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Vortex Vanes: A Low Cost Sediment Excluder

by

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ABSTRACT

The vortex vane sediment excluder is a simple device for preventing excessive sediment loads from entering an irrigation intake. The excluder is relatively cheap to construct and does not require a weir across the river, so it can be used where there is insufficient head available for traditional methods of hydraulic flushing. The excluder consists of a pair of vanes constructed in the river bed.

Theoretical and laboratory studies have shown that the device can be designed to work successfully, and an initial design method has been developed. The feasibility of the excluder at a field scale chiefly hinges on the criterion that it should not become blocked by the river bed sediment. Theoretical and laboratory work on this aspect of the vortex vane's operation has produced the prediction that the vanes will not suffer from blockage at typical field conditions.

A field study is now recommended.

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APPENDIX

1 INTRODUCTION

A vortex vane sediment excluder is a low cost device for preventing excessive sediment loads from entering irrigation canal systems. The excluder is suitable for offtakes with no diversion weir. This technical note presents some theoretical analyses and laboratory experiments on the device, the purpose of the work is firstly to assess the feasibility of vortex vane sediment excluders and secondly to develop a design method. The study has shown that, under suitable conditions, the device can be used to reduce significantly the quantities of sediment entering a canal intake.

The note is presented in three parts. Firstly, the rest of this chapter presents a description of the sediment excluder and gives some background to the study. The second part, Chapters 2 and 3, describe the theoretical and laboratory work which has been performed at Hydraulics Research; and the final part presents the conclusions and recommendations for further work.

1.1 Description

A vortex vane sediment excluder consists of two vertical vanes constructed on the river bed immediately upstream of a canal intake. In plan, the vanes start at the river bank and they take a curved line out into the river so that they are at an angle to the flow passing into the canal, they also diverge slightly. The vanes do not extend up to the water surface; the height of the more downstream vane is approximately a third of the water depth, while the elevation of the upstream vane is approximately the bed elevation. The vortex flow between the vanes prevents sediment deposition in the space between them. Vortex vanes are suitable for sites where there is no weir across the river at the offtake.

A sketch of the device is given in Figure 1, it shows the layout of a canal intake and the two vanes.

The vanes exclude sediment from the offtake canal by trapping the water which is flowing near the bed. The water is collected in the vortex between the vanes and so is carried away from the canal offtake. It is this flow moving near the bed which carries both the bed load and the highest concentration of suspended sediment; the flow which passes into the offtake canal has much lower sediment concentrations.

Figure 2 demonstrates the principles by which vortex vanes exclude sediment. It shows the streamlines of the water passing into the offtake canal and into the vortex between the vanes. Figure 2 also gives the streamlines at a cross-section through the vanes.

The vortex between the vanes is a result of the momentum of the water flowing into it. To maintain its momentum, the water is forced into the helical path which forms the vortex.

Vortex vanes have not been constructed in the field so their field performance in excluding sediment has not yet been measured. An estimate of the likely field performance of the device is that it would halve the sand sized sediment loads entering an offtake canal. It would have no effect on the quantities of silt and clay abstracted with the irrigation water, because the concentration of these sediments is uniform through the flow depth.

1.2 Background

Vortex vanes have great potential as a sediment control device for two reasons. Firstly, they are cheap to construct, and secondly the device is suitable for free offtakes, where the water is abstracted from a river without the aid of a diversion weir. Therefore vortex vanes are feasible where conventional techniques of hydraulic flushing are not feasible.

The only mention of vortex vane sediment excluders in the literature is found in a paper by Karunaratne (1975). Karunaratne undertook a model study to evolve a suitable sediment exclusion structure for a diversion in Sri Lanka. He found that the use of two submerged vanes, suitably curved and spaced, were effective in excluding sediment from entering the canal. A plan view of the structure tested by Karunaratne is shown in Figure 3; the chief difference between this structure at that sketched in Figures 1 and 2 is that the vanes are parallel.

Karunaratne does not refer to this design being used elsewhere, we also understand that the device was not finally adopted for the prototype diversion.

Both the potential of vortex vanes for low cost sediment exclusion at free offtakes and the indications of the model study described above, have encouraged the work presented in this note. The starting point for the work was provided by a draft report by Sanmuganathan (1981).

The overall objectives for the work are:

(i) To assess if vortex vanes are a feasible means of sediment control at an irrigation intake;

(ii) To develop a reliable design method; and

(iii) To develop a method to predict the reduction in sediment quantities entering an offtake after a vortex vane sediment excluder has been constructed.

2 THEORETICAL ANALYSIS

2.1 Flow field

Before the overall objectives listed above can be met, an analysis of the flow field between the vanes must first be performed. The analysis is presented below. Reference should also be made to Figure 4, which shows the meaning of the symbols.

Firstly we assume that the elevation of the top of the vortex is the elevation of the dividing streamline between the flow passing over and entering the vortex. Also assuming that:

(i) the vortex completely fills the gap between the vanes; and

(ii) that the bed upstream of the vanes is level with the top of the upstream vane, we obtain:

$$a = r_2 \tag{1}$$

$$b = r_2 - r_1 \tag{2}$$

where

a = half of the vane spacing
r_2 = radius of vortex
b = height above bed of dividing streamline
r_1 = vertical distance from centre of vortex to top of upstream vane

The component of the velocity of the flow entering the vortex which is parallel to the vane axis, v_a, and the component normal to the axis, v_n, are:

$$v_a = v_b \cos\Theta \tag{3}$$

$$v_n = v_b \sin\Theta \tag{4}$$

where

v_b = overall velocity of flow entering vortex
Θ = angle between flow entering vortex and vane axis

Using the principle of the conservation of mass between the flow entering the vortex and the flow in the vortex, we obtain:

$$\frac{dq}{dx} = b\ v_n \tag{5}$$

where

q = discharge between vanes (axial direction)
x = distance along axis of vanes, $x=0$ at the river bank end of the vanes

If we assume that the axial flow between the vanes only occurs within the vortex itself then:

$$q = v_v\ \pi\ a^2 \tag{6}$$

where

v_v = mean velocity of flow between vanes, parallel to vane axis

A momentum balance between the flow entering the vortex and the flow in the vortex gives (assuming no friction losses):

$$\frac{d\ (v_v\ q)}{dx} = v_a\ v_n\ b \tag{7}$$

Substituting Equation 6 into Equation 5 and 7 produces:

$$\frac{d\ (v_v^2\ a^2)}{dx} = \frac{d\ (v_v\ a^2)}{dx}\ v_a \tag{8}$$

Expanding Equation 8:

$$\frac{1}{a^2}\ \frac{da^2}{dx} = \frac{dv_v}{dx}\left[\frac{v_a - 2\ v_v}{v_v\ (v_v - v_a)}\right] \tag{9}$$

If we assume that v_a is a constant, then after integrating we obtain:

$$a^2\ (v_v^2 - v_v\ v_a) = A \tag{10}$$

where A is the constant of integration, the boundary condition is:

$q = 0$ at $x = 0$

hence $A = 0$

Equation 10 now reduces to:

$$v_v = v_a \tag{11}$$

This is an important result which is useful for the design of the layout of vortex vanes, see Section 2.2.

The vortex between the vanes can be assumed to be a forced vortex, where the rotation speed is constant with distance from the centre of the vortex. By taking moments about the centre, we can equate the angular momentum of the vortex with the angular momentum of the water entering it:

$$\int_{r_1}^{r_2} \rho \, v_n^2 \, r \, dr = \frac{dM}{dx} \tag{12}$$

where

r = radial distance from the centre of the vortex
ρ = density of water
M = angular momentum of fluid in vortex:-

$$M = \int_0^{r_2} 2 \, \pi \, r^3 \, \rho \, \omega \, v_v \, dr \tag{13}$$

where

ω = speed of rotation of vortex.

Equations 12 and 13 give:

$$\frac{d \, (\omega \, v_v \, r_2^4)}{dx} = \frac{(r_2^2 - r_1^2) v_n^2}{\pi} \tag{14}$$

If we substitute Equations 1, 2 and 11 in Equation 14, we obtain:

$$\frac{d \, (\omega \, v_a \, a^4)}{dx} = \frac{(2 \, a \, b - b^2) \, v_n^2}{\pi} \tag{15}$$

Equation 15 can be used, together with the functions for v_a, v_n, a and b, to predict the speed of rotation of the vortex, ω. The value of $a\omega$, the rotational velocity at the edge of the vortex, can then be used to assess the ability of the vortex to prevent sediment deposition between the vanes.

For example, if the simple case of $\Theta = 45°$ and uniform near-bed flow velocities is taken, then integration of Equation 15 yields:

$$a\omega = v_n \left(\frac{4}{3} - \frac{b}{a} + \frac{2}{3}\frac{b^3}{a^3}\right) \qquad (16)$$

Here the boundary condition was obtained by balancing the angular momentum in a short section near the bank. This shows that the velocity driving the vortex is the approaching near-bed velocity.

2.2 Design of vanes

The analysis presented in Section 2.1 is used in this section to provide guidelines for the design of vortex vanes.

A design method for a vortex vane sediment excluder would need to set:

(i) the elevation of the upstream vane;
(ii) the elevation of the downstream vane;
(iii) the vane spacing;
(iv) the length of the vanes; and
(v) the position of the axis of the vanes.

The analysis of Section 2.1 suggests that the axial flow between the vanes is driven by the velocity component, v_a, while the rotation of the vortex by the

component, v_n. Both the axial flow and the rotation are vital to the operation of the excluder; the axial flow is the discharge which carries sediment away from the offtake, and the rotation of the vortex is the mechanism which prevents sediment deposition between the vanes. We desire to maximize both v_a and v_n. This is best done by setting $\Theta = 45°$, hence both v_a and v_n are set at 71% of their maximum possible values.

The decision to make $\Theta = 45\%$ sets the layout of the vanes. The centreline between the vanes starts at a point on the river bank a little upstream from the offtake, the path of the centreline is then set so that it crosses all the near-bed streamlines at 45°. The downstream end of the vanes is reached when the centreline is well downstream of the offtake.

If the elevation of the upstream vane is set higher than the river bed, then a ramp will form upstream of the vane and nothing will be gained. Therefore the vane's elevation should be set at river bed level.

The elevation of the downstream vane is related to the value of b, the height of the dividing streamline, so the required value of b at each point along the vane must be known. Firstly a uniform trapping of sediment laden water across the vanes is achieved if the value of b is constant (see equation 5), so this is recommended.

The performance of the excluder is directly related to the value of b: as b increases, more sediment laden water is carried away from the offtake and so lower concentrations of sediment enter the offtake canal. Section 2.4 presents an analysis which quantifies the effect. The required reduction in sediment concentrations entering the offtake sets the value of b. Equations 5 and 6 give:

$$\frac{d\,(v_v\,a^2)}{dx} = \frac{b\,v_n}{\pi} \tag{17}$$

Substituting the result of the axial momentum analysis, $v_v = v_a$, we get:

$$a^2 = \frac{b}{\pi\,v_a} \int v_n\,dx \tag{18}$$

Equation 18 can be used to calculate the vane spacing of a vortex vane sediment excluder. Current metering in the river before an excluder is built could give v_n and v_a, which are functions of the distance along the vanes. Alternatively they may be predicted using an assumption of a logarithmic velocity profile. The equation will then give a value for 2a, the vane spacing, at each value of x.

For a complete solution, the integral in Equation 18 requires a value for a when x = 0. The criteria $v_v = v_a$ would suggest that a must tend to zero as x tends to zero. However, values of a smaller than b would require that the elevation of the centre of the vortex is above the upstream vane, which is not possible. Therefore it is recommended to set a = 0 at x = 0 for Equation 18, but to ensure a minimum value of a = b in practice.

The elevation of the downstream vane is the final parameter to be set in the vane design. If there were no friction losses, and the vanes were designed correctly, then a downstream vane elevation equal to the elevation of the dividing streamline could be expected to perform well. However the friction losses cause a higher downstream vane height to be required. This aspect of vortex vane design is discussed further in the next chapter.

2.3 Self-cleaning of vanes

During the dry season water levels in a river may be so low that the vortex between the vanes of an excluder will not form. As a consequence sediment will deposit between the vanes, therefore the flow at the beginning of the wet season must be able to scour out the vanes. This may be considered as a worst possible case, in some rivers the flow will be capable of maintaining a vortex between the vanes in all seasons.

Direct modelling of the flow patterns and scour between the vanes as they self-clean would be an extremely complex exercise and is not attempted here. However, it is investigated by comparing it to a similar phenomenon: scour at bridge piers.

Local scour around piers is the result of vortex systems which develop as the river flow is deflected around the pier. The main vortex system which contributes to the formation of scour holes originates at the upstream nose of the pier, where the flow acquires a downward or diving component which impinges on the stream bed. As bed material is removed by the flow, a spiral roller develops within the hole formed, which spirals around the side of the pier. In plan, the vortex system has a horseshoe shape and is frequently referred to as a horseshoe vortex (Fig 5).

The scour hole will increase in size until an equilibrium depth is reached. The equilibrium depth is dependent on which of the following scour conditions prevail:

(i) Clear water scour, where bed movement occurs only adjacent to the piers. The equilibrium depth is reached when the shear stresses at the surface of the scour hole are insufficient to eject particles.

(ii) Sediment transporting scour, where the whole river bed is in motion. The equilibrium depth is reached when the amount of sediment entering the scour hole is balanced by the amount leaving.

The depth at which the equilibrium condition is reached is greatest at the transition between the clear water and sediment transporting conditions, ie at the threshold of movement. The approach velocity then equals $\bar{u}_c$, the average critical velocity for initiating sediment movement.

Chiew and Melville (1987) present the factors which affect the local scour depth at bridge piers; firstly the ratio of scour depth to pier width is a basic function of $\bar{u}/\bar{u}_c$ (where $\bar{u}$ is mean velocity in the river). The function is shown in Figure 6 which is reproduced from their paper. There are two forms of the function, one for ripple forming sediments and one for non-ripple forming sediments, but both functions have a peak when $\bar{u} = \bar{u}_c$. The size of the sediment particles also affects the scour depth if the ratio of particle diameter to pier width is more than $^1/_{50}$. Finally, the scour is reduced if the ratio of river depth to pier width is less than 4.

The work of Chiew and Melville can be applied to scour at vortex vanes. In this case it is not the mean velocity, $\bar{u}$, which drives the vortex but, as was seen in Section 2.1, the velocity of the near-bed flow entering the vortex, v_b. The parameter which is equivalent to the bridge pier width can be taken as the spacing between the vanes at their downstream end. However, the exact dimension chosen is not important because the ratio of particle diameter to a typical dimension of the structure is always likely to be less than $^1/_{50}$. Also the ratio of depth to pier width in bridge pier scour is not expected to be important for vortex vanes because the scour is a near-bed phenomenon, and is not greatly affected by the height of the free surface.

The analogy between scour at vortex vanes and bridge piers should not be stretched too far, however the discussion above indicates that Figure 6 can be applied to vortex vanes if v_b is used instead of $\bar{u}$. The exact equivalence of the ratio of scour depth to pier width is not clear, but appreciable scour can be expected to occur at vortex vanes when the near bed velocity v_b is equal to or greater than $\bar{u}_c$, the mean velocity at initiation of sediment movement. Figure 6 suggests that when the value of v_b falls below $\bar{u}_c$ the scour will soon reduce. Therefore the self-cleaning criterion is taken as: v_b must be equal to or greater than $\bar{u}_c$. The value of $\bar{u}_c$ depends only on sediment and fluid properties and the flow depth, in this work it is predicted using the method of Van Rijn (1984).

Laboratory testing of this tentative prediction method for self-cleaning is reported in Section 3.4.

2.4 Performance of excluder

An important component in the design of vortex vanes is the prediction of their performance, this section explains a theory which can be used to make the performance prediction.

Sanmuganathan has developed a theory which can be used to predict the performance of vortex tube and tunnel type sediment extractors (Sanmuganathan, 1976). The theory has been tested at six sediment extractors in India, Nepal and Indonesia, good agreement was found between predicted and measured performance at each extractor (Atkinson, 1987). The theory has been adapted to conditions at vortex vane sediment excluders, and is briefly described below.

If the mean sediment concentration in the river is known, then the value of the ratio P is required, where

$$P = \frac{\text{sediment concentration entering canal}}{\text{sediment concentration being carried by river}}$$

The theory predicts P by first using formulae for the variation of sediment concentration with height above the bed, and for the velocity at each height. The theory then predicts the sediment flux at each height by taking the product of the concentration and velocity profiles.

The total quantity of sediment, per unit width, in the flow approaching the vanes is the integral of the sediment flux from the bed to the water surface, including the bed load. Meanwhile the proportion of this sediment which passes into the canal is the same integral but from the elevation of the dividing streamline to the water surface. The ratio of these two integrals is the ratio P. The appendix presents in detail the equations which form the predictive theory.

A value for b, which is needed in the design method to set the vane spacing and downstream vane height, can be obtained using the predictive theory. A graph of P against b can be plotted using Equations 11 and 12 of the Appendix, the value of b corresponding to the required value of P can then be read from the graph.

The laboratory experiments described in the next chapter did not include a testing of the theory just outlined, they were aimed chiefly at predicting whether or not vortex vane sediment excluders are feasible in the field. Testing of the performance prediction theory did not need to be included in these initial laboratory experiments because feasibility of vortex vanes does not hinge on this aspect as critically as on other aspects of the design. Also the flow and self-cleaning theories presented in Sections 2.1 and 2.3 were completely untested prior to the experiments, while the performance prediction theory is similar to one for sediment extractors, which has been verified both in the laboratory and in the field.

3 EXPERIMENTAL WORK

The principal objective for the experimental work was to determine if the vortex vane sediment excluder is a viable means of sediment control in field conditions. The questions to be answered included:

(i) Can an excluder be designed to trap the bed layer of the flow into a vortex and carry it to the downstream end of the vortex?

(ii) What criteria can be applied to assess whether or not a particular design for a vortex vane will be self-cleaning?

(iii) How accurate is the design theory presented in Chapter 2?

(iv) What height of downstream vane is required?

3.1 Qualitative observations

Initial tests in a 2.44m wide laboratory flume with no sediment showed that the vanes could trap the bed layer of the flow and carry it in a vortex to the end of the vanes. The vanes were 70 cm long and 18.4 cm wide at maximum, and had an elevation of downstream vane of 3.7 cm. The flow depth was approximately 10 cm. Dye was injected into the flow near the bed just upstream of the vanes, it was carried into a vortex between the vanes and did not pass over the downstream vane but was discharged at the end of the vanes.

A similar excluder also appeared to perform well in a sand bed flume (D_{50} = 0.205 mm) of 5m width and about 10 cm flow depth. The space between the vanes was initially filled with the flume sand, after half an hour of running the flow had washed sand from between the vanes and a vortex had formed. The vortex was seen to trap the bed load approaching it and to discharge it into a scour hole which the flow had formed at the downstream end of the vanes. High sediment concentrations could be seen being carried away downstream from the end of the vanes.

3.2 Design of vanes for the experiments

The theory presented in Chapter 2 was used to recommend that the centreline of the vanes was kept at 45° to the approaching flow near the bed (Section 2.2). The flow into the offtake canal distorts the near bed flow patterns, but according to the theory, has no other effect on the vortex vane design. Therefore to simplify the experiment, and to test each aspect of vane design separately, it was decided initially to test a vane with no offtake. When there is no offtake the flow is parallel to the flume wall, so a vane with a straight axis at 45° to the flow was designed.

The near bed velocity was assumed to be constant across the flume and the value of b was chosen at 0.0375m. Equation 18 then gave the following formula for the vane spacing:

$$2a = 2\sqrt{0.0119\ x} \qquad (19)$$

The length of vanes was set at 70cm, hence their maximum spacing was 18.4 cm, and initially the downstream vane height was set at 0.0375m above the upstream vane elevation.

3.3 Experiments with a fixed bed

3.3.1 Description of experiments

The first set of experiments were designed to verify the flow theory and to investigate the required height for the downstream vane. They were performed in a fixed bed flume: its width was 2.44 m, its flow depth ranged from about 0.15m to about 0.25m, and the maximum discharge was about 0.25 m^3/s.

Firstly the range of conditions required for the tests were derived using dimensional analysis. When the effect of the sediment is neglected, dimensional analysis shows that the flow conditions in the vortex depend on:

- the geometry of the vanes;
- the Froude number of flow in the river;
- a Reynolds number for the flow between the vanes;
- the ratio of a vane dimension to flow depth; and
- the shape of the velocity profile upstream of the vanes (the shape can be described by the ratio $\bar{u}/u_*$)

In the field the Reynolds number will be high (about 500 000) so the flow will be rough turbulent, a typical value for Reynolds number in the experiments was 30 000, therefore the flow in the model was also rough turbulent and so Reynolds number effects can be ignored.

Tests were run under five conditions, the values for Froude number, flow depth and the ratio $\bar{u}/u_*$ are given in the following table:

TEST CONDITION	FLOW PER m WIDTH (m^2/s)	FROUDE NUMBER	FLOW DEPTH (m)	$\bar{u}/u_*$
1	0.099	0.28	0.235	11.6
2	0.101	0.48	0.165	9.1
3	0.102	0.44	0.175	33.2
4	0.098	0.26	0.242	30.9
5	0.092	0.27	0.223	13.7

Velocities in flume. For each condition, the velocities were measured on five verticals at a cross-section upstream of the vanes. Five points were measured on each vertical using a minature propeller current meter. The data for the five verticals were averaged, and a logarithmic velocity profile was fitted to the bottom three points. The bottom three points only were used because the purpose of fitting a profile was to determine the velocity of the flow entering the vortex. The values of $\bar{u}/u_*$ shown in the table above were derived from these fitted velocity profiles, the averaged data is shown in Table 1.

Discharge between vanes. For each condition, the axial flow velocities between the vanes were measured, this was repeated for a range of downstream vane heights. Again a minature current meter was used, the rotation speed of the propeller was so much greater than the rotation speed of the vortex that the latter's effect was negligible. To convert the axial flow velocities into discharges the flow area was required. The flow area was estimated as the rectangular area between the vanes from the flat bed up to the elevation of the dividing streamline. A small allowance for the region outside the vortex near the base of the vanes was subtracted from the area to account for the slow flow in the region. The discharge between the vanes is given in Table 2 for all conditions.

Height of dividing streamline. The height of the dividing streamline between the flow passing into the vortex and passing over the vanes was determined indirectly from the velocity measurements. The measured discharge between the vanes was equated to the discharge flowing below the dividing streamline upstream of the vanes. The average velocities in the region below the dividing streamline are tabulated in Table 2, together with the dividing streamline heights.

3.3.2 Verification of flow theory

The flow theory gave the result that the axial velocity between the vanes, v_v, should be equal to the component of the velocity approaching the vortex which was parallel to the vane axis, v_a. However, the theory did not give guidance on the required height of the downstream vane. If the height was not sufficient then axial velocities in the vortex smaller than v_a could be expected. Also friction losses would produce values of v_v smaller than v_a.

The data of Table 2 shows that there is a good correlation between v_v and the velocity of the flow below the dividing streamline (v_b). Figure 7 gives a plot of v_v against v_b for two cases: a downstream vane height of 0.037m and a height of 0.077m. The figure shows that v_v is proportional to v_b for the whole range of values for Froude number and $\bar{u}/u_*$. The data for the other vane heights could not all fit on the one graph, but the rest of the data does show the same pattern. This result is encouraging because it helps to verify the theory which predicts that, for a certain vane geometry, the flow between the vanes depends only on the near-bed velocity of the flow upstream; and not on the Froude number or the variation of velocity with depth.

All the data was used to obtain a mean value of the ratio v_v/v_a for each downstream vane height. Because the angle of the vanes was 45°, this ratio is $v_v/(v_b \text{ Cos } 45°)$. Mean values for v_v/v_a are plotted in Figure 8 against downstream vane height.

Figure 8 should be considered together with Figure 9 which shows the measured height of the dividing streamline plotted against downstream vane height. Mean values for all five conditions are shown in Figure 9 (individual values varied by up to ± 15% from the mean, but the scatter appeared to be random as there were no trends with either Froude number or $\bar{u}/u_*$.)

The design conditions according to the flow theory are plotted as dotted lines on Figures 8 and 9, they are $v_y/v_a = 1$ and dividing streamline height = 0.037m. Figure 9 shows that design conditions are obtained with a downstream vane height, h, equal to 0.058m, however Figure 8 shows that at h = 0.058m the ratio v_v/v_a is 0.8. In fact we do expect v_v to be less than v_a and there are two reasons. Firstly friction causes a drop in velocity, and secondly the flow area is greater than is assumed in the design theory. The theory assumes that the flow area is the cross-sectional area of a cylindrical vortex, while the actual cross-section between the vanes is rectangular. When the design discharge is flowing between the vanes the increased area causes the velocity to be less than its design value.

Overall, the experimental results show that the theory can be used to design a vane which will extract the required quantity of water from the bed layer of the flow. The results show that the elevation of the downstream vane should be set at about 1.6b.

Initial measurements also suggested that water was flowing into the vortex evenly along the length of the vanes.

3.4 Experiments with a mobile bed

Further experiments were performed in a flume with a bed material of fine sand.

The median sand size was 0.205 mm, the flume width 5m and typical depths and flow intensities were about 0.14m and 0.07 m^2/s respectively. The objectives for these experiments were firstly to test the overall operation of a vortex vane sediment excluder in conditions where bed levels around the vanes were set by the natural processes of scour and sediment transport. The excluder did appear to perform well in these conditions as described in Section 3.1 above. The second objective was to establish a criterion to assess the flow conditions required to allow a vortex vane to self-clean, more specifically: to test the self-cleaning criterion outlined in Section 2.3

The vortex vane used in the mobile bed experiments was similar in layout to that used in the fixed bed experiments, however its length was 60cm, its design value of b was 3cm and its downstream vane height (above the elevation of the upstream vane) was 5cm, that is 1.67b.

The space between the vanes was filled with the flume bed material and the flume was run with a relatively high downstream water level. The downstream water level was gradually lowered until the vanes self-cleaned. Measurements made at the best estimate of the threshold condition were used to compare predicted and observed values of the near-bed velocity required for self-cleaning. The measurements are recorded in Table 3 and the comparison is summarised in Table 4. For the sediment sizes coarser than the original fine sand, the sediment was only placed between and around the vortex vanes, the rest of the flume contained the original fine sand.

Table 4 shows that the method presented in Section 2.3 does give good predictions of the near-bed velocity which is required to achieve self-cleaning. The ratio of observed to predicted near-bed velocity had a mean of 1.02 and a standard deviation of 0.17.

Chiew and Melville (1987) do not report field data in their paper on bridge pier scour so caution is necessary when applying the self-cleaning criterion, which is adapted from it, at a field scale. To investigate the performance of the theory at a slightly larger scale the experiments just described were repeated on a similar model scaled up by a factor of 1.67. Limitations of laboratory facilities prevented complete equivalence of the experiments, the larger flume did not have a sediment feeding device so sediment could only be placed in the locality of the vanes. Therefore the experiments at the larger scale were for clear water scour. The result was nevertheless encouraging: predicted near-bed velocities were, on average, 11% greater than with the previous experiments, while observed velocities were on average only 2% greater. The result is within expected scatter. The data from these tests has been included in Tables 3 and 4.

When the prediction theory is applied to a typical set of field scale conditions it indicates that vortex vane sediment excluders are likely to self-clean, provided that the river bed does not contain material coarser than about 2mm.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 Feasibility of vortex vanes

The vortex vane sediment excluder appears to be a viable means of sediment control at a field scale. The work reported here has shown that the vanes do operate as expected, it has also predicted that they can self-clean even after material has deposited between them, and that the vane's performance is likely to be satisfactory. More definite conclusions are now not possible without a field trial.

Laboratory work has shown that the vanes do trap a vortex and they do carry the bed layer of the flow away from the bank where an intake would be sited. Experiments in a mobile bed flume showed that the vortex between the vanes could carry high sediment concentrations.

Research into the vital self-cleaning criterion for vortex vanes has produced the prediction that the river flow will be able to scour out the space between the vanes, even if they are initially blocked with river bed sediment. If the river bed contains very coarse material such as gravel, then the vanes may not self-clean. The theory which produced these predictions has been tested in the laboratory at two scales to provide confidence in its performance at a field scale. However, the theory still cannot be used with complete confidence without field verification.

The performance of vortex vanes in the field, in terms of reduction of sediment loads entering an offtake, can be assessed using the theory presented in Section 2.4 and in the Appendix. The theory has not been verified at vortex vanes, but it is similar to a performance prediction method for sediment extractors which has been verified successfully at six field sites. Experience gained from fieldwork on sediment extractors suggests that the mean sediment concentration entering a canal would be reduced to roughly half the mean concentration in the river. At an intake with no sediment control measures, the concentrations entering a canal are usually significantly greater than those in the river.

4.2 Design of vortex vanes

The theoretical and laboratory work described in this technical note has produced the following design recommendations for vortex vane sediment excluders. They may subsequently be adjusted in the light of field experience.

(a) The upstream vane should be set at an elevation equal to the mean bed level of the river.

(b) The vane spacing and the elevation of the downstream vanes are set by the required height of the dividing streamline b. The value of b is set by the required performance of the device, the method of calculation for b is summarised in Section 2.4 and is presented in more detail in the Appendix.

(c) The spacing between the vanes is set by use of Equation 18.

(d) The elevation of the downstream vane, relative to the upstream vane, should be set at about 1.6b.

(e) The centreline of the vanes should be set so that it is always at 45° to the direction of the near-bed flow.

(f) The length of the vanes should be set so that they extend past the point where flow discharging from them could enter the intake.

4.3 Recommendations

The clear recommendation of this technical note is that a field trial of a vortex vane sediment excluder is now required. Further laboratory work may shed more light on the design aspects of the device, but its feasibility for field conditions can now only be assessed in the field. The field trial should be targeted to:

(a) verify the performance prediction theory for vortex vanes;

(b) test the self-cleaning criterion;

(c) refine some of the design aspects; and

(d) investigate the danger of the vanes preventing dry season flows from entering the intake.

5 ACKNOWLEDGEMENTS

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TABLES.

TABLE 1: Velocity measurements upstream of vanes (fixed bed experiment)

	MEASURED VELOCITY (m/s), mean of the values over five verticals				
CONDITION NUMBER	1	2	3	4	5
HEIGHT ABOVE THE BED (mm)					
7	0.170	0.207	0.482	0.316	0.211
41		0.623	0.590		
55	0.421			0.401	0.403
82		0.725	0.603		
110	0.463			0.417	0.455
123		0.708	0.616		
165	0.479	0.696	0.580	0.431	0.467
220	0.464			0.411	0.449

TABLE 2: Summary of results of fixed bed experiments

CONDITION	HEIGHT OF DOWNSTREAM VANE * (m)	AXIAL VELOCITY BETWEEN VANES, v_v (m/s)	MEASURED DISCHARGE FLOWING BETWEEN VANES (m^3/s)	HEIGHT OF DIVIDING STREAM LINE (m)	VELOCITY BELOW DIVIDING STREAM LINE, v_b (m/s)	RATIO $v_v/v_a = v_v/(v_b \cos 45°)$
1	0.037	0.095	0.0027	0.024	0.24	0.56
	0.057	0.196	0.0062	0.043	0.29	0.94
	0.067	0.234	0.0077	0.051	0.31	1.07
	0.077	0.242	0.0080	0.053	0.31	1.10
	0.087	0.286	0.0099	0.062	0.33	1.24
2	0.037	0.146	0.0042	0.025	0.36	0.57
	0.057	0.185	0.0058	0.031	0.40	0.66
	0.067	0.252	0.0082	0.040	0.44	0.81
	0.077	0.321	0.0110	0.049	0.47	0.96
	0.087	0.355	0.0124	0.053	0.49	1.03
3	0.037	0.238	0.0073	0.028	0.52	0.65
	0.057	0.314	0.0102	0.039	0.53	0.83
	0.067	0.340	0.0113	0.043	0.54	0.90
	0.077	0.343	0.0114	0.043	0.54	0.90
	0.087	0.378	0.0129	0.048	0.54	0.99
4	0.037	0.172	0.0054	0.032	0.35	0.71
	0.057	0.172	0.0054	0.032	0.35	0.71
	0.067	0.191	0.0061	0.035	0.35	0.77
	0.077	0.214	0.0070	0.040	0.35	0.86
	0.087	0.239	0.0081	0.045	0.36	0.95
5	0.037	0.130	0.0040	0.030	0.28	0.66
	0.077	0.204	0.0068	0.045	0.31	0.93

* height is relative to elevation of the upstream vane

TABLE 3: **Measured conditions at threshold of self-cleaning**

		TEST CONDITION: Size of Sand Between Vanes (mm) D_{50} =			
		0.205	0.205 (Note 1)	0.25	0.82
FIRST EXPERIMENT					
Flow depth (m)		0.14	0.137	0.132	0.163
Velocity (m/s) at following heights above bed (mm) (Note 2)	7	0.18	0.073	0.093	0.20
	14	0.41	0.35	0.29	0.32
	28	0.46	0.40	0.36	0.47
	56	0.56	0.48	0.43	0.53
Fitted values for $\bar{u}$ and u_* (m/s) (Note 3)	$\bar{u}$ =	0.567	0.497	0.433	0.562
	u_* =	0.069	0.073	0.062	0.067
Near-bed velocity (m/s) (Note 4)		0.313	0.239	0.218	0.294
LARGER SCALE EXPERIMENTS					
Flow depth (m)		0.300	-	0.315	0.315
Velocity (m/s) at following heights above bed (mm) (Note 2)	15	0.32	-	0.34	0.35
	45	0.37	-	0.41	0.42
	85	0.52	-	0.54	0.55
	155	0.64	-	0.67	0.67
Fitted values for $\bar{u}$ and u_* (m/s) (Note 3)	$\bar{u}$ =	0.57	-	0.61	0.61
	u_* =	0.077	-	0.080	0.072
Near-bed velocity (m/s) (Note 4)		0.243	-	0.261	0.290

NOTES

1. Repeat measurements.
2. Average over 3 verticals.
3. The velocity data was fitted to the velocity profile shape which is used in the Appendix.
4. Derived by integrating the fitted velocity profile from the bed to the design height of dividing streamline, b. (Below the height where u = 0, zero velocity is assumed).

TABLE 4: **Comparison of predicted and observed near-bed velocity at self-cleaning conditions**

First Experiment:-

SEDIMENT SIZE		THRESHOLD CONDITIONS FOR SELF-CLEANING			RATIO:
D_{50} (mm)	D_{90} (mm)	FLOW DEPTH (m)	MEASURED NEAR-BED VELOCITY (m/s)	PRECITED NEAR-BED VELOCITY (m/s)	OBSERVED / PREDICTED
0.205	0.25	0.14	0.313	0.245	1.28
0.205	0.25	0.14	0.239	0.245	0.98
0.25	0.42	0.132	0.218	0.235	0.93
0.82	1.03	0.163	0.294	0.328	0.90
					Mean 1.02 sd 0.17

Larger Scale Experiment:-

SEDIMENT SIZE		THRESHOLD CONDITIONS FOR SELF-CLEANING			INCREASE FROM FIRST EXPERIMENT (%)	
D_{50} (mm)	D_{90} (mm)	FLOW DEPTH (m)	MEASURED NEAR-BED VELOCITY (m/s)	PRECITED NEAR-BED VELOCITY (m/s)	OBSERVED	PREDICTED
0.205	0.25	0.30	0.243	0.269	-12	+10
0.25	0.42	0.32	0.261	0.264	+20	+12
0.82	1.03	0.32	0.290	0.362	-1	+10
				MEAN	+2	+11

FIGURES.

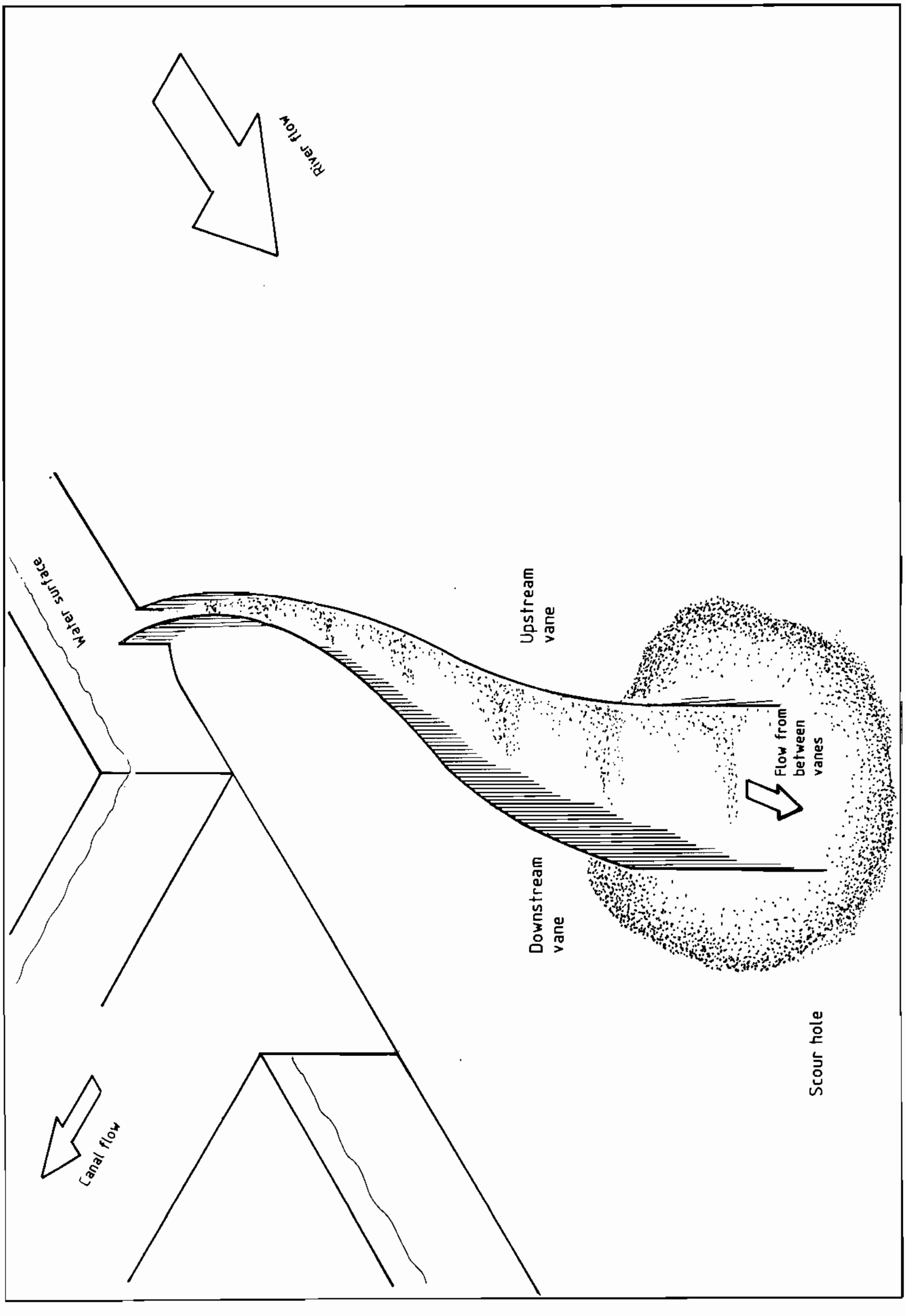

Fig 1 Layout of a vortex vane sediment excluder

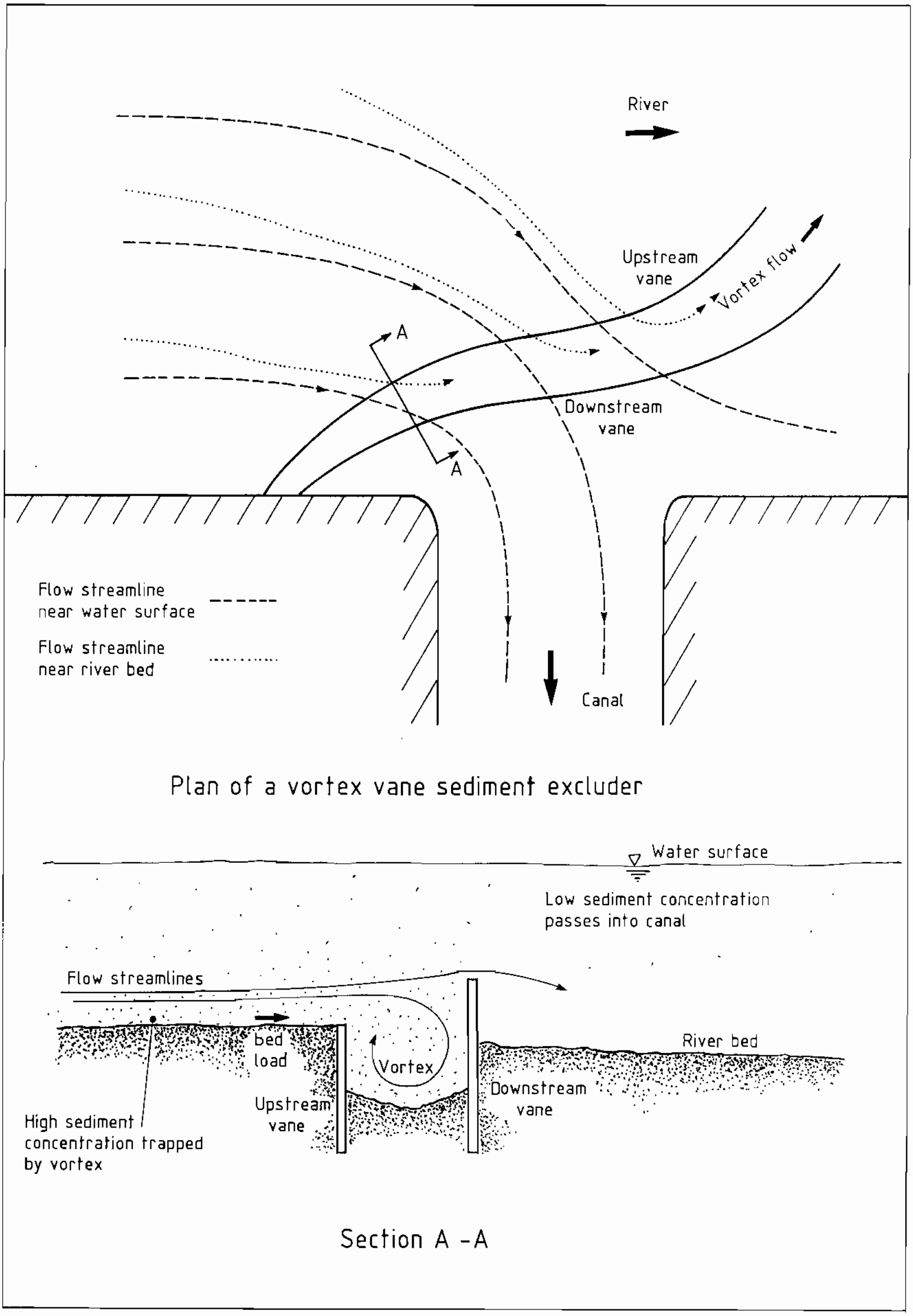

Fig 2 How vortex vanes exclude sediment

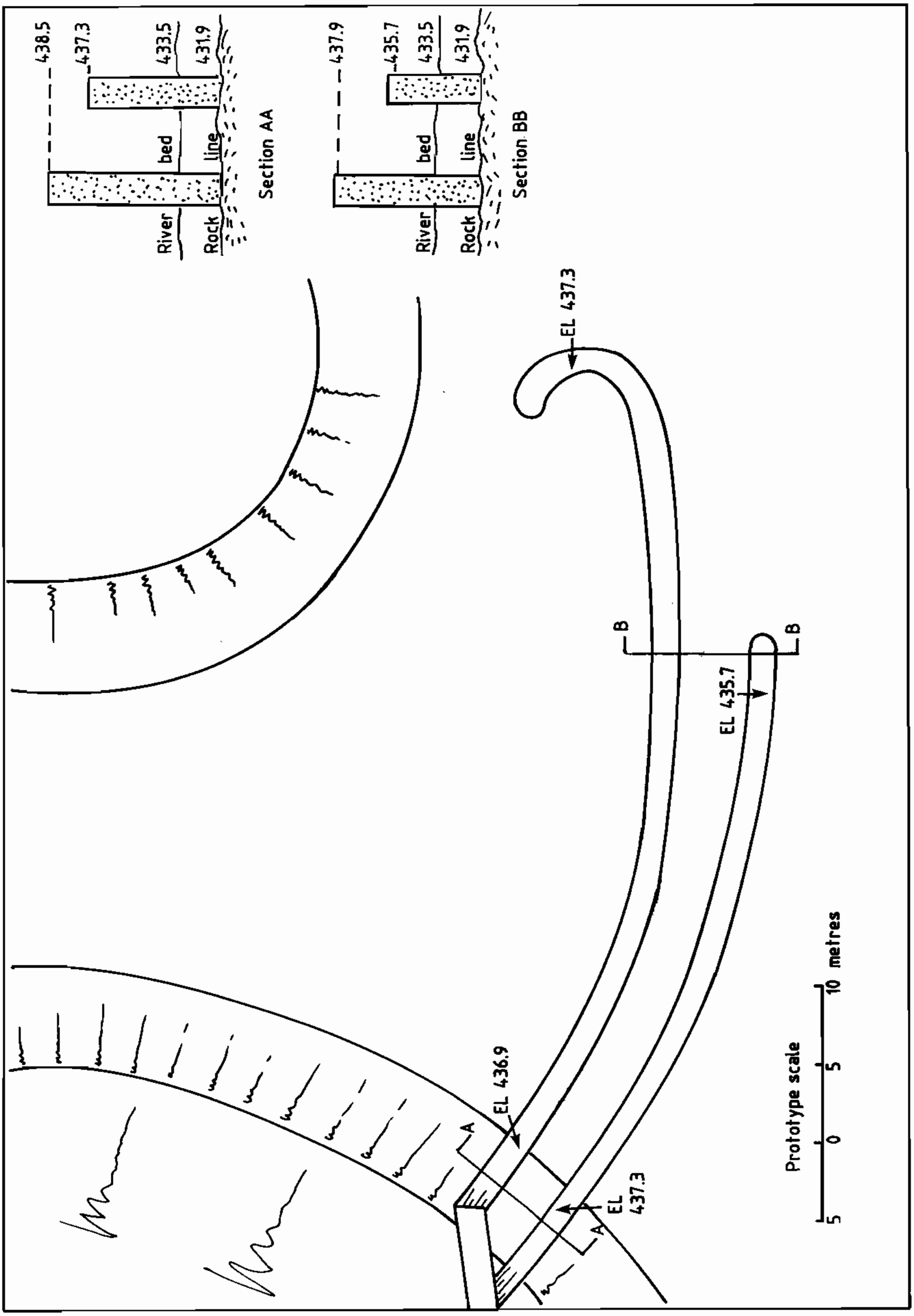

Fig 3 Plan of sediment controlling structure, model of Polgolla diversion

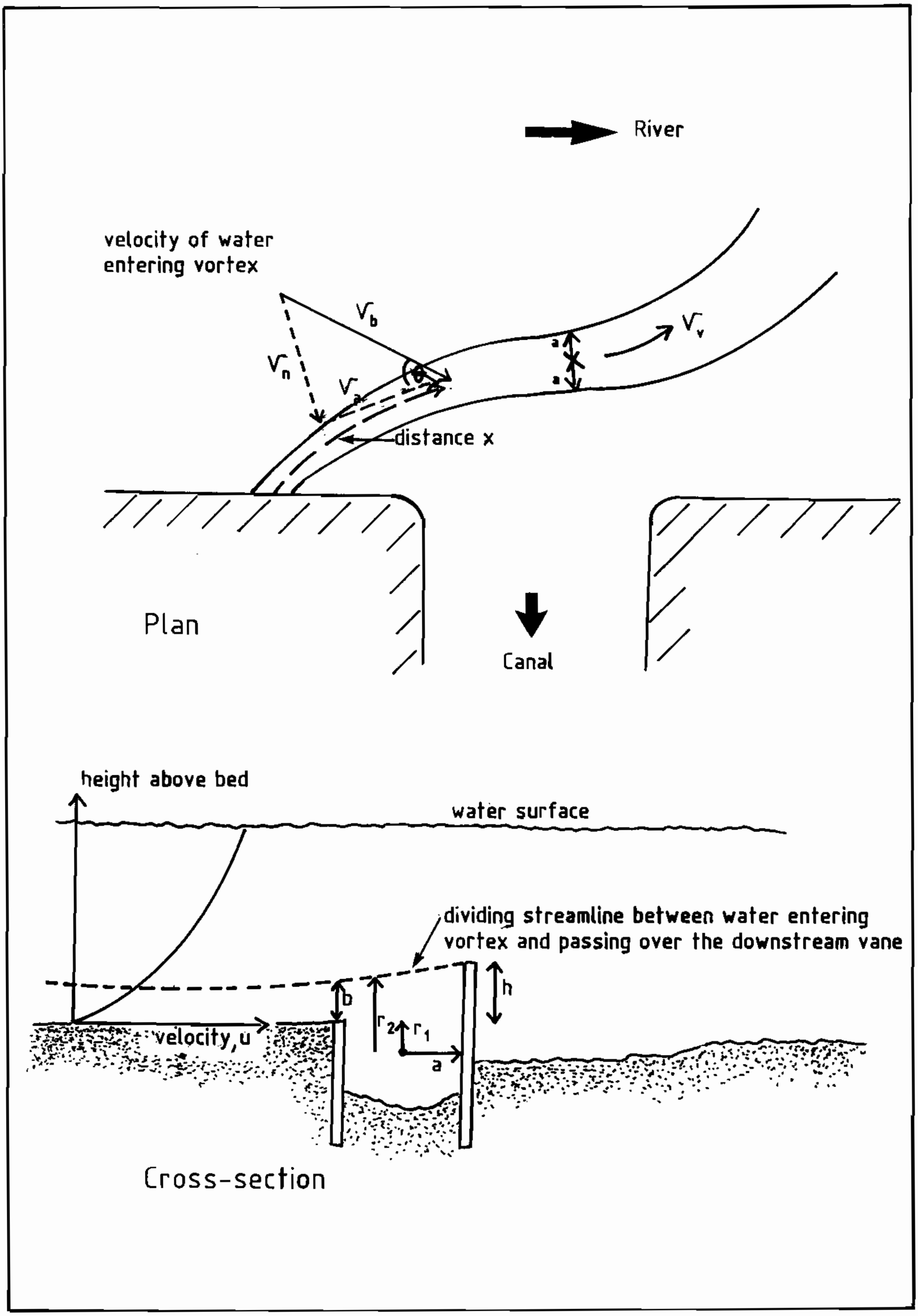

Fig 4 Symbols used in the flow analysis

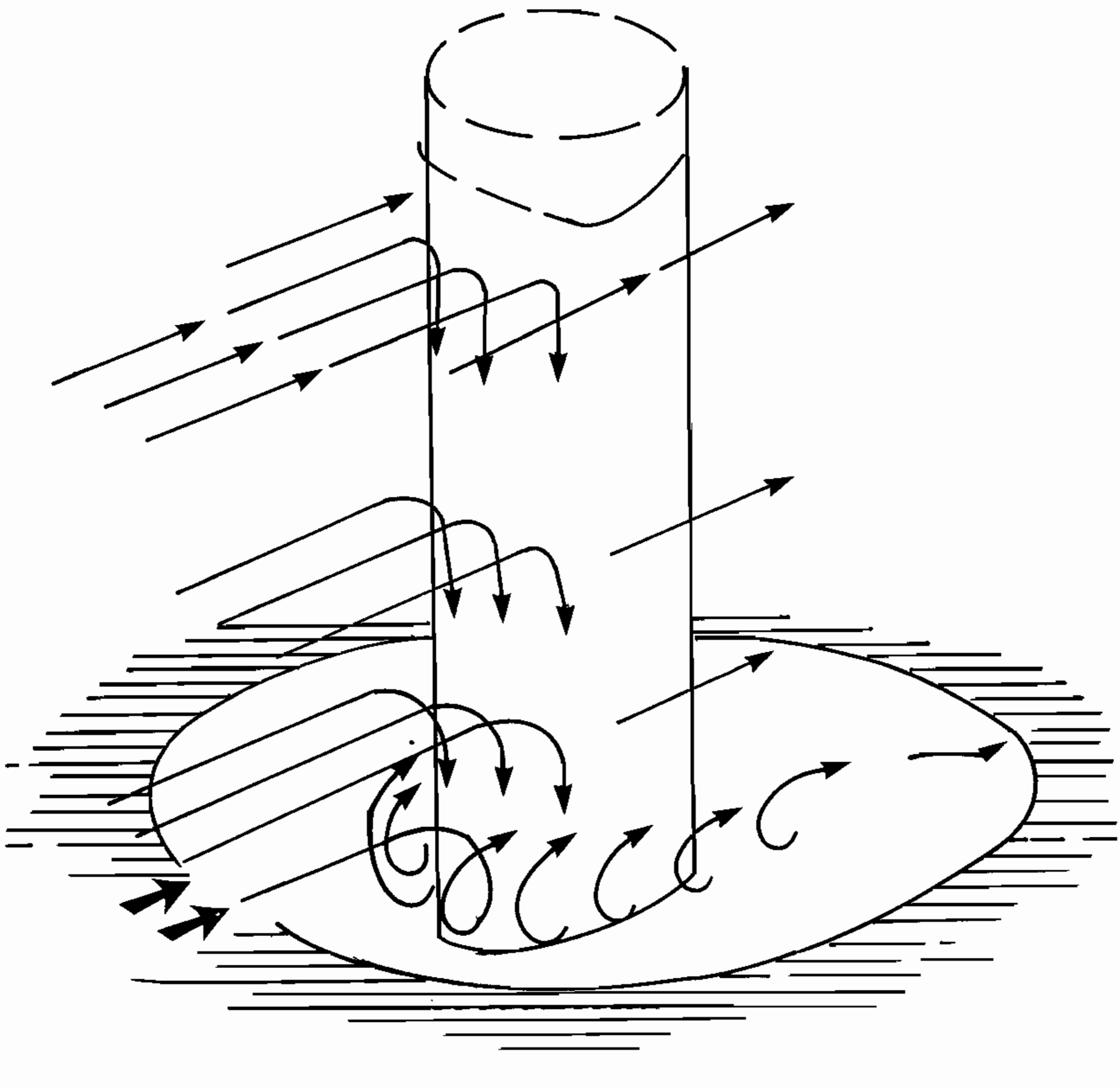

Fig 5 Horseshoe vortex formation at a cylindrical pier

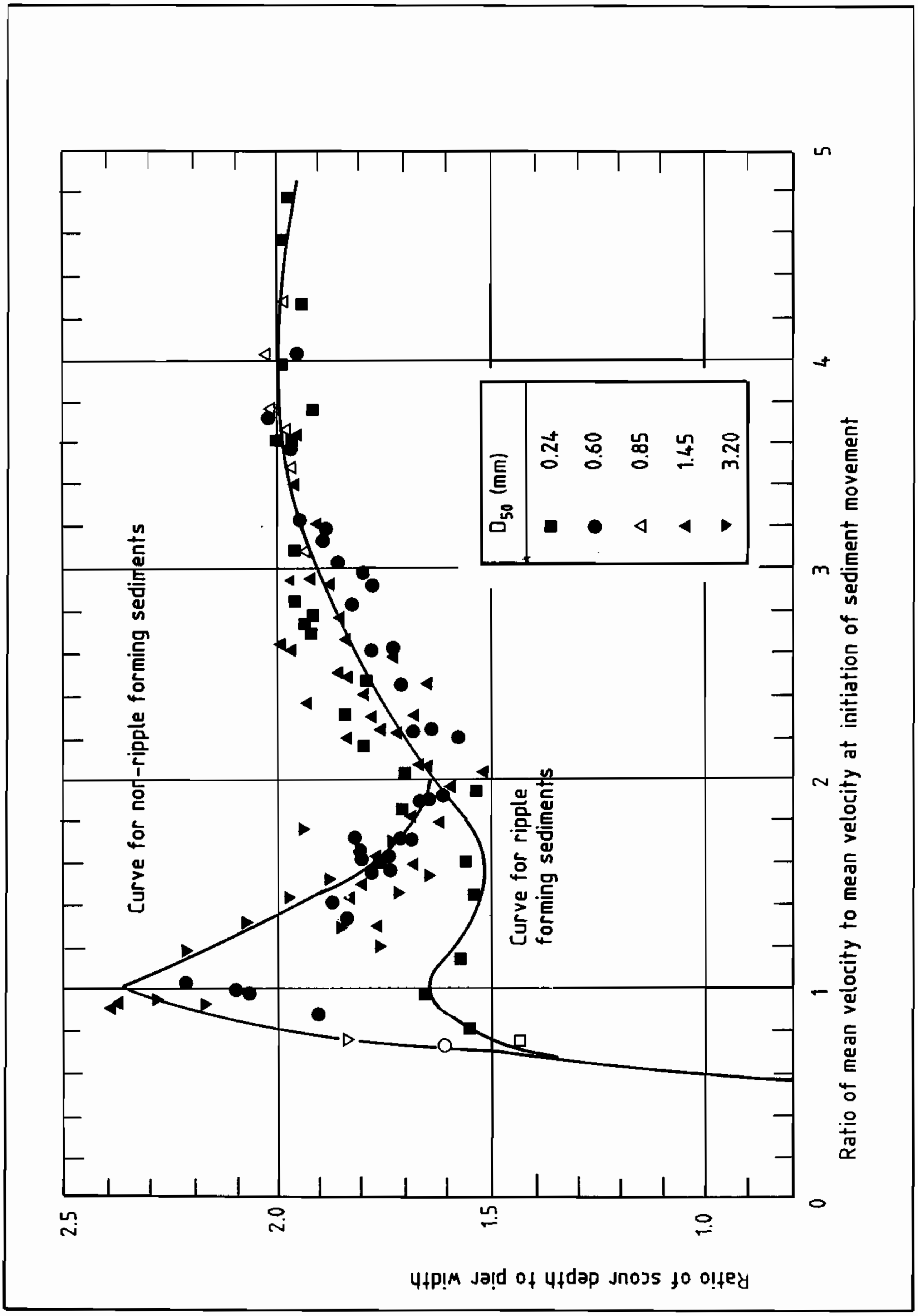

Fig 6 Relative equilibrium scour depth versus $\bar{u}/\bar{u}_c$ where the effects of flow depth and sediment size are not present (from Chiew & Melville)

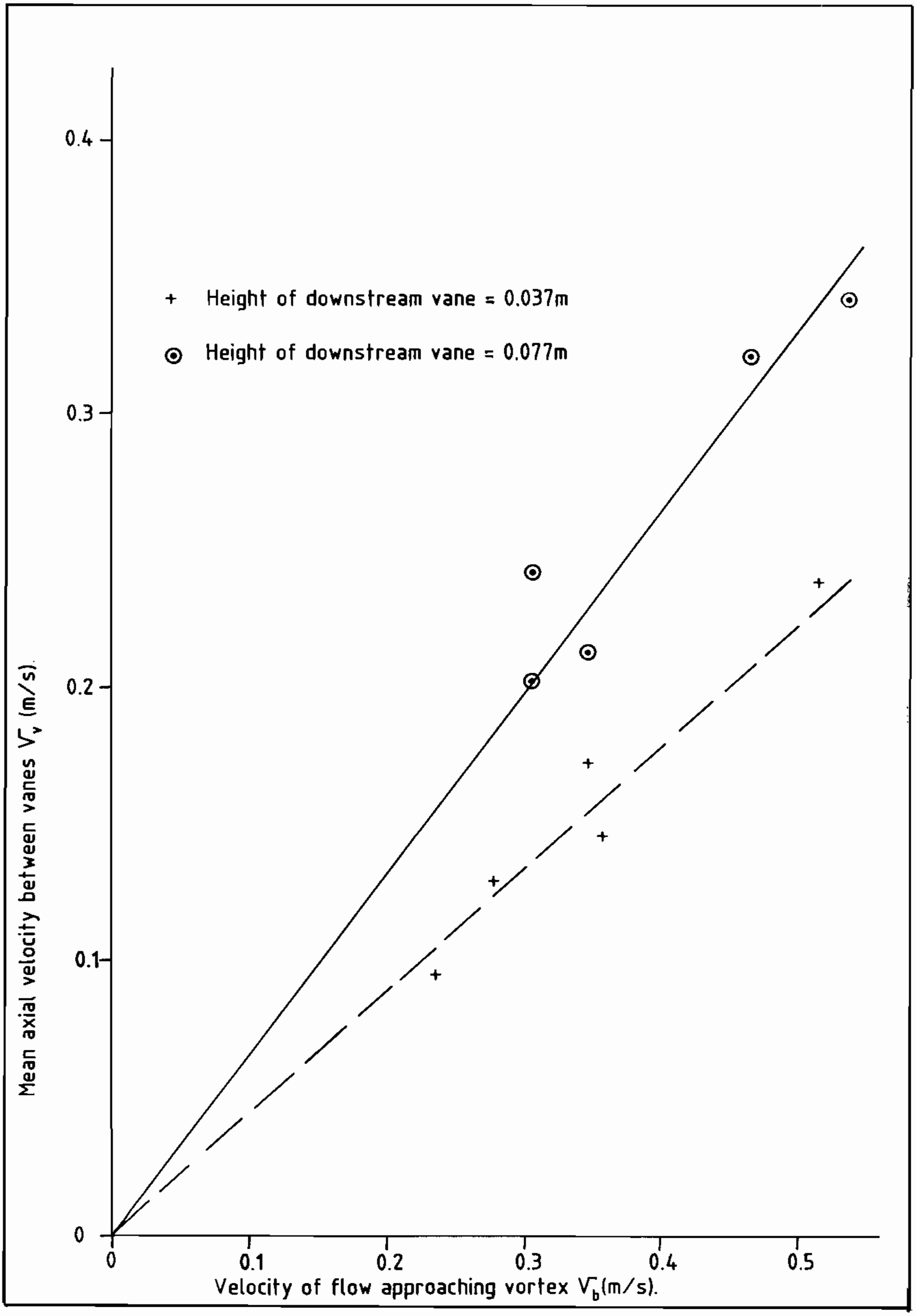

Fig 7 Plot of axial velocity between vanes against velocity approaching vanes

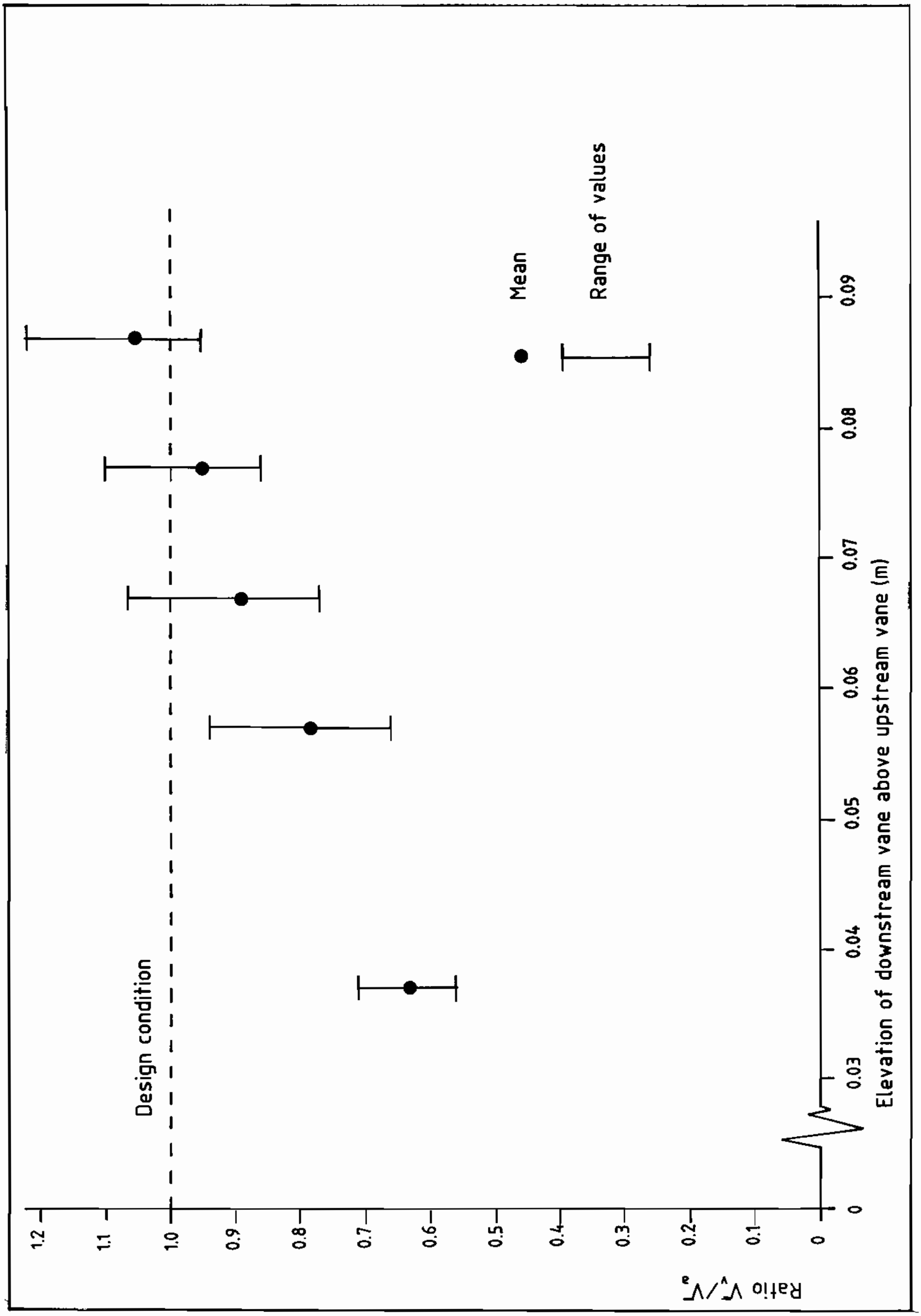

Fig 8 Ratio $\bar{V}_v/\bar{V}_a$ against height of downstream vane

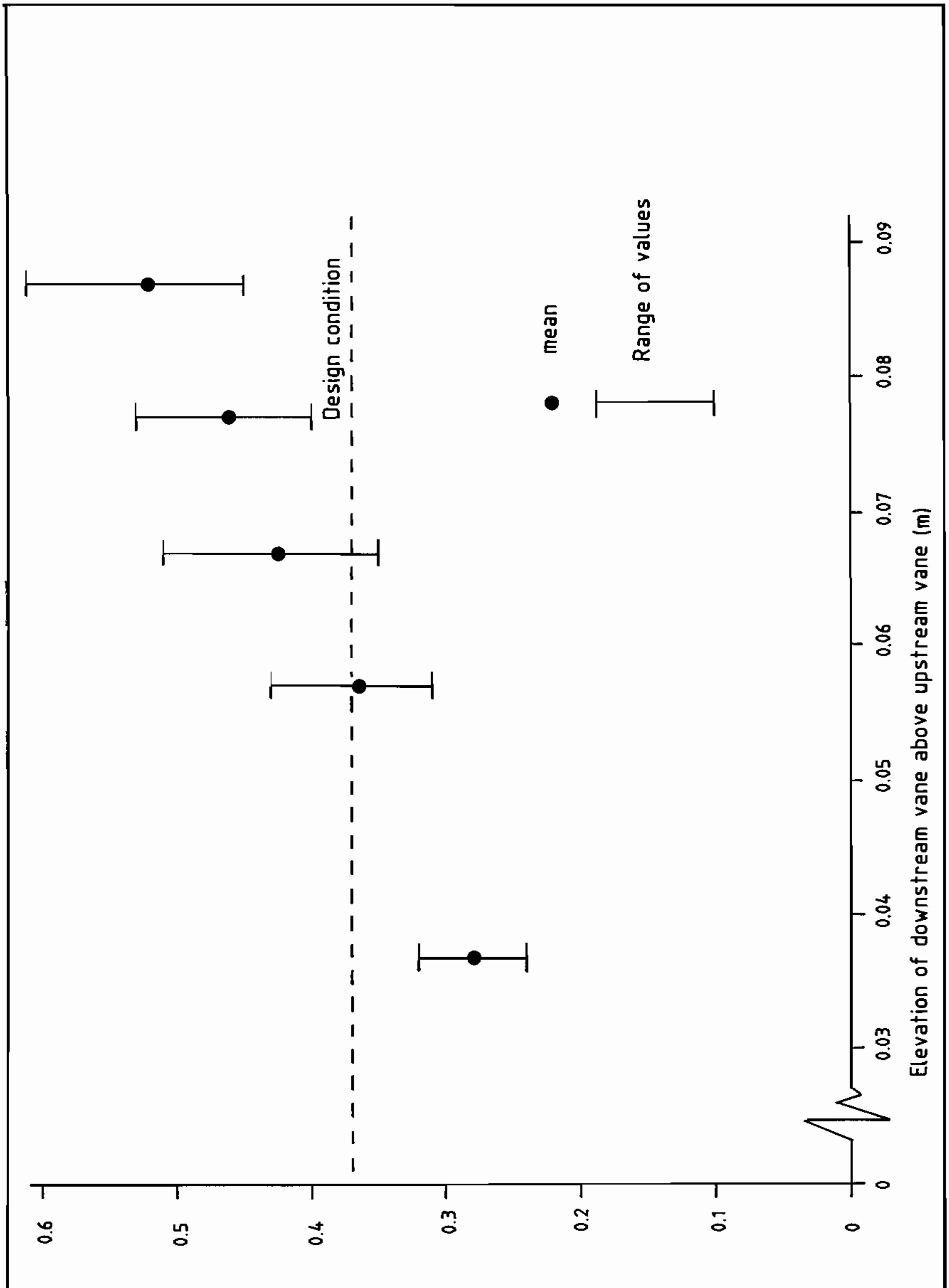

Fig 9 Height of dividing streamline as a function of downstream vane height

APPENDICES.

APPENDIX: Equations for the prediction of excluder performance

The velocity profile of the flow approaching the vanes is assumed to be logarithmic

$$\frac{u}{u_*} = \frac{\bar{u}}{u_*} + \frac{1}{\kappa}\left(\ln(\frac{z}{d}) + 1\right) \qquad (1)$$

where u is the velocity at height z above bed, $\bar{u}$ is the mean velocity, u_* is the shear velocity, κ is von Karman's constant and d the depth of flow.

Turbulent fluctuations maintain sediment in suspension against gravity, the balance is described:

$$C\,V_s + \varepsilon_s \frac{\partial C}{\partial z} = 0 \qquad (2)$$

where C is the concentration, V_s the settling velocity and ε_s the sediment diffusion coefficient, it is assumed (Lane & Kalinski, 1942) to be $du_*/15$. The solution to Equation 2 is the sediment concentration profile:

$$\frac{C}{C_a} = \mathrm{Exp}\left[-15\,\frac{V_s}{u_*}\,(\frac{z}{d} - a)\right] \qquad (3)$$

where a is the height of the bed layer relative to the depth, it is taken as the greater of twice the grain diameter or the height at which $u = u_*$ (to avoid negative velocities near the bed). C_a is the concentration at that point.

Within the bed layer the bed load is described (Einstein, 1950):

$$u = u_*^{`} \qquad \text{and} \qquad (4)$$

$$\frac{C}{C_a} = 1 \qquad (5)$$

$u_*^{`}$, the grain shear velocity, is a variable used to predict the shear velocity from the flow parameters d and $\bar{u}$.

It is determined together with u_*, by iterating between Engelund's rough turbulent flow equation:

$$\frac{\bar{u}}{u_*} = 2.5\,\ln\,(\frac{d^{`}}{2D65}) + 6.0 \qquad (6)$$

$$\text{where } d^{`} = d(\frac{u_*^{`}}{u_*})^2 \qquad \text{by definition} \qquad (7)$$

and his relationship (Engelund, 1966) between:

$$Y = \left[\frac{u_*^2}{g\,(S_g - 1)\,D_{35}}\right] \qquad \text{and} \qquad (8)$$

$$Y` = \left[\frac{u_*^{`2}}{g\ (S_g-1)\ D_{35}}\right] \tag{9}$$

where Y and Y` are entrainment functions, S_g is the specific gravity of the sediment, g is acceleration due to gravity, and D_{35} and D_{65} are bed sediment grain sizes.

The water velocity and the sediment concentration profiles have now been determined, their product is the sediment flux. The integral of this flux from the bed to the depth b is the sediment load into the vortex between the vanes (per unit width of river), where b is the height of the dividing streamline. The integral from the bed to the free surface is the total load per unit width, and the integral from b to the surface is the load passing over the vanes. The ratio of the load passing over the extractor to the total load is the performance ratio, P. So using suffix j for a particular size fraction of the sediment in suspension, we get:

$$P_j = \frac{\int_b^d C_{aj}\ e^{-15 \frac{V_{sj}}{u_*} (z/d-a)}\ u\ dz}{\int_0^{ad} C_{aj}\ u\ dz + \int_{ad}^d C_{aj}\ e^{-15 \frac{V_{sj}}{u_*} (z/d-a)}\ u\ dz} \tag{10}$$

C_{aj} cancels so:

$$P_j = \frac{\int_b^d \left[\frac{\bar{u}}{u_*} + \frac{1}{\kappa}\left(\ln(\frac{z}{d})+1\right)\right] e^{-15 \frac{V_{sj}}{u_*} (\frac{z}{d} -a)} dz}{a\ u_*^`/u_* + \int_{ad}^d \left[\frac{\bar{u}}{u_*} + \frac{1}{\kappa}\left(\ln(\frac{z}{d})+1\right)\right] e^{-V_{sj}/u_* (\frac{z}{d} -a)} dz} \tag{11}$$

The overall value of P is then:

$$P = \frac{1}{M} \sum_{j=1}^{M} P_j \tag{12}$$

where M is the number of size fractions.

The value of P calculated above is derived for conditions at a single position along the vane axis. The bed will form at the elevation of the upstream vane so the depth will be constant along the vanes, however $\bar{u}$ may well vary. Here a single value for $\bar{u}$ is assumed, so P is the overall prediction of performance ratio.

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