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Obstacles in streams and their roles as hydraulic structures

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ABSTRACT: Any obstacle in a channel, such as a bridge pier or woody debris, exerts a drag force on the flow that reduces its momentum. In sub-critical flow this causes the water level upstream to be higher than downstream and flooding danger to be enhanced. A literature survey shows that there have been few definitive results for this problem, which is explained by the theory developed here. The principle of momentum conservation is applied to an arbitrary body, including the force due to free surface variation around the obstacle. An explicit formula for backwater height is developed. This depends on flow rate, which means that the obstacle, even if it does not seem to change the overall flow, is strictly a control. The theory including free surface effects explains the apparent scatter of experimental results, as it predicts that the force on the obstacle can oscillate rapidly with flow velocity because of the wave-like nature of the surface disturbance. This is rather unusual in river and canal engineering, and means that where there is a significant obstacle, the common problem of the numerical simulation of the stream is difficult and uncertain. It is suggested that a new type of hydraulic structure could be developed in some situations, one that minimises the overall force, especially by suppressing free surface drag. Overall, this is a problem of such complexity that for the foreseeable future only experimental investigations will provide accurate results for a particular situation.

1 INTRODUCTION

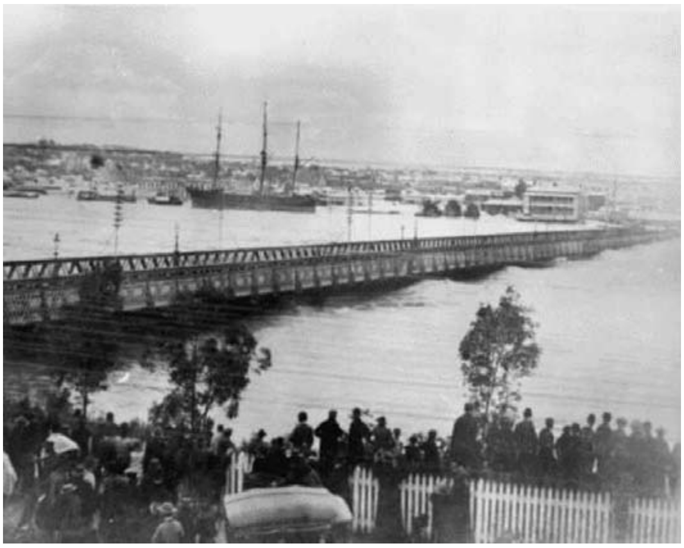

Figure 1. Brisbane, Australia – February 1893 – the Victoria Bridge before being heavily damaged by the floodwaters and floating debris. Note the local mounds upstream of the piers

In this work, an obstacle is considered to be an object in a stream that does not change the overall nature of the flow, but can have finite effects on the hydraulics of the stream. The purpose here is to show that such things can and should be considered to be hydraulic structures.

When water in a river or canal flows past an obstacle such as a bridge pier, piles, woody debris, or boundary roughnesses, there are accompanying momentum and energy losses, and the water level is different upstream and downstream of the obstacle. The problem of calculating that difference is important. There is a greater danger of flooding in the usual case of sub-critical flow, when the water surface is higher upstream of an obstacle. This can be of considerable social importance. In super-critical flow, the water level is higher downstream, and the problems such as the design of stilling basins are more hydraulically than socially significant.

Figure 1 shows the local effects caused by a bridge in a severe flood. The more important hydraulic effect, the mean backwater caused by the bridge, is not observable here.

The resistance force of an obstacle on a flow causes momentum loss to that flow, which takes place immediately. The momentum deficit is initially confined to the line of the obstacle, but as the turbulent flow proceeds downstream, the deficit is dispersed laterally. The force, and hence the momentum loss to a flow, can be simply measured experimentally, incorporating all the physical processes on the object.

In comparison, there are almost no energy losses at the front face of an obstacle, but because of the sudden local diversion of the flow, as the water flows downstream, interacting flows and turbulent flow processes cause energy losses for some distance. Unlike the momentum loss they are distributed and are more difficult to measure or calculate, as they take place in boundary layers, shear layers, separation zones, vortices, and subsequent turbulent decay in the wake.

For this reason the problem can be quantified more easily using momentum than energy as the underlying principle, and in recent decades it has been preferred. In contrast to the use of these rational principles, several well-known early research works followed only empirical procedures, which are still referred to, even if they are not used. Another difficulty in practice has been that experimental work has reported wide fluctuations in results, and it has been difficult to use them for a particular application.

The intention of this paper is to present a description of the application of momentum and drag force considerations to the problem of obstacles in streams, and to include an explicit model of the effects of the variation of the free surface around any obstacle that approaches or pierces the surface. This explains why previous experiments have shown large amounts of scatter, as the force on the obstacle can vary rapidly with the fluid velocity. Some comparisons are made with recent experiments, and conclusions are drawn that it seems to be necessary to rely on experiment in each case. A suggestion is that it may be worthwhile incorporating drag-reducing structures on bridge piers.

2 LITERATURE SURVEY

Despite its importance, the interaction between obstacles and channel flow has not been investigated as much as it might. The hydraulic jump, for example, has received much more attention, even though it is of less economic importance. There has been confusion as to the dominant governing physical processes, although in recent decades there has been a movement towards using the momentum principle. Few results have been obtained for solving practical problems, because they depend on the detailed geometry of the obstacle in each case. This has not generally been recognised.

Yarnell (1934) presented a survey of early works on bridge piers, beginning with Dubuat in 1786. He also conducted a large number of tests, obtaining values of the coefficients used in various formulae for both piers and groups of piles in rectangular channels. There was a wide scatter in the results, and none yielded anything like a constant coefficient for a particular pier shape. Textbooks quote his work, but only in the form of an empirical formula based on a few pier shapes that he presented in an appendix. The results cannot be used with any accuracy.

Rehbock (see for example Montes 1998) also conducted a large experimental programme on a variety of pier shapes and river conditions, with the results published between 1917 and 1921. He also presented an empirical formula, but its use for a general problem seems difficult. Escande (1939) developed an energy-based theory for a rectangular channel section and vertical-faced obstacles such as piers that provided a more rational approximation to the mechanics of the problem, although parts of the method were arbitrary.

In the 1960s use of momentum and drag concepts were introduced. Hsieh (1964) measured the resistance coefficients of vertical circular cylinders in a rectangular channel, but did not consider the water level differences caused by the cylinders, and no general method for estimating the coefficient for particular dimensions was presented. Henderson (1966, §7.5) described how one could in principle solve the backwater problem by using Escande's approach. He went on to say that a more realistic approach would be to use momentum between sections upstream and downstream of the obstacle, and that the difference between them could be expressed in terms of drag force on the pier. He considered this to be just a "useful theoretical exercise" in view of the absence of detailed knowledge of the drag coefficient. It will be seen below that the problem of determining that coefficient is even more complicated than he suggested.

Ranga Raju, Rana, Asawa, & Pillai (1983) applied the momentum approach to the problem of a rectangular channel with vertical cylinders as obstacles. They obtained a quadratic approximation for the backwater. To test this they conducted experiments in a rectangular channel with vertical cylinders. Good agreement was found between the experiments and their explicit solution.

Montes (1998) described the empirical expressions of Rehbock and Yarnell and noted that it was not difficult to develop a more rational approach. He used the momentum principle and the drag force on the pier for a rectangular channel and pier cross-sections. A cubic equation, similar to Ranga Raju *et al.*, was linearised for small backwater, which gave an explicit solution. Montes also noted that there is, in general, the phenomenon of wave drag to be considered, where some of the loss of momentum at the obstacle appears as a train of "ship" waves which also possess momentum.

Charbeneau & Holley (2001) brought the problem rather more up-to-date. They conducted a large-scale laboratory investigation and presented a detailed momentum analysis that included friction both in the channel and on the obstacle, as well as gravitational slope effects, although both were subsequently neglected. Two series of experiments were performed on vertical cylinders – the first directly measured the resistance coefficient of several piers while the second series examined the water level variation. A number of results were presented.

Fenton (2003) used momentum conservation to develop an approximate approach similar to Montes but for general problems of arbitrary bodies in arbitrary cross-sections. The momentum equation was linearised in terms of the resistance term, giving a first-order explicit solution for the change in surface level due to the resistance force exerted, which helped reveal the factors governing the problem. However, no results for drag coefficient were given, and there still seems to be room for considerable progress in developing methods to solve the problem.

3 FORCE BETWEEN FLOW AND OBSTACLE

3.1 *Physical model*

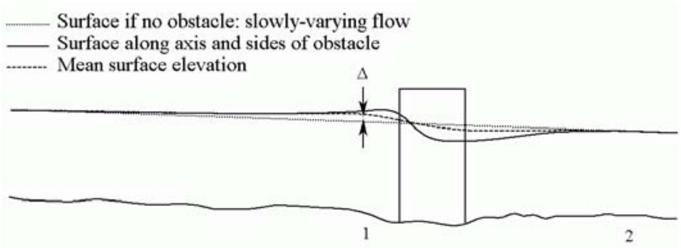

Figure 2. Longitudinal section - the physical problem, showing backwater at and upstream of an obstacle

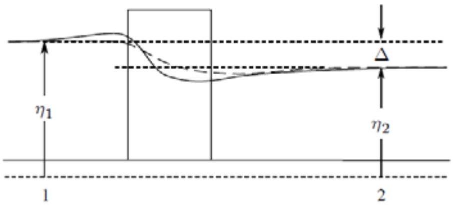

Figure 3. Longitudinal section – the idealised problem, uniform horizontal channel with no friction

Figure 2 shows a typical surface-piercing obstacle in sub-critical flow. Local dynamical effects cause the water surface in the vicinity of the obstacle to be disturbed. Just upstream of the pier there is a local mound due to Bernoulli stagnation effects, as already seen in Figure 1, while around the sides of the pier the water level drops where the velocity is large. The local effect has been thought to be of no great importance for the hydraulics of the problem, as it quickly decays away from the pier. More important has been thought to be the change of the *mean* water level across the whole channel, shown by the dashed line. Just behind the obstacle it has risen to some excess ("backwater") height Δ, which slowly disappears upstream, but which effect can extend for a long way. This is important for flood studies, and the economic consequences can be important. It will be seen below that the flow details around the structure are indeed important to the overall problem.

Figure 3 shows the physical idealisation adopted to model the problem, with the usual simplification in open channel hydraulics for local rapidly-varying solutions using the approximation of flow in a horizontal prismatic frictionless channel. The results can be applied to unsteady and non-uniform flows. The origin for surface level height η shown in Figure 3 is arbitrary, for here channels of arbitrary cross-section are considered, and the bed of a natural stream might be poorly defined such that the origin cannot be placed there. It is necessary to introduce two sections where momentum will be evaluated. Section 1 is a short distance upstream of the obstacle, just behind the local stagnation mound, where the free surface is sensibly horizontal across the channel. Section 2 is further downstream, where effects of the obstacle in the form of a region of velocity defect in the cross section have already dissipated.

3.2 *Drag force*

The flow near the obstacle is so complicated that for practical problems at the time of writing it is not feasible to calculate accurately the force on it from fluid-mechanical principles. The horizontal force T on the obstacle could be expressed using the empirical expression from conventional fluid mechanics for the drag on a bluff body (see for example, §7.6 of: White 2003): $T = \frac{1}{2}\rho C_D u^2 a$. where ρ is fluid density, C_D is drag coefficient, u is the local horizontal fluid velocity impinging on the object, and a is the projected area of the object normal to the flow direction. For numerical values of C_D, reference can be made to most introductory fluid mechanics books; however they are almost all for the case of fully-immersed bodies with no free-surface effects. The present problem of a finite body in a shear flow with a free surface is more complicated. Here, the incident flow velocity can vary markedly over the body, and depends on the body's position in the flow, whether near the bottom, the surface, or extending across the whole flow. The obstacle may be a compound body with different components, such as the piers and roadway of a bridge. Each element may be subject to a different velocity. For example, the appropriate velocity for a vertical pier extending over the whole depth would be the mean velocity in the flow; that for a bridge deck would be the surface velocity, rather greater than the mean; while for a vane or block near the bed it would be rather smaller.

Free surface effects can make the problem rather more complicated. If the body penetrates or approaches the surface, the water level on the immediate upstream side of the body will be greater than on the downstream side, and there is a net difference in pressure force, greater than that for a submerged body.

All this means that conventional drag coefficients obtained for submerged objects in a uniform flow cannot be used alone. Such coefficients C_{D} can be used for the viscous and form drag, in association with the mean velocity Q/A over the channel, where Q is discharge and the channel cross-sectional area A. The viscous and form drag force T_{D} on the obstacle is written as

$$T_{\mathrm{D}} = \frac{1}{2}\rho\gamma C_{\mathrm{D}} a \frac{Q^2}{A^2}, \text{ where } \gamma = \frac{A}{Q^2}\int_a u^2 da , \qquad (1)$$

a dimensionless coefficient γ for the mean square of the velocity over the obstacle.

3.3 *Free surface resistance*

As already indicated by Figure 2 and Figure 3, part of the resistance of the obstacle to a flow is determined by the shape of the free surface immediately around the obstacle. The disturbance to the free surface means that ultimately a system of waves is produced and in the context of ships its momentum is called wave drag. Here, the cause of those waves is used in the nomenclature: "free surface resistance".

The contributions to the force on the flow from the front and rear of a body tend to cancel each other. The extent of this depends on the difference between water levels fore and aft, which depends partly on the nature of the free surface and the length of the waves around the body. That length is an increasing function of the velocity of the flow. Considering a slow flow, when the wavelengths are short, there may be several wave crests and troughs around a pier, although they might be barely perceptible. For a faster flow both the height and wavelength are larger. Hence, the water level at the rear of the body will be an oscillatory function of velocity, and so is the net force. This is well-known for the resistance of ships (*e.g.* Hoerner 1958, §11, p12; Tupper 2004, p150), which are designed to minimise such forces. For bluff objects such as bridge piers, however, this should be a rather more marked effect. The magnitude of the forces will increase like the square of velocity, oscillating in sign, but the resistance coefficient, proportional to the ratio of the force to the square of the velocity, should show finite oscillations as the flow velocity and wave length increases.

A simple hydraulic model is now attempted here. It is assumed that the force due to surface variation around an object is the difference between the hydrostatic force on the upstream face of a surface-piercing object and the hydrostatic force on the downstream face. The hydrostatic force on an arbitrary surface is $\rho g a \bar{h}$, where a is the projected area of the surface perpendicular to the force direction, in a streamwise direction here, and $\bar{h}$ is the depth of the centroid of the projected area (see almost any book on elementary hydraulics). The force on the upstream (US) projection of an obstacle is then $\rho g \left(a\bar{h}\right)_{\mathrm{US}}$. A more complete analysis would allow for non-hydrostatic flow effects. The upstream directed force on the downstream (DS) projection of area, is $\rho g \left(a\bar{h}\right)_{\mathrm{DS}}$, hence the net force on the obstacle due to free surface (FS) effects is

$$T_{\mathrm{FS}} = \rho g \left(a\bar{h}\right)_{\mathrm{US}} - \rho g \left(a\bar{h}\right)_{\mathrm{DS}} . \qquad (2)$$

For the usual case of small backwater relative to the depth, this can be written as the first term of a Taylor series which can be shown to give

$$T_{\mathrm{FS}} = \rho g a \left(\eta_{\mathrm{US}} - \eta_{\mathrm{DS}}\right). \qquad (3)$$

The water level difference between the upstream and downstream ends of the obstacle thus plays a crucial role. The water level upstream can be assumed to be $\eta_{\mathrm{US}} \approx \eta_2 + \Delta$. However, the surface level immediately downstream of the obstacle is not in general equal to η_2, the mean water surface further downstream, nevertheless it plays a crucial role in determining the force on the obstacle. A simple approximation is made, which will be found to provide an adequate model. A dimensionless free surface resistance coefficient C_{S} is introduced, and the assumption is made that

$$\eta_{\mathrm{US}} - \eta_{\mathrm{DS}} = C_{\mathrm{S}}\Delta , \qquad (4)$$

where C_{S} is a function of the dimensions and hydraulic parameters. The assumption is now made that the water surface around the pier is in the form of a cosine wave such that the coefficient is given by

$$C_{\mathrm{S}} = c_{\mathrm{s}}\left(1 - \cos kl\right). \qquad (5)$$

where $k = 2\pi/\lambda$, in which λ is wavelength, and where l is an effective obstacle streamwise length.

For small waves on a slow flow one can imagine short periodic waves around the side of an obstacle. For more dramatic flows, such as that shown in Figure 2 a cosine wave is a gross approximation. However, the coefficient c_s will be of the order of 1. The problem is now to determine k. If a dimensional analysis were performed for small short two-dimensional waves of wavenumber k on a flow of speed U, the relationship would be established that $\Pi = kU^2 / g =$ constant. Hence we write $kl = \kappa gl / U^2$, where κ is a coefficient, also roughly equal to 1. From equation (5) the approximate form for the function C_S becomes

$$C_S = c_s\left(1-\cos\frac{\kappa gl}{U^2}\right) = c_s\left(1-\cos\frac{\kappa}{F^2}\frac{l}{h}\right), \tag{6}$$

giving an expression for C_S as a function of the geometric and flow quantities of l/h, the ratio of streamwise dimension to water depth, and the square of the Froude number F^2. The equation shows, that for small F, the argument of the cosine function becomes large and small variations in F will cause the cosine function and hence C_S to oscillate between values of 0 when a crest is also near the downstream face, and a local maximum of $2c_s$, when a trough is there.

Combining equation (1) for the fluid dynamic drag, and equations (3) and (4) for the free surface resistance, gives an expression for the total resistance force of an obstacle on a flow

$$T = \frac{1}{2}\rho\gamma C_D a\frac{Q^2}{A^2} + \rho g a C_S \Delta, \tag{7}$$

where C_D is a conventional drag coefficient, combining the effects of viscous and form drag, such that values could be taken from tables for totally-immersed bodies, while C_S is a function of flow and geometry, as given by equation (6), allowing for the effects of free surface deformation around the body, and which is yet to be investigated.

4 APPLICATION OF MOMENTUM THEOREM

The principle of conservation of momentum between sections 1 and 2 is applied to give an equation for the unknown water surface level at 1. The momentum flux M at any section of gradually-varying open channel flow is, assuming a hydrostatic pressure distribution,

$$M = \rho\left(gA\bar{h} + \overline{\int_A u^2\,dA}\right), \tag{8}$$

where g is gravitational acceleration, A is cross-sectional area of the channel, $\bar{h}$ is the depth of the centroid of the section below the surface (such that $A\bar{h}$ is the first moment of area of the section about a transverse axis at the water level), and Q is the discharge. The term $\overline{\int_A u^2\,dA}$ is the time mean value of the integral over the section of the square of the velocity. This term is approximated by $\beta Q^2/A$, where β is a Boussinesq coefficient allowing for non-uniformity of velocity over the section and for effects of turbulence, such that it is larger than traditionally thought (Fenton, 2005). Hence

$$M = \rho\left(gA\bar{h} + \beta\frac{Q^2}{A}\right). \tag{9}$$

The conservation of momentum theorem states that $M_1 - M_2 - T = 0$, in terms of the force T of the obstacle on the fluid and the difference in the momentum flux between the section upstream of the obstacle at station 1 and one at 2 further downstream. Substituting the expression for T from equation (7) at section 1, and the expression (9) for M at 1 and 2, and dividing by ρ:

$$\left(gA\bar{h} + \beta\frac{Q^2}{A}\right)_1 - \left(gA\bar{h} + \beta\frac{Q^2}{A}\right)_2 - \frac{1}{2}\gamma C_D a_1\frac{Q^2}{A_1^2} - g a_1 C_S\left(\eta_1 - \eta_2\right) = 0. \tag{10}$$

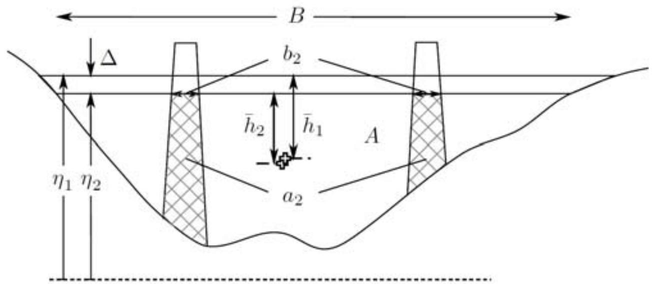

Figure 4. Transverse section showing physical dimensions corresponding to water levels at 1 and 2

The geometrical quantities are as shown in Figure 4. For sub-critical flow and known downstream conditions at point 2, and where both A_1 and $\bar{h}_1$ are known functions of η_1, equation (10) is a transcendental equation for η_1 that could be solved numerically for a given flow and channel cross-section. For super-critical flow, where the flow depth increases after the obstacle, one would calculate η_2 from knowledge of *upstream* conditions at 1.

Numerical solution of equation (10) is not difficult, and any root-finding method could be used, such as trial-and-error, bisection, or the secant method. Newton's method is a quickly convergent iterative method. It will be found here, however, that an approximate procedure provides more insight.

5 APPROXIMATE EXPLICIT SOLUTIONS

Whereas the above procedure is of considerable generality, it has used approximations and relies on knowledge of numerical values of coefficients that are usually not available. It is in keeping with the nature of the present work to approximate that equation, to give an explicit approximation for the backwater Δ, which provides more insight into the problem.

For sub-critical flow, the momentum equation (10) is now re-written with all quantities at section 1 expressed as series in terms of the known downstream conditions at section 2, which are as shown in Figure 4. The change of surface elevation Δ is considered to be relatively small, so that all geometric properties with the water at elevation η_1, can be obtained in terms of the properties at the downstream elevation η_2 and Δ. Substituting into the momentum equation (10) and using power series operations gives a linear equation for the dimensionless backwater $\Delta/\left(A_2/B_2\right)$, where A_2/B_2 is the mean depth. At the first level of approximation, with just a linear term in Δ, performing similar operations in the supercritical case for conditions at 1 in terms of values at 2, the same expression is obtained, with subscripts reversed. Hence, generalising by dropping all subscripts, the explicit solution for dimensionless backwater $\Delta/(A/B)$ is obtained:

$$\frac{\Delta}{A/B}=\frac{1}{2}\gamma C_{\mathrm{D}}\sigma\frac{F^2}{1-\beta F^2-\sigma C_{\mathrm{S}}}, \tag{11}$$

which is in terms of the velocity distribution parameter γ, the drag coefficient C_{D}, the blockage ratio $\sigma=a/A$ (the fraction of area of obstacle to stream area), the Froude number $F^2=Q^2B/gA^3$, the Boussinesq momentum coefficient β, and the free surface resistance coefficient C_{S}.

This provides some insight into the behaviour of the channel. For subcritical Froude numbers, $\beta F^2+\sigma C_{\mathrm{S}}<1$ the denominator will be positive and so will be the backwater Δ. On the other hand, if the flow is super-critical, $\beta F^2+\sigma C_{\mathrm{S}}>1$, Δ is negative and the surface rises. Clearly, if the flow is near critical, $\beta F^2+\sigma C_{\mathrm{S}}\approx 1$, the change in depth will be large, when the theory, and all hydrostatic theory, breaks down.

Equation (11) has an important implication, that an obstacle causes a change in surface elevation, which means that computational solutions should introduce a computational point at the obstacle, so that continuity of functions on either side can be maintained. This also means that the obstacle, even if it does not seem to change the overall flow, does strictly form a control on a channel, where the depth upstream depends on the flow, and as such is an hydraulic structure.

6 EXPERIMENTAL RESULTS

Earlier experimenters such as Yarnell and Rehbock devised their own forms of drag coefficient, which make comparison rather difficult. Charbeneau & Holley (2001) did present results in terms of the drag coefficient as it is conventionally known, but which included the effects of the free surface deformation, such that the results were in terms of what will be termed the *apparent* drag coefficient $C_{\mathrm{D}}^{/}$ here. In this work, all results are presented in terms of that quantity multiplied by the square of the Froude number $C_{\mathrm{D}}^{/}F^2$, as those terms always occur together in the physical equations, and it is proportional to the force on an obstacle and to the backwater caused by it. Presenting results in terms of just $C_{\mathrm{D}}^{/}$ causes them to look unnecessarily variable for small Froude number, when all measured quantities are small and errors are relatively large.

Figure 5 shows typical experimental results from Charbeneau & Holley (2001) for $C_{\mathrm{D}}^{/}F^2$ plotted as a function of F^2. It can be seen that there is apparently a scatter of results across a well-defined band. Here it is claimed that that is due to the effects of the free surface drag, which shows the complicated variation with Froude number as given in equations (6) and (7). The adequacy of this model can be tested from equations (1), (6), (7) and (11), and by making linearising approximations it is possible to generate equations that can be solved in a least-squares sense to give the (assumed constant) values of the fluid dynamic drag coefficient C_{D}, and the quantities c_s and κ as used in equation (6) for a particular type of pier. This makes the results depend on the dimensionless diameter of the pier D/h, where in this case of a circular cylinder both the cross-stream and downstream dimensions of the obstacle are the diameter D.

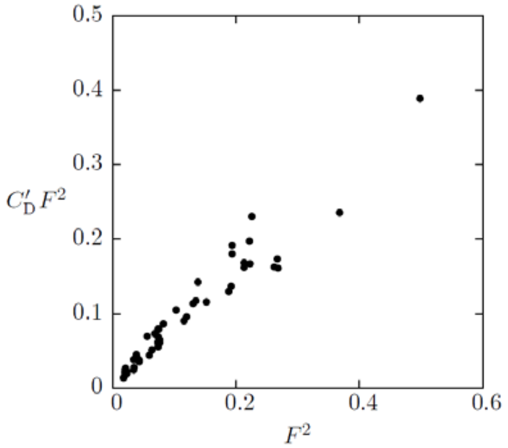

Figure 5. Experimental results from Charbeneau & Holley for a smooth circular cylinder, with $C_D' F^2$ plotted against F^2 showing apparent scatter of results.

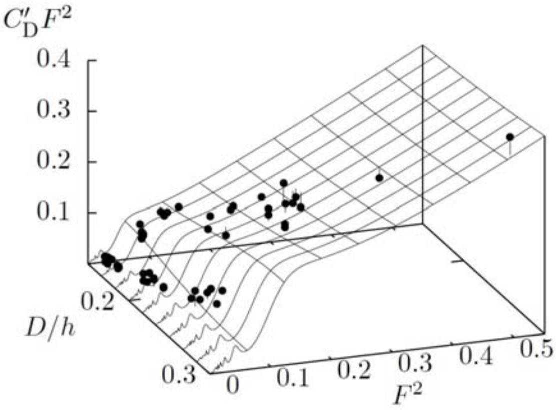

Figure 6. The results of Figure 5 re-plotted with aspect ratio D/h as parameter, compared with present theory shown by the surface. The short vertical lines, where visible, show the difference between theory & experiment.

Results are as shown in Figure 6, where the view parallel to the D/h axis is the same as Figure 5. The values fitted to the experimental points and used to plot the figure were $C_D = 0.61$, $c_s = 4.1$, and $\kappa = 1.3$. Experimental points are still shown by black circles, while the theory is shown by the surface, and short vertical lines show where the points should lie on the surface if predicted by theory. Almost all such lines are obscured by the points themselves, and it can be seen that the theory gives a good approximation to the variation of the force (and hence backwater) with F^2 and with D/h. The figure also shows that for small F, the argument of the cosine function becomes large and small variations in F will cause the cosine function and hence C_S to oscillate between values of 0 (when a crest is also near the downstream face) and $2c_s$, when a trough is there, explaining the banded nature of the results on Figure 5.

The rapidly-oscillating variation of drag, and hence backwater, with Froude number has implications for practice that in any particular situation it is difficult to calculate the actual amount of backwater with any accuracy. This may have important consequences for simulation software. For design or flood hazard estimation, however, the worst case $C_S = 2c_s$ is clearly the case to use.

7 REDUCTION OF RESISTANCE

In many cases the backwater generated by an obstacle is not a problem. Where it is, when upstream flooding is cause for concern, measures to reduce the drag of the obstacle might be considered. In general, adding appurtenances to in-stream structures might create more problems, such as reducing navigation width, or capturing more floating debris. However, some of the following could be considered:

Bulbous bow (*e.g.* Figure 7a): a bulb at the bow of a ship can reduce drag considerably by reducing the waves generated at the bow. The effect depends on the degree of submergence. However, that which might function for piers, and would not be possible for a ship, would be a floating bulb or streamlined hull-shaped object, which could slide up and down a pier, maintaining its position relative to the water level, or it could be fixed just below the deck.

Surface plate (*e.g.* Figure 7b): Hoerner (1958, p10-16) mentioned the possibility of "ventilation" of vertical surface-piercing objects, particularly at very high speeds, whereby the water separates from the sides of the object such that air penetrates deep down into the cavity behind the obstacle. Hoerner wrote: "To prevent air entrainment or ‘leakage’ along a surface-piercing strut, a fence-like ‘subsurface plate’ can be used (as for example in outboard motor shafts). Adding such a plate closely below the water surface, ventilation is completely eliminated". In structures in streams, almost always the Froude number would be much less than those considered by Hoerner. However, the possibility of suppressing surface disturbances in the vicinity of the pier might be further investigated. A possibility might then be, to attach a horizontal plate to a pier just below the deck of a bridge, so as to reduce drag before water reaches the level of the deck. Another might be to have a plate floating on the water that could slide up and down the pier.

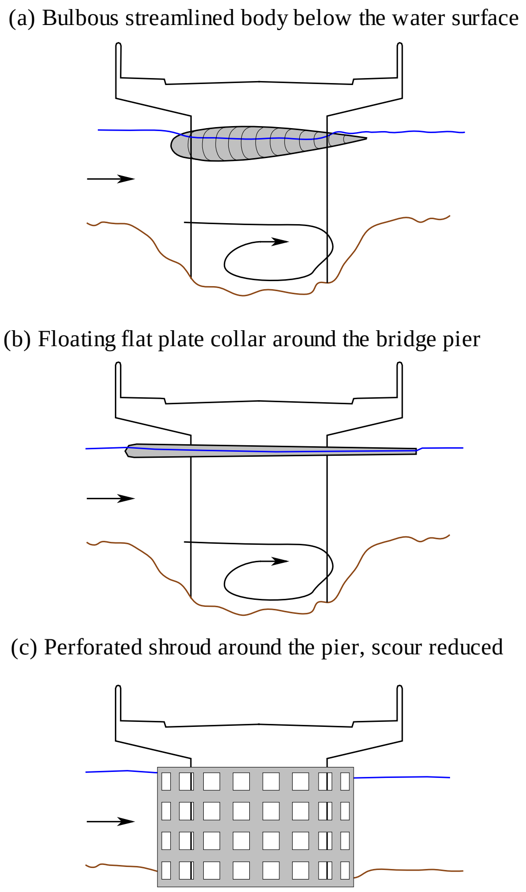

Figure 7. Possible structures to minimise free surface resistance

Perforated shroud (*e.g.* Figure 7c): on the Ekofisk oil production platform in the North Sea, such a structure was used to reduce wave forces. It is unlikely that such a structure, with an abrupt un-streamlined nature, could reduce drag in a steady stream flow. However one possible effect might be to break up the horseshoe vortex that forms ahead of piers and which can lead to large scour. Figure 7c has been drawn, possibly optimistically, with little scour, while in Figure 7a&b the conventional scour hole, with its resulting separated flow and enhanced drag force on the exposed lower part of the pier, has been shown. It is possible that a hybrid structure, combining both a flat plate collar and shroud, might be best.

8 CONCLUSIONS

It has been shown that a momentum and drag force approach can describe the change of water level due to an obstacle in a stream such as a bridge pier. For such obstacles which pierce the surface, it is shown that the additional force due to deformation of the free surface leads to complicated behaviour. The variation of force, and hence backwater, shows a highly-oscillatory dependence on the speed of the flow. As the precise nature of this is difficult to establish with any great accuracy, it means that the numerical simulation of streams where such effects occur, which probably means most, is liable to inaccuracy. While the simulation process is very sensitive, the design and hazard evaluation process is not so, and remains fairly robust.

It is suggested that the details of such backwater problems are too complicated to be solved by conventional hydraulics or even by computational fluid mechanics in this age. It seems that the best solution is, in apparent opposition to modern tendencies, to use experiments to solve practical problems.

Finally, some possible structures are suggested for the reduction of the force on bridge piers and resultant upstream flooding.

9 REFERENCES

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