70.6	11000	156	1 1 1	60 79 102	14.5 14.5 14.5	7.1 7.1 7.1	0.200 0.150 0.100	- - -	1.300 1.350 1.400	- - -	- 72 90	Lateral ducts Lateral ducts Lateral ducts
Guri	Venezuela	Marcano et al (1984) Volkart et al (1984a)	*	*	250	1 1 1 1	59 * * *	51.3 15.5 15.5 22.2	0.9 8.9 17.0 15.4	0.100 0.400 0.750 0.400	0.210 - - -	1.790 - - -	- - - -	- - - -	Transverse ducts in offset Separation d/s of pier Separation d/s of pier Wall wedges and wall slots
Karakaya	Turkey	Volkart et al (1984a)	140	17000	121	1 1	29 85	59.7 59.7	1.1 *	0.150 0.200	- -	1.500 1.000	- -	- 33	Lateral ducts Slot in floor
King Talal	Jordan	De Fazio et al (1983)	*	*	62	1 1	44 86	19.3 19.7	- 7.8	- 0.740	0.700 -	0.700 -	- -	- ~ 200	Wall wedges Wall wedges
Manila Water Supply III	Philippines	Volkart et al (1984a)	25	3000	120	1 1 1 1	55 70 87 89	20.3 20.3 1.7 1.7	5.2 6.5 4.9 6.5	0.200 0.250 0.300 0.400	- - - -	1.258 1.258 - -	- - 1.400 1.400	- 40 60 60	Lateral ducts Lateral ducts Floor slot Floor slot
Restitucion (Mantaro I.3)	Peru	Volkart et al (1984a)	4	96	24	7	24-205	41.5-30.9	*	0.060	-	0.700	-	18-55	Lateral ducts
San Roque	Philippines	Eccher et al (1982)	105	12800	122	7	61-126	14.0-7.7	9.6	0.500	0.750	-	-	50,60	Transverse ducts in offset
Souapiti	Guinea	Dejoux et al (1983)	80	4500	56	1 1	54 64	12.4 12.4	5.7 5.7	(0.200) (0.130)	- -	- -	* *	- 46	Slot in floor Slot in floor
Toktogul	USSR	Oskolkov et al (1979) Volkart et al (1984a)	*	2340	*	1 1	44 114	54.5 ~ 17	9.5 9.5	0.150 0.070	2.000 0.280	- -	- *	- ~ 65	Separation d/s of pier Slot in floor
Ullum	Argentina	De Fazio et al (1983)	*	*	73	1	~ 50	14.0	14.0	1.000	-	-	-	-	Wall wedges
Uribante	Venezuela	Coleman et al (1983)	*	1100	*	1 1	~ 35 ~ 90	7.1 21.8	4.7 5.0	0.250 0.300	- -	0.400 0.985	- -	- 153	Lateral ducts Lateral ducts
Ust-Ilim	USSR	Volkart et al (1984a)	*	9700	*	1	30	55.0	9.5	(0.600)	-	-	-	-	Lateral ducts

Notes: - Not applicable

* Not known

TABLE 3: (CONT'D)

B. TUNNELS AND BOTTOM OUTLETS

DAM	COUNTRY	REFERENCE	TUNNEL WIDTH B (m)	TUNNEL DIAMETER D (m)	MAXIMUM DISCHARGE Q (m ³ /s)	NO OF AERATORS OF SAME TYPE	DEPTH BELOW RESERVOIR LEVEL H _s (m)	CHANNEL SLOPE θ (deg)	RAMP SLOPE φ (deg)	RAMP HEIGHT h, (h ₁) (m)	OFFSET DEPTH u, (u ₁) (m)	GROOVE DEPTH t, (t ₁) (m)	SLOT DEPTH v, (v ₁) (m)	HORIZ SPACING OF AERATORS L _h (m)	METHOD OF AIR SUPPLY
Calacuccia	France	Quintela (1980)	2	-	100	Several	≥ 61	4.2	-	-	0.070	-	-	10	-
Colbun	Chile	Montero et al (1986)	*	-	730	1 2	108 ~ 110	* *	- 5.7	- 0.300	0.600 0.500	- -	- -	- 117	Wall slots Wall offsets; wall wedges and slots
Fengjiashan	China	Pan et al (1984)	7.2	-	1140	1 1	57 ~ 65	26.6 0.9	3.8 7.1	(0.600) 0.175	- *	- ~	(0.600) -	- ~ 50	Floor slot Lateral ducts
Libby	U S A	McGee (1984)	3 x 3.0	-	*	3	*	~ 12	3.2	(0.038)	-	-	(0.762)	-	Wall slots
M'Dez	Morocco	Billore et al (1979)	-	7	500	1	*	*	-	-	1.100	-	-	-	*
M'Jara	*	Quintela et al (1979)	-	11/10	*	1	*	*	-	0.200	1.100	-	-	-	Wall offsets and corner fillets
Mica	Canada	Quintela (1980)	-	11	1000	1 1	61 191	~ 0 *	- -	- -	2.75 4.50	- -	- -	- ~ 210	Air shaft Ventilation gallery
Navajo	U S A	Beichley et al (1975)	1.270	-	51	1	96	~ 0	6.3	0.051	-	-	-	-	0.305m square wall slots
Palisades	U S A	Beichley et al (1975)	6 x 2.29	-	6 x 184	6	72	~ 0	4.8	0.064	-	-	-	-	Wall wedges and 0.305m square slots
Pueblo	U S A	Beichley et al (1975)	3 x 1.83	-	3 x 87	3	40	~ 0	-	-	0.305	-	-	-	0.152m deep wall slots
Tarbela (Tunnel 3)	Pakistan	Lowe et al (1979) Pinto (1984)	2 x 5.64	-	2690	2	120	1.5	7.3	0.137	-	-	*	-	Lateral ducts
Yellowtail	U S A	Colegate (1971) Hamilton (1984)	-	~ 10	> 425	1	103	55.0	6.3	(0.076 max)	-	-	0.914	-	Horseshoe-type tapering slot

Notes: ~ Not applicable
* Not known

FIGURES

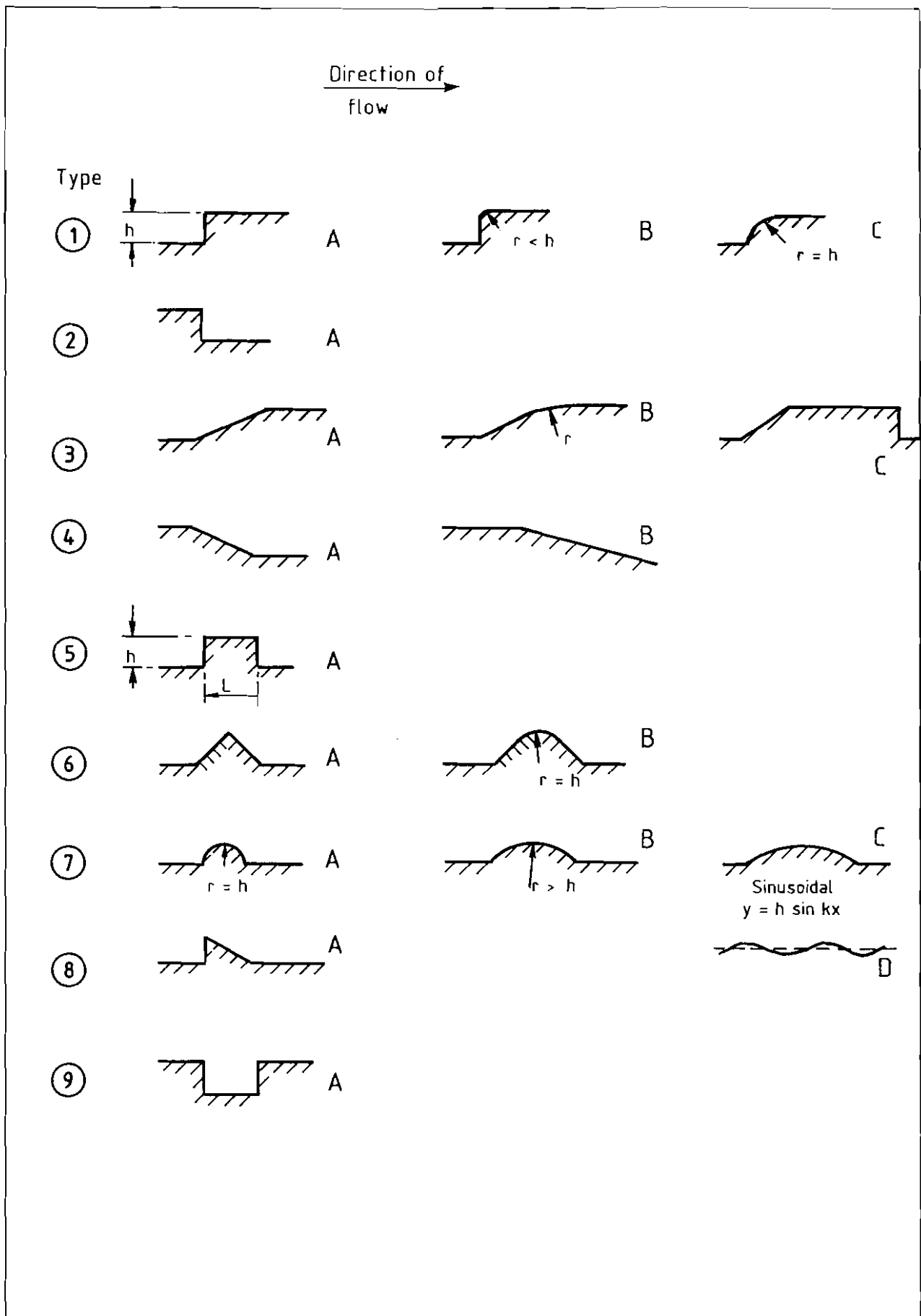


Fig 1 Types of surface irregularity

From : Falvey (1983)

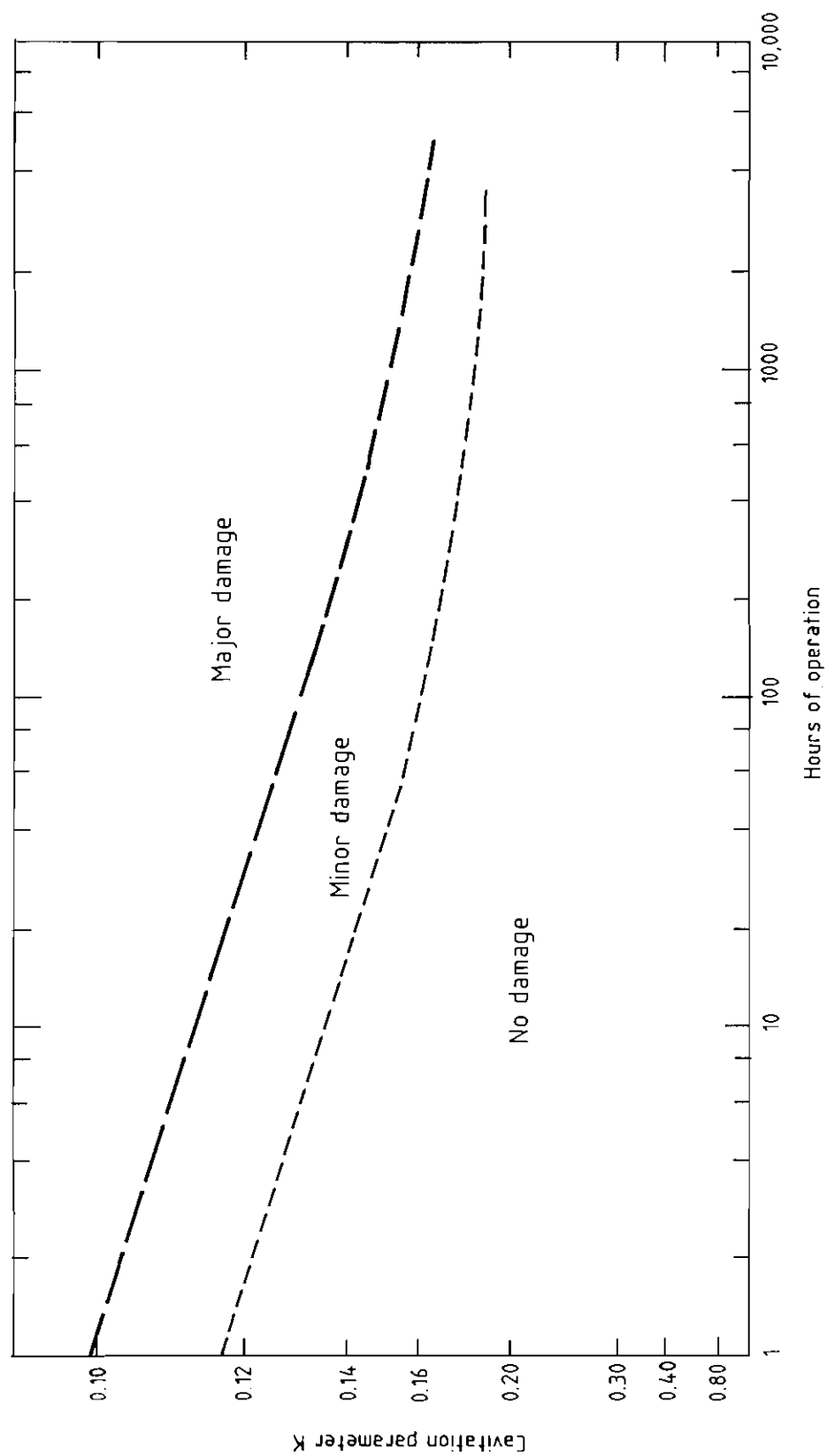


Fig 2 Cavitation damage curve

From : Oskolkov & Semenov (1979)

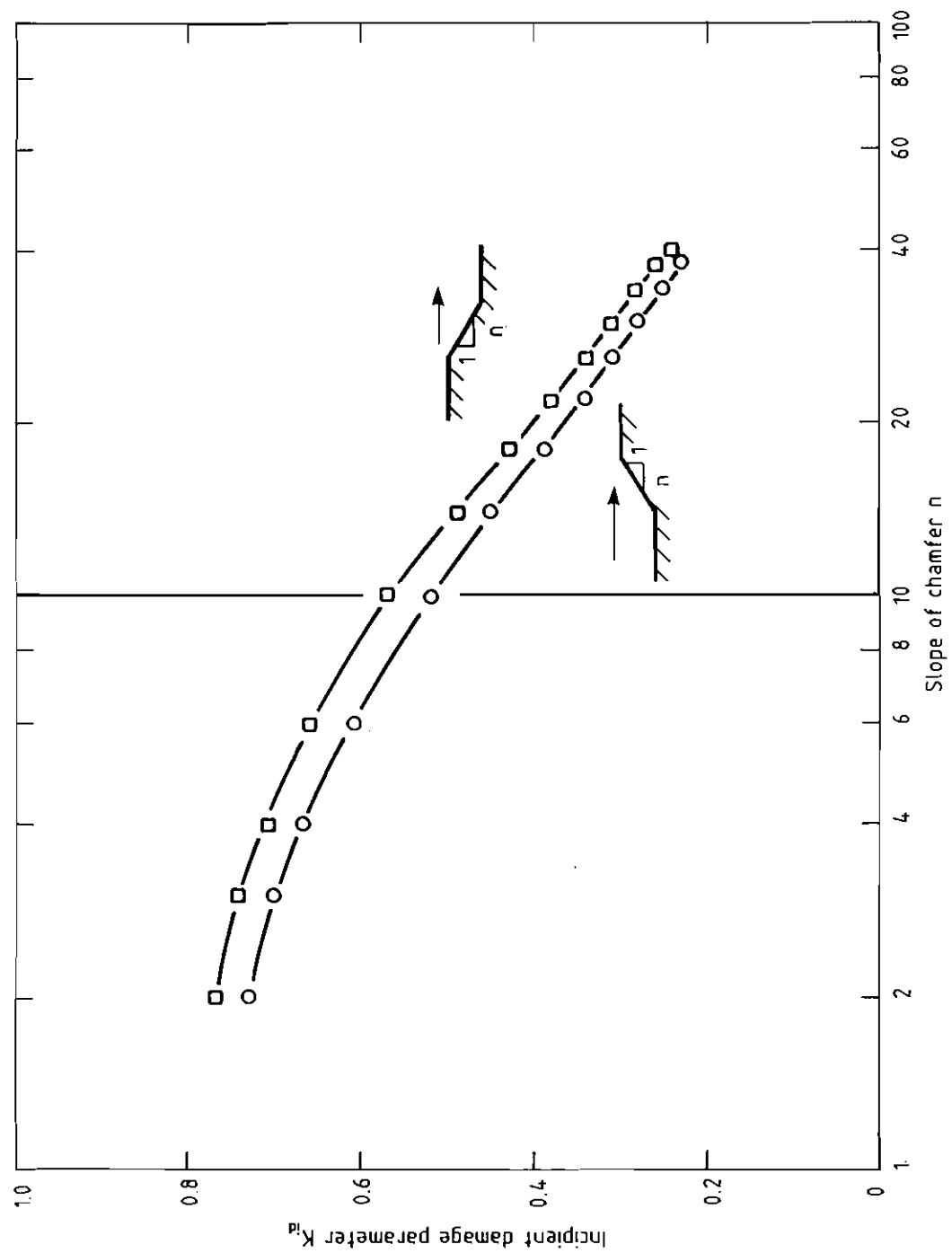


Fig 3 Values of K_{id} for surface irregularities

From : Wang & Chou (1979)

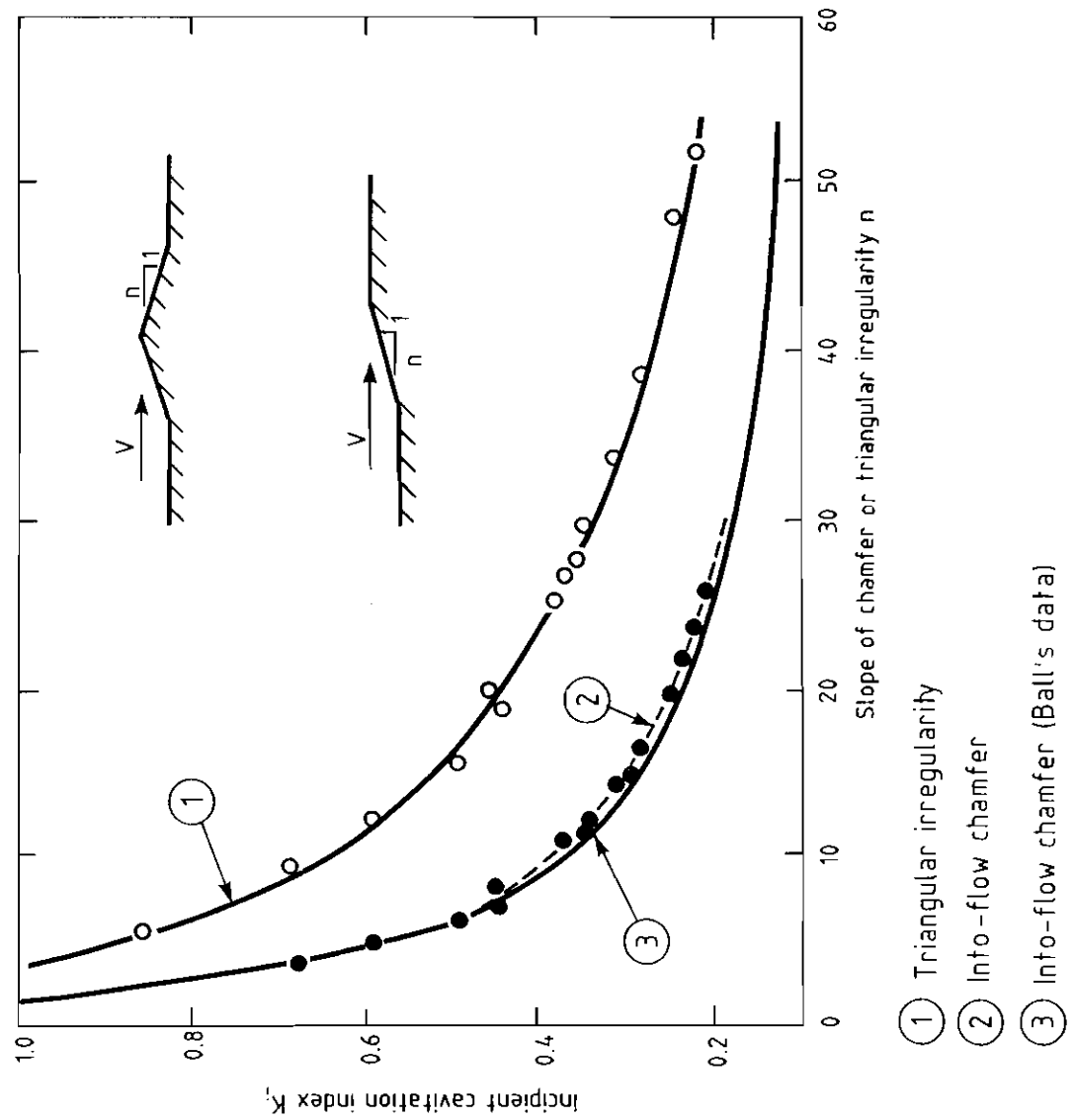


Fig 4 Values of K_i for surface irregularities

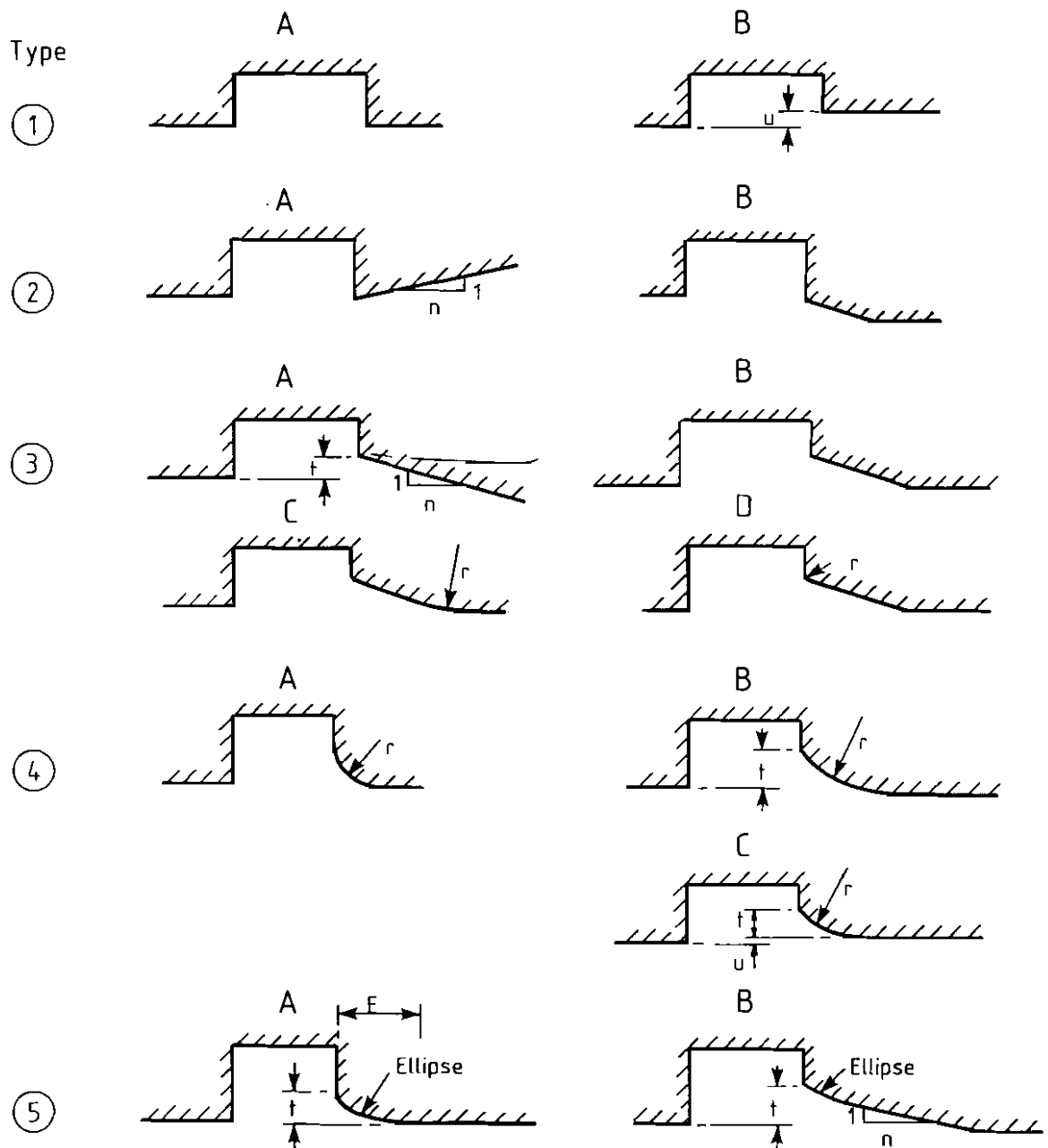
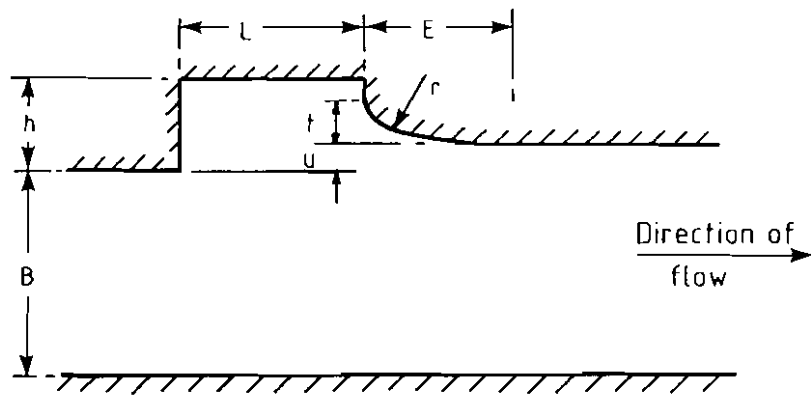


Fig 5 Types of gate slot

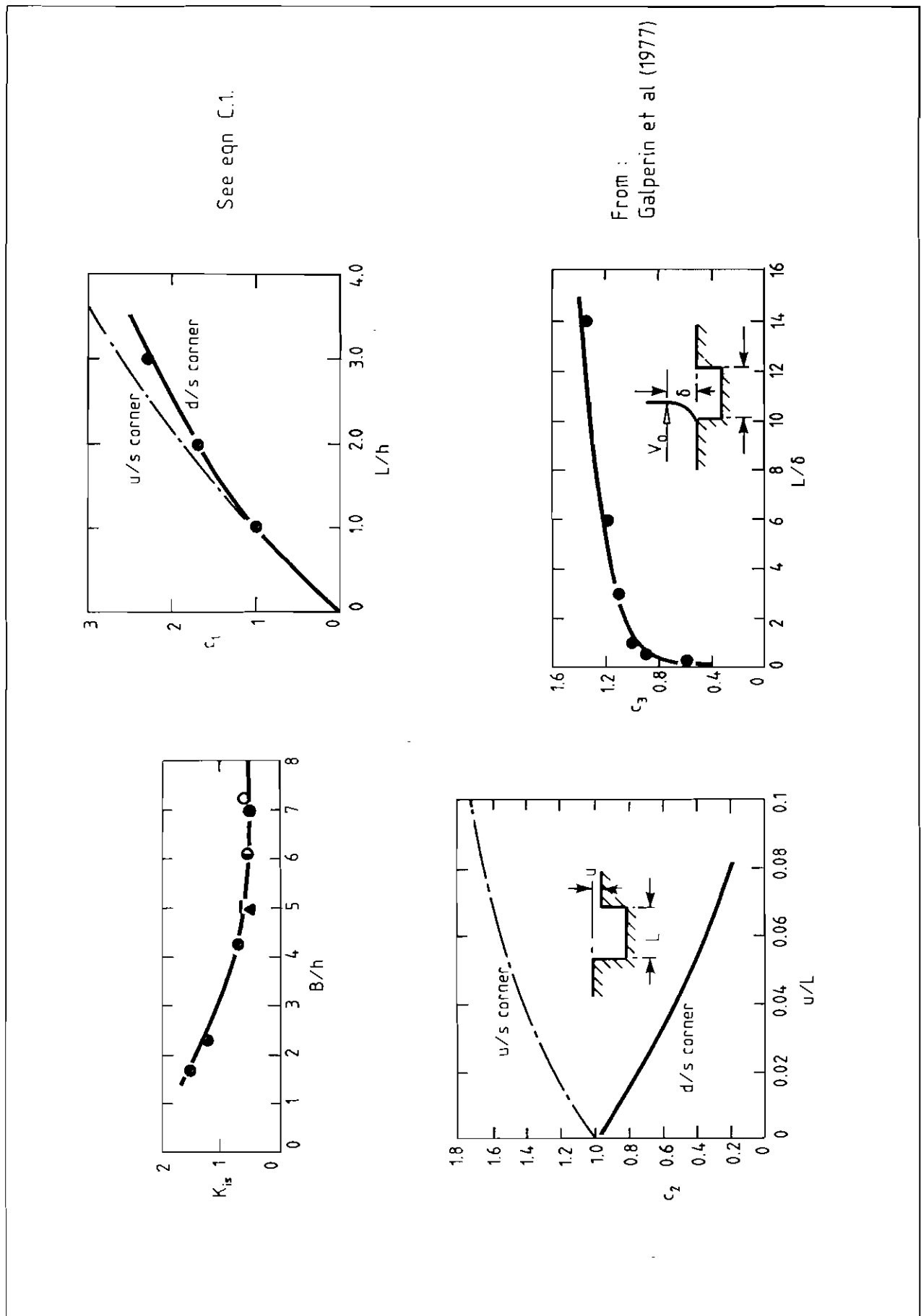


Fig 6 Cavitation parameters of gate slots

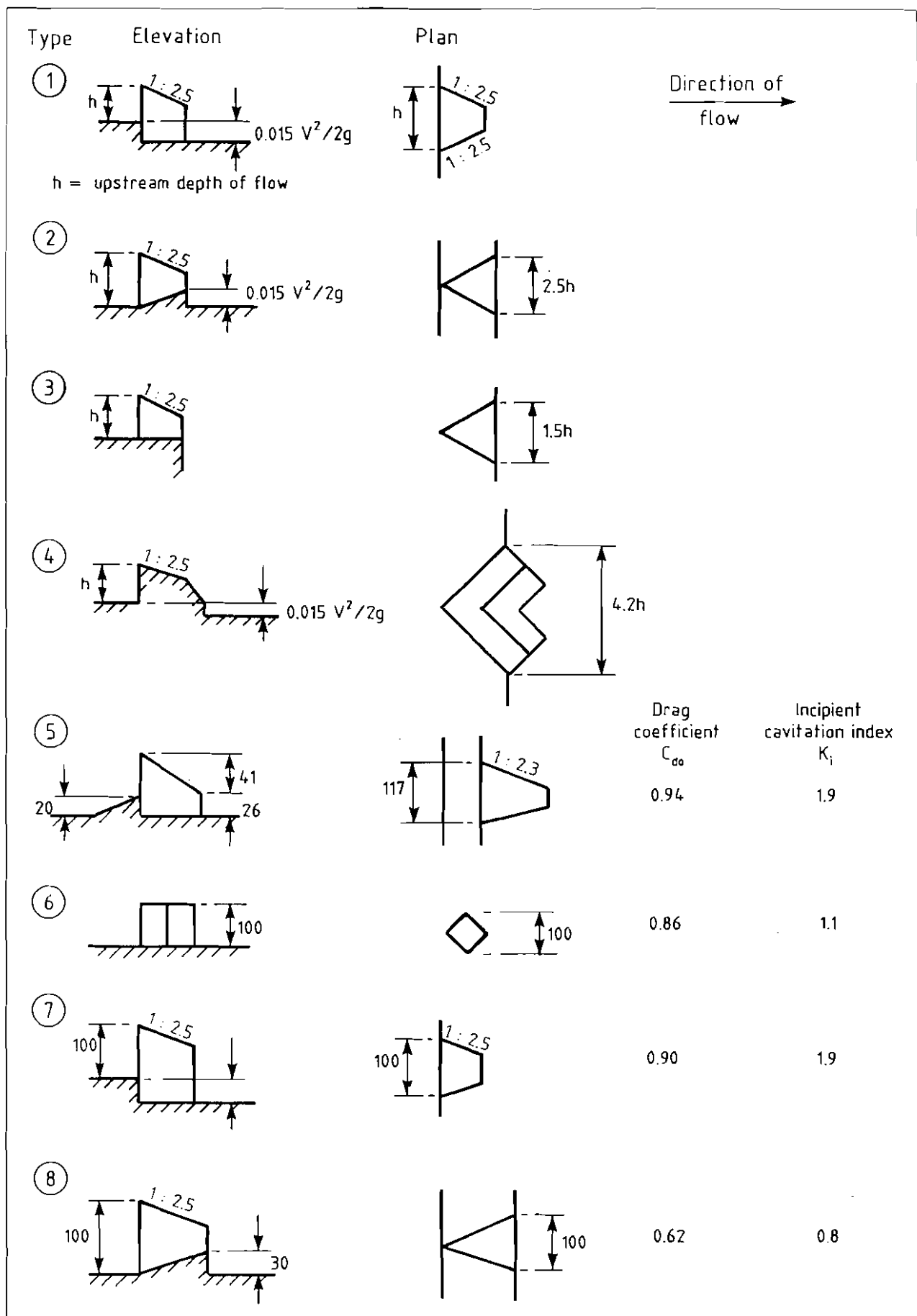
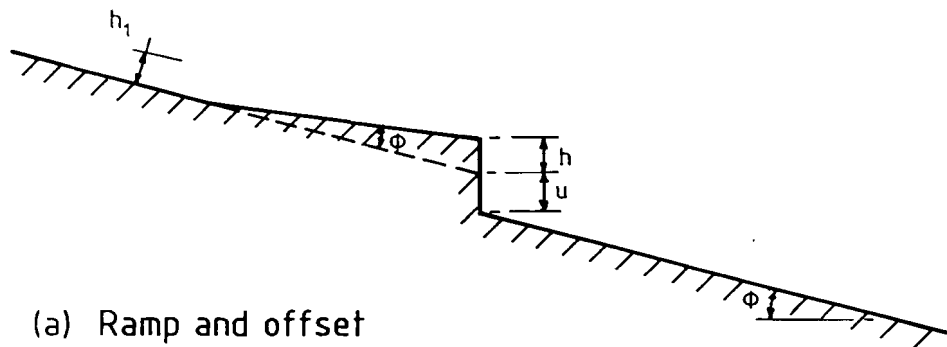
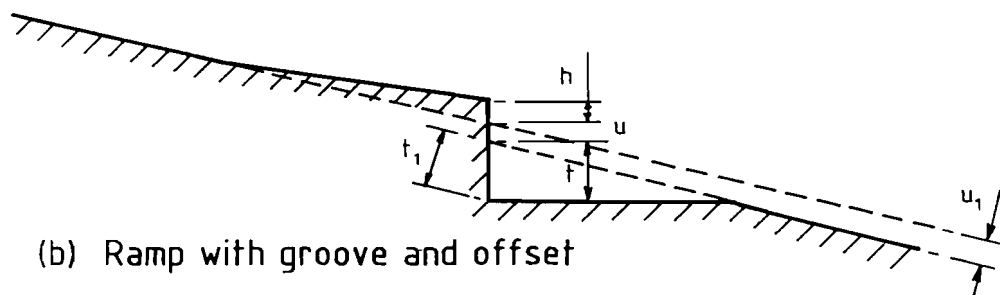


Fig 7 Types of baffle block

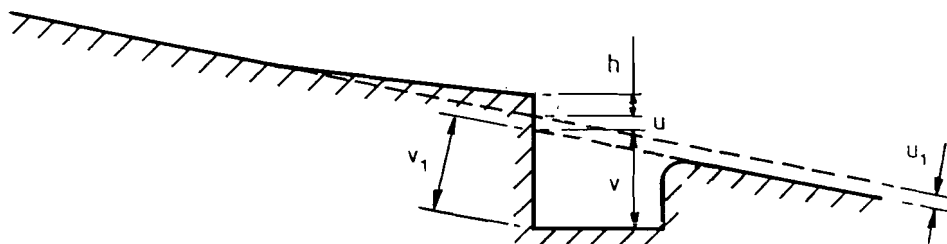


(a) Ramp and offset

Ramp only $t = 0$
Offset only $h = 0$

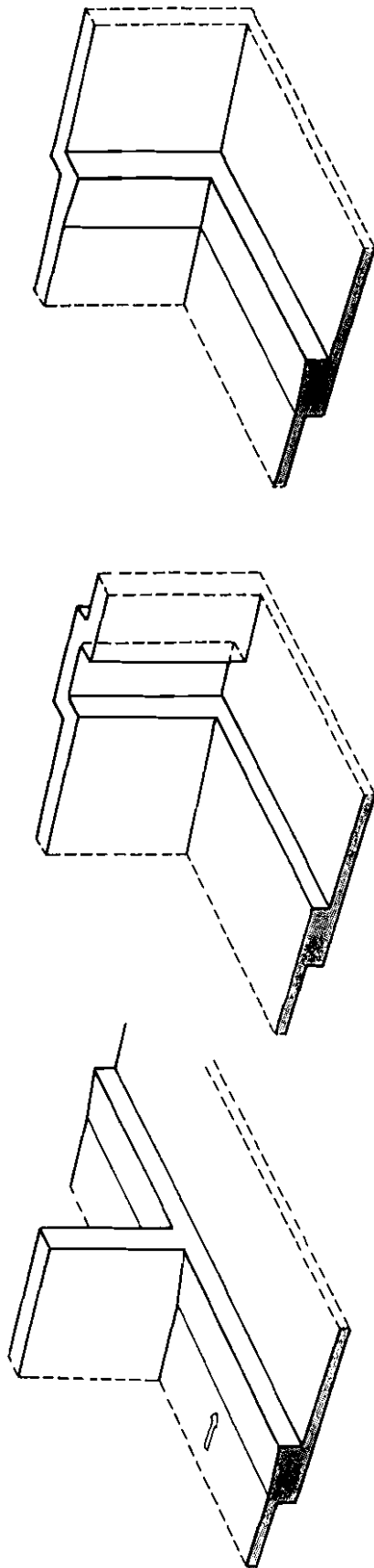


(b) Ramp with groove and offset

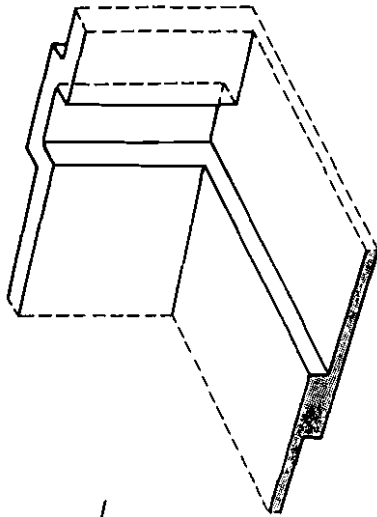


(c) Ramp with slot and offset

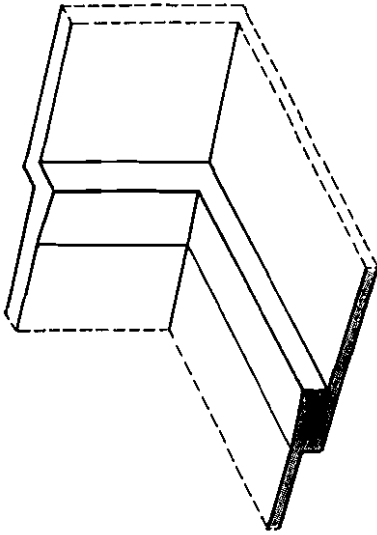
Fig 8 Types of aerator.



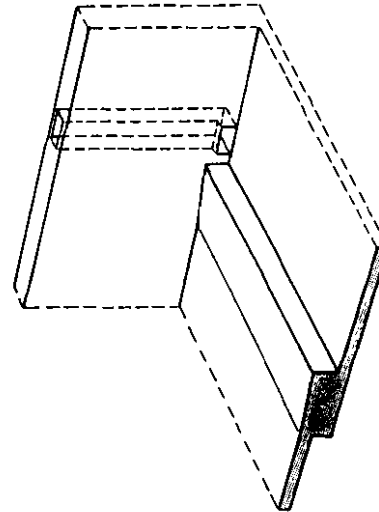
(A) Separation downstream of pier



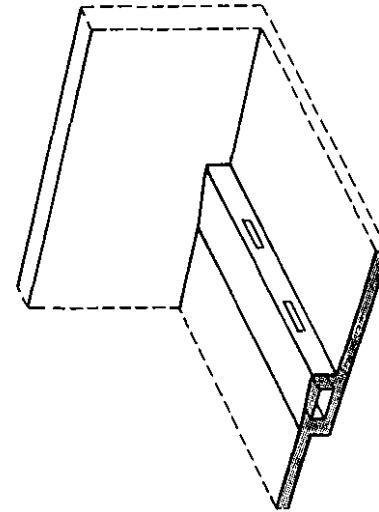
(B) Wall slot



(C) Deflector and offset in wall

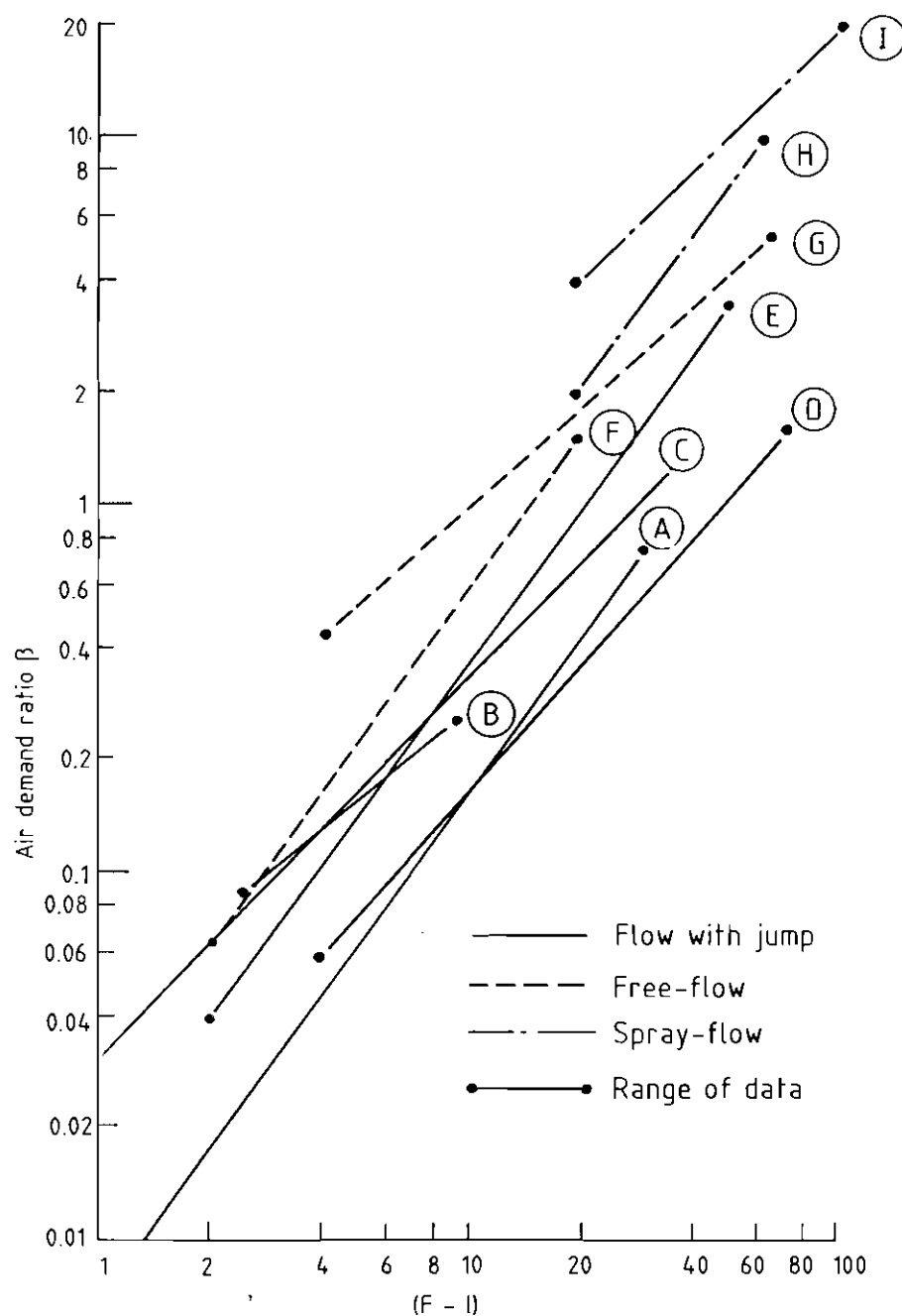


(D) Lateral duct



(E) Transverse duct and slots

Fig 9 Types of air supply system



Key	Reference	Eqn.	Key	Reference	Eqn.
A	Kalinske & Robertson	F.52	E	Haindl	F.69
B	Campbell & Guyton	F.54	F	Wisner	F.59
C	U.S. Army Corps Engrs.	F.55	G	Sharma	F.62
D	Ouazar & Lejeune	F.67	H	Wisner	F.60
			I	Sharma	F.63

Fig 10 Comparison of predicted air demands in tunnels

A P P E N D I C E S

APPENDIX A

SYMBOLS

A	Cross-sectional area of flow
A_a	Cross-sectional area of air duct
A_e	Effective cross-sectional area of air duct (Eqn F.64)
A_m	Maximum cross-sectional area of aerated flow
A_t	Total cross-sectional area of tunnel
A_w	Cross-sectional area of non-aerated flow
a	Amplitude of undulation; coefficients in Eqns 14, B.16, B.19 and B.38
a_1, a_2, a_3, a_4	Coefficients in Eqn F.46
B	Surface width of flow, or width of channel
b	Coefficients in Eqns B.16 and B.39
C	Concentration of air
C_1	Concentration of air in terms of volumes
C_2	Concentration of air in terms of volumetric flow rates
$\bar{C}, \bar{C}_1, \bar{C}_2$	Mean concentrations of air (depth-averaged)
C_d	Drag coefficient (with cavitation)
C_{do}	Drag coefficient without cavitation
C_f	Skin friction coefficient
C_p	Pressure coefficient (Eqn B.1)
C_{pm}	Minimum pressure coefficient
c	Coefficients in Eqn B.16 and F.39
c_1, c_2, c_3	Coefficients in Eqn C.1
D	Diameter of pipe or tunnel
D_d	Downstream diameter
D_o	Diameter of orifice
D_u	Upstream diameter
d	Depth of flow measured normal to bed
d_c	Depth of flow at vena contracta
d_e	Equivalent water depth for aerated flow (Eqn F.10)
d_T	Transition depth in aerated flow
d_w	Depth of non-aerated flow
E	Length of transition downstream of gate slot (see Fig 5)
E_e	Euler number (Eqn F.38)
e	Stabilised depth of cavitation erosion

F	Froude number ($= V/(gA/B)^{\frac{1}{2}}$)
F_l	Value of F just upstream of hydraulic jump
F_c	Value of F at vena contracta
F_e	Equivalent Froude number for aerated flow (Eqn F.13)
F_k	Value of F for start of air entrainment
F_r	Froude number based on hydraulic radius (Eqn F.20)
F_w	Froude number based on characteristic length (Eqn F.17)
f	Frequency of vortex shedding
G	Constant in Eqn F.56
g	Acceleration due to gravity
H	Total head
H_i	Static pressure head at point of incipient cavitation
H_s	Vertical distance below level of reservoir surface
H_{v5}	Vickers Hardness of material for applied load of 5kg
h	Height of step, irregularity or baffle block; depth of offset or gate slot; vertical height of ramp
h_l	Height of ramp measured normal to invert of channel
h_m	Maximum height of aerated flow
I	Cavitation intensity (Eqn 5)
I_o	Parameter for inception of air entrainment (Eqn F.11)
i	Energy gradient of flow
j	Parameter for rate of decrease of air concentration (Eqn F.50)
K	Cavitation index
K_c	Critical value of K (corresponding to continuous but light cavitation noise)
K_i	Incipient cavitation index (Eqn 3)
K_d	Value of K for desinent cavitation
K_{id}	Value of K for incipient damage
K_{il}	Local value of K_i
K_{ip}	Value of K_i estimated from pressure measurements
K_{ir}	Value of K_i for a rectangular gate slot
K_{is}	Value of K_i for a square-shaped gate slot
k	Entrainment constant for aerator (Eqn F.40)
k_s	Nikuradse sand roughness
L	Length of irregularity or gate slot
L_a	Length of air duct
L_c	Length of air cavity produced by aerator
L_h	Horizontal distance between adjacent aerators
L_i	Distance to inception of self-aeration, measured from upstream end of channel

M	Compressive strength of material
m	Coefficient in Eqn 14
m_a	Overall head-loss factor for air duct (Eqn F.58)
N	Number of vortices in gate slot
n	Manning roughness coefficient; slope of surface relative to incident flow (n units parallel to flow to 1 unit normal to flow)
P	Total pressure
P_u	Upstream total pressure
p	Static pressure
P_o	Static pressure at reference point 0
P_l	Static pressure at general point 1
P_c	Critical static pressure for growth of nuclei
P_d	Downstream static pressure
P_v	Vapour pressure of liquid
Δp	Pressure difference across jet (positive if pressure on upper surface is greater than pressure on lower surface)
Q	Volumetric flow rate
Q_a	Volumetric flow rate of air
Q_w	Volumetric flow rate of water
q	Volumetric flow rate of water per unit width
q_a	Volumetric flow rate of air per unit width
R	Hydraulic radius (flow area/wetted perimeter)
R_a	Value of R for air duct
R_c	Cavitation resistance (= [rate of loss of weight/unit area] ⁻¹)
R_e	Reynolds number
R_w	Value of R for non-aerated flow
r	Radius of curvature
r_b	Radius of bubble
r_e	External radius
r_i	Internal radius
S	Strouhal number (Equation C.3)
s	Area of opening of gate; geometric scale ratio (prototype/model)
T	Incubation period for cavitation damage
t	Dimension at downstream end of gate slot (Fig 5); vertical depth of groove at aerator (Fig 8)
t_l	Depth of groove at aerator measured normal to invert of channel
U	Constant in Eqn E.1

u	Offset of downstream wall away from flow (Fig 5); vertical offset of channel floor at aerator (Fig 8)
u_l	Offset of channel floor at aerator measured normal to invert
V	Flow velocity
$\bar{V}$	Mean velocity of water in aerated flow (Equation F.9)
V_{90}	Water velocity at point above bed where air concentration is 90%
V_*	Shear velocity ($= (gRl)^{\frac{1}{2}}$)
V_{aw}	Mean velocity of air-water mixture
V_b	Rise velocity of air bubble
V_d	Velocity at downstream end of air cavity produced by aerator
V_e	Net velocity of air entrainment
V_{in}	Volumetric rate of inflow of air per unit surface area of flow
V_k	Velocity for start of air entrainment
V_o	Velocity at reference point O
V_T	Allowable flow velocity for incubation period T
V_w	Non-aerated flow velocity
v	Volume; vertical depth of slot at aerator (Fig 8)
v_l	Depth of slot at aerator measured normal to invert of channel
v_a	Volume of air
v_w	Volume of water
W	Weber number (Eqn F.18)
W_e	Weber number (Eqn F.38)
w	Overall step height at aerator ($= h + t$, or $h + u$)
X	Scale effect (ratio of prototype value to model value transformed according to Froude criterion)
X_u	Dimensionless parameter (Eqn F.37)
x	Distance measured parallel to surface of channel
y	Distance measured normal to surface of channel
y_{90}	Value of y at which air concentration is 90%
z	Vertical elevation of point above reference level
α	Angle of chamfer relative to incident flow
β	Ratio of volumetric flow rate of air to volumetric flow rate of water
γ	Volume of cavitation erosion with air as proportion of volume of erosion without air

δ	Proportionate change in time-averaged velocity; thickness of boundary layer; vertical roughness index at aerator
ϵ	Proportionate fluctuation in velocity
ζ	Velocity head coefficients for losses in air duct
η	Dimensionless parameter (Eqn B.36)
θ	Angle of channel to horizontal
λ	Wavelength of undulation; Darcy-Weisbach friction factor ($= 8gR1/V^2$)
λ_a	Friction factor for aerated flow
λ_w	Friction factor for non-aerated flow
ν	Kinematic viscosity of liquid
ξ	Factor in Eqn D.1
ρ	Density of liquid
ρ_a	Density of air
σ	Surface tension of liquid
τ_o	Average shear stress
ϕ	Angle of ramp of aerator relative to channel
ψ	Scale factor in Eqn G.3
Ω	Channel shape parameter (Eqns F.14 a, b)

APPENDIX B

CAVITATION AT SURFACE IRREGULARITIES

B.1 General

Most studies have been concerned with determining values of the parameter K_i (see Equations 3 and 4) for incipient (or desinent) cavitation at surface irregularities. Results have been obtained:

1. theoretically;
2. by laboratory experiments,
3. by field tests and observations.

Generally the various values of K_i for a particular type of excrescence are in reasonable agreement, but direct comparisons between experiments are not always possible because of different definitions of the characteristic pressure and velocity (p_o and V_o in Equation 3), and different means of identifying the limit of cavitation (by eye, by sound or by increase in turbulence levels).

B.2 Theoretical studies

Most results in this category apply to streamlined types of irregularity for which the flow remains attached to the surface. Values of the pressure coefficient

$$C_P = \frac{p - p_o}{\frac{1}{2} \rho V_o^2} \quad (B.1)$$

along the boundary are determined theoretically, usually by means of potential flow theory. It is then assumed that when cavitation begins the minimum pressure on the surface is equal to the vapour pressure p_v of the liquid; thus from Equation 3 the inception parameter is given by

$$K_i = -C_{pm} \quad (B.2)$$

where C_{pm} is the minimum value of the pressure coefficient on the irregularity. This approach neglects the effect of boundary-layer development and the influence of turbulent pressure fluctuations which will tend to result in higher-than-predicted values of K_i .

Rosanov et al (1965) describe results obtained by conformal transformation for streamlined irregularities consisting of circular arcs (Type 7B in Figure 1). For flow with a free surface, the critical cavitation number was found to be

$$K_i = 4h/L \quad (B.3)$$

where h is the height of the irregularity and L is its length. The formula was checked experimentally for a value of $h/L = 0.38$. Xu & Zhou (1982) also used conformal transformations to calculate the minimum pressure coefficients for irregularity Types 1D and 7B in both open channels and pressure conduits. Theoretical and experimental results were presented graphically in the form

$$C_{pm} = fn(r/h, d/h) \quad (B.4)$$

where d is the depth of flow.

Zhuravliova (1983) studied flow over different types of smoothly undulating surface, and concluded that the most severe case was provided by sinusoidal variations of type

$$y = a \sin (2 \pi x / \lambda) \quad (\text{B.5})$$

in which a is the amplitude of the undulation and λ is its wave length. The corresponding value of the pressure coefficient is

$$C_p = - \frac{4 \pi a}{\lambda} \phi (d / \lambda) \sin (2 \pi x / \lambda) \quad (\text{B.6})$$

where d is the flow depth; the value of V_o used in calculating C_p from Equation B.1 is the undisturbed average velocity upstream of the undulation. For free-surface flow

$$\phi (d / \lambda) = \tanh (2d / \lambda) \quad (\text{B.7a})$$

and for flow under pressure

$$\phi (d / \lambda) = \coth (2d / \lambda) \quad (\text{B.7b})$$

If the depth of flow $d > 2\lambda$, the minimum value of the pressure coefficient is approximately

$$C_{pm} = - \frac{4 \pi a}{\lambda} \quad (\text{B.8})$$

Comparisons with experimental measurements showed that the criterion for the inception of cavitation was given by

$$K_i = -C_{pm} + 0.05 \quad (\text{B.9})$$

where C_{pm} is the theoretically-predicted value, and the 0.05 term takes account of the effect of turbulent pressure fluctuations.

Zhou et al (1984) used a finite element method to predict values of C_{pm} for four types of irregularity

(Types 1D, 3B, 6B, 7B in Figure 1) on the invert of a pressure conduit. The irregularities were assumed to have rounded edges of radius r . The results were presented graphically, and for Types 1D and 7B were given in the form

$$C_{pm} = \text{fn } (r/h, d/h) \quad (\text{B.10})$$

For both types the magnitudes of C_{pm} were fairly similar, and decreased rapidly with r/h in the range $r/h < 40$; beyond this limit the values were almost independent of r/h and varied between $C_{pm} = -0.6$ at $d/h = 6$ and $C_{pm} = -0.2$ at $d/h = 20$. In the case of irregularity Types 3B and 6B it was assumed that the radius of curvature r was equal to the height h . Results were presented in the form

$$C_{pm} = \text{fn } (n, d/h) \quad (\text{B.11})$$

where n defines the slope of the irregularity (n units parallel to the flow to 1 unit normal to the flow). The magnitudes of C_{pm} for Types 3B and 6B were fairly similar, and in both cases became almost constant for $n > 30$; in this range values varied from about $C_{pm} = -0.6$ at $d/h = 5$ to $C_{pm} = -0.1$ at $d/h = 20$. Results were also obtained for groups of irregularities at different longitudinal spacings.

These various theoretical results apply to two-dimensional irregularities, and the values of L/h need to be quite large for the assumption of no flow separation to be valid. They are therefore not suitable for estimating the cavitation potential of typical construction faults, such as those at mis-aligned joints, but can be used to define permissible tolerances for remedial works.

In the case of separated flows, Johnson (1963) suggested that a reasonable estimate of the cavitation parameter is given by

$$K_i = 1-2 C_{pm} \quad (B.12)$$

where C_{pm} is the pressure coefficient at the point on the surface at which the flow separates. This result is obtained by assuming that the minimum pressure in the fluid occurs at the centre of a forced vortex core formed at the point of separation.

B.3 Laboratory studies

Experiments to determine the conditions for incipient cavitation have been carried out using cavitation tunnels (pressure flow) and vacuum test rigs (free-surface flow), usually with the ambient pressure reduced below atmospheric.

Ball (1963) provided curves for determining the limit of cavitation for into-the-flow offsets and chamfers (irregularity types 1A, 1B, 1C, 3A in Figure 1). The curves are expressed in dimensional form, and give the static pressure head H_i for incipient cavitation as a function of the following variables:

$$\text{Type 1A} \quad : H_i = \text{fn} (V_o, h) \quad (B.13a)$$

$$\text{Types 1B, 1C: } H_i = \text{fn} (V_o, h, r) \quad (B.13b)$$

$$\text{Type 3A} \quad : H_i = \text{fn} (V_o, n) \quad (B.13c)$$

where V_o is the average flow velocity. Analysis of the graphs suggests that the corresponding values of the cavitation parameter K_i do not vary greatly with flow velocity for a given shape and size of irregularity. However, in the case of the three Type 1 irregularities there is a strong dependence of K_i on the height h of the offset. Given this behaviour, it is perhaps surprising that the values of K_i for the Type 3A chamfer appear to depend only upon the slope

n. Approximate values of K_1 for the irregularities are given in Table 2, but it is stressed that these have been determined from the graphs and not from the original data. Falvey (1984) mentions that Ball's experiments were carried out in a water tunnel measuring 102mm high by 152mm wide, and that the thickness of the boundary layer was about 2mm.

Johnson (1963) gives values of K_1 for a sharp-edged offset away from the flow (Type 2A in Figure 1). The graphical results can be described approximately by

$$K_1 = 0.1 + 0.035h \quad , \quad h \leq 20\text{mm} \quad (\text{B.14a})$$

$$K_1 = 0.8 + 0.002(h-20) \quad , \quad h > 20\text{mm} \quad (\text{B.14b})$$

where the depth h of the offset is in mm.

Rosanov et al (1965) provide data for four types of irregularity as follows:

Irregularity Type	K_1
1A	2.2
2A	1.1
7A	1.4
8A	1.6

The values of K_1 were calculated using the average flow velocity in the contracted section. No mention is made of any variation of K_1 with the height of the irregularity. The into-the-flow offset (Type 1A) was also tested with positive and negative slopes of 1:5 and 1:10 downstream of the step; the largest value of $K_1 = 2.4$ occurred with a slope of 1:10 away from the flow. In the case of the offset Type 2A, varying the slope upstream of the step did not alter K_1 from the figure of 1.1.

Galperin et al (1977) defined values of K_1 using the undisturbed flow velocity at the level of the top of the irregularity and obtained

Irregularity Type	K_i
7B(r/h=4.6)	0.92
8A	1.76

It was found that these values were not dependent on the height h of the irregularity relative to the thickness δ of the boundary layer (for $h/\delta \leq 2.5$). Results for a chamfer into the flow (Type 3A in Figure 1) can be approximated by

$$K_i = \sqrt{3} \quad , \quad n < \sqrt{3} \quad (\text{B.15a})$$

$$K_i = 3/n \quad , \quad \sqrt{3} \leq n \leq 6 \quad (\text{B.15b})$$

where the slope of the chamfer is n units parallel to the flow to 1 unit normal to the flow.

Arndt et al (1979) analysed data for six types of irregularity, and found that the value of K for desinent cavitation, K_d , depended upon the Reynolds number and upon the height h of the excrescence relative to the boundary layer thickness δ . Results were fitted to an equation of the form

$$K_d = a (h/\delta)^b (V_o \delta / \nu)^c \quad (\text{B.16})$$

where V_o is the velocity outside the boundary layer. The coefficients a , b and c vary according to the type of irregularity as follows:

Irregularity Type	a	b	c
5A (L=h)	0.00117	0.737	0.550
6A (L=h)	0.00328	0.632	0.451
7A	0.0108	0.439	0.298
7B (r/h=4.6)	0.041	0.344	0.267
8A (L=h)	0.152	0.361	0.196
9A	0.000314	0.041	0.510

Falvey (1982) combined data for into-flow chamfers (Type 3A) obtained by Colegate (1977) and Jin et al (1980) which showed that

$$K_1 = 1.8 n^{-0.7}, \quad n \geq 5 \quad (\text{B.17})$$

In the case of abrupt chamfers with $n \leq 1$, the value of K_1 depends only upon the height h of the chamfer, this dependency is described approximately by

$$K_1 = 1.17 h^{0.14}; \quad 2\text{mm} \leq h \leq 19\text{mm} \quad (\text{B.18})$$

where h is in mm. In the range $1 < n < 5$, K_1 varies with both the height and slope of the chamfer. Falvey mentions that the data were obtained with virtually no boundary layer, so the limiting velocity corresponding to K_1 is the local value at the level of the irregularity. These results are in reasonable agreement with those of Galperin et al (see Equations B.15a, b).

Keller & Koch (1982) studied cavitation conditions for a square block mounted on the floor of a rectangular channel and subject to supercritical free-surface flows. The ratio of the block height to the upstream water depth was kept constant at 0.142. At Froude numbers of $F < 2$, it was found that increasing the amount of turbulence in the flow increased the value of K_1 ; for $F > 2$, the results were little affected by the degree of turbulence. The values of K_1 reached a maximum of $K_1 = 2.6$ at $F = 2.11$, and then decreased to $K_1 = 2.0$ at $F = 3.24$. This indicates that cavitation characteristics may be modified if irregularities are large enough to cause an interaction with the free surface.

Liu (1983) found that values of K_1 for three types of

irregularity could be described by an equation of the form

$$K_1 = a h^{0.326} \quad (B.19)$$

where the height of the irregularity is in mm, and the constant a has the following values:

Irregularity Type	a
1A	1.02
1B ($r/h = 0.5$)	0.88
6A	0.70

The heights of the irregularities studied in the tests varied between 1mm and 15mm. Results were also obtained for into-flow chamfers (Type 3A) for which

$$K_1 = 2.9 n^{-0.96}, \quad \text{for } 2 \leq n \leq 12 \quad (B.20)$$

The chamfers tested all had a height of $h = 10\text{mm}$.

Kudriashov et al (1983) investigated the inception of cavitation at changes in channel slope away from the flow (irregularity type 4B). Results for three deflection angles were

α	K_1
5°	0.3
16°	1.1
31°	1.25

Experiments on chamfers angled away from the flow (irregularity type 4A) were also carried out by Demiröz & Acatay (1985). For deflection angles of $\alpha < 20^\circ$, the flow remained attached to the boundary, and pressures were measured by surface tappings. At larger deflection angles the flow separated, and pressures were calculated from measurements of velocity within the fluid obtained using a

Laser-Doppler anemometer. For non-separating flows, the measured values of K_i were independent of the depth of the chamfer and fitted the equation

$$K_i = 0.16 + 0.015 \alpha, \quad \text{for } 10^\circ \leq \alpha \leq 20^\circ \quad (\text{B.21})$$

where the angle α is in degrees. When the flow separated, K_i was almost independent of α but varied with the depth h of the chamfer

h (mm)	Value of K_i	
	$\alpha = 25^\circ$	$\alpha = 90^\circ$
10	0.52	0.52
20	0.72	0.75
40	0.82	0.85

For angles between $20^\circ < \alpha < 25^\circ$, K_i depended upon both α and h . These values of K_i are lower than those obtained by Kudriashov et al (1983) who determined the onset of cavitation directly.

Scheur (1985) determined the conditions for incipient cavitation for five types of irregularity with heights varying between 5mm and 20mm. The values of K_i obtained at a free stream velocity of 8m/s for irregularities of height $h = 10\text{mm}$ were

Irregularity type	$(K_i)_{10}$
1A	1.90
2A	1.40
5A	1.80
7A	1.65
8A	1.95

Values of K_i for other heights were related to those for $h = 10\text{mm}$ by the following factors

Height h (mm)	$(K_i)_h / (K_i)_{10}$
5.0	0.81
7.1	0.92
10.0	1.00
14.1	1.08
20.0	1.18

The results for the rectangular rib (Type 5A) were also expressed in the form

$$K_i = 0.075 (h/\delta)^{0.335} (V_o \delta / \nu)^{0.270} \quad (\text{B.22})$$

This equation is similar in type to the one used by Arndt et al (1979) (see Equation B.16), but the coefficients have significantly different values.

Experimental data for into-flow chamfers (Type 3A) were presented by Novikova & Semenov (1985). The values of K_i were calculated using the velocity at the level of the top of the chamfer, and were represented by the following equations

$$K_i = 2.3n^{-0.7}, \quad \text{for } n > 1 \quad (\text{B.23})$$

$$K_i = 2.3, \quad \text{for } n \leq 1 \quad (\text{B.24})$$

These values are higher than those found by Galperin et al (1977) and Falvey (1982), although it is noteworthy that the exponent of n in Equation B.23 is the same as in Falvey's Equation B.17.

The information given so far applies to two-dimensional irregularities. Zharov & Kudryashov (1977) tested three-dimensional irregularities of Type 3C (see Figure 1) both singly and in groups. The height h of the excrescences was varied from 3mm to 10mm, and the chamfer angle α from 15° to 90° (where $n = \cot \alpha$). All the results were well described by the formula

$$K_1 = 2.0 \sin \alpha \quad (\text{B.25})$$

with no dependence on h . The characteristic velocity was taken as that at height h in the absence of the projection.

If an irregularity does not project outside the boundary layer, the velocity V at the level of the tip of the excrescence is given according to Galperin et al (1977) by

$$V/V_* = 9.8 \{1 + 0.57 \ln (h/k_s)\} \quad (\text{B.26})$$

where k_s is the Nikuradse sand roughness, and where the shear velocity V_* is related to the shear stress τ_o at the surface by

$$\tau_o = \rho V_*^2 \quad (\text{B.27})$$

Turbulent pressure fluctuations in a boundary layer can cause cavitation to occur on plane surfaces. Arndt et al (1979) found (for desinent cavitation) that

$$K_1 = 16 C_f \quad (\text{B.28})$$

where the skin friction coefficient C_f is defined by

$$C_f = \frac{\tau_o}{\frac{1}{2} \rho V_o^2} \quad (\text{B.29})$$

For rough-turbulent flow over a plane surface, the value of C_f at a distance x from the start of the boundary layer can be estimated from

$$C_f = 0.0493 (x/k_s)^{-0.304} \quad (\text{B.30})$$

An alternative formula for determining the skin friction coefficient is given by Duncan et al (1962, p330) as

$$C_f = [2.87 + 1.58 \log_{10}(x/k_s)]^{-2.5} \quad (B.31)$$

Cavitation can also be produced when there is a sudden change in surface roughness, as for example at the end of a section of concrete channel protected by a steel lining. According to Kudriashov et al (1983), if the downstream roughness height k_2 is much greater than the upstream value k_1 , then the cavitation potential of the discontinuity is equivalent to an into-flow chamfer of height k_2 and slope $n = 10$.

All the results described so far apply to uniform flows over irregularities on plane surfaces. Values of the cavitation parameter for non-uniform conditions can be calculated by means of the so-called "addition theorem" described by Arndt et al (1979). Let K_{11} be the local value of the incipient cavitation index for an irregularity on a plane surface. Now let the irregularity be placed at a point where the local pressure and velocity (p , V) are different from the free-stream values (p_o , V_o); the pressure coefficient C_p for the point can be calculated from Equation (B.1). It can then be shown from Bernoulli's equation that the cavitation index for the irregularity, defined in terms of free-stream conditions, is given by

$$K_1 = -C_p + K_{11} (1 - C_p) \quad (B.32)$$

The validity of this result has been checked experimentally.

Li (1982) describes a method for designing the sectional profile of a spillway so as to reduce or

eliminate the possibility of cavitation. Suitable profiles are obtained by varying the radius of curvature so as to maintain a constant value of the cavitation index K (Equation 2) along the surface of the spillway, alternatively the profile may be selected so as to keep the pressure at the bed constant.

The presence of sediment in water influences the occurrence of cavitation. Liu (1983) carried out experiments with a circular cylinder to determine how the limit of incipient cavitation varied with sediment concentration. For concentrations up to 10kg/m^3 , the values of K_i were slightly higher than for clear water; increasing the concentration from 10kg/m^3 to 70kg/m^3 decreased K_i to about 80% of its clear-water value; above 70kg/m^3 the values of K_i remained approximately constant. Research reported by Lin et al (1987) also showed that sediment accelerated the rate of cavitation pitting, but did not alter the final depth of erosion.

It is convenient to include in this section experimental information about cavitation at bends in circular pipes. Kudriashov et al (1983) found that measurements of incipient cavitation fitted the formula

$$K_i = 1.1 \left[\frac{2(r_e - r_i)}{(r_e + r_i)} \right]^{0.7} \quad (\text{B.33})$$

where r_i and r_e are respectively the internal and external radii of curvature of the pipe.

Tullis (1981) studied cavitation in 90° bends with nominal diameters of 75, 150 and 300mm. Flow conditions were determined for incipient cavitation (light and intermittent noise) and critical cavitation

(continuous but light noise). The critical cavitation criterion was recommended for design as it corresponds to the point beyond which pitting of the pipe surface begins. Pipe size was found to have a significant effect on the values of the cavitation parameters. The results for incipient and critical conditions were described respectively by

$$K_1 = 0.325 D^{0.46} \quad (B.34)$$

$$K_c = 0.378 D^{0.40} \quad (B.35)$$

where the pipe diameter D is in mm; the value of pressure used to calculate K_1 and K_c from Equation 2 was the total pressure upstream of the bend (static plus velocity head). Although this work is not strictly relevant to conditions in tunnel spillways, it does indicate that models of such structures may be subject to important scale effects.

B.4 Field studies

Most field data concerning allowable irregularities on prototype structures have been obtained from surveys carried out after cavitation damage had occurred. However, two systematic studies at full scale have been made to study the onset and development of cavitation, and these are described at the end of this section.

Wagner (1967) describes cavitation damage downstream of gates in the diversion tunnel of Glen Canyon Dam (USA). The gates were used to control flows with heads of up to about 102m. Erosion due to cavitation was found at the following places:

1. minor irregularities in the steel liner fitted downstream of the gates caused damage to a maximum depth of 10mm;

2. irregularities in application of paint coating;
3. offsets into the flow of more than 0.8mm caused cavitation at flow velocities of 41m/s.

Surface depressions of less than 3mm did not lead to damage; depressions of 6mm resulted in some removal of the paint coating and minor pitting.

Galperin et al (1977) give details of cavitation damage which occurred at several large dams. Supkhun Dam (Korea) has a spillway slope of 1:0.78 and an overall head of about 100m, and was designed for unit discharges of up to $64\text{m}^3/\text{s}/\text{m}$. Cavitation damage occurred during the first operating season and originated at horizontal construction joints; 200 cavities with depths exceeding 0.1m were noted, and the total volume of erosion was 1100m^3 . After twelve years of service the volume had increased to $10,000\text{m}^3$, and the maximum depth of erosion was 2.4m.

The spillway of Bratsk Dam (USSR) has a slope of 1:0.8 and an overall head of 95m, and at normal reservoir level the unit discharge is $30.5\text{m}^3/\text{s}/\text{m}$. The strength of the concrete varied between 34MPa and 54MPa with an average of 44MPa. Imperfections in surface finish found after construction included stepped drops of up to 80mm due to displacement of formwork, undulations, and isolated irregularities such as holes and lumps of concrete. Cavitation erosion occurred first at the largest irregularities subjected to the highest velocities. The biggest hole was downstream of a 60-80mm high projection, and measured 7.5m wide by 10.5m long with a maximum depth of 1.2m. The maximum rate of erosion observed was

18mm/day. Cavitation damage also originated at design features such as drain holes.

The construction of Krasnoyarsk Dam (USSR) benefited from the experience obtained at Bratsk. The spillway has a slope of 1:0.8, an overall head of about 82m, and a unit discharge of $59\text{m}^3/\text{s}/\text{m}$ at normal reservoir level; the strength of the concrete was 52-53MPa. An improved surface finish was obtained by changes in the design of the formwork, and remaining surface imperfections were ground to chamfers with slopes of between 1:5 and 1:13. Despite these precautions, some cavitation damage did still occur, but it was less severe than at Bratsk, with the maximum rate of erosion being reduced to 1mm/day.

Lowe et al (1979) document cavitation damage which occurred at Tarbela Dam (Pakistan) on chutes downstream of two tunnels (Nos 3 and 4) controlled by radial gates. The profiles of the chutes were designed to give approximately atmospheric pressure on the lower surfaces. Causes of the cavitation were:

1. patches of mortar left by mistake: after repair with ordinary concrete, no further damage occurred;
2. irregularities in the floor: steps of 1.6-2.4mm at transition from steel to concrete surface, and 3mm high humps with slope changes of about 1:20;
3. joints designed with offsets away from the flow of 13-19mm, and double cracks at control joints.

The damage due to item 2 started at velocities of about 47-49m/s, indicating values of K for incipient

damage of approximately $K_{1d} = 0.08$. This suggests that use of Ball's laboratory data (see Section B.3 and Table 2) for design will err on the conservative side. In item 3 the construction of the joints was changed and the offsets eliminated.

Aksoy & Ethembabaoglu (1979) give details of cavitation problems in the spillway channels of Keban Dam (Turkey). Damage occurred at incorrectly constructed transverse joints which had offsets away from the flow of up to 50mm; the design value of unit discharge was $14.5 \text{ m}^3/\text{s}/\text{m}$ width of channel and the total head was about 120m. No damage took place in regions where there was fully-developed air entrainment.

The mechanism by which a series of cavitation holes forms downstream of a step was described by Vorobiyov (1983). Based on prototype measurements, a rather complex empirical equation was obtained for predicting the rate of loss of material from the first hole, and then from the subsequent ones; as the holes develop, those downstream can eventually become larger than the one adjacent to the step. The empirical equation was also used to scale results from model to prototype. The following recommendations were made for the maximum volume of erosion that should be allowed behind each step for varying thicknesses of lining:

<u>Lining thickness (m)</u>	<u>Allowable erosion (m^3)</u>
0.2	0.6
0.5	3
1.0	6

The figures are not related to the transverse width of the step, but are apparently based on measurements of erosion caused by typical types of imperfection that occur on prototype surfaces.

Falvey (1983) collected data on cavitation at seven major dams, and observed that the incidence of damage depended both on the value of the cavitation parameter K and on the length of time that the structure was operated under these conditions. Results were presented in graphical form and are reproduced in Figure 2; two curves are given which delimit regions in which no damage, minor damage or major damage can be expected. The following suggestions were also made on the precautions which should be taken according to the value of K occurring on a hydraulic structure:

<u>Value of K</u>	<u>Precaution</u>
$1.8 \leq K$	No surface protection needed
$0.25 \leq K < 1.8$	Treat surfaces (eg by grinding irregularities to flat chamfers)
$0.17 \leq K < 0.25$	Modify design (eg increase pressures by decreasing amount of curvature)
$0.12 \leq K < 0.17$	Add aerators (for $K < 0.25$ if design cannot be modified)
$K < 0.12$	Abandon design

Cassidy & Elder (1984) cite the results of a survey carried out by ICOLD (1980). Out of 71 large dams operating for more than 100 days, 52 suffered no damage, 9 slight erosion (< 20mm depth), 2 moderate erosion (20mm to 100mm), and 8 serious erosion (from 100mm to several metres). Flow velocity was the parameter that showed the strongest correlation with damage: of 12 chute or tunnel spillways operating at more than 30m/s, five suffered serious erosion and four slight or moderate erosion. Discharge per unit width was a less reliable indicator, but the risk of damage did appear to increase when $q > 50\text{m}^3/\text{s/m}$. Many of the problems were caused by construction faults (eg joints and projecting reinforcement), and most were

successfully repaired using fibrous or epoxy concrete. Out of nine spillways equipped with aerators (see Section F.3), six still suffered cavitation damage (two seriously). In order to calculate cavitation parameters, it is necessary to estimate the surface roughness of the spillway surface; the best concrete finish that can be obtained without steel troweling is probably in the range of 0.8mm to 1.1mm.

According to Zhang (1984), cavitation damage on chute spillways is mostly likely at the toe where the vertical transition curve ends. This is the region where the boundary shear stress tends to be a maximum, and where irregularities are presumably most exposed to local high velocity flows. This argument does not take account of self-aeration effects which can prevent cavitation damage near the bottom of chute spillways. Zhang correlated model and prototype data, and concluded that the worst conditions for cavitation occur when the following parameter has the value

$$\eta = \frac{q}{\sqrt{g} H_s^{3/2}} \approx 0.015 \quad (\text{B.36})$$

where q is the unit discharge, g the acceleration due to gravity, and H_s the height of the reservoir surface above the point in question.

There would not appear to be any fundamental reason why the potential for cavitation should be greatest when the parameter η has a certain value. However if one considers, for a particular spillway, the conditions which produce the maximum velocity in the vicinity of a surface irregularity, then it can be seen that the effects of H_s and q are interrelated in a rather complex way. As one moves down the spillway, the head H_s and therefore the average flow velocity

increase, but the boundary layer also thickens; therefore the maximum velocity at an irregularity may occur at some intermediate point on the spillway. As the unit discharge q increases, the distance needed for the boundary layer to become fully developed also increases. Therefore, it is possible to envisage that cavitation conditions could be most severe when a parameter containing q and H_s has a certain value; the value of the parameter would be determined by additional factors such as the shape of the spillway, its surface roughness, and the type of irregularity.

As mentioned at the beginning of this Section, two systematic studies of cavitation on spillways have been carried out at full scale. Galperin et al (1977) and Oskolkov & Semenov (1979) describe results of field tests using "indicators" of various heights and slopes (equivalent to irregularity types 3A and 4A in Figure 1) placed on the surface of a spillway. Such indicators may be made of the same materials as the surface, or from a softer material so as to accelerate the tests. The conditions for incipient cavitation may be identified by the removal of a thin film of easily-erodible material applied to the surface of the indicator. Controlled discharges are then used to determine the height and slope of irregularity which will cause incipient cavitation (K_i) or incipient cavitation damage (K_{id}). Figure 3 is based on tests at Bratsk Dam (USSR), and shows how the value of K_{id} , for the start of cavitation erosion, varies with the slope of the chamfer. Perhaps surprisingly, the chamfers angled away from the flow have slightly higher values of K_{id} than those directed towards the flow.

The second systematic study was carried out by Wang & Chou (1979) who obtained comprehensive field data from measurements on Feng Man, Zhe Xi and Liu Jia Xia Dams

(China); the first two have chute spillways and the third a tunnel spillway. Between 1953 and 1975 the Feng Man spillway operated nine times, and on each occasion some cavitation damage occurred; the overall head above the toe of the spillway reached about 68m, and the maximum unit discharge was $69\text{m}^3/\text{s}/\text{m}$.

Cavitation originated at faults at transverse construction joints, which took the form of sloping offsets and triangular-shaped irregularities (Types 3B and 6B in Figure 1). The largest area of damage measured 35m^2 , and the maximum depth of erosion was 1.2m. In 1963 and 1964 tests were carried out in which symmetrical triangular concrete blocks of various heights (up to 100mm) and slopes ($n = 5$ to 20) were mounted on the spillway, and the resulting cavitation damage noted. Measurements of pressure at the apex of each block showed that no erosion took place until the time-averaged pressure fell to -7m of water head below atmospheric, and that erosion occurred rapidly once the pressure dropped to -9.7m . The double amplitude of the pressure fluctuations at an offset away from the flow was found to be 10.7% of the average velocity head.

Wang & Chou provide detailed profiles of the irregularities and the resulting cavitation holes that occurred at the three dams. Based on these observations, the following empirical equation was derived for predicting the stabilised depth of cavitation erosion

$$e = (V_o/a)^b \quad (\text{B.37})$$

where e is the depth in mm, V_o is the flow velocity in m/s at the level of the irregularity, and the constants a and b are given by

$$a = 9.6 I^{-0.32} \quad (\text{B.38})$$

$$b = 7.6 I^{0.44} \quad (\text{B.39})$$

I is a measure of the intensity of cavitation, as defined in Equation 5. Equation B.37 is based on data for concrete with a compressive strength of about 20-25MPa. On Feng Man Dam the time for the erosion to reach an equilibrium depth was about 200 hours. In order to calculate values of I in the prototype, it was necessary to make estimates of the inception parameter K_i . Tests on a 1:30 scale model were therefore carried out to determine the minimum pressures at chamfers and triangular irregularities (Types 3A and 6A in Figure 1). The results shown in Figure 4 were then obtained by assuming $K_i = -C_{pm}$ (see Section B.2), and allowing for pressure fluctuations of $\pm 5\%$ of the velocity head. Comparison with Ball's data for chamfers (see above) showed good agreement provided K_i was defined in terms of the velocity at the level of the irregularity.

Wang & Chou suggest that it is unreasonable to use K_i as a design parameter for hydraulic structures, because it is usually possible to accept a limited amount of surface damage. They therefore propose that design be based on a value of $I = 0.2$ (ie $K = 0.8K_i$); Equation B.37 then gives

$$e = (V_o/16.1)^{3.4} \quad (\text{B.40})$$

where again e is in mm and V_o in m/s.

APPENDIX C

TUNNELS AND GATES

C.1 Tunnel inlets Sub-atmospheric pressures can occur at inlets to tunnels due to

1. convergence of the flow
2. curvature of the boundaries
3. turbulent pressure fluctuations in the boundary layers
4. flow separation

In tunnels with high-velocity flows the pressures may become low enough to cause cavitation and damage to the walls. Surface irregularities also are particularly liable to cause cavitation erosion in sections of tunnel downstream of vertical bends.

Galperin et al (1977) describe damage which occurred at the intakes to the bottom sluices of Bratsk Dam (USSR). Subsequent calculations showed that the mean pressures along the walls of the inlets would have been low enough to produce cavitation, even without taking the effect of turbulent fluctuations into account. However, predicted pressure distributions or pressure measurements in models can be misleading if the flow separates, because the lowest pressures will occur away from the boundaries.

Yan et al (1982) carried out model tests to determine the causes of cavitation damage at the inlet to a short spillway tunnel. Downstream conditions caused the tunnel to flow full, and flow separation in the inlet was found to occur due to its unfavourable geometry and to jets issuing from gate shafts in the roof of the tunnel.

Hsu & Zhao (1982) used the technique of conformal transformation to calculate the pressure distribution in two-dimensional inlets having level inverts and converging roofs of circular or elliptical shape. The results were found to agree with experimental measurements except in those regions where flow separation occurred.

Zhu et al (1982) used the relaxation method to determine pressure variations in square tunnels having axisymmetric circular inlets. The values of pressure coefficient agreed satisfactorily with experimental data. Tests were also carried out to determine pressure distributions and head losses for rectangular inlets with a level invert and converging side walls and roof of elliptical section.

C.2 Prototype data on gates

Cavitation is a recognised danger at high-head gates such as those which are used to control flows in low-level outlet tunnels in dams. The cavities are often formed at points where the flow separates from a boundary, such as at the lip of a gate or at the corners of a slot. If a gate is partially submerged on the downstream side, cavitation can occur in the intense shear layer formed between the high-velocity jet and the more static water above it. The cavities generated at a gate may not collapse and cause damage until they have been carried some distance downstream by the flow. Also surface irregularities on tunnel walls just downstream of gates are particularly liable to cause cavitation because the boundary layers have not developed sufficiently to protect the irregularities from high local velocities.

Significant improvements in performance can often be obtained by quite small changes in the configuration of a gate or its slot, but these details usually need

to be studied in a model. Stainless steel linings are sometimes used downstream of gates to protect concrete surfaces from cavitation damage. Due to the high cost of such linings, it is necessary to keep their length as short as possible. However, steel is not immune from cavitation damage, and problems can be caused by inadequate fixing and by the sudden change in surface finish at the downstream end of the lining.

Some examples will now be given of cavitation damage in prototype installations. Destenay & Bernard (1968) provide an interesting survey of French experience. Of 400 hydro-electric schemes, 21 suffered some erosion due to cavitation. These structures tended to be those which had operated at high flows for long periods. This figure of 21 included one surface spillway, one mid-level outlet and two bottom outlets. Four cases were caused by cavitation at gate slots: the erosion was fairly localised and its depth was typically 100mm. The most serious damage occurred in the bottom outlet of Serre-Ponçon Dam (France). The tunnel was protected by a 20mm thick steel lining for a distance of 15m downstream of the control gate. After operating at heads of up to 85m, a hole formed 10m downstream of the end of the lining, and reached a depth of 4m with a volume of 360m^3 . The cavitation may have been caused by the transition in tunnel shape from rectangular to circular. The damage was repaired, but after further operation at heads of up to 105m, a new hole 2m deep formed close to the end of the steel lining. Some damage of the lining was also caused by cavitation at the gate slot.

Schmitt (1971) describes problems at Kinzua and Madden Dams (USA) which occurred downstream of gate slots near the entrances to the low-level tunnels. Cavitation was caused by an interaction between the

flow in the tunnel and a high-velocity jet travelling down the vertical gate shaft, which was open at its top end to the reservoir. The problem was solved by preventing flow down the shaft.

Vinnogg (1971) provides details of two tunnels in Norway which were damaged by cavitation. The control gates were operated $1/3$ - and $2/3$ -open for more than 60 days in each condition. Cavitation originated at the gate slots and caused erosion, which in turn led to worse damage further downstream.

Galperin et al (1977) give examples of serious cavitation damage which illustrate the wide range of possible causes. For gated structures, these included : inadequate surface smoothness of walls and liners; insufficient length of steel lining; blockage of an aeration device at a radial gate; provision of an insufficient air supply; gap cavitation at radial and leaf gates, and failure to follow procedures regarding symmetrical gate operation.

Cavitation damage in the sluices of Libby and Dworshak Dams (USA) is described by Regan et al (1979). The dams are of similar design, and each has three sluices which are controlled by radial tainter gates and which discharge on to a chute spillway. At Libby Dam, steel liners were used close to the gates but cavitation damage occurred further downstream. At Dworshak, one sluice was unlined, one was protected by a 0.9mm thick epoxy paint layer, and the third by a 13mm thick layer of epoxy grout. All three sluices, including the linings, were damaged. The vertical profiles of the sluices were designed to conform to the trajectories of free jets. Inadequacies in these profiles and in their construction were believed to have been the cause of the cavitation.

Jin et al (1980) obtained data on the performance of 158 gates and slots installed in 85 different projects in China. Of the former total, 85 were operating gates, 44 were emergency gates and 29 were service gates for penstocks; 32 of the gates have been subject to some cavitation damage. The following conclusions were drawn from the study:

1. more damage occurs with operating gates than emergency ones due to higher velocities, lower pressures and more frequent operations;
2. gate slots near the upstream ends of tunnels are more liable to damage because curvature of the entrance walls produces low pressures;
3. damage is more likely with partially-open gates;
4. damage is likely to occur at plain rectangular slots if the operating head exceeds 30m;
5. gate slots with length/depth ratios (L/h , see Figure 5) greater than 2.5 or in the range 0.8-1.2 are liable to cause damage.

Erosion downstream of three control gates led to the collapse of a 13.7m diameter tunnel (No 2) at Tarbela Dam (Pakistan) in 1974. The main damage occurred on the invert of the tunnel over a distance of about 45m and reached a depth of 5m. Kenn & Garrod (1981) concluded that this erosion was the result of cavities originating in vertical shear layers, which formed at the downstream ends of the walls separating the three

gates. The divide walls themselves were also damaged, possibly by cavitation in horizontal shear layers caused by the gates operating under partially submerged conditions. Erosion started when the velocity in the tunnel exceeded about 30m/s.

Lesleighter (1983) describes cavitation which occurred at Dartmouth Dam (Australia) in a 3m x 1.5m tunnel downstream of control gates operating at heads of up to 160m. The design, which was based on the results of a model test, included a stainless steel liner and the use of compressed air injected into the flow. Despite these precautions, cavitation caused denting of the steel lining. After further model testing, ramps were added to the sidewalls to produce increased aeration of the water.

Sharma & Goel (1983) give details of damage in a 7.62m diameter tunnel forming part of the Beas Sutlej Link Project (India). Cavitation resulted from flow separating at the downstream end of a central dividing wall. Negative pressures of 3-4m head of water were measured, and erosion reached a depth of 125-400mm. The problem was remedied by supplying air to a number of nipples fitted to the surface of the divide wall. The concrete was repaired using 75mm thick epoxy mortar with two coats of epoxy paint.

Shengzhong (1984) reports damage in the slots of two gates at Liujiaxia Dam (China). Cavitation occurred when the operating head exceeded about 50m, and originated at the point where the gate rail formed a notch in the downstream face of each slot. The problem was studied in a model, and solved by filling in the notch to give a rounded corner.

In Canada serious cavitation damage was reported by Yung & Pataky (1986) to have occurred at the gate slots of two spillways and also downstream of a bulkhead gate in a low-level outlet. At Terzaghi Dam (Canada) low-level gated outlets discharging through a plug in the diversion tunnel caused cavitation erosion downstream. As a result steel constrictors were installed in the outlets downstream of the gates, and these satisfactorily prevented further damage.

These examples suggest that cavitation in tunnels can be due to a variety of factors, and that often the cause is specific to the particular project. Remedial measures also differ, and include use of alternative lining materials, modifications to the flow geometry and injection of air.

C.3 Design of gates Horizontal loads on vertical lift gates are transferred to rails or bearing plates, which are usually placed in vertical slots in the side walls so as to remove them from regions of high-velocity flow. Cavitation problems can be avoided completely by locating the slots on the upstream side of the gate, but this leads to structural difficulties and is not common. Alternatively, with slots on the downstream side, sliding plates can be fitted to the gate in order to close off each slot and present a smooth boundary to the flow. However, this solution requires deep wells to accept the cover plates when the gate is in its closed position. Therefore, in most cases, the gate slots are located on the downstream side of vertical gates and are open to the flow. Several model studies have been carried out to establish suitable shapes of slot for cavitation-free operation.

Ball (1959) describes the results of extensive studies carried out by the US Bureau of Reclamation. Designs were tested in water or air tunnels by measuring pressures around the perimeters of the slots; some typical shapes are shown in Figure 5. The lowest pressures occurred either on the downstream face of the slot, or on the channel wall adjacent to it. Changes which raised the pressure in the slot tended to lower it on the downstream wall, and vice versa. Restricting the amount of circulation in the slot by keeping it as narrow as possible was beneficial.

Ball found that a simple rectangular slot (Type 1A) was satisfactory for heads of up to 10m; the pressure in the slot (relative to the free-stream value) was positive, but negative on the downstream wall. Adding a deflector at the upstream edge lowered pressures in the slot, and would not be satisfactory unless the deflector were large enough to produce strong aeration. Offsetting or sloping the downstream wall away from the flow (Types 1B and 2A) did not improve the overall performance. Type 3C with a converging wall and rounded transition ($n = 24$, $r \geq 300\text{mm}$) was fairly good, but the best designs studied were Type 4b (radiused transition, $100 \leq r/t \leq 250$) and Type 5A (elliptical transition, $E/t = 4$ or 5). As already mentioned, the slots were evaluated by measuring pressure changes. However, the rectangular slot was also studied in a cavitation tunnel: cavitation was found to occur at a higher value of K than predicted, probably because the surface tapings did not record the minimum pressure in the flow.

Rosanov et al (1965) used a cavitation tunnel to test several types of gate slot. Values of the inception parameter K_i were given separately for the upstream and downstream corners of the slot. For a sharp-edged

upstream corner (as all those in Fig 5) $K_1 = 1.15$; rounding the edge reduced the value slightly to $K_1 = 1.05$. Results for various types of downstream corner are as follows:

Type	K_1
1A	2.2-3.0
3D (n = 12)	0.55
4A (r/L = 0.3)	1.0
4A (r/L = 0.5)	0.7
4A (r/L = 1.0)	0.3
5A (E/L = 1.0)	<0.3

Values are also given in this reference for several more unusual slots with deflectors, air pipes and dentations.

Three designs of vertical slot were tested by Adami (1974) under conditions of free-surface flow : Type 1A (with $L/h = 1.0-2.5$); Type 4A (with $L/h = 1.32$, $r/L = 0.22$); Type 1B (with $L/h = 1.0-2.5$, $t/h = 0.40$). Pressures in the slots were measured by means of tappings, and tests were performed with and without a partially-open gate upstream of the slots. The measurements indicated that the pressures in the slots were close to hydrostatic under all the conditions studied; the largest negative departure from hydrostatic pressure was equivalent to -0.059 times the velocity head of the flow. It was concluded that cavitation should not occur provided sufficient air was supplied to maintain atmospheric pressure above the free surface of the flow.

Galperin et al (1977) analysed the results of several studies on cavitation at sharp-edged gate slots. The effects of various geometric factors on the value of

K_1 were presented in the form

$$K_i = c_1 c_2 c_3 K_{is} \quad (C.1)$$

in which K_{is} is the value for incipient cavitation at the upstream or downstream edge of a square-shaped slot; K_{is} depends only upon the depth h of the slot relative to the width B of the conduit. The factors c_1 , c_2 , c_3 take account respectively of the length-to-depth ratio of the slot, the amount of any offset in the downstream wall, and the relative thickness δ of the boundary layer; δ was calculated from the boundary layer equation for smooth-turbulent flow:

$$\delta/x = 0.37 (V_o x / \nu)^{-0.2} \quad (C.2)$$

where x is the longitudinal distance from the start of the boundary layer. The experimental results are reproduced graphically in Fig 6. These show that the size of the conduit has a significant effect on K_{is} if $B/h < 5$, and that reducing the size of the conduit increases K_{is} . Use of an offset increases the pressure at the downstream edge of the slot and thereby reduces the tendency there for cavitation. However, an offset also raises the value of K_1 for the upstream edge; this is because the offset weakens the vortex in the slot and intensifies the eddies formed by the flow separating at the upstream edge. Cavities generated at the upstream edge will not cause damage until the cavitation plume extends far enough to reach the downstream face of the slot; measurements indicate that this occurs when the cavitation number K of the flow is less than $0.6 K_i$. Results such as these apply when a gate is fully open and the flow past the slot is approximately two-dimensional.

Galperin et al also give data for leaf gates that are partially open. If the supporting mechanism of the gate does not fully occupy the slot, downward flow will occur within the slot and will increase the value of K_1 . Cavitation damage tends to occur first on the wall immediately downstream of the slot, at the levels of the gate lip and the floor. The latter damage is due to the downward flow in the slot which develops into a spiral vortex that is drawn out at floor level. At gate openings of less than 60% the damage on the wall tends to be concentrated near the floor. For gates discharging under submerged conditions, typical values of K_1 (calculated it is thought for a reference point in the jet just downstream of the gate) can vary between $K_1 = 1.0$ at a gate opening of 35% and $K_1 = 2.5$ at an opening of 90%. For gates discharging freely, the values are lower and in the range $K_1 = 0.3-1.0$. For partially-open gates, offsetting the wall downstream of the gate slot is only beneficial in reducing K_1 if there is free-surface flow downstream of the gate.

Serious cavitation can be caused by high pressure flow through small gaps at seals and at gates that are just opening or closing. Cavities may be generated in the gap itself due to flow separation at the upstream end, or in the turbulent shear layer bounding the high-velocity flow downstream of the gap. The value of K_1 depends upon the shape of the gap, and according to Gaperin et al can vary from about 3.5-4.0 for a sharp-edged entrance to 0.4-0.5 for a smoothly-shaped one. Gate seals should therefore have rounded profiles on the upstream side. Tests showed that seals with gaps of less than 0.1mm are safe for short periods; gaps of more than 2mm can cause serious erosion, and the seals may themselves be damaged by vibrations induced by unstable cavity formation.

Radial gates have the advantage of not requiring slots, but they can be difficult to operate under partially-submerged conditions because the trunnions are subjected to fluctuating flow forces. Under these conditions (such as occur in navigation locks), a reverse radial gate may be more suitable. The seals of a radial gate can be attached to the gate (which allows the conduit walls to be kept smooth), or offsets can be introduced in the sides and floor of the conduit to accept recessed seals; the latter type are either inflatable or the gate is pressed tight against them by means of special cams. Galperin et al describe results of cavitation tests with three types of radial gate. For a normal radial gate with attached seals, cavitation under submerged conditions occurs along the bottom edge of the gate, and is particularly intense at the side walls. Values of K_1 varied between about $K_1 = 1.1$ at gate openings of up to 60% and $K_1 = 1.4$ at an opening of 80%. Cavities are also generated downstream of the gate in the shear layer between the jet and the surface roller. Under free-flow conditions, cavitation is generated only at surface irregularities. In the case of a reverse radial gate, cavitation again occurs at the bottom edge but is more influenced by the shape of the lip; for a sharp knife edge $K_1 \approx 2$ and for a streamlined one $K_1 \approx 1.3$. For a normal radial gate with recessed seals, cavitation develops at the offsets in the conduit walls in a similar way to cavitation at the upstream edge of a slot. Under submerged conditions, K_1 was found to vary from about 1.2 to 1.8 as the gate opening was increased from 20% to 60%. For free-flow conditions, the maximum value of K_1 was about 0.3 at a gate opening of 50%.

Galperin et al concluded that, from the point-of-view of cavitation, radial gates have an advantage over

leaf gates only under free-flow conditions, and then only in those cases where the conduit walls cannot be offset downstream of the slots required for the leaf gates. Aeration of gate seats was recommended as a means of preventing damage due to cavitation at gates and at surface irregularities on the downstream walls of conduits (see Section F.4).

Mean and fluctuating pressures were measured by Ethembabaoglu (1978, 1979) in slots of Type 1A, 1B, 5A and 5B. The length-to-depth ratio was varied for values of $L/h \leq 5$. The elliptical transition (Type 5A with $t/h = 0.2$ and $E = h$) gave the best performance of those tested, confirming the findings of Ball and Rosanov described previously. The largest pressure fluctuations occurred at the downstream edge of each slot, and were maximum for length ratios of $3.0 \leq L/h \leq 3.5$; the maximum root mean square pressure fluctuation was $0.24 (\rho V_o^2/2)$, where V_o is the undisturbed flow velocity. The frequency of the vortices which formed in the slot was predicted quite well by the theoretical formula

$$S = \frac{fL}{V_o} = 0.5 (N + 1/4) \quad (C.3)$$

where N is the number of vortices in the slot. One vortex occurred when $L/h < 1.2$, and two for $L/h > 1.2$ (up to the value of $L/h = 5$ studied in the tests).

Jin et al (1980) carried out extensive tests in a cavitation tunnel to determine how the parameter K_1 varies with the geometry of the gate slot. Two sources of cavitation can exist simultaneously in a slot: "fixed" cavitation due to flow separation, and "vortex" cavitation due to the formation of one or more vortices in the slot. In narrow slots (eg, 0.75

$\leq L/h \leq 1.5$) vortex cavitation predominates and determines the overall K_1 value of the slot. In wider slots (eg, $2.0 \leq L/h \leq 3.5$) the vortex becomes weaker and the K_1 value is determined by the fixed cavitation.

The tests showed that, for satisfactory performance, gate slots should have length/depth ratios in the range $1.4 \leq L/h \leq 2.5$; best results are obtained if $1.6 \leq L/h \leq 1.8$. Measurements of K_{1r} for plain rectangular gate slots of Type 1A were described by the empirical formula

$$K_{1r} = 0.38 (L/h) \quad , \quad \text{for } 1.5 \leq L/H \leq 3.5 \quad (C.4)$$

The values of the cavitation parameter were calculated using the average velocity and pressure just upstream of the slot.

Slots of Type 3B with offsets and sloping downstream walls have lower values of K_1 than plain rectangular slots. Values can be calculated from the empirical relation

$$\frac{K_1}{K_{1r}} = 0.128 (t/L)^{-0.36} \quad (C.5)$$

where t is the amount of the offset and K_{1r} is the value for the plain slot given by Equation C.4. The slots which were tested had downstream walls with a slope of $n = 12$. It was recommended that the amount of offset should be in the range $0.05 \leq t/L \leq 0.08$.

Tests with slots of Type 2B showed that cavitation will develop on the downstream sloping wall at

$$K_1 = 1.0 n^{-0.7} \quad (C.6)$$

When n becomes large, other features of the slot predominate and determine its overall value of K_1 . If

the downstream wall is to be protected with steel, it was recommended that the slope should be in the range $10 \leq n \leq 12$.

Rounding the downstream edge of the slot (as in Type 4A) gave lower critical cavitation numbers than the corresponding rectangular slot. The results were described by the empirical equation

$$\frac{K_i}{K_{ir}} = 0.436 (r/L)^{-0.18} \quad (C.7)$$

where r is the radius of the edge and K_{ir} is obtained from Equation C.4. Based on Equations C.5 and C.7 it was found that the combined effect of an offset and a rounded edge (as for Type 3D) could be approximated by

$$\frac{K_i}{K_{ir}} = 0.128 \left[(t/L) + \frac{(r/L)^{\frac{1}{2}}}{30} \right]^{-0.36} \quad (C.8)$$

This result shows that an offset is normally more effective in reducing the value of K_i than rounding.

Overall, Jin et al concluded that a simple rectangular slot will be suitable if the cavitation number of the flow has a value of $K > 1$. However, if $K < 0.4$, then particular care is needed in the design, model testing and construction of the gate slot. Comparison of the model and prototype performance of gate slots for two hydro-electric schemes indicated that the models overestimated the actual values of K_i by between 7% and 16%. For design, it was recommended that a safety factor of 20% be adopted.

Sharma & Goel (1983) stress the importance of removing the downstream channel wall from the cavitation

collapse zone. Gate slots of Type 3A ($t/L = 0.1-0.2$, $n \geq 10$) and Type 4B ($t/L = 0.1-0.2$, $r/L \geq 5$) are recommended. The authors also discuss suitable shapes of gate lips. Lips should be designed so that either the flow separates cleanly at the upstream end of the lip, or remains attached until it reaches the downstream edge. If the flow separates and then re-attaches to the lip a short distance downstream, the flow becomes unstable and may produce cavitation and also damaging vibration.

Measurements of mean and fluctuating pressures in rectangular gate slots of Type 1A were made by Yue (1984). Five types of flow pattern were observed according to the length/depth ratio of the slot, which was varied between $L/h = 0.25-8.0$. Measurements of the velocity profiles showed that the free-stream flow expanded into the slot at an angle of about 10° relative to the floor of the channel.

Naudascher & Locher (1974) studied the flow-induced vibrations of small rectangular walls projecting from a plane surface. The walls were similar in shape to irregularity Type 5A in Fig 1, with values of $L/h = 1$ and 3; the width of the tunnel was $6h$. With the square wall the flow separated cleanly, but for $L/h = 3$ there was unstable re-attachment which resulted in the rms forces being increased by a factor of 2.5; stable re-attachment occurred when $L/h > 4.5$. Cavitation started at a value of about $K_1 = 4$ for both shapes of wall (defined using the velocity and static pressure upstream of the wall). The effect of oscillating the walls in the direction transverse to the flow was also investigated: this increased the forces considerably in the case of the square wall, but had little effect when there was unstable re-attachment. The results of the study give an

indication of the loads which might occur on a vertical lift gate when almost fully open.

APPENDIX D

ENERGY DISSIPATORS

This section is concerned with the particular problems of energy dissipators in which high levels of turbulence can result in cavitation.

Bowers & Tsai (1969) describe results from model studies of spillway stilling basins. Maximum pressure fluctuations occur downstream of the toe of the hydraulic jump, and can be up to 40% of the incoming velocity head. If drainage pipes below the surface of a spillway discharge into a stilling basin, there is a danger that positive pressure peaks in the basin could result in large uplift forces on the spillway slabs. Negative fluctuations can lead to cavitation if the pressures drop close to vapour pressure.

Narayanan (1980) analysed data on pressure fluctuations in hydraulic jumps, and concluded that the rms variation was about 0.05 times the upstream velocity head. The probability or intermittency of pressures reaching vapour pressure (and hence producing cavitation) was calculated by assuming that the variations followed a normal distribution.

Measurements of the pressure fluctuations beneath free and forced hydraulic jumps were made by Akbari et al (1982). For free jumps on plain horizontal floors, the maximum rms pressure variations decreased from about 5.3% of the upstream velocity head at a Froude number of $F_1 = 6.2$ to 3.0% at $F_1 = 11.5$. In the case of forced jumps produced by a sill, the maximum rms fluctuations varied from about 5% to 8%, increasing as the sill was moved closer to the toe of the jump; for a given configuration, the relative degree of turbulence decreased as F_1 was made larger.

Lopardo et al (1982, 1984, 1985) compared measurements of pressure fluctuations in a prototype stilling basin and a 1:50 scale Froudian model. The rms values of the fluctuations and the probabilities of occurrence of different amplitudes were well predicted by the model. The incidence of cavitation damage in the prototype also correlated satisfactorily with the model measurements; the results suggested that cavitation may occur if the instantaneous pressure falls below vapour pressure for more than 0.1% of the time (in the first two papers, Lopardo et al referred to a limiting intermittency of 2%). In general the pressure variations were not distributed symmetrically about the mean value (cf Narayanan's assumption above). Tests on a 1:60 model of a second stilling basin showed that the positive pressure fluctuations were larger than the negative ones as long as the flow remained attached to the spillway channel. However, in separation zones (eg downstream of baffle blocks, sills etc) the situation was reversed, and the negative fluctuations became bigger than the positive ones. Evidence from the prototype suggests that models may tend to overestimate somewhat the amount of this asymmetry. The maximum rms values of the pressure fluctuations on the floor of the basin varied between about 5% and 9% of the velocity head entering the jump, depending upon the layout of the basin and upon the entrance conditions. A pressure tapping in the downstream face of a chute block indicated an rms variation equal to 27% of the incoming velocity head.

Baffle blocks and other appurtenances used in stilling basins need to have large drag coefficients to be effective. However, the turbulence generated by the blocks also tends to make them liable to cavitation damage. Careful design is therefore needed to reconcile the conflicting demands of good drag and cavitation characteristics.

Research on the cavitation performance of baffle blocks appears to have been mainly concentrated in the USSR. Quintela & Ramos (1980) give a useful summary of some of the Russian work which is not otherwise readily available.

Iuditski (1965) studied cavitation at baffle blocks at Novosibirsk Dam (USSR) using a 1:53 scale model in a vacuum test rig. Points at which cavitation pressures were recorded in the model coincided with those at which damage had occurred in the prototype. Flow separation at the upstream face of the blocks caused erosion along the sides, while separation at the downstream corners produced damage on the adjacent areas of floor.

Pressure measurements at baffle blocks tend to underestimate the value of the incipient cavitation parameter because the lowest pressures do not occur at the surface of the block. Rosanov et al (1965) found that the true K_i is related to the value K_{ip} obtained from pressure measurements (allowing for fluctuations) by

$$K_i = \xi K_{ip} \quad (D.1)$$

where $\xi = 1.8$ for cubic shapes and $\xi = 1.45$ for pyramidal and rhombic shapes.

Rozanov et al (1971) give values of the inception parameter K_i for various types of block. For a cube of side 100mm set normal to the flow $K_i = 2.2$, while rotating it through 45° reduces the figure to $K_i = 1.1$ (calculated using the depth of water above the block and the velocity of flow entering the jump). Rounding the corners lowers the value of K_i , but also reduces the drag coefficient. Damage can also be controlled by injecting air or water into the separation zones.

Comparative tests were carried out in a cavitation tunnel (no free surface) and a vacuum test rig which allowed the hydraulic jump to be reproduced: the cavitation tunnel gave values of K_1 lower by about 10-20%. Laboratory and field measurements indicated that the greatest rate of damage to the blocks occurred at a cavitation intensity of about $I = 0.7$ (see Equation 5).

Galperin et al (1977) describe pressure measurements made on four types of truncated pyramidal baffle block; the slopes of the upstream and downstream faces were respectively 1:1 and 1:0.5 (vertical : horizontal). The sides of three of the blocks were sloped outwards in the direction of flow so as to facilitate the passage of ice and floating debris. This sloping gave rise to lower (ie more adverse) pressures than a fourth baffle with parallel sides. Rounding the upstream corners of pyramidal blocks was recommended to reduce the danger of cavitation (radius = 0.05 times overall block width). The transverse distance between adjacent baffles was found not to affect the value of K_1 unless the clear distance was less than 1.5 times the block width; reducing the spacing reduced K_1 . Galperin et al also give results for six types of wedge block which may be installed in sills at the downstream ends of stilling basins to increase the amount of energy dissipation; the values of K_1 varied from 1.91 to 1.05. Jet splitters may be used at the downstream end of a spillway to form a slotted lip which breaks up the flow into upper and lower jets. Tests showed that serious vortex cavitation will begin along the sides of such splitters at about $K_1 = 0.7$; rounding the longitudinal edges of the splitters (radius = 0.07 times width of splitter) reduced K_1 to about 0.15.

In general the most favourable cavitation characteristics for baffle blocks are obtained by placing a downstream step in the floor, and sloping the top and sides of the block away from the flow so that cavities are prevented from collapsing against any solid surfaces. The concept can be extended to the design of supercavitating blocks in which the flow separates to form a fixed cavity which extends downstream of the block. Oskolkov & Semenov (1979) give details of four types of supercavitating block, and these are reproduced in Figure 7 (Types 1-4).

Rozanova & Ariel (1983) measured the drag coefficients of four kinds of baffle block (Types 5-8 in Figure 7); note that although Types 2 and 8 are similar in shape, they have different proportions. The tests showed that the drag coefficient of a block was constant for values of $K > K_i$, but decreased when cavitation occurred. The results were found to fit the formula

$$\frac{C_d}{C_{do}} = \left(\frac{K}{K_i}\right)^{0.6}, \quad 0.4 \leq \frac{K}{K_i} \leq 1.0 \quad (D.2)$$

where C_d and C_{do} are respectively the drag coefficients with and without cavitation. Values of C_{do} and K_i for the four shapes tested are given in Figure 7.

Jin (1983) tested four designs of baffle block, of which one was of supercavitating type. The experiments were carried out using free-surface flows with Froude numbers between 4.8 and 7.8. Measurements were made of the cavitation index K_i and also of the mean and fluctuating pressures on the surface of the blocks. The pressure fluctuations varied between 0.51 and 0.23 times the upstream velocity head, depending upon the shape of the block and the Froude number of the flow.

Energy can be dissipated in high-head tunnels by means of sudden expansions which convert kinetic energy into turbulence. Cavities are liable to be formed around the perimeter of the high velocity jet, and can damage the walls of the chamber if they are too close.

Tests on cylindrical expansions were carried out in a cavitation tank by Rouse & Jezdinsky (1965, 1966). The condition of incipient cavitation was determined acoustically for different ratios of the upstream and downstream pipe diameters, D_u and D_d . Values of the incipient cavitation index (calculated using the velocity and static pressure upstream of the expansion) ranged from $K_i = 0.6$ at $D_u/D_d \approx 0$ to $K_i = 0.45$ at $D_u/D_d = 0.6$. However, the more important criterion is the parameter K_{id} at which damage starts to occur on the chamber walls: values were in the range of $K_{id} = 0.08$ to 0.15 , so that the use of K_i for design should provide a considerable safety factor. Large positive pressure fluctuations take place just upstream of the point at which the high-velocity jet reattaches to the chamber wall, and these can give rise to damaging structural vibrations.

Russell & Ball (1967) used a 1:56.6 model to study the design of a dissipator for Mica Dam in which three conduits discharged into a single expansion chamber. The cavitation parameter was defined as

$$K = \frac{p_d - p_v}{p_u - p_d} \quad (D.3)$$

in which p_u is the upstream total pressure and p_d is the downstream static pressure. Values of K_i proved to be larger than expected, and were sensitive to changes in the spatial configuration of the three conduits. The model was tested under heads close to those in the prototype (about 140m). Incipient

cavitation occurred in the range of $K_i = 2.5$ to 3.0 and damage started at $K_{id} = 0.6$.

Ripken & Hayakawa (1972) studied the performance of a jet-valve dissipator using a model with an 83mm diameter orifice discharging into a 152mm diameter chamber. The cavitation parameter was defined as

$$K = \frac{P_d - P_v}{P_u - P_d} \quad (D.4)$$

Cavitation started between $K_i = 1.7$ and 2.3 , and damage at the wall occurred at $K_{id} = 0.58$. The amount of damage was reduced by adding vortex generators around the perimeter of the orifice. This permitted a reduction in the length of the expansion chamber, but increased the value of K_i . The different definitions of K used in these various studies make it difficult to compare results without having access to the original data.

Scale effects in modelling cavitation in sudden enlargements were investigated by Ball et al (1975). The limit of incipient cavitation was found to vary with changes in size but not with changes in the pressure at which the tests were carried out.

However, exactly the opposite applies to the limit of incipient damage, which was defined to be a rate of 1 pit/in²/minute on soft aluminium. This definition is a convenient measure for experimental work, but may itself be subject to a type of scale effect because the volumes of the pits increase as the size of the model increases.

Information on the related topic of cavitation at pipe orifices is provided by Tullis & Govindarajan (1973). The ratio of orifice diameter to pipe diameter, D_o/D ,

was varied between 0.33 and 0.88 in pipes with diameters ranging from 27.4mm to 587mm. Cavitation was detected by changes in the intensity of turbulence recorded by an accelerometer. Values of the incipient cavitation parameter (defined according to Equation D.4) varied from about $K_i = 1.5$ at $D_o/D = 0.4$ to $K_i = 11$ at $D_o/D = 0.8$. Scale effects were found due to changes in size, but not due to changes in pressure or velocity.

APPENDIX E

CAVITATION RESISTANCE OF MATERIALS

E.1 Concrete

Inozemtsev et al (1965) carried out a comprehensive investigation of the factors affecting the resistance of different concretes. Samples were tested in a laboratory water tunnel by placing them downstream of a cylinder which generated cavities in its wake; the flow velocity in the plane of the cylinder was 26.4m/s. The rate of loss of weight was recorded, and a test was terminated if the depth of erosion reached 5mm.

Good resistance characteristics of concrete were found to be associated with a high compressive strength and a low water/cement ratio. The cavitation resistance is determined by the internal cohesion of the binder and by the adhesion between the binder and the aggregate; the strength of the aggregate itself is not usually a factor. Large, dense aggregates produce low resistance because the forces of adhesion are weak; best results are obtained if the aggregate is porous, if the cement and aggregate are as similar in size as possible, and if the aggregate reacts chemically with the cement.

Of the ordinary concretes tested, the highest resistance occurred with cement clinker aggregate (loss rate of 3.1g/hour) and the lowest with gravel aggregate (32g/hour); crushed limestone and crushed granite were intermediate. Grinding of the cement also improved the erosion properties, and the optimum fineness was found to be 4000cm²/g. Fine-grained vibromix concrete and concrete with crushed granite and autoclave curing were about 25 times more resistant than gravel concrete.

Plastic concretes were also tested and were found to have resistances that were 10-100 times higher than normal cement concretes. The loss rates for epoxy-polyester plastic concretes with sand and graphite aggregates were between 0.03 and 0.21g/hour. The best results were obtained with an epoxy-thiokol plastic concrete which had a performance similar to that of steel, and showed no weight loss after 12 hours. A coating of epoxy resin improved the cavitation resistance of ordinary concrete, and was more effective than using FA monomer.

The effect of surface finish on the rate of cavitation damage was investigated by Thiruvengadam (1960). Similar samples of granite were polished and then roughened to different degrees. It was found that the smoother the surface, the lower was the initial rate of weight loss due to cavitation. However, polishing gives only a temporary benefit since cavitation attack will eventually roughen the surface anyway.

Kenn (1971) tested samples of concrete in a cavitation rig similar in type to that used by Inozemtsev et al (see above). Compressive strengths of 41.5MPa and 20.7MPa were obtained with water/cement ratios of 0.60 and 0.80 respectively; the aggregate size was 10mm. The cavitation resistance of the normal 41.5MPa concrete was significantly higher than that of the half-strength material. It was also found that the amount of damage could be much reduced by protecting the concrete with a 6mm thick layer of Renfor cement or Renfor tropical grout.

Galperin et al (1971) give data on the relationship between the flow velocity in a structure and the compressive strength of concrete needed to resist cavitation. The results were shown graphically but can be approximated by

$$V = U + 0.43M \quad (E.1)$$

where V is the allowable velocity in m/s and M is the compressive strength in MPa. For compressive strengths in the range $20 \leq M \leq 50$ MPa, the constant U has a value of approximately $U = 1.5$ m/s.

Kudriashov et al (1983) also presented data on allowable flow velocities adjacent to concrete surfaces. The results agreed with the form of Equation (E.1), but the value of the constant was approximately $U = 3.0$ m/s for compressive strengths of $20 \leq M \leq 50$ MPa. According to Novikova & Semenov (1985), the allowable velocities given by Kudriashov et al are for an incubation period of 48 hours. Allowable velocities V_T for other periods T (in hours) can be calculated from

$$V_T = \frac{V}{2} [1 + (48/T)^{1/8}] \quad (E.2)$$

The use of steel-fibre concrete to repair cavitation damage at Libby Dam (USA) is described by Schrader & Munch (1976). The original concrete which was eroded was of good quality with a water/cement ratio of 0.34-0.42 and a compressive strength at 90 days of 43.1 MPa. This was replaced with concrete containing 1% of 25mm long steel fibres (0.36-0.40 water/cement ratio, 19mm maximum aggregate size, 433kg/m³ of cement and about 5% entrained air). The compressive strength at 28 days was 48.0-55.0 MPa, and at 90 days exceeded 67.1 MPa. The material was stiff unless vibrated, but was placed successfully and had an appearance and surface texture similar to that of the original concrete. Fibrous concrete was also used for repairs at Dworshak Dam (USA), and Regan et al (1979) report that no significant erosion of the new material occurred.

At Dworshak Dam some of the fibrous concrete was also polymerized to increase further its durability. Details of the technique are given by Murray & Schultheis (1977) and by Stebbins (1978), and consisted essentially of soaking an area of cured concrete with a monomer which was then polymerized by the application of heat. The constituents by weight of the monomer were 95% methylmethacrylate (MMA), 5% trimethylolpropane trimethacrylate (TMPTMA, cross-linking agent) and 0.5% catalyst. Before applying the monomer it was necessary to dry the concrete, and this was done by using infra-red lamps to heat it to a temperature between 127°C and 150°C for 8 to 10 hours. The concrete was then allowed to cool to 38°C, after which it was soaked with monomer for 5 to 6 hours. Polymerization was achieved by heating for 2 hours to a temperature between 65°C and 99°C using water or dry steam. The technique was carried out on both horizontal and vertical areas of concrete and was considered viable, although it did require careful control. The fibrous concrete was polymerized to a depth of up to 38mm, and this increased its compressive strength from 55MPa to about 140MPa.

Galperin et al (1977) explain how a denser finish to the concrete surface of the spillway at Krasnoyarsk Dam (USSR) was obtained using absorptive and vacuum formwork. The absorptive panels were lined with timber-fibre sheets covered with dense coarse calico, and were used successfully for the straight sections of the spillway. The vacuum forms were used for the curved sections of the spillway bucket, but movements of the panels gave rise to steps of up to 30-40mm in height. Galperin et al also give test results which showed that adding a relatively small amount of a polymer to concrete could increase its cavitation resistance by a factor of up to 50. Gunita

(shotcrete) was also found to have good cavitation-resisting properties.

Lowe et al (1979) describe comparative cavitation tests on different concretes which were carried out in connection with the repairs to Tarbela Dam (Pakistan). Regular concrete (with a 28 day compressive strength of 31.0MPa) eroded to a depth of 75mm three times as quickly as did steel-fibre concrete (41.4MPa at 28 days) and polymerized ordinary concrete. In the case of polymerized fibrous concrete the depth of erosion did not exceed 25mm. With the fibrous concrete it was possible to use a higher cement ratio because the steel fibres prevented the crazing which would otherwise have occurred.

Details of the remedial works carried out at Tarbela Dam are given by Chao (1980). Damaged areas were initially repaired using regular concrete (with a compressive strength of 41.4MPa) and two coats of epoxy seal. Some of this concrete subsequently failed due to cracking and was replaced with 27.6MPa concrete. The epoxy seal also failed due to poor adhesion. A total of 6000m³ of fibrous concrete was used to reinstate some of the floor slabs of the stilling basins, and in conjunction with an aeration slot performed satisfactorily at flow velocities up to 47m/s.

Jiang & Chen (1982) tested samples of concrete in a cavitation tunnel to investigate how the cavitation resistance was affected by factors such as the water/cement ratio, the use of additives and the age of the concrete. It was found that the cavitation resistance R_c (defined as the inverse of the rate of loss of weight per unit area) varied with the water/cement ratio (W/C) as

$$R_c \propto (W/C)^{-4.83} \quad (E.3)$$

and with the compressive strength M as

$$R_c \propto M^{4.84} \quad (E.4)$$

Preece & Hansson (1983) carried out tests which showed that the cavitation resistance of ordinary concrete could be improved by using a sulphate-resistant portland cement containing silica particles (known commercially as "Densit"). These particles have a size of about $0.1\mu\text{m}$ (compared with the $100\mu\text{m}$ of normal cement particles), and therefore produce a dense mortar which is able to fill the interstices of the aggregate and thus give a strong bond.

Schrader (1983) surveyed the practical aspects of constructing concrete structures to avoid or resist cavitation. Unwanted offsets at joints are sometimes caused by the difficulty of allowing fully for shrinkage, differences in heat of hydration, etc. Tight tolerances do not necessarily prevent the occurrence of significant slope changes. As an example, a limit of 1.5mm deviation per 300mm could result in a slope change of 1/25, while a seemingly less severe criterion of 6mm per 3000mm would restrict the change to 1/60. Designers need to take account of the practical problems of placing concrete when designing reinforcement. If placement is difficult, a contractor will tend to use a finer aggregate and a higher water content, which reduces the strength of the concrete and increases the amount of heating and shrinkage.

Attempting to obtain a smooth finish by overworking the newly-placed concrete with a trowel produces a softer surface that is liable to craze. Grinding to

remove irregularities can be detrimental because it takes away parts of the aggregate which may then be plucked out more easily by the flow; the sudden change in surface roughness may also promote cavitation downstream.

Great care is needed when patching. Where possible the new material should be of the same mix as the surrounding concrete; ideally the two materials should have the same mortar and aggregate, similar surface texture and equal coefficients of shrinkage and thermal expansion. If the patch is harder than the surrounding concrete, it will tend to project above it. Patches can also shrink away from the base material, and thus be plucked out completely by the flow.

Although epoxy materials have a good cavitation resistance, they may fail due to the formation of a "glue-line" at the edges of the surrounding concrete. Water or vapour pressure, or the effects of differential expansion or shrinkage can cause the concrete below the glue-line to fail so that the epoxy is lost in a lump; it is therefore important to obtain good continuity at the joint. The difference in surface texture between epoxy materials and concrete can be considerable, and may give rise to cavitation.

Polymerizing concrete increases its strength and cavitation resistance by a factor of three, and is effective in producing a good bond at joints and repairs. However, it is also expensive. Steel-fibre concrete has proved successful, but may still be eroded by the grinding action of debris (eg in stilling basins). Adding 0.5-1.5% by volume of steel fibres increases the cavitation resistance by a factor of three, but has little effect on strength. The

fibres are effective because they enable the concrete to absorb high-frequency impacts without suffering fatigue failure.

Zheng (1984) measured the cavitation resistance of bitumen mortar, and showed that, under certain conditions, it was slightly higher than that of ordinary cement mortar. Unlike most other materials, the resistance of the bitumen mortar was found to increase as its elastic modulus decreased.

The American Concrete Institute is preparing a guide on the erosion of concrete which includes sections on cavitation damage and methods of repair, but at the time of writing this had not been published.

E.2 Metals

A considerable amount of laboratory work has been carried out to compare the resistance of different metals to cavitation. Mousson (1937) tested a large number of steels and other metals in a venturi tunnel using water at 20°C, and measured the loss of volume which occurred after 16 hours. The results show that the amount of damage varies with the chemical content of the metal and also with the method of forming (eg cast, rolled or forged). A small selection of the data is given below to illustrate the range of values obtained. The values of volume loss are only relative since they are specific to the type of equipment and intensity of cavitation used in the tests.

<u>Metal</u>	<u>Volume loss after 16 hours</u> (mm ³)
aluminium alloy	3420
phosphor copper bronze	1370
cast iron	636
Mn bronze	300

Low-alloyed steels

0.30% rolled carbon steel	135.0
0.33% cast carbon steel	62.4
0.22% forged carbon steel	36.8
cast Cr Mo steel	4.7

High-alloyed steels

14% Cr forged stainless steel	167.3
15% Cr Ni cast stainless steel	113.0
17% Cr rolled stainless steel	103.0
forged Monel steel	26.6
cast Stellite steel	2.1
rolled Stellite steel	0.9

Mousson's results together with data from other sources are available in convenient form in Chapter 9 of the book by Knapp et al (1970).

Abelev et al (1971) tested samples of different steels and protective coatings in venturi tunnels with flow velocities of up to 60m/s. The results were as follows:

carbon steel	~ pitting all over surface after 25 hours
stainless steel (1X18H9T)	- no erosion after 200 hours
epoxy-thiokol over carbon steel	- upper layers damaged after 40 hours
rubber over carbon steel	- slight breaking away after 100 hours
nyrite over carbon steel	- slight erosion after 200 hours

Although steel linings are often used in tunnels downstream of high-head gates, Locher & Hsu (1984) mention that armouring chute blocks and baffle blocks in stilling basins has not proved successful because of the difficulties of fixing.

Li & Huang (1985) studied the relationship between the cavitation resistance of eight different metals and their ultimate resilience. The results were found to fit the formula

$$\frac{\Delta v}{\Delta t} = 3.4 H_{v5}^{-2.145} \quad (E.5)$$

where $\Delta v / \Delta t$ is the rate of volume loss of the test sample in mm^3/h , and H_{v5} is (believed to be) the Vickers Hardness of the material, measured using an applied load of 5kg.

An ICOLD Committee (1986) found that there were no definite guidelines on how far steel linings should be extended downstream of orifices or gates. It was suggested that, if the flow velocity exceeds 25m/s, steel protection should be provided for the following distances:

floor - 50 R

full wetted height of side walls - 15 R

half wetted height of side walls - 30 R

where R is the hydraulic radius of the orifice or gate opening. Steel linings in flip buckets and stilling basins should be well drained and tied back to the concrete in order to resist the jetting action of the flow.

E.3 Epoxy and polyester resins

A useful guide to the properties and uses of these resins is given by Tabor (1978). Polyester resins belong to the group known as alkyds or glyptals, and they develop their strength by the formation of connections between similar molecules. The reaction is inhibited by the presence of other trace chemicals, and is started by the addition of a catalyst. The resin can be made easier to use by adding a diluent which has similar connectors and therefore takes part in the reaction.

By contrast epoxy resins consist of two different chemicals with "epoxide" groups which react, when brought together, to form a solid. The liquid resin has a good affinity for concrete and so forms a strong bond. The amount of hardener needs to be measured accurately so as to ensure that all the resin can be converted. The rate of reaction is affected by temperature, and can be increased by the addition of a chemical accelerator.

Resins can be used directly as adhesives and surface coatings, or they can be mixed with inert mineral fillers or aggregates to produce mortars. Epoxy and polyester resins have fairly similar properties: compressive strengths about 2.5 times that of portland cement mortar or concrete; Young's moduli approximately 0.1-0.3 times that of concrete; coefficients of thermal expansion about 3 times that of concrete. Resins also tend to creep under load much more than conventional materials. The properties of resin mortars can, however, be varied considerably by the choice of suitable fillers. Some epoxy resins may not cure if moisture is present, and surfactants must be added to obtain a bond under water. The design of a resin or mortar requires specialist knowledge, and should be tailored to the needs of each particular job. Also the standards of control needed

on site are higher than are normally encountered when working with conventional concrete.

References in the literature suggest that epoxy materials have not performed well in hydraulic structures subject to high velocity flows. It is possible, however, that the failures may have received more attention than the successes.

Wagner & Jabara (1971) report USBR experience on seven dams which suffered cavitation damage. Nearly all the repairs carried out with epoxies or epoxy mortars subsequently failed.

Galperin et al (1977) describe the use of epoxies at Krasnoyarsk Dam (USSR) to rectify surface imperfections found after construction. Holes up to 50mm deep were filled with an epoxy-based plastic mix which performed well. An epoxy-based cement mix was used for holes 50-100mm deep, but many of the repairs failed and caused serious cavitation erosion downstream. Holes deeper than 100mm were filled using concrete (containing 5-20mm size crushed rock) on an epoxy base. A protective layer of epoxy paint was also applied to the surface of the spillway bucket; this was found to delay but not prevent the start of cavitation damage.

Examples of the use of epoxies at Tarbela Dam (Pakistan) are given by Lowe et al (1979) and Chao (1980). The floor and a wall of Tunnel 3A were repaired with ordinary concrete finished with a layer of epoxy concrete. This failed after three years and was replaced with a steel lining. Epoxy coats were applied to concrete surfaces in the stilling basins, but failed as a result of poor bond. Sinmast P-103 paste proved satisfactory for repairing areas where the depth of erosion did not exceed 6mm. However,

where epoxy mortar was used for deeper areas of damage, the concrete below the repair pulled away from it due to the different thermal expansions of the two materials. Patches on walls exposed to direct sunlight failed within a matter of days.

Problems with epoxies are attributed by Warner (1980) to:

1. poor surface preparation (dirt, wet);
2. poor mixing;
3. too much heat generation;
4. unsuitable formulation of epoxy;
5. formulation not compatible with moisture (either present naturally or generated by heat).

At Dworshak Dam (USA) an area of 3m^2 of concrete wall was coated with epoxy mortar. The coating had to be applied three times; on the first occasion the epoxy was improperly mixed, and on the second there was a lack of bond in wet areas. After completion the surface had to be ground to remove sags. Epoxy mortar was also used to repair the stilling basin. Bad weather and insufficient time prevented a satisfactory job (presence of moisture, poor mixing and preparation). Approximately 20% of the epoxy material was lost after 53 days service, and 80% had gone within a few more months.

E.4 Plastics and other materials

Hobbs used flow past a cylinder to study the cavitation resistance of plastics and other materials. Most of the plastics showed little damage, and so were not rated on the basis of weight loss, but visually as follows.

<u>Excellent</u>	monocast nylon
	nylon 66
	high-impact polythene

<u>Very good</u>	"alkathene" polythene "propathene" polypropylene aluminium bronze
<u>Good</u>	nylatron GS stainless steel
<u>Fair</u>	fluorocarbon PTFE "darvic" vinyl high-tensile brass
<u>Bad</u>	penton K51 aluminium alloy
<u>Very bad</u>	perspex acrylic resin.

Although nylon performed well, it has poor fatigue properties and absorbs water. Good cavitation resistance was found to correlate in most cases with a high value of the quantity (tensile strength)^{2/} (elastic modulus); penton and perspex did not fit the pattern.

Inozemtsev et al (1965) mention that sheet rubber is effective in preventing cavitation damage, but that no reliable means of fixing it has been devised. Thin coatings of synthetic rubber increase the life of concrete by a factor of between 3 and 20, but their cavitation resistance is still only 1/10 to 1/20 that of steel.

According to Kenn (1968) the best lining materials are stainless steel, neoprene and thiokol rubber, and these have better cavitation-resisting properties than epoxy and phenolic resins.

Results of tests on some lining materials carried out by Abelev et al (1971) have already been mentioned in Section E.2.

Wagner & Jabara (1971) reported that a neoprene compound was found in US Bureau of Reclamation experience to be the only suitable coating material. A thickness of 70mm was required, and this was built up in 2mm thick layers applied by brush, with a waiting period of up to two hours between each application.

The cavitation resistance of various polymeric materials was studied by Barletta & Ball (1983). No clear relationship was found between resistance and any single mechanical or chemical property. The performance of the materials was rated as follows:

<u>Best</u>	heterogeneous polymers (eg polyamide 6.6 plus polyethylene, and polyacetal plus polyethylene)
<u>Intermediate</u>	homogeneous polymers
<u>Worst</u>	polyurethane and polycarbonate.

Fibre-reinforced and fibre-filled polymers were less resistant than the homogeneous matrix materials alone.

Results of abrasion tests on a polyurethane resin (Sikaflex KW2) were described in an ICOLD (1986) survey. The resin was applied as a protective layer to concrete at Khasm el Girba Dam in the form of a 14mm thick mortar layer and an 8mm thick wearing coat of the neat resin. Laboratory tests showed that the abrasion resistance of neat Sikaflex was intermediate between neat epoxy and steel; the elasticity of the resin may enable it to resist cavitation damage, but test data are not available.

APPENDIX F

AIR ENTRAINMENT

F.1 Effect on cavitation

The presence of air in water lowers the pressures generated by collapsing cavities, and can thereby reduce the amount of damage that they cause. Peterka (1953) studied this beneficial effect of air using concrete samples in a venturi tunnel at flow velocities of about 30m/s. The weight loss due to erosion was approximately halved when the air concentration was $C = 1\%$, and became negligible for $C > 7.4\%$. These conclusions were confirmed by later work by Russell & Sheehan (1974) and by Oskolkov & Semonkov (1979) who found that an air concentration of $C = 7-8\%$ was sufficient to prevent damage to concrete at flow velocities of up to 45m/s.

Reference has already been made in Section E.1 to the data presented by Galperin et al (1971) and Kudriashov et al (1983) on allowable flow velocities for concrete. Tests were also carried out to determine how the amount of air β affects the allowable velocity, where β is defined as:

$$\beta = Q_a / Q_w \quad (F.1)$$

and Q_a is the flow rate of air and Q_w that of the water. The results of both studies can be approximated by Equation (E.1), but correspond to different values of the constant U, as follows:

Amount of Air $\beta(\%)$	Constant U (m/s)	
	Galperin (1971)	Kudriashov (1983)
0	1.5	3
4	7	11
8	14	18

Vorobiyov (1983) found that the volume of cavitation erosion was reduced by a factor γ which varied with the air concentration C (%) as

$$\gamma = 0.11 - 0.01 C, \quad 2\% \leq C \leq 11\% \quad (\text{F.2})$$

A theoretical description of the effect of air on collapsing cavities was provided by Huang et al (1985). The model reproduces the unsymmetrical collapse of cavities near solid boundaries, and shows that entrained air reduces the peak pressures by decreasing the speed of sound in the liquid.

Air tends to be entrained naturally at the surface of a high velocity flow and becomes dispersed through the depth by turbulent mixing. The above results indicate that cavitation damage may be prevented if the resulting air concentration at the bed reaches a value of about 7%. It is therefore important to be able to predict the amount and distribution of air entrained by flow on a spillway. If there is insufficient natural entrainment to prevent cavitation, it is possible to add air to the flow by means of aerators constructed in the floor and walls of the channel or tunnel.

An important factor affecting self-aeration and also the performance of aerators is the rise velocity of air bubbles in water. Data from various sources are summarised by McKeogh et al (1983) as follows

$$v_b = 0.625 r_b^2, \quad r_b \leq 0.4\text{mm} \quad (\text{F.3a})$$

$$v_b = \{(0.01 r_b) + (0.079/r_b)\}^{\frac{1}{2}}, \quad 1\text{mm} \leq r_b \leq 5\text{mm} \quad (\text{F.3b})$$

$$v_b = 0.1 r_b^{\frac{1}{2}}, \quad r_b \geq 10\text{mm} \quad (\text{F.3c})$$

where v_b is the rise velocity in m/s and r_b is the radius of the bubble in mm.

F.2 Self-aeration

Air concentration can be defined in terms of the volumes of air and water, ie

$$C_1 = \frac{v_a}{v_a + v_w} \quad (F.4)$$

or in terms of their flow rates, ie

$$C_2 = \frac{Q_a}{Q_a + Q_w} \quad (F.5)$$

The two definitions are compatible only if the air and water travel at the same velocity (speed and direction). This is a reasonable assumption if the bubbles are small enough for their slip velocity and rise velocity to be small compared with that of the fluid. The choice of definition is usually determined by the experimental technique used to measure the concentration: Equation F.4 would be appropriate for a device that measures the size and number of bubbles in a given volume; Equation F.5 would be suitable where the total rates of air and water supply are known. The symbol C will be used in cases where the concentration is not defined precisely. Results for aerators are sometimes presented in terms of the ratio β in Equation F.1; clearly at low concentrations β and C_2 are nearly equal. A separate problem of definition occurs where a turbulent water surface causes an instrument to be periodically in and out of the flow; in these conditions it may be difficult to determine what proportion of a measurement is due to air bubbles in water and what is due to air above the free surface.

There is general agreement that air entrainment on a spillway starts when the boundary layer grows sufficiently for its thickness δ to be nearly equal to the depth of flow d . Turbulent clumps of liquid then break through the free surface and fall back again, thereby entraining air. The distance along the channel required for this to occur is called the inception length L_1 ; some authors assume that at the point of inception $d = \delta$, while others assume $d = 1.2\delta$ since turbulent eddies can be projected from below the free surface. Downstream of the point of inception three zones can be defined. In the "developing partially-aerated zone" the mechanism of turbulent diffusion causes some of the entrained air to spread downwards as it is carried along by the flow. When air becomes present at the bed, the flow enters the "developing fully-aerated zone" in which the depth of water, the amount of air and its distribution pattern within the flow all continue to vary with distance. Finally, if the channel is long enough and of constant slope, the flow reaches the "uniform aerated zone" where there is no further change in depth or in the vertical profile of air concentration.

A large amount of research has been carried out on self-aeration, and in this review it is appropriate to concentrate mainly on the more recent work. A classic series of experiments on air entrainment in a rough channel was performed by Straub & Anderson (1958), while Anderson (1965) gives corresponding results for a smooth channel. Tests were conducted in a 15.2m long flume with unit discharges up to $0.9\text{m}^3/\text{s}/\text{m}$ and slopes up to 75° . Measurements were made to determine the mean concentration of the air and its distribution with depth for conditions of uniform aerated flow. Below a certain transition depth d_T it was found that the flow consisted mainly of air bubbles in water, while above this depth it was predominantly water

droplets in air; d_T was identified as the point where the rate of change of local air concentration with depth (dC/dy) was maximum. The measured air distributions above and below d_T were able to be fitted to two separate theoretical equations by choosing suitable values of certain coefficients. Based on these and other data, an ASCE Task Committee (1961) recommended the following formula for predicting the mean air concentration (averaged over depth) in rough channels.

$$\bar{C}_1 = 0.743 \log_{10} (\sin \theta / q^{1/5}) + 0.723 \quad (F.6)$$

where θ is the angle of the channel to the horizontal and q is the unit discharge in $m^3/s/m$. The corresponding result for flow in a smooth channel was found by Anderson to be

$$\bar{C}_1 = 0.503 (\sin \theta / q^{2/3}) \quad (F.7)$$

Values of the Darcy-Weisbach friction factor λ were calculated from the equation:

$$\lambda = (8gd_T \sin \theta) / \bar{V}^2 \quad (F.8)$$

where d_T is the transition depth defined previously and $\bar{V}$ is the mean velocity of the water such that:

$$\bar{V} = q / d_e \quad (F.9)$$

Here, d_e is the equivalent water depth calculated from:

$$d_e = \int_0^{\infty} (1-C) dy \quad (F.10)$$

On this basis, it was found that air entrainment did not alter the flow resistance of the rough channel ($\lambda = 0.0315$), but did reduce that of the smooth channel from $\lambda = 0.0204$ to $\lambda = 0.0110$.

A series of fairly similar experiments was carried out by Lakshmana Rao et al (1970), Gangadhariah et al (1970) and Lakshmana Rao & Gangadhariah (1971), a summary of which is given by Lakshmana Rao & Kobus. The data on the variation of air concentration with depth were fitted to different theoretical equations from those used by Straub & Anderson (see above), but again it was necessary to choose suitable values for certain coefficients. For the inception of air entrainment, it was suggested that in addition to the boundary layer reaching the surface, it is necessary for the turbulent fluctuations to have sufficient energy to overcome the force of surface tension; the criterion for this was found to be

$$I_o = \frac{(\rho d V^2 / \sigma)}{(V_* d / \nu)} > 56 \quad (F.11)$$

where V is the average flow velocity, V_* the shear velocity at the bed and σ the surface tension. The following equation was obtained for the mean air concentration in uniform aerated flow

$$1 - \bar{C} = \frac{1}{1 + \Omega F_e^{3/2}} \quad (F.12)$$

where the equivalent Froude number F_e is defined as

$$F_e = \frac{\bar{V}}{(g d_e \cos \theta)^{1/2}} \quad (F.13)$$

and $\bar{V}$ and d_e are respectively the mean velocity and

equivalent water depth calculated from Equations F.9 and F.10. The coefficient Ω is given by:

$$\Omega = 1.35n \quad , \quad \text{for rectangular channels (F.14a)}$$

$$\Omega = 2.16n \quad , \quad \text{for trapezoidal channels (F.14b)}$$

with n being the Manning roughness coefficient of the channel. In the experiments, values of n for aerated flows were determined from an analogue of Equation F.8 used by Straub & Anderson, ie:

$$n = (d_T^{2/3} \sin \theta) / \bar{V} \quad (F.15)$$

Application of Equation F.12 to find $\bar{C}$ in a design situation is not straightforward because values of d_e , $\bar{V}$ and possibly n need to be found first.

The position of the critical point at which air entrainment starts depends on the unit discharge. Galperin et al (1977) give the following field data for high-head spillways:

Unit discharge (m ³ /s/m)	Distance from spillway crest (m)
4.2	30
5.6	40
7.2	50
9.0	60
11.0	70
13.4	80
15.8	90
18.5	100

Observations at Bratsk and Krasnoyarsk Dams (USSR) showed that areas which were eroded when the flow was not aerated did not suffer damage at lower flows when the flow was self-aerated.

Thandaveswara & Lakshmana Rao (1978) studied the region of developing aeration, between the point of

inception and the establishment of uniform flow, using a channel with unit discharges of up to $0.20 \text{ m}^3/\text{s/m}$ and slopes between 15.3° and 30.7° . The measurements indicated that in the developing fully-aerated zone (see above) the air concentration reached a minimum above the bed and not at the bed as other researchers have found. If this finding were confirmed, it would be significant when determining whether the air concentration on the floor of a channel is sufficient to prevent cavitation damage.

Falvey (1979, 1980) correlated Straub & Anderson's data with measurements from four prototype structures (three chutes and one spillway) to obtain the following equation for the mean air concentration in uniform aerated flow

$$\bar{C}_1 = 0.05 F_r - \frac{(\sin \theta)^{\frac{1}{2}} W}{63 F_w}, \quad 0 \leq \bar{C}_1 \leq 0.6 \quad (\text{F.16})$$

where the Froude number is given by:

$$F_w = \frac{V_w}{(gL_w)^{\frac{1}{2}}} \quad (\text{F.17})$$

and the Weber number by:

$$W = V_w \left(\frac{\rho L_w}{\sigma} \right)^{\frac{1}{2}} \quad (\text{F.18})$$

The length dimension L_w is not precisely defined in these references, and it is unclear whether it should be the flow depth, the hydraulic depth (area/surface width), or the hydraulic radius (area/wetted perimeter). The values of V_w and L_w are calculated as though the flow were not aerated. Although the surface tension σ was included in the correlation, its value is likely to have been approximately constant

within the data set used. Air entrainment leads to bulking of the flow, and the depth for design is sometimes assumed to be equal to $d_w/(1-\bar{C})$. However, Falvey (1979) points out that it is not a very useful parameter, because turbulence causes water to rise well above this level.

Wang (1981) used experimental data on mean air concentrations to compare the predictions of six existing formulae, but found that the minimum standard deviation was given by a new equation

$$1-\bar{C} = 0.937 \left[F_r \left(\frac{n^6 g^3}{R_w} \right)^{1/6} \left(\frac{B}{d_w} \right) \right]^{-0.088} \quad (F.19)$$

$$\text{where } F_r = \frac{V_w}{(gR_w)^{1/2}} \quad (F.20)$$

B is the width of the channel, and the depth d_w and the hydraulic radius R_w are calculated assuming non-aerated flow.

Volkart (1982) studied air entrainment in steep partially-filled pipes, and obtained both model and prototype data for pipe diameters up to 900mm and slopes up to 45°. The resulting equation for the mean air concentration was

$$1-\bar{C}_2 = \frac{1}{1+0.02(F_r-6.0)^{1.5}} \quad , \quad 6.0 \leq F_r \leq 15 \quad (F.21)$$

where F_r is calculated from Equation F.20 using the non-aerated flow parameters. The mean velocity V_{aw} of the air-water mixture was given by

$$V_{aw} = (1-\bar{C}_2)^{2.09} V_w \quad (F.22)$$

The area of flow A_m corresponding to the maximum height h_m reached occasionally by the aerated water surface was related to the non-aerated flow area A_w by

$$\frac{A_m - A_w}{A_w} = -2.0 \ln (1 - \bar{C}_2) \quad (F.23)$$

To prevent slug flow occurring in a pipe it was recommended that $h_m/D < 0.9$.

Bruschin (1982) compared Falvey's Equation F.16 and Volkart's Equation F.21 for mean air concentration, and concluded that Equation F.16 did not give reasonable predictions for prototype conditions, possibly due to the second term on the right-hand side not being valid.

Wang (1984) used measured data on mean air concentrations to obtain the following best-fit equation.

$$\bar{C} = 0.538 \left[\frac{nV}{R^{2/3}} - 0.02 \right] \quad (F.24)$$

where n is the Manning roughness of the channel.

An important line of research on air entrainment has stemmed from prototype measurements carried out by Cain & Wood (1981 a,b) on Aviemore Dam (New Zealand). Instruments were developed to determine profiles of air concentration and water velocity along the spillway and also the size of the air bubbles. The spillway slope is 45° , and data were obtained for unit discharges of up to $3.15 \text{ m}^3/\text{s}/\text{m}$; the channel was not long enough to give conditions of uniform aerated flow. Measurements of the point of inception of air entrainment were found to correspond reasonably with

the empirical equation due to Bauer (1954) for the growth of the boundary layer thickness

$$\frac{\delta}{x} = 0.0254 \left(\frac{x}{k_s} \right)^{-0.135} \quad (\text{F.25})$$

where k_s is the equivalent sand roughness of the channel. Downstream of the point of inception it was found that the non-dimensional velocity profile did not vary with the amount of entrained air, and had the form

$$\frac{v}{v_{90}} = \left(\frac{y}{y_{90}} \right)^{0.158}, \quad C \geq 0 \quad (\text{F.26})$$

where the subscript 90 refers to the point above the bed where the air concentration is 90%. This contradicts the results of other investigators (eg, Straub & Anderson, Lakshmana Rao et al, see above) who found that the velocity did not increase steadily with level, but reached a maximum below the surface of the flow. Cain & Wood suggest that the difference arises because they measured the velocity of the water while other investigators measured that of the air-water mixture; if this is the case it suggests that the two phases travel at significantly different speeds, contrary to what is often assumed.

Discrepancies such as these between different studies may be due to the measuring instruments having different operating principles. Most measurements of the velocity and concentration of aerated flows are indirect, and the results may not therefore be exactly comparable. Details of some of these instruments are given in Section G.3.

Wood et al (1983) assumed that the formula for the growth of a boundary layer was similar in form to Bauer's Equation F.25, but evaluated the coefficients

using Equation F.26 together with numerical results obtained by Keller & Rastogi (1977) for the point of inception on standard spillways. This procedure gave

$$\frac{\delta}{x} = 0.0212 \left(\frac{x}{H_s} \right)^{0.11} \left(\frac{x}{k_s} \right)^{-0.10} \quad (\text{F.27})$$

where H_s is the vertical distance from the upstream total energy line to the surface of the water in the spillway. The form of the equation allows it to be applied to channels of non-uniform slope.

Wood (1983) re-analysed Straub & Anderson's data, and concluded that uniform aerated flow was not in fact achieved in all the tests. Where equilibrium conditions were reached, Wood found that the mean air concentration and the distribution of the air through the depth of the flow were uniquely determined by the slope of the channel. The variation of $\bar{C}$ with channel slope was as follows:

Slope	$\bar{C}$
7.5°	0.137
15°	0.245
22.5°	0.302
30°	0.410
37.5°	0.560
45°	0.618
60°	0.675
75°	0.715

The data also indicate that in order to obtain a local air concentration at the bed of about 7% (so as to avoid cavitation damage), the mean air concentration needs to be about 30% and the slope of the channel about 22.5°. This result applies only after the flow has travelled sufficiently far along the channel for uniform conditions to be attained. Upstream, in the region of developing aerated flow, the air

concentration at the bed will be lower than the final equilibrium value.

Wood (1985) demonstrates how results from his earlier work can be used to produce a numerical model for predicting air concentrations along the length of a spillway. The point of inception is identified by assuming that entrainment starts when the depth of flow is equal to 1.2 times the thickness of the boundary layer. The entrainment of air into the flow is described in terms of a net entrainment velocity V_e where

$$\frac{d(q \bar{C})}{dx} = V_e \quad (F.28)$$

$$V_e = (\bar{C}_e - \bar{C}) V_b \cos \theta \quad (F.29)$$

Here $\bar{C}_e$ is the equilibrium mean air concentration for the given spillway slope, $\bar{C}$ is the local value of the mean concentration, and V_b is the rise velocity of the air bubbles. Calibration of this model against Cain & Wood's data (see above) indicated a value for the rise velocity of $V_b = 0.17\text{m/s}$. The development of the aerated flow along the spillway is then determined using the gradually-varied flow equation and information on the effect of air on channel roughness obtained from a re-analysis of Straub & Anderson's results. As mentioned above, Straub & Anderson used Equation F.8 to determine values of the friction factor λ , and found that air entrainment did not appear to alter the resistance of their rough channel. Wood calculated values of λ from the alternative formula

$$\lambda = (8g d_e \sin \theta) / \bar{V}^2 \quad (F.30)$$

where d_e is the equivalent water depth given by Equation F.10. On this basis (which appears more logical), it was found that the presence of air reduced the flow resistance.

Ackers & Priestley (1985) developed a model for predicting air entrainment on spillways which is based on the same information as used by Wood (1985), but with some detailed differences in approach. The point of inception is found numerically by computing the growth of the boundary layer until its thickness is equal to the depth of flow. The effect of air concentration on flow resistance was evaluated from Straub & Anderson's data (using the same method as Wood) and expressed in the form

$$\lambda_a / \lambda_w = 1 - 1.9 \bar{C}^2, \quad \bar{C} < 0.65 \quad (\text{F.31})$$

$$\lambda_a / \lambda_w = 0.2, \quad \bar{C} \geq 0.65 \quad (\text{F.32})$$

where λ_a and λ_w are the friction factors for aerated and non-aerated flow respectively. The change in mean air concentration in the region of developing aeration is calculated from the gradually-varied flow equation and the continuity relation

$$\frac{d}{dx} \{ \bar{C} / (1 - \bar{C}) \} = \frac{V_e}{q} \quad (\text{F.33})$$

This differs from Wood's Equation F.28; the definition of concentration in Equation F.5 shows that F.33 is correct.

The net entrainment velocity V_e of the air was assumed to be given by

$$v_e = v_b \left\{ \left(\frac{v_{in}}{v_b} \right) - \bar{C} \cos \theta \right\} \quad (F.34)$$

where v_{in} is the volumetric rate at which air is entrained into the flow per unit surface area, and $v_b \bar{C} \cos \theta$ is the corresponding rate at which air escapes due to its buoyancy (cf Equation F.29). Two hypotheses were considered for the quantity (v_{in}/v_b) : either that it depended only on the slope of the channel or only on the value of the local Froude number; comparison with some of Straub & Anderson's data suggested that the second hypothesis was slightly superior.

An equation for estimating the point of inception of air entrainment on a spillway can be obtained by using Equation F.25 for the velocity distribution in the boundary layer, and by assuming that inception occurs when the depth of flow is just equal to the thickness of the boundary layer. Combining with Equation F.27 then gives the following result for the distance L_1 (measured along the spillway) from the crest to the point of inception.

$$L_i = 37.3 \left[\frac{g^{0.50} H_s^{0.39} k_s^{0.10}}{q} \right]^{-1/1.01} \quad (F.35)$$

With minor differences in the coefficients, this equation is equivalent to one which Wood (1985) similarly obtained for spillways of constant slope; the derivation of Equation F.35 suggests that the latter may also be valid for cases of varying slope. Comparison of Equation F.35 with the prototype measurements of L_1 given by Galperin et al (1977), see above, shows reasonable qualitative agreement. A quantitative comparison cannot be made because the slope of the prototype spillway was not stated; the

equation would fit the data well if the slope were about 26° and the surface roughness were $k_s = 1\text{mm}$. It can be seen from Equation F.35 that the inception length is not very sensitive to changes in roughness.

F.3 Aerators on spillways

Aerators are being increasingly used to protect the spillways of high-head dams from cavitation damage. Their use is appropriate where the standards of surface finish needed to avoid cavitation are too high to be achievable and there is insufficient entrained air in the flow to prevent erosion by collapsing cavities.

Air can be injected by means of pumps, but most aerators work by producing a region of sub-atmospheric pressure which draws air naturally into the flow. This is achieved by means of a ramp, slot or offset which causes the flow to separate from part of the boundary and form a stable pocket of air.

Requirements of an effective aeration system are that:

1. Its air demand should be sufficient to give local air concentrations at the boundaries that are high enough to prevent cavitation damage (typically $C > 7\%$);
2. The air cavity produced by the device should remain stable over the full range of operating conditions and should not tend to fill with water;
3. The aerator should not produce too great a disturbance of the flow or an excessive amount of spray;

4. The spacing between successive aerators should be such that the local air concentration at the floor does not fall below the amount required to provide protection against cavitation damage.

The air demand depends upon the velocity and depth of the water, and upon the geometry of the aerator and the system of ducting which supplies it with air. Model tests are usually carried out to study the behaviour of the flow around an aerator. The phenomenon of air entrainment is subject to significant scale effects, so small models cannot normally provide accurate predictions of air demand.

An aerator initially produces a high concentration of air near the boundary, but the distribution becomes more uniform as the bubbles are carried downstream by the flow. The transverse movement of the air is determined by two effects : turbulent diffusion away from areas of high concentration, and buoyancy forces due to pressure gradients. Gravity gives rise to an upward-directed buoyancy force, but this may be counteracted by the effects of flow curvature.

Aerators can consist of deflectors, offsets, notches or slots used either singly or in combination; the elements of some typical designs are shown in Figure 8. Means of supplying air to an aerator are shown in Figure 9 and include:

1. use of a separation zone formed downstream of a pier or divide wall;
2. offsets or deflectors at the side walls which allow a flow of air from the surface to the floor of the channel;

3. ducts discharging air at the base of the side walls;
4. a duct beneath the floor of the channel connecting to a horizontal slot or to the downstream face of a vertical offset.

The design of each aeration system tends to be specific to the particular application, and data on some prototype installations (built or planned) are given in Table 3.

Hay & White (1975) tested two types of aerator as part of a more general model study to determine whether aeration would increase the efficiency of stilling basins, and reduce the amount of scour in downstream erodible channels. The first type consisted of a number of individual aerators, each of which comprised a small semi-circular notch in the spillway surface with a tear-shaped deflector upstream. A double row of this design of aerator gave mean air concentrations of up to $\bar{C} = 15\%$. The second type consisted of a continuous slot across the spillway with downstream a large-radius transition to the smooth profile of the channel; this produced values of up to $\bar{C} = 25\%$. Adding air to the flow gave more stable conditions in the stilling basin and reduced the amount of downstream scour for basins of simple design (but not for the more complicated USBR Type III).

According to Oskolkov & Semenov (1979) the height of offset needed to produce an adequate length of air cavity is typically in the range 1.5 - 2.5m, but can be up to 5-7m; an advantage of offsets is that they produce relatively little flow disturbance.

Deflectors produce stronger aeration than offsets, and normally need to be only about 0.1 - 0.8m high. These suggested sizes of offsets and deflectors are larger

than have been used in most prototype installations (see Table 3).

Prusza et al (1983) give recommendations on the design of aerators based on Russian experience and work carried out for Guri Dam (Venezuela). An aerator needs to produce local air concentrations of more than 7-8% in a 150-200mm thick layer adjacent to the floor and walls of a channel. In order to prevent atomisation of the flow the mean air concentration should not exceed $\bar{C} = 40-50\%$; at this limit the length of cavity produced by the aerator will be about 3-5 times the depth of flow. At low discharges the length of air cavity ought not to be more than 20-25% of its length at the maximum discharge. If a ramp is used on a concave surface, there must be a straight length of channel upstream of the aerator equal to at least 3 times the depth of flow. As the velocity of flow on a spillway increases, the required height and angle of ramp both decrease. If air is supplied via a gallery, either an offset or an offset with a ramp is recommended; the total cross-sectional area of the outlets of the air ducts should not be less than that of the gallery. If a larger flow of air is needed, this is best achieved by means of additional deflectors in the side walls; these are capable of providing a transverse supply of air in channels up to 50m wide. For all types of aerators it may be necessary to add corner wedges at the junctions of the walls and the floor so as to promote a clean flow separation and reduce the amount of surface disturbance.

Pinto & Neidert (1983b) studied the distribution of pressure in the flow at a ramp aerator. Regions of high pressure occurred on the surface of the ramp (due to the curvature of the flow) and at the point where the separated jet reattached to the floor of the

channel. The rapid variations in longitudinal pressure induced turbulence in the flow which entrained air on the underside of the separated jet and also at the free surface. The rise in pressure at the reattachment point caused the air to move upwards from the floor of the channel. Volkart & Chervet (1983) found that this effect could reduce the local air concentration at the bed to less than 10%, but the accompanying rise in pressure was sufficient to prevent cavitation. Immediately downstream of the reattachment zone, the air concentration at the floor increased rapidly due to turbulent mixing of the entrained air.

Model tests on several types of aerator were carried out by Volkart & Chervet (1983) for San Roque Dam (Phillipines). The best results were obtained with a plain deflector or a smaller deflector plus offset. A ramp combined with a slot (see Fig 8c) was not successful because falling droplets caused the slot to fill with water; the addition of drainage holes failed to solve the problem. Offsets alone did not produce a strong enough air demand.

Volkart & Rutschmann (1984a) mention that although plain deflectors can produce a good length of air cavity, they tend to work satisfactorily for only a limited range of flows. A combined deflector and offset was considered to give the best results.

The efficiency of an aerator can be increased by using "turbulisers" to break up the flow passing over an air cavity. For an offset, Galperin et al (1977) recommended the use of an upstream deflector with triangular slots arranged to produce a transverse saw-tooth pattern; the height of the teeth should be $1/10$ of the thickness of the boundary layer, and their transverse spacing should be at least 1.5 times their

height. Model tests showed that such a device increased the amount of entrained air by up to 20%.

The length of air cavity produced by an aerator is an important factor affecting its performance. Several theoretical methods of predicting this length have been developed by assuming the flow to be irrotational. Schwarz & Nutt (1963) studied the trajectory of falling nappes, but the results can be applied to jets formed by deflectors or offsets; equations for the horizontal and vertical co-ordinates are given separately, with the time of travel as the common parameter. It is assumed that the initial velocity and angle of projection are known, and that the thickness of the nappe is small so that it behaves effectively as a solid jet of liquid. Account is taken of gravity and any pressure difference between the upper and lower surfaces of the nappe. Effects of surface tension and air resistance are not included.

Pan et al (1980) determined the trajectory of a solid jet downstream of a deflector, but the solution does not take account of any pressure difference between the upper and lower surfaces. Three correction factors were introduced into the equations. The first allows for the fact that internal pressures in a jet cause the angle at which flow separates from a ramp to be less than that of the ramp itself; the reduction in angle was found theoretically using the method of conformal transformation (ignoring gravity). The two other factors were determined from a comparison with experimental data, and take account of the effects of energy losses and air resistance.

Wei & De Fazio (1982) and De Fazio & Wei (1983) solved Laplace's equation numerically by the finite element method to find the length of cavity downstream of an aerator. The flow upstream of the ramp is assumed to

be uniform, but allowance can be made for curvature of the spillway surface and differences in pressure across the jet. Comparison with model and prototype data for Guri Dam showed reasonable agreement.

Yen et al (1984) determined the flow around aerators by solving Laplace's equation numerically using three different models based on (i) the two-dimensional finite element method (FEM), (ii) the three-dimensional FEM, and (iii) the two-dimensional boundary-integral equation method (BIEM). In each case allowance could be made for a pressure difference across the nappe, but the shape of the lower surface was assumed to be a parabolic curve. Results were compared with data from a model of a deflector in a circular tunnel. The 2-D BIEM model was the least accurate and the 3-D FEM was slightly superior to the 2-D version. All three models overestimated the length of the cavity by a factor of about 1.8.

Shi et al (1983) carried out experiments with different heights of deflector to measure the trajectory of the jet, the pressure pattern on the channel floor, and the amount and distribution of air entrained into the flow. The following regression equation was obtained for the cavity length L_c , defined as the distance between the end of the ramp and the point on the floor where the local air concentration reach 60%,

$$\frac{L_c}{d} = 0.155 + 2.961 X_u - 1.674 X_u^{-1} \quad (F.36)$$

where

$$X_u = \frac{v}{(gd)^{\frac{1}{2}}} \cdot \frac{(h_1/d)^{\frac{1}{2}}}{\cos \theta \cos \phi} \quad (F.37)$$

and V and d are the velocity and depth of flow upstream of the aerator; the other quantities are defined in Figure 8 (note that h_1 is measured normal to the channel, whereas h is measured vertically).

Wood (1985) mentions a method used by Tan (1984) to estimate the cavity length produced by an offset, but the latter reference has not been studied for this review.

Predicting the air demand is the most important and the most difficult aspect of designing an aerator. Model and prototype studies carried out by Pinto (1979), Pinto et al (1982) and Pinto & Neidert (1982, 1983a) have led to a better understanding of the factors involved. Use of dimensional analysis suggested that the rate of air demand (q_a) per unit width of channel should depend upon the following parameters:

$$\frac{q_a}{VL_c} = \text{fn} \left[F = \frac{V}{(gd)^{\frac{1}{2}}} ; R_e = \frac{VL_c}{\nu} ; W_e = \frac{V}{(\sigma/\rho L_c)^{\frac{1}{2}}} ; \right. \\ \left. E_e = \frac{V}{(\Delta p/\rho)^{\frac{1}{2}}} ; \frac{L_c}{d} ; \frac{h}{d} ; \frac{t}{d} ; \theta, \phi \right] \quad (\text{F.38})$$

where the first four quantities on the right-hand side are the Froude, Reynolds, Weber and Euler numbers respectively; Δp is the pressure difference between the upper and lower surfaces of the jet. The Euler and Froude numbers influence the length and curvature of the jet, while the value of the Weber number determines whether it breaks up into a spray and thus entrains air strongly.

The air demand cannot be considered in isolation from the head-loss characteristics of the air supply system, which can be expressed in the general form

$$Q_a = c A_a (\Delta p / \rho_a)^{\frac{1}{2}} \quad (F.39)$$

where Q_a is the total rate of air flow, ρ_a is its density, A_a is the cross-sectional area of the duct and c is (normally) constant for a particular arrangement. For a given flow velocity, the rate of air entrainment on the underside of the jet depends upon the length L_c of the cavity, which in turn is affected by the pressure difference Δp : increasing Δp decreases L_c and vice versa. The value of Δp adjusts until the air demand of the jet matches the rate of flow through the air duct. If air is supplied to the cavity from lateral outlets in the side wall, there will be a variation of Δp across the width of the channel; the difference is largest at the outlet and decreases towards the centre of the channel.

Pinto et al (1982) determined values of the parameter q_a / VL_c for the aerators at Foz do Areia Dam (Brazil): the air demand ratio $\beta = Q_a / Q$ was obtained from prototype measurements, the cavity length L_c from a 1:50 scale model and the depth of flow d by means of calculations. Over a six-fold range of water discharges it was found that the quantity q_a / VL_c was approximately constant, i.e.

$$q_a = kVL_c \quad (F.40)$$

where $k = 0.033$ for air supplied laterally from both sides of the channel (70.6m wide) and $k = 0.023$ with air supplied from only one side. However, later model tests which Pinto & Neidert (1983a) carried out over a wider range of conditions showed that k was not in

fact a constant, but varied significantly with F , E_e and h/d . Values of F and h/d for a particular dam do not alter greatly with flow conditions, but the significance of the Euler number E_e shows that the characteristics of the air supply system have an important effect on the amount of entrainment. The influence of surface tension can be neglected if the value of the Weber number $W_e > 1000$ approximately (see Equation F.38).

Pan et al (1980) carried out a laboratory study of ramp aerators which lends support to the later work of Pinto et al described above. Vertical and longitudinal measurements of air concentration were made in order to determine how the air was entrained upwards into the flow from the cavity created by the aerator. The length L_c of the cavity was taken as being the distance from the aerator to the point on the floor of the channel where the air concentration decreased to 60%. Based on the vertical profile of air concentration at the downstream end of the cavity, the rate of flow of entrained air was calculated to be

$$Q_a = 0.022 V_d L_c \quad (F.41)$$

where V_d is the flow velocity at the end of the cavity (not at the aerator). This result agreed well with the model data, and has a similar form to Equation F.40 which was determined from prototype measurements.

Pan & Shao (1984) also considered an alternative approach to predicting the air demand that would not require prior determination of the cavity length. Analysis of laboratory and prototype data, in terms of the non-dimensional parameter X_u defined in Equation

F.37, led to the following empirical equation for the air demand produced by a ramp and/or slot (but no offset) in a channel of constant slope.

$$\beta = \frac{Q_a}{Q_w} = -0.0678 + 0.0982 X_u - 0.0039 X_u^2,$$

for $X_u > 1$ (F.42)

This result may not be generally applicable because it does not take account of the head-loss characteristics of the air supply system. On a channel of varying slope, the air demand is altered by the effect of centripetal pressure.

Model tests for four aerators to be used on the spillway of Laiban dam (Philippines) were described by Koschitzky et al (1984). It was found that, provided the air supply system did not limit the amount of entrainment, the air demand ratio β for a given aerator depended only upon the Froude number of the flow (regardless of the absolute values of velocity and water depth). The results also showed that the presence of an aerator upstream tended to increase the amount of air entrained at an aerator downstream.

Useful prototype data on the performance of four aerators tested on chutes 1 and 3 of Guri dam (Venezuela) are given by Marcano & Castillejo (1984). The values of the entrainment parameter k in Equation F.40 were found to be approximately constant for each aerator, and varied between $k = 0.011$ for a 0.10m high ramp plus 2.0m deep groove and offset, and $k = 0.073$ for a 0.75m high ramp. It was found that it was difficult to predict or to reproduce correctly in a model the under pressures that occurred at the prototype aerators. As a result, the models tended to over-estimate the lengths of the air cavities.

Bruschin (1985) analysed the Foz do Areia data together with results from a model of Piedra del Aguila Dam (Argentina). Using the overall step height w instead of L_c as the characteristic length led to the following formula for the air-demand ratio

$$\beta = 0.0334 F \left(\frac{w}{d} \right)^{\frac{1}{2}} \quad (F.43)$$

This result does not take account of the under-surface pressure, and its validity has been questioned by De Fazio & Wei (1985).

Wood (1985) also studied the Foz do Areia data and produced the following equation for determining the value of the factor k in Equation F.40.

$$k = 0.0079 (F - F_k) \quad (F.44)$$

where the value of the Froude number F_k at the start of air entrainment is given by

$$F_k = 4.3 \left[1 + 4.7 \left(\frac{\Delta p}{\rho g d} \right) \left(\frac{h}{d} \right) \right] \quad (F.45)$$

Model tests of an aerator with an offset, but no deflector ($h = 0$) for Clyde Dam (New Zealand) gave lower values of k than predicted by Equation F.44.

Low (1986) describes model tests on aerators for the spillway of Clyde Dam (New Zealand) carried out at a scale of 1:15. Results are given for aerators of the type shown in Figure 8c (but without the rounded corner) for ramp angles of $\phi = 4^\circ$ and 5.7° and a spillway slope of 1:0.8. The measured air demands were approximated by an equation of the form:

$$\beta = a_1 (F - a_2) - a_3 \left(\frac{\Delta p}{\rho g d} \right)^{a_4} \quad (F.46)$$

where the first term on the right-hand side describes the effect of flow velocity and the second term the effect of the sub-atmospheric pressure in the air cavity. The factors a_1 , a_2 , a_3 and a_4 depended on the geometry of the aerator. Use of a dentated ramp upstream of the slot reduced the tendency for β to decrease as the pressure difference Δp was increased (i.e. it had the effect of reducing the value of a_3 in Equation F.46). Since the tests were carried out on a sectional model, it was not possible to determine directly the total air demand for an aerator spanning the full width of the spillway. The problem is complex because the pressure difference Δp in the air cavity varies with transverse distance from the ducts in the side walls of the spillway. Low describes a theoretical model of the air supply system which enables the total air demand to be calculated using the data from the sectional model. Measurements were also made of the vertical distribution of air in the flow downstream of the aerators. These showed that the air concentration close to the bed decreased fairly rapidly downstream of the reattachment point of the flow. In model terms, the concentration at a height of 10mm above the bed decreased to $C = 10\%$ within a distance that varied from about 0.1-1.0m for Froude numbers between $F = 7.0$ and 13.4.

Bretschneider (1986) tested models of slot-type aerators to determine the critical flow velocity V_k for the start of air entrainment. The best-fit correlation obtained for five sizes of square slot was:

$$\left(\frac{V_k d}{\nu}\right) = 185 \left(\frac{\rho V_k^2 d}{\sigma}\right)^{0.75} \quad (F.47)$$

where the bracketed term on the left-hand side is a type of Reynolds number and that on the right-hand

side a type of Weber number. However, the form of the correlation was not fully tested because the fluid properties (ρ , ν , σ) were not varied. For water at 20°C, Equation F.47 becomes

$$V_k = 18.2 \sqrt{d} \quad (\text{F.48})$$

where V_k is in m/s and d in m. If gravity is assumed to be implicit in the factor 18.2, then this result is equivalent to a critical Froude number for air entrainment of $F_k = 5.8$.

Bruschin (1987) proposed an alternative type of entrainment function to that given by Equation F.40. The characteristic length is postulated to be a certain vertical "roughness" index δ rather than the cavity length L_c . The proposed equation has the form:

$$Q_a = \delta (V - V_k) \quad (\text{F.49})$$

Use of some prototype data, together with an assumed threshold velocity of $V_k = 1\text{m/s}$, gave values of $\delta = 0.2\text{--}0.4\text{m}$. The factors which may influence δ were not discussed.

Pinto (1986) used photographs of flow conditions in the Foz do Areia spillway to estimate the amount of bulking and hence the total amount of air entrainment. At the downstream end of the spillway the mean air concentration was calculated to vary between about 39% and 47% for unit water discharges ranging from 20.8 to 120 $\text{m}^3/\text{s}/\text{m}$. The longitudinal flow profiles showed that most of the air entrainment occurred over a distance of about 20–30m downstream of each of the three aerators. However, the aerators themselves supplied only a relatively small proportion of the total air in the flow (of the order of 25% or less).

Most of the air appeared to be entrained at the surface as a result of the very strong flow turbulence generated by the aerators; this entrainment was distinct from the normal self-aeration considered in Section F.2. These findings suggest that factors not highlighted by model tests may contribute to the effectiveness of aerators in preventing cavitation damage.

A recognised problem with reduced-scale models of aerators is that they may significantly underestimate the air demand in the prototype. This topic is considered in detail in Section G.2.

The required spacing between successive aerators is determined by the rate at which the local air concentration near the floor of the channel decreases with distance. Data for Bratsk Dam (USSR) given by Kudriashov et al (1983) showed that the mean air concentration decreased at a rate of 0.4% per metre of channel, but the loss rate is believed to vary with the slope and flow velocity (Bratsk spillway has a steeper-than-usual slope of 51°).

Prusza et al (1983) summarise Russian information on aeration and give the following loss rates for different types of channel

Straight section	0.15 - 0.20% per metre
Concave section (bucket)	0.50 - 0.60% per metre
Convex section	0.15 - 0.20% per metre

Model data for San Roque Dam presented by Volkart & Chervet (1983) showed that the local air concentration near the bed decreased from about 50% to less than 10% in a distance of about 15m, for flow velocities in the range of 25 - 32m/s (in prototype terms). However, the loss rate is likely to be subject to significant

scale effects. It was found that the required spacing between aerators depended on the flow velocity in the spillway and not on the discharge of water per unit width.

Volkart & Rutschmann (1984a) quote Semenov & Lentjaev (1973) as giving the loss rate for a straight channel as 0.5 - 0.8% per metre and for a channel with concave curvature 1.2 - 1.5% per metre. Distances between aerators are typically in the range 30-100m.

Hamilton (1984) suggested that the loss rate might be expected to be proportional to the local air concentration, i.e.

$$\frac{dC}{dx} = -jC \quad (F.50)$$

leading to an equation of the form

$$\frac{C}{C_o} = e^{-j(x-x_o)} \quad (F.51)$$

Data on the decrease of air concentration near the floor of Bratsk Dam ($C = 85\%$ to 35% in $53m$) gives a value of $j = 0.017$ per metre.

Cui (1985) measured both the vertical and longitudinal variation of air concentration downstream of aerators. An exponential type of equation was fitted to the data on the longitudinal decrease of concentration, but the form of the equation suggests that it may be specific to the particular study.

When designing an aeration system it is necessary to choose a figure for the maximum air velocity in the ducts in order to avoid compressibility problems and

objectionable noise. Limiting velocities recommended or used by various authors are as follows:

Reference	Maximum Air Velocity (m/s)
Falvey (1980)	30 (continuous operation)
Haendl (1984)	40
Billore et al (1979)	50
Coleman et al (1983)	50
Eccher & Siegenthaler (1982)	60
Falvey (1980)	90 (short duration)
Prusza et al (1983)	100 - 120

Design pressures at aerators supplied by air ducts are typically in the range $\Delta p = 0.5\text{m}$ to 2.0m head of water below atmospheric pressure. Where side-wall deflectors are used to supply air to aerators, the pressure differences are normally smaller ($< 0.5\text{m}$ head of water).

Aerators are reported to have been successful in preventing cavitation damage at the following dams:

Bratsk, Calacuccia, Emborcação ($V \leq 35\text{m/s}$), Foz do Areia ($V \leq 43\text{m/s}$), Grand Coulee, Guri ($Q_w \leq 10\,000\text{m}^3/\text{s}$), Heart Butte, Mica, Nurek, Tarbela (tunnel no 3) and Yellowtail.

F.4 Aeration in tunnels

The high speed flow of water downstream of gates in tunnels leads to air entrainment at the free surface and also a flow of air in the space above it, the velocity of which may sometimes be greater than that of the water itself. What may be termed this "natural" air demand is usually met by means of a system of galleries or ducts connecting to the gate shaft. Aerators may also be used to prevent cavitation damage to the floor and walls of the tunnel. The devices operate in a similar way to those on spillways; side deflectors are often provided in

the walls to allow air to flow from the surface to the invert of the tunnel. The additional "forced" air demand can thus be supplied by means of the gate shaft and its connecting ducts.

The natural air demand created by the high velocity flow in a closed conduit will be considered first. Falvey (1980) gives a useful guide to the subject and describes the various types of air-water flow that can occur. It is important to distinguish cases where a tunnel downstream of a gate flows part-full over its full length from those where the tunnel is sealed by a hydraulic jump; in the latter cases the air flow is determined by the amount of entrainment in the jump and by the capacity of the flow to transport the bubbles of air along the tunnel.

Kalinske & Robertson (1943) used model data for the air demand in tunnels with hydraulic jumps to obtain the formula

$$\beta = \frac{Q_a}{Q_w} = 0.0066 (F_1 - 1)^{1.4}, \text{ for } 1.5 \leq F_1 \leq 30 \quad (\text{F.52})$$

where the Froude number just upstream of the jump is given by

$$F_1 = \frac{V}{(gA/B)^{\frac{1}{2}}} \quad (\text{F.53})$$

Falvey (1980) demonstrates satisfactory agreement between Equation F.52 and measurements from three prototype tunnels for values of $2.5 \leq F_1 \leq 50$.

Campbell & Guyton (1953) compared Kalinske & Robertson's formula with data from five different dams, and found that it under-predicted the air demand. The maximum rates of air flow (Q_a) occurred

at gate openings of about 80%, and the upper limit to the field data for tunnels with jumps was given by

$$\beta = 0.04 (F_c - 1)^{0.85}, \text{ for } 3.5 \leq F_c \leq 10 \quad (\text{F.54})$$

where F_c is the value of the Froude number at the vena contracta.

The US Army Corps of Engineers (1964) reviewed model and prototype information on air demand, and recommended the following equation for flows with hydraulic jumps

$$\beta = 0.03 (F_c - 1)^{1.06} \quad (\text{F.55})$$

Uppal et al (1965) carried out tests on a 1:17 scale model of a 2.59m diameter tunnel of horseshoe cross-section downstream of a control gate. The tunnel flowed part-full for gate openings less than 90%, and measured air demands were greater than predicted by Equations F.52 and F.54. The maximum value of β occurred at a 40% gate opening and the maximum air flow Q_a at a 60% opening.

Levin (1965) analysed information from previous studies of air demand in tunnels with jumps, and proposed the formula

$$\beta = G \left[\left(\frac{2H}{d_c} \right)^{\frac{1}{2}} - 1 \right] \quad (\text{F.56})$$

where H is the total head upstream of the gate and d_c is the depth of flow at the vena contracta downstream of the gate; for $H/d_c \gg 1$, the quantity $(2H/d_c)^{\frac{1}{2}}$ is approximately equal to F_c . For a circular tunnel with carefully designed gate slots, $G = 0.025 - 0.040$. Where there is a gradual transition from a rectangular

to a circular cross-section downstream of a gate, then $G = 0.040 - 0.060$. If the transition is less gradual and flow separation occurs, $G = 0.08 - 0.12$. The rate of flow in the air supply system is given by

$$Q_a = 28 m_a A_a (2 \Delta p / \rho)^{\frac{1}{2}} \quad (F.57)$$

where

$$m_a = [\Sigma \zeta + (\lambda L_a / 4 R_a)]^{-\frac{1}{2}} \quad (F.58)$$

and $\Sigma \zeta$ is the sum of the velocity head coefficients for form losses in the duct, λ is the Darcy-Weisbach friction factor, L_a is the length of the duct, and A_a and R_a are respectively its cross-sectional area and hydraulic radius.

Field data for tunnels flowing part full, without a jump, were obtained by Wisner (1965) who fitted the following equation to the measurements of air demand

$$\beta = 0.024 (F_c - 1)^{1.4}, \text{ for } 3 \leq F_c \leq 20 \quad (F.59)$$

At small gate openings the slots give rise to a spray-type flow which entrains air more strongly, and for this condition the air demand is given by

$$\beta = 0.033 (F_c - 1)^{1.4}, \text{ for } 20 \leq F_c \leq 60 \quad (F.60)$$

Lysne & Guttormsen (1971) measured the air demand in high-head tunnels in two Norwegian dams. Spray formation at gate openings of 5-10% produced the largest values of β , but the rates of air flow increased steadily as the gates were opened. The upper bound to the data was described by the equation

$$\beta \leq 1.2 (s/A)^{-0.2} - 1 \quad (F.61)$$

where s is the area of opening of the gate and A is the cross-sectional area of the tunnel. Pressures downstream of the gates were 80-90% of atmospheric pressure, and this reduction needs to be taken into account when calculating values of the cavitation parameter K (see Equation (2)).

Sharma (1976) studied air entrainment in a rectangular conduit 0.1m x 0.15m and also made use of some prototype data. For flow with a hydraulic jump, it was found that Kalinske & Robertson's Equation F.52 gave reasonable results if the value of the Froude number was calculated at the vena contracta (F_c) instead of just upstream of the jump (F_1). This avoids the problem of having to estimate separately the air entrainment along the free surface as well as at the jump itself. Sharma also studied the case of part-full flow without a jump and obtained the relation

$$\beta = 0.09 F_c, \text{ for } 5 \leq F_c \leq 60 \quad (F.62)$$

For spray-type flow at small gate openings, the air demand was given by

$$\beta = 0.2 F_c, \text{ for } 20 \leq F_c \leq 100 \quad (F.63)$$

Rabben et al (1983), Rabben (1984) and Rabben & Rouvé (1984) give results of model tests to determine the air demand downstream of a gate in a rectangular tunnel. The air demands were found to depend on the size and headloss characteristics of the air ducts, as described by an effective area

$$A_e = \frac{A_a}{(1 + \sum \zeta)^{\frac{1}{2}}} \quad (F.64)$$

where A_a is the cross-sectional area of the duct and $\sum \zeta$ is the sum of the various head-loss coefficients. Tests were carried out on three geometrically similar models, the largest having tunnels of height 0.25m and 0.32m upstream and downstream of the vertical gate. For the case of flow with a hydraulic jump, the air demand in the largest model was given by:

$$\beta = 0.04 (A_e/A_t)^{0.11} (F_c - 1)^{0.70}; 4 \leq F_c \leq 100 \quad (F.65)$$

where A_t is the total downstream area of the tunnel. For free flow downstream of the gate, the corresponding result was:

$$\beta = 1.57 (A_e/A_t)^{0.94} (F_c - 1)^{0.52}; F_c \leq 40 \quad (F.66)$$

The results were compared with data from the two smaller models, which relative to the largest one had scale ratios of 1:1.333 and 1:2.0. For the case of flow with a hydraulic jump, it was found that the Froude criterion correctly scaled the air demands; Equation F.65 may therefore be valid outside the experimental range. On the other hand, the results for the case of free flow showed that the air demands did not scale according to the Froude criterion; Equation F.66 should not therefore be used directly, although Rabben & Rouvé (1984) do give a method for estimating the appropriate scale factor. Tests were also carried out on an aerator consisting of an offset in the floor of the tunnel downstream of the gate; as in the case of free flow, it was found that the air demands were subject to scale effects. These discrepancies were believed to occur because the Froude scaling did not reproduce correctly the formation of spray.

Ouazar & Lejeune (1984) analysed prototype data on air entrainment in tunnels with jumps, and obtained the relation

$$\beta = 0.012 (F_c - 1)^{1.135} ; \quad 5 \leq F_c \leq 75 \quad (\text{F.67})$$

Model tests were also carried out in a gated conduit measuring 100mm x 150mm in section, and equipped with a vacuum system to reproduce the pressure reductions correctly. Measurements of air demands for flows with jumps in this and other models were fitted by the equation

$$\beta = 0.0085 (F_c - 1)^{1.191} \quad (\text{F.68})$$

Comparison with Equation F.67 shows that the amount of air entrainment in models tends to be proportionately lower than in prototype tunnels. Tests were also made with the model tunnel flowing freely, and it was found that the air demand ratio β depended upon the flow velocity and not the Froude number. This indicates that Froude scaling may not be appropriate for modelling air entrainment in tunnels flowing freely.

Haindl (1984) carried out experiments on the entrainment of air by a jump in a rectangular conduit measuring 0.266m x 0.200m. Some of the tests gave higher values of β than Equation F.52, and inclusion of Campbell & Guyton's field data led to the following formula for the maximum air-water ratio

$$\beta = 0.015 (F_1 - 1)^{1.4} , \quad \text{for } 3 \leq F_1 \leq 50 \quad (\text{F.69})$$

Laboratory experiments to determine the amounts of air entrained by hydraulic jumps in a closed conduit were carried out by Ahmed et al (1984). The cross-section of the conduit measured 0.14m x 0.14m, and tests were

done at slopes of 90°, 60°, 45°, 30° and 10°. Measurements were made of the total rate of air entrainment at the toe of the jump and the net rate at which it was transported downstream by the flow. Analysis of the data from many tests led to the following equation for the net air demand:

$$\beta = 0.00234 F_1^2 [1 + 4.87 \exp \{-0.35(F_1 - 1)\}] \left[1 - \frac{V_k}{V}\right]^3$$

$$; 2 \leq F_1 \leq 18 \quad (F.70)$$

Here V is the velocity of the jet entering the jump, F_1 is the corresponding Froude number (see Equation F.53), and V_k is the flow velocity at which air entrainment starts; note that the slope of the conduit was not found to be significant. The equation was developed assuming a fixed value of $V_k = 0.8\text{m/s}$. The last bracketed term on the right-hand side of the equation may help to explain why air demands in models can be subject to scale effects. At high flow velocities, such as occur in prototype tunnels, this term tends towards unity; in Froudean models the velocities are lower, and the last term may become significantly less than unity. Comparison of this laboratory equation with prototype data would help to establish its general validity. It should be noted that the result is based on conditions just upstream of the jump, whereas most of the others described in this section relate to conditions at the vena contracta formed just downstream of a gate.

The "natural" air demands predicted by some of the equations described above are compared in Figure 10, and it can be seen that there are quite substantial differences between some of them. Overall, it appears that, for a given Froude number, the value of β is greater if the tunnel flows part full than if it is

sealed by a jump. Spray flow produces the highest values of β , but since it occurs at small gate openings it will not normally give rise to the maximum rate of air flow, Q_a . The presence of air in tunnels flowing full can cause undesirable pressure shocks, and it may need to be removed by means of deaeration chambers.

Details of aerators in various prototype tunnels (built or planned) are given in Table 3. An aerator was added to the 9.76m diameter tunnel of Yellowtail Dam to prevent cavitation damage that had been found to occur at the start of a vertical bend. Model studies carried out by Colegate (1971) showed that the shape of the aerator required careful design. A slot around the perimeter of the conduit filled too easily with water and thus did not aerate efficiently; narrowing the top of the slot made the problem worse. A deflector was therefore added upstream of the slot, and was successful in keeping it clear of water at all discharges. However, the deflector produced fins of water downstream, and it was necessary to ensure that these were not large enough to seal the pipe. It had been intended to add two other aerators, one near the head of the tunnel and the other at the downstream end of the vertical bend. However, the model tests showed that they would not operate satisfactorily, and they were therefore not adopted.

Based on USBR experience on seven tunnel spillways, Wagner & Jabara (1971) recommended the use of offsets as aerators. On the floor of the channel, the amount of offset should be $1/6$ of the gate width, while at the side walls it should be $1/12$ of the gate width. If larger offsets are used, fins of water may seal the tunnel or overtop the side walls.

Beichley & King (1975) describe aerators used in three US high-head tunnels and make the following recommendations:

1. For new designs, wall and floor offsets are normally better than air slots and deflectors. The latter may be the only solution for existing structures;
2. Offsets should be a minimum of 100mm (1/6 of gate frame width at floor, 1/12 at side walls). Air slots are not required with offsets;
3. Wall deflectors need to be used in conjunction with air slots if the downstream sides of the tunnel are parallel. The wall deflectors should not project more than 25mm into the flow with a slope of 1:30;
4. Floor deflectors should start at the end of the gate frame, have a rise of at least 50mm, and a slope not exceeding 1:9 (6.3°);
5. Air slots should be square in cross-section. A size of 300mm x 300mm should be adequate for gates measuring up to 1.2m x 2.3m with heads of up to 100m;
6. The downstream edge of an air slot should be offset 25-50mm away from the flow, and any transition should be made with slopes not greater than 1:20 (for $V < 12\text{m/s}$), 1:50 ($V < 27\text{m/s}$) and 1:100 ($V < 37\text{m/s}$).

Rabben et al (1983) studied air entrainment in a model of a tunnel with a floor offset located downstream of a gate. The air demand was found to be linearly

related to the length L_c of the cavity formed by the offset

$$\beta = -0.066 + 0.032 L_c/d_c \quad (F.71)$$

where d_c is the depth of flow at the vena contracta. The equation is valid for values of $L_c/d_c \leq 20$ and $4 \leq F_c \leq 18$; for $L_c/d_c > 20$ the jet breaks up and the air cavity is no longer sealed.

Hart (1982) and McGee (1984) describe prototype measurements at Libby Dam (USA) of air demand in three sluices, each measuring 3m x 6.7m high and controlled by a tainter gate. Cavitation damage had occurred previously, so an aerator, consisting of a deflector and air slot (see Table 3), was fitted immediately downstream of each gate. The total air demands (natural plus forced) for part-full flow without a jump were found to be in reasonable agreement with Sharma's Equations F.62 and F.63, which do not take account of the effect of an aerator. The lowest pressure in the aerators was about -1.3m head of water, and the maximum value of β was approximately 3.3 (i.e. $C \approx 77\%$).

Measurements of prototype air demands at Krasnoyarsk and Zeia Dams (USSR) are described by Abelev et al (1983). The design of the temporary outlet tunnel for each dam was similar, and included a step aerator downstream of the tainter gate, with air provided by ducts from the gate shaft. In the case of the earlier Krasnoyarsk Dam, the air supply system was not adequate; air velocities in the ducts reached 130 m/s, and cavitation occurred downstream of the aerator. The tunnels flowed part-full downstream of the gates, and the air demands (natural plus forced) were higher than predicted by Wisner's Equation F.59. The data for the two dams were fitted by the formula

$$\beta = 0.11 (F-1), \text{ for } 2.5 \leq F \leq 16 \quad (\text{F.72})$$

where F is calculated using the area and depth of opening of the gate.

Vernet & Larrea (1985) give model and prototype measurements of air entrainment for an aerator used at Alicura Dam (Argentina). The aerator consists of a deflector and air slot, and is positioned 50m downstream of a gate at the point where the steel lining to the 6.55m x 3.7m high channel ends (the channel is formed in a 9m diameter tunnel). The tunnel flows part-full, and the air demand (natural plus forced) was in reasonable agreement with Sharma's Equation F.62 and greater than predicted by Wisner's Equation F.59. However, for the case of spray flow, the measured value was close to Wisner's Equation F.60 and lower than given by Sharma's Equation F.63. It should be remembered that these formulae relate to the entrainment which occurs at the surface of the flow, and do not allow for the additional demand created by an aerator.

Montero et al (1986) describe the design of three aerators used in the bottom outlet of Colbun Dam (Chile). The outlet has a capacity of 730m³/s with flow velocities of up to 45m/s. Control gates in twin lined tunnels discharge into a rectangular channel formed inside a larger diversion tunnel, which is of oval cross-section. Tests were carried out on a 1:30 model of the complete outlet and a 1:18 model of the gate section. A stepped aerator with wall slots was located 4m downstream of the gates. A second aerator with a combined floor ramp and step was placed 117m downstream of the gates, at the point where the flow discharged from the rectangular channel into the original diversion tunnel. The third aerator was located a further 117m downstream, and consisted of a

floor ramp and side slots formed in the walls of the diversion tunnel. The effectiveness of the aerators was demonstrated by the fact that irregularities in the diversion tunnel and failure of an epoxy mortar repair in the rectangular channel did not cause any cavitation damage after 324 days of operation at flows of up to $688\text{m}^3/\text{s}$.

Factors affecting the performance of types of aerator used downstream of radial gates were investigated by Pan & Shao (1986). The aerators consisted of floor offsets (with and without ramps), and wall offsets which were curved in elevation to accommodate the upstream profile of the gate. The geometric factors which were varied in the tests were the size of the offsets, the angle of the ramps and the slope of the rectangular channel downstream of the aerator. Complicated semi-empirical formulae were developed to predict the critical Froude number for the start of aeration, and the lengths of the air cavities produced at the floor and the side walls. Formulae, based on Equation F.41 and using these cavity lengths, were also given for estimating the overall air demand of the aerator.

If an aerator does not function as intended, or if the flow conditions are outside its correct operating range, it may fill with water and not entrain air. Steps and lateral offsets may then act as large scale irregularities causing cavitation. Zhu (1984) tested a model of a tunnel with a stepped aerator downstream of a radial gate. It was found that the upstream head at which cavitation would begin at the step was considerably affected by the slope of the tunnel downstream of the step: decreasing the slope increased the value of the safe operating head.

APPENDIX G

MODELLING AND INSTRUMENTATION

G.1 Cavitation

Many aspects of modelling cavitation have been dealt with in Section 2 and Appendices B to F, and detailed descriptions of studies already mentioned will not be repeated here. Studies of cavitation can be carried out at a reduced scale in three main ways.

The first type of model is operated at atmospheric pressure according to the specified scaling law (usually Froudian). Measurements are made to determine the points of minimum pressure along the boundaries of the flow. Assuming the model and prototype to have equal values of the pressure coefficient C_p (Equation B.1), it is possible to predict whether pressures in the prototype will fall to the vapour pressure of the water and thus give rise to cavitation. This method can be used to determine the limit of incipient cavitation (see 2.2) provided:

1. the flow remains attached to the boundaries and the instruments are located at the points of minimum pressure;
2. measurements are made of both fluctuating and mean pressures;
3. the degree of turbulence and the boundary layer development are similar in model and prototype.

If the flow separates from a boundary, the lowest pressure will tend to occur in the body of the fluid, and the method will therefore under-estimate the likelihood of cavitation. Results which predict pressures below the vapour pressure of the liquid are

therefore not reliable, although they do of course indicate a serious danger of cavitation. In such tests it is necessary to ensure that the response time of the instrumentation is short enough to measure the fluctuating pressures accurately. Information is limited on levels of turbulence in prototype flows, and it may be difficult to reproduce these correctly in a model. Despite these potential problems, tests at atmospheric pressure can be useful in comparing the relative performances of different designs.

The second kind of test is carried out in a cavitation tunnel, in which the pressure in the working section is reduced below atmospheric so as to obtain equal values in model and prototype of the parameter K defined in Equation 2. Since the working section flows full, this method is suitable for studying only those situations in which free-surface effects are not important, e.g. gate slots in tunnels and small irregularities in spillway channels. With this approach it is possible to detect incipient cavitation directly, investigate the changes in flow which occur as the cavitation becomes more intense, and measure the amount of damage caused. However, all three of these aspects are subject to scale effects which are not well understood, particularly when the results are to be applied to large hydraulic structures.

The third way of studying cavitation is to use a vacuum test rig in which the air pressure can be reduced below atmospheric. This allows models with free-surface flows to be operated at prototype values of K . Such facilities are appropriate for models of spillways and stilling basins in which free-surface effects have a significant influence on the behaviour of the flow. However, vacuum test rigs can be difficult and expensive to construct.

The inception and development of cavitation are affected by the size and number of gas and dust nuclei in the water. Keller (1972) demonstrated the importance of nucleus size on conditions for incipient cavitation about a streamlined body. Fresh tap water gave $K_1 = 0.36$, whereas similar water which had been filtered and left to stand for one hour gave $K_1 = 0.036$. Although the overall gas contents of the two samples were nearly equal, measurements made using a focused laser beam showed that the fresh tap water contained many more large nuclei (with radii of the order of $8\mu\text{m}$ or greater). Keller (1984) demonstrated that repeatable results with water samples of different quality could be obtained if K_1 were calculated using p_c , the critical pressure for cavity growth (see Section 2.2), instead of the vapour pressure, p_v . The value of p_c for each water sample was found by producing a vortex within a specially designed nozzle, and determining the pressure at which cavitation started in the core of the vortex. This type of technique offers the prospect of more consistent laboratory results. However, in order to apply the results reliably, it will be necessary also to obtain information on the cavitation properties of water under prototype conditions.

The limits of cavitation are themselves influenced by the way in which they are measured (e.g. visually, acoustically, by changes in turbulence levels, or by the rate of pitting on a sample of soft material). Tests can compare the relative resistances of different materials, but it is difficult to predict the amount of damage which might occur in a prototype. Studies have been carried out in the USSR using "weak" model materials which are intended to reproduce the properties of those in the prototype (see for example Rozanov & Rozanova (1981)).

However the physical characteristics which contribute to a good cavitation resistance cannot yet be quantified, particularly in the case of a non-homogeneous substance such as concrete. Until this can be done, modelling of materials will remain fairly qualitative.

Although cavitation tunnels and vacuum test rigs enable models to be operated at prototype values of K , the results may still be subject to scale effects. Such models generally indicate correctly the points at which cavitation will occur in a prototype. However, there is conflicting evidence about whether the value of a parameter such as the limit of incipient cavitation K_1 is affected by the pressure, velocity and scale at which the tests are carried out.

Robertson (1963) suggested that in the case of bluff bodies the value of K_1 is initially equal to the minimum value of the pressure coefficient on the surface of the body (i.e. $K_1 = -C_{pm}$, see Equation B.2), and that it increases as the log of the Reynolds number. For streamlined shapes K_1 starts below $-C_{pm}$ and rises asymptotically towards this value as $V\sqrt{L}$ increases (where L is the characteristic length).

Several laboratory studies using models of different scales have indicated that K_1 increases with size, but is not affected by changes in pressure or flow velocity. Examples mentioned in Section B.3 and Appendix D include cavitation in orifices (see Tullis & Govindarajan (1973)), sudden enlargements (Ball et al (1975)) and 90° bends (Tullis (1981)). The fact that K_1 varied with size but not velocity indicates that the scale effects in these cases were not determined simply by the Reynolds number.

Liu (1984) considered the stresses causing a cavity to expand or contract, and thereby developed a theoretical equation which describes the effect of scale changes on the cavitation parameters. Let the geometric scale of a model be 1:s, and the values of K measured in the model for incipient and desinent cavitation be $(K_i)_m$ and $(K_d)_m$ respectively. The equation suggests that the prototype values of K_i and K_d are given approximately by:

$$(K_i)_p \approx \frac{1}{s} [(K_i)_m + (s-1) (K_d)_m] \quad (G.1)$$

$$(K_d)_p \approx (K_d)_m \quad (G.2)$$

Interestingly, the theoretical results suggest that conditions for desinent cavitation are not subject to significant scale effect. However, the equations have not been checked against experimental data.

Keller (1984) studied scale effects for incipient cavitation around axially-symmetric bodies. The following relationship was found between values of K_i for two bodies of similar shape but different size D

$$\frac{(K_i)_1}{(K_i)_2} = \psi^{\log_2 (D_1/D_2)} \quad (G.3)$$

where the factor ψ varies between about 1.1 for bodies with streamlined upstream ends and 1.45 for bodies with blunt ends. Changes in velocity altered the values of K_i for the bluff bodies but not for the streamlined ones.

It seems possible that the scale effects identified in studies such as these may be linked to the way in which the limits of cavitation are identified. A visual determination of incipient cavitation usually

depends upon the size at which cavities can first be seen by the human eye; alternatively the limit may be based upon a certain level or frequency of cavitation noise. These criteria are normally kept constant, but in fact they ought to be varied according to the scale of the model: for example, limiting cavity size proportional to model size, or noise intensity proportional to flow energy. Support for this contention is provided by the results of Ball et al (1975) for sudden enlargements. As mentioned above, K_i (based on noise levels) varied with size, but not with velocity and pressure. Values of the parameter K_{id} for the start of cavitation damage were also measured, using the rate of pitting per unit area as the criterion. The results showed that K_{id} was not dependent upon size, but did vary with pressure. The lack of size effect may be because the criterion correctly allowed for the change in scale by using the number of pits per unit area rather than the total number of pits.

Arndt (1981) suggested that cavitation in turbulent shear flows is subject to scale effects for two reasons. Firstly, as the scale increases, nuclei become responsive to a wider range of pressure fluctuations. Secondly, the deviations from mean pressure become larger as the Reynolds number increases. Information on turbulence in shear flows is limited, but measurements indicate that the pressure fluctuations corresponding to given velocity fluctuations are larger than occur in isotropic turbulence.

Hammitt (1975a) surveyed the problem of scale effects in cavitation testing, including those due to changes in temperature, fluid density and viscosity, but was not able to draw any firm conclusions.

Evidence from prototype installations is more encouraging, and suggests that models can correctly predict the occurrence and extent of cavitation damage at local features such as gates, baffle blocks and surface irregularities. Scale effects are difficult to identify precisely, but models do not appear to have under-estimated the danger of cavitation in prototypes. However, the comparisons may not be conclusive because cavitation is not usually identified in a prototype until damage occurs (i.e. $K < K_{1d}$), whereas most model studies use incipient cavitation as the design criterion ($K \geq K_1 > K_{1d}$).

G.2 Air entrainment The fact that water will not entrain air unless the velocity and turbulence of the flow are great enough demonstrates clearly that prototype air demands can be underestimated by models which are too small. However, it is necessary to distinguish between air which is entrained into the flow and air which is drawn along above the free surface. The former is the phenomenon which needs to be reproduced correctly for flows on spillways, and at aerators and hydraulic jumps. The flow of air above the free surface is important, however, in tunnels because it makes up a significant proportion of the total air demand.

Laboratory measurements by Ervine et al (1980) on falling jets showed that the minimum velocity required to entrain air varied from 0.8m/s at a turbulence level of 8% to 2.5m/s at a level of 1%. By contrast, Bruschin (1985) analysed prototype data for the aerators at Foz do Areia Dam, and estimated the minimum velocity for entrainment to be 11.3m/s.

The following non-dimensional criteria for the start of air entrainment have been described earlier in this review:

self-aeration on spillways	$I_o > 56$,	Equation F.11
self-aeration in pipes	$F_r > 6$,	Equation F.21
aerators	$W_e > 1000$,	Equation F.38
aerators	$F > F_k$,	Equation F.45
aerators	$F > 5.8$,	Equation F.48

Self-aeration cannot be reproduced satisfactorily in complete models of dam spillways because it is not possible to scale the inception lengths correctly and because the velocities are not usually high enough. However, numerical models based on prototype data, such as those developed by Wood (1985) and Ackers & Priestley (1985) (see Section F.3), offer a means of estimating whether the concentration of entrained air near the bed of a channel will be sufficient to prevent cavitation damage.

Larger-scale models of particular parts of dams, such as aerators and gated tunnels, have been used to estimate prototype air demands. The case of gated tunnels will be considered first.

Harshbarger et al (1977) carried out 1:20 scale model and prototype tests on a tunnel flowing part-full, and did not find any scale effects in the measured air demands. Galperin et al (1977) also give data which showed that a 1:20 model of a gated tunnel with free outflow satisfactorily predicted the amount of air entrained in the prototype. The velocity of the water in the model was 6.5m/s.

Falvey (1980) suggests that models can be used successfully provided all the air- and water-flow passages are correctly reproduced. It is particularly important to obtain the correct head-loss

characteristics for the air-supply system. If its design has not been determined at the time of testing, the performance of the model should be measured for a range of possible characteristics.

Abelev et al (1983) compared model and prototype measurements of air demand in two gated tunnels, each equipped with an aerator. The scales of the models were 1:34 and 1:36, and it was found that the predicted air flow rates (based on Froudian scaling) varied from about 25% to 50% of those in the prototypes.

Vernet & Larrea (1985) consider that air demand in tunnels can be predicted satisfactorily provided the scale of the model is not less than about 1:30. Model tests were carried out for a free-flowing tunnel equipped with an aerator; the flow to the aerator was assessed to be about 20% of the total air demand. Using a model scale of 1:25, it was found that the predicted flow rates of air were about 90% of those in the prototype.

Evidence from studies of aerators suggests that they need to be modelled at larger scales than gated tunnels in order to give reliable estimates of air demand. Aerators entrain air strongly when the water surface above the cavity breaks up into a spray; it is likely that a higher velocity and level of turbulence are required to produce this spray than to draw air along a tunnel flowing partly full. Aerators are normally tested using sectional models, but in relatively narrow flumes the boundary layers on the walls may have a disproportionate effect on the amount of entrainment.

Data from 1:6 and 1:25 scale models of an aerator are presented by Galperin et al (1977). At low

discharges, the air demand in the 1:6 model was up to twice that in the 1:25 model, but at higher discharges the two models gave similar results.

Quintela (1980) describes Russian studies carried out in connection with Nurek Dam (USSR). Eight aerators were fitted to a tunnel discharging on to a chute spillway. Tests of a 1:35 scale model predicted air demands that were only about 20-25% of those subsequently measured in the prototype.

Pinto & Neidert (1982) investigated the effect of scale when studying aerators for Foz do Areia Dam (Brazil). Sectional models were tested in a 150mm wide flume at scales of 1:50, 30, 15 and 8; also a 1:30 general model was used to reproduce one half of the prototype spillway which is 70.6m wide. The predicted air demands (based on Froudian scaling) in the 1:8 and 1:15 models were found to be in good agreement with measurements made in the prototype. The 1:30 and 1:50 models underestimated the entrainment, but the differences relative to the prototype became smaller as the water discharge increased. However, the results also show that the 1:30 general model gave air demands that were only 40% of those in the 1:30 sectional model. This suggests that the agreement between the two larger scale sectional models and the prototype may have been enhanced by increased entrainment at the side walls of the flume.

Zagustin et al (1982) and Zagustin & Castillejo (1983) carried out comparative tests on the ramp-type aerators to be used in chute no 3 of Guri Dam (Argentina). Sectional models at scales of 1:50, 40, 30, 25, 15 and 10 were installed in series in a 300mm wide flume. Predicted air demands from the 1:10 and 1:15 models were found to be in satisfactory agreement

with prototype measurements, while the 1:20 model gave values that were about 10% low. Since the width of each model was the same, the effect of the side walls on the amount of entrainment may have increased as the scale became larger. Measured cavity lengths in the 1:50 model were found to be 20-30% greater than those in the prototype; this was due to the fact that the amount of suction at the aerator was too small in the model.

In connection with studies for San Roque Dam (Philippines), Volkart & Chervet (1983) investigated size effects by testing models of an aerator with a combined ramp and offset at scales of 1:30, 25, 21.43 and 18.75. Each model represented a prototype width of 2.25m, so that in the tests the widths varied from 75mm to 120mm; the proportionate effect of the side walls therefore remained the same in all the tests. Prototype data were not available, so it was not possible to determine the overall scale effects precisely. However, comparing the various results and expressing them in terms of the air demand in the 1:18.75 model gave the following factors

Scale	Air demand ratio
1:1	106% (estimated)
1:18.75	100%
1:21.43	96%
1:25	89%
1:30	73%

The values of the ratios varied somewhat with the flow conditions, and those given above are the mean figures. The maximum average air concentration achieved in these model tests was 5.8%.

Pan & Shao (1984) carried out tests on two ramp aerators used in a rectangular spillway tunnel (measuring 7.2m wide by 11.0m high) at Fengjiashan Dam

(China). A model of the whole tunnel was constructed at a scale of 1:40, together with partial models (each 200mm wide) at scales of 1:30, 20, 15 and 12. Air demands in the prototype tunnel were also measured at five discharges up to 548m³/s, and were found to vary between $\beta = 0.15-0.30$ for Froude numbers of $F = 6.0-8.5$. The results showed that the 1:40 and 1:30 models underestimated the air demands, but that the larger models agreed quite well. From the tests it was concluded that a model of an aerator will predict the air demand correctly if the following limits are satisfied

$$V > 6-7\text{m/s} \quad (\text{G.4})$$

$$R_e = (VL_c / \nu) > 3.5 \times 10^6 \quad (\text{G.5})$$

where L_c is the length of the air cavity. It was also considered that a model which meets these requirements will not be subject to scale effects due to surface tension. However, problems do remain in modelling correctly how the air introduced by an aerator diffuses into the flow downstream of the point of reattachment.

Volkart & Rutschmann (1984b) measured air entrainment in a small spillway at Grande Dixence power plant (Switzerland); the spillway measured 0.80m by 0.80m in section, and tests were carried out both with and without a ramp deflector. The results were compared with measurements in models with scales varying from 1:6 to 1:18.75. The models were operated so as to obtain the correct Froudean velocities, but not necessarily the correct flow depths. Also, the model channels were made relatively wider than in the prototype so as to allow for the effects of wall roughness. All the models under-estimated both the jet length and the air demand produced by the

prototype aerator. No simple relation was found for scaling the model results correctly. In order to minimise modelling errors, the pressure distribution and velocity profile at the prototype ramp need to be carefully reproduced in the model.

Sakhuja et al (1984) analysed the relationship between measured air demands in models and prototypes for aerators and gated tunnels. They found that the scale effect X (defined as the prototype air demand divided by the model demand transformed according to the Froude criterion) was related to the geometric scale s (prototype/model) by:

$$\log_{10} X = 0.0048 (s-1) \quad (G.6)$$

On the basis of experimental evidence such as that described in Section F.1, it is generally accepted that local air concentrations of about 5-10% are sufficient to prevent damage by collapsing cavities. However, experiments carried out by Clyde & Tullis (1983) on cavitation at orifices in pipes suggest that the limiting air concentration is itself subject to scale effects. Tests to determine the onset of cavitation were performed first without the addition of air; the limit was detected by a sudden change in the level of turbulence. Air was then injected, and the velocity increased until the level of turbulence was the same as it was at the onset of cavitation without air. The results showed that, for a given flow velocity and orifice ratio, the amount of air needed decreased rapidly with pipe size : for example at $V = 2.33\text{m/s}$, the concentration required in a 76mm diameter pipe was $C = 6.1\%$ whereas in a 610mm pipe it was $C = 0.16\%$. Using as a parameter the rate of air flow/unit length of perimeter correlated the data better than did the concentration. It was also found

that the required amount of air increased considerably as the flow velocity was increased.

G.3 Instrumentation for aerated flows Specialised instruments are needed to study aerated flows. The main quantities to be measured are the air concentration and the velocity of flow. A summary of some of the techniques is given by Lakshmana Rao & Kobus.

In the case of concentration, it is necessary to distinguish between methods which measure the volume of air bubbles per unit volume of water from those which record the relative rates of flow of air and water (see Section F.2). The first group includes gamma ray attenuation equipment (see for example Babb & Aus (1981)), instruments which measure the change in conductivity of water due to the presence of bubbles (e.g. Cain & Wood (1981a)), and methods based on the attenuation of a beam of light (see Lakshmana Rao & Kobus). The second group includes probes used to abstract samples of air-water mixtures, which are then separated into their two components. Vischer et al (1982) explain how it is necessary to ensure that the rate of abstraction is equal to the velocity of flow, which itself partly depends upon the air concentration; it is therefore necessary to draw off the samples at several different rates in order to determine the true flow velocity and air concentration. Having obtained a sample, the amount of dissolved air can be found by measuring the conductivity of the water, which is affected by the partial pressure of the dissolved oxygen. The total amount of air (free + dissolved) can be determined using equipment such as the Van Slyke apparatus, or the newer Brand apparatus described by Mohammad & Hutton (1986).

A separate class of instruments for measuring concentration works by recording the proportionate lengths of time that a probe is in air and in water. The signal may be produced by hot-film techniques (eg Babb & Aus (1981)), or by the change in resistance which occurs when the tip of an insulated probe passes through an air bubble (White & Hay (1975)). These devices in fact function by detecting the air-water interfaces, and there would seem to be a problem of deciding precisely what quantity they actually measure if the air and water phases do not travel at the same speed.

Another type of instrument is the twin-wire gauge developed by Halbronn (1951). This consisted of two 0.3mm diameter wires insulated from each other and twisted to form a thin tube. The electrical resistance of the gauge depends upon the proportionate length of the tube that is in contact with water, so in aerated flow the resistance is directly related to the air concentration.

Conventional pitot tubes have been used to determine the velocity of aerated flows, and Vischer et al (1982) found that they were satisfactory for air concentrations of up to 10%. Various authors have differed on how results from pitot tubes should be interpreted (see Lakshmana Rao & Kobus) : the problems centre on how the density and velocity of air-water mixtures should be defined. Cain & Wood (1981a) show that the presence of air in water can reduce the speed of sound in the mixture to the order of 20m/s, so that compressibility effects may need to be taken into account when analysing data from pitot tubes.

An alternative method for determining velocity is to measure the time delay between signals from two probes which respond to the passage of air bubbles, and which

are mounted parallel to the flow and a known distance apart; the time delay is normally obtained by cross-correlating the two signals. If the probes are close together, they will respond to the same set of bubbles, but the time difference will be small. If the probes are further apart, the time delay can be measured more accurately, but the correlation will be determined by larger-scale variations in the flow rather than by the passage of individual bubbles. Vischer et al (1982) used an instrument with probes 10mm apart for laboratory work, whereas Cain & Wood (1981a) adopted a separation of 101.6mm for field measurements on Aviemore Dam. Cain & Wood argued that their equipment measured the velocity of water, but the principle of the method suggests that it does in fact register the velocity of the air-water interfaces. When the air concentration is very low, the velocity of the interfaces is equal to that of the air bubbles; conversely at very high concentrations, the velocity is that of the water droplets. When there are approximately equal volumes of air and water and the two phases move at different speeds, it is difficult to determine or define the velocity at which the interfaces between the air and water will move.

A third method of velocity measurement was used by Straub & Anderson (1958), and involved injecting a salt solution into the flow and measuring its time of travel over a known distance; since the salt is transported by the water, this technique gives an estimate of the average water velocity.

APPENDIX H

FUTURE RESEARCH

Further research that would be of benefit in the design of hydraulic structures will be considered under some of the headings used earlier in this review.

1. Mechanism of Cavitation

When studied in detail, almost every aspect of cavitation is found to be imperfectly understood. Fundamental research, both theoretical and experimental, can therefore be expected to continue in universities on a broad front. Particular topics that would be relevant to civil engineering hydraulics are:

- (a) role of nuclei in the growth of cavities, particularly in large-scale structures such as tunnels and spillways;
- (b) generation of cavities in turbulent shear flows;
- (c) motion of cavities and mechanisms of collapse;
- (d) pressures and forces produced by cavities collapsing near solid boundaries;
- (e) concentration of air needed to prevent cavitation damage, and variation of required concentration with velocity and scale.

2. Cavitation at Surface Irregularities

A considerable amount of laboratory work has been carried out on cavitation at various types of irregularity. In general, values obtained by different researchers for the incipient cavitation index K_i are in reasonable agreement, and enable designers to assess the likelihood of damage and to specify suitable tolerances for surface finish. Some uncertainties in the results remain, for example whether the value of K_i for a chamfer depends upon its height as well as its slope. However, it is unlikely that further testing would resolve these questions entirely because of the difficulties of obtaining exactly equivalent conditions in different laboratories (eg gas content of the water and the number and size of nuclei). More importantly, the types of fault which occur in prototype structures tend to be irregular and three-dimensional, and will seldom correspond exactly to those tested in laboratories. Possible areas for new research are:

- (a) model and prototype tests to determine conditions for the start of cavitation damage at surface irregularities (ie values of K_{id} instead of the more conservative inception parameter K_i);
- (b) studies to identify types of construction joint which are less liable to cause cavitation problems on spillways.

3. Tunnels and Gates

Several studies have reached similar conclusions about the features of gate slots which are desirable in order to minimise the danger of cavitation. Although further research might provide more detailed

recommendations, it is unlikely that they would remove the need to test models of major structures, since each scheme tends to have special requirements that prevent the adoption of standard designs. Topics which warrant further investigation are:

- (a) alternative gate designs which would eliminate the need for slots on the downstream side;
- (b) new materials for lining tunnels as cheaper alternatives to stainless steel.

4. Energy Dissipators

Outside of the USSR, little research appears to have been carried out on the design of supercavitating baffle blocks for stilling basins. The reasons for this are not evident from the literature, but it could be because: (1) western designers avoid the use of appurtenances in high-head stilling basins; (2) in such situations they choose alternative types of energy dissipator; (3) flow aeration is normally sufficient to prevent cavitation damage at the foot of spillways. Baffle blocks permit shorter stilling basins, and their increased use could produce cost savings. Views should therefore be sought from the civil engineering profession about the need for:

- (a) Research on types of supercavitating baffle block for use in hydraulic jump stilling basins.

In order to reproduce free-surface effects correctly, this work would need to be carried out in a vacuum test rig, which the UK does not at present possess.

5. Materials

Results from cavitation testing of materials tend to be affected by the type of equipment used and the particular laboratory conditions. It is therefore recognised that such studies do not give very precise estimates of how much damage can be expected to occur in a prototype. However, comparative tests carried out under similar conditions do assist designers to choose between different materials according to the perceived level of cavitation risk. Such work has been carried out for a wide range of steels, but there are relatively few results for concrete and these are difficult to compare. There is therefore a requirement for:

- (a) systematic studies to establish a comparative scale of cavitation resistance for a range of ordinary concretes, special concretes (eg steel-fibre and epoxy concretes) and epoxy fillers. The method used should reproduce as closely as possible the type of cavitation which occurs in prototype structures: vortex-shedding techniques are therefore preferable to vibratory or drop-impact methods.

6. Self-Aeration

Self-aeration on spillways is important in its own right, and in relation to cavitation because the presence of entrained air in a flow may prevent damage from collapsing cavities. It is not feasible to predict self-aeration by means of physical models, and the best way forward appears to be the development of numerical models based on laboratory and prototype information. At present the amount of experimental data is limited, and covers only a limited range of

unit discharges ($< 3.2 \text{ m}^3/\text{s per m}$). The following work is therefore needed:

- (a) measurements of aerated flows on prototype spillways for unit discharges greater than $5 \text{ m}^3/\text{s per metre width of channel}$.

It is appreciated that this proposal would be difficult and expensive to achieve, but without such data it will not be possible to verify numerical models and obtain reliable predictions for high-discharge spillways.

7. Aeration in Tunnels

Comparative data from model and prototype tests on gated tunnels indicate that carefully-constructed models of suitable scale can give satisfactory estimates of air demand. A number of equations for predicting air demand are available, but give contradictory estimates. Before any new basic research is carried out, it is recommended that:

- (a) available model and prototype information on gated tunnels should be critically reviewed in order to determine whether sufficient data already exist to make reliable predictions of air demand.

8. Aerators

Aerators are being increasingly used to prevent cavitation damage in tunnels and spillways.

In the case of tunnels, some general recommendations have been produced for the design of aerators incorporating floor- and wall-deflectors. However it is likely that model tests will continue to be needed

because small variations in gate configuration can significantly alter the flow conditions at an aerator.

In the case of spillways, model studies for individual schemes have led to the use of a variety of different types of aerator. However, since flow conditions in a spillway can be defined in terms of a few variables (eg velocity, depth and channel slope), a systematic programme of research should enable the most effective configurations to be identified. It should also be possible to define standard designs whose dimensions could be selected according to the particular flow conditions on a spillway. This would reduce the costs of individual model studies of dams, and would make efficient use of prototype data, since the performance of aerators on different dams could be compared on a similar basis against results from the laboratory studies. Objectives of an integrated programme of experimental research should be to determine:

- (a) length of air cavity formed at an aerator as a function of (i) flow conditions, (ii) geometry of the aerator, and (iii) head-loss characteristics of the air supply system;
- (b) most suitable theoretical method for predicting length of air cavity;
- (c) relationship between air demand, cavity length and flow conditions at aerator;
- (d) effect on air demand of changes in scale;
- (e) effect of side walls on air demand;
- (f) effect of aerators on aeration at free surface;

- (g) distribution of entrained air in flow downstream of aerators;
- (h) rate of loss of air from flow downstream of aerators.

APPENDIX I

REFERENCES

Abbreviations

ASCE	- American Society of Civil Engineers
ASME	- American Society of Mechanical Engineers
BHRA	- British Hydromechanics Research Association
CIRIA	- Construction Industry Research and Information Association
DFG	- Deutsche Forschungsgemeinschaft
DVWK	- Deutscher Verband für Wasserwirtschaft und Kulturbau e.V.
ETH	- Eidgenössischen Technischen Hochschule
ICE	- Institution of Civil Engineers
ICOLD	- International Commission on Large Dams
ISCME	- International Society of Computational Methods in Engineering
IWHR	- Institute of Water Conservancy and Hydroelectric Power Research

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