Design of outfalls in tidal waters

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In proposing the use of the finite element method for the prediction of dispersion in tidal waters, the Authors do not fully justify the use of a two-dimensional analysis, nor do they give any indication of the effect of such an assumption on the accuracy of the results. The problem of jet dispersal in still waters could be analysed with some degree of certainty in two dimensions but with the introduction of tidal movements having variations of velocity with depth this is no longer the case. The example quoted makes no provision for river flows down the estuary although no doubt this could be incorporated in the model if required. Have the Authors considered the implications of the movement upstream of a salt water wedge underlying a downstream flow of fresh water?

53. The Paper sets out two examples which have been analysed; it appears that the results given for the Haringvliet Estuary are solely those derived from the mathematical model. Have the Authors any field observations to confirm the accuracy of these results?

54. The scale of the characteristic dispersion plume from an outfall at depths of 9 or 10 metres is so small in relation to the area of the tidal model that it is difficult to see how the large elements used would enable any significant contribution to be made to the study of dispersion. It would appear that a much smaller net would be necessary, at least in the immediate vicinity of the outfall.

55. The Authors are concerned primarily with the development of a method of analysis which will lead to a better appreciation of the dispersion of effluent discharged through a sea outfall. However, they state that research is necessary into the problem of the continuing pollution of the seas by effluents. Although they quote the range of concentrations of certain pesticides which are lethal to fish, the Authors make no mention of the known accumulation of some of these pesticides with time in the food chain, which leads to increasingly concentrated forms. Thus the presence of quantities less than the quoted lethal dosage in the effluent is no guarantee that permanent damage will not be done to fish or other forms of aquatic life.

56. Estuaries and shorelines are invariably spawning grounds for fish and it is these same estuaries and shorelines which are steadily becoming more and more polluted. The oxygen content of parts of the Baltic is already reported to be virtually exhausted and the closure of a number of Italian holiday resorts in 1971 because of the fear of disease from polluted seas and beaches could well be a preview of what may recur on a larger scale if effluent discharge into the Mediterranean continues unabated.

57. However much or little effluent is discharged to the sea, it will affect the ecological balance. It is only now that, due to the accumulation of the past and the ever increasing rates of present discharges to the oceans, noticeable effects are becom-

ing apparent. The warnings are clear and the continuing use of the sea and coastal waters as dumping grounds for effluents must stop. The Authors should give priority of place in future research to methods of treatment to permit alternative methods of disposal or re-use.

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Unfortunately the Authors have not mentioned much of the relevant work carried out in recent years. For example Fan¹³ and Fan and Brooks¹⁴ have extended the basic theory by considering movement and stratification of the receiving water. Information is now available on the effects of interference between jets^{15,16} in a multiport system and on the effect of interaction of a single jet with the sea floor.¹⁷ Studies have also been made of slot jets,¹⁸ which relate to multiport systems, and of methods of increasing dilution by use of appurtenances.¹⁹ The possibility of physical models received little attention in the Authors' treatment although methods of selection scales for both surface and submerged outfalls are now available.²⁰⁻²²

59. Unwanted heat is an important pollutant which may be introduced to tidal estuaries from cooling water outfalls of thermal or nuclear power stations. The large flows involved mean that thermal effluents are normally discharged through surface outfalls, and considerable sums have been spent to ensure that these are suitably located.²³ An appreciation of the information available on this aspect²⁴⁻²⁸ would lead to greater understanding.

60. Specific points must be considered. The Authors have not explained the note ' S_m according to present theory' which is presented on Fig. 1. In fact this figure represents a comparison between the work of Rawn *et al.*⁴ and that of Abraham.⁵ The figure appeared first in a report by Abraham⁵ and later in a paper of his.²⁷

61. In § 12 the Authors mention optimum dilution. This is a misnomer because the points indicated are, in fact, points of minimum dilution. At each stated value of Y_0/D (Table 2 or Table 3) any increase or decrease in F will increase the dilution above the quoted value. This is best illustrated by Figs 1 and 2 which show that, as F increases, the dilution (at any constant value of Y_0/D) first decreases to a minimum and then increases. Reasons for this phenomenon have been given elsewhere.²⁸

62. In § 13 the Authors appear to indicate that because the dilutions quoted in Tables 2 and 3 fall short of the required dilution, recourse must be made to dispersion. This is misleading in so far as they imply that these dilutions represent optimum design criteria. A comparison of Table 3 and Fig. 2 shows that this is in error. For example the Authors quote $S_0=40$ for $Y_0/D=120$ and $F=30$. This is true. However, if F is reduced to 1.8 while Y_0/D is held constant, S_0 is increased to 120. Thus reduction in F by 15 times increases dilution three times and better dilution is obtained at the same site with a decrease of head losses and associated costs. Any design must take into account the variation in discharge and hence be acceptable over a range of F values.

63. The Authors state that their diagrams relate to quiescent, homogeneous conditions but they do not say that turbulence in the ambient fluid may significantly affect dilution. For example, it is known²⁹ that, at a site where Fig. 1 indicated dilutions of the order of 26-28, the actual dilution measured after installation of the outfall exceeded 1000. This was an extreme case of an outfall in a relatively narrow tidal channel with a 5 knot current but data for slower currents are also available.³⁰

64. In § 16 the Authors state that X_0 is the radius of the plume near the discharge point, but in Fig. 3 X_0 is given as the radius at the surface. Fig. 3 also shows the surface diameter as $L/4$, a value reiterated in § 15. Could the Authors clarify this point? Calculation of secondary dilution requires a value for X_0 which in turn depends on L , the length of the trajectory. How do the Authors propose that L should be calculated as it is dependent on Y_0/D and F ?

65. The term 'plume' or 'buoyant plume' is normally reserved for discharge without momentum. This is a theoretical concept but is closely approximated by the

discharge of heated air or gases from chimneys³¹ and cooling towers.³² The Authors refer throughout to what has been more appropriately described as a 'buoyant jet', i.e. a jet discharged with both buoyancy and momentum.

66. The Authors' 'Froude number' F is more appropriately termed a 'densimetric' Froude number or 'internal' Froude number. These terms have become accepted as standard terminology and serve to indicate that buoyancy effects are present.

Mr P. Ackers, Binnie & Partners

The Paper makes little reference to the spread of a surface layer of reduced density in situations where stratification may persist and the Authors do not give details of the considerable effect of tidal currents on the buoyant plume itself.

68. Much research on the former topic has been stimulated by concern for thermal pollution arising from power station cooling systems, but the spread of a surface field in calm waters is the same irrespective of the source of the density deficiency. Barr²² has described the basic phenomena of density spread, and Sharp²⁰ considered the two-dimensional spread of a surface field. Fig. 3 shows an embryo surface field but the Authors indicate eddy diffusion as the removal agency. However, in a calm fluid the density gradient at the interface may prevent mixing between the underlying sea and the field of diluted sewage. Density spread is then the removal agency.

69. Typically outfalls around the British coastline discharge into a tidal stream. Unless tidal velocities are very low, the ambient current modifies the dilution in the plume to such an extent that it could be misleading to ignore it in design. Although relatively little has been published on this subject, Pearce³³ and Abraham³⁴ have suggested modifications to the basic theory of buoyant plumes, the former deducing that in the case of a vertical nozzle the additional dilution due to the current is proportional to the square of the ratio of ambient velocity to nozzle velocity. Perhaps the most relevant information for design is that obtained by Agg and Wakeford³⁵ through field surveys of the performance of many existing British outfalls. Their comprehensive measurements enabled them to compare the static water dilution

Fig. 11. Additional dilution at the surface due to ambient current (after Agg and Wakeford³⁵)

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factor according to Abraham with the actual dilution. The additional dilution factor was then related to the velocity ratio.

70. Dimensional analysis suggests the following functional relationship for turbulent conditions

$$S_0 = f \left\{ \frac{V}{\sqrt{g'D}}; \frac{Y_0}{D}; \frac{V_a}{V} \right\} \quad \dots \dots \dots (9)$$

The nomenclature is the Authors, except that $g' = g\Delta s/s$ and V_a is the ambient velocity. Y_0 is strictly the vertical distance from the diffuser outlet to the underside of surface field, rather than the depth of water at the outfall.

71. Agg and Wakeford³⁵ took the more restrictive function

$$S_0 = f_0 \left\{ \frac{V}{\sqrt{g'D}}; \frac{Y_0}{D} \right\} \times f_a \left\{ \frac{V_a}{V} \right\} \quad \dots \dots \dots (10)$$

where f_0 is the function for static water. The additional factor represented by a function of velocity ratio is shown in Fig. 11. However, I have replaced the simple power law suggested by Agg and Wakeford (the broken line) by the S curve. This links in to a minimum value of about 1.5 that applies when $V_a/V < 0.1$ and a maximum value of 20 as V_a/V approaches 5. The minimum value probably arises because the averaged observed dilutions at the boil plotted in the figure are less than the centre line values, but the only reason for suggesting an upper limit is that the observations do not warrant extrapolation beyond an added dilution of 20.

72. Unfortunately the combination of the Abraham type of plot (Fig. 2) with the function for ambient velocity makes the designer's task more difficult. The usual conclusion for static water that dilution at the surface is maximized by reducing the diffuser diameters to a minimum consistent with acceptable head loss is no longer correct; the gain from an increased velocity ratio will in most cases exceed the loss due to a lower densimetric Froude number.

73. The usefulness of Fig. 2 for static conditions can be enhanced by adding grids for the parameters $4Q/\pi n\sqrt{g'Y_0^{2.5}}$ and h_e/Y_0 where Q is the total discharge, n is the number of outlets and h_e is the head loss at exit. This has been done on Fig. 12. The dimensionless discharge expressed thus does not vary with nozzle diameter, so that for a given discharge per outlet in a given depth of water reducing the nozzle diameter moves the design point to the right and upwards along one of the dimensionless discharge lines. The calm water dilution factor is relatively insensitive to a tenfold increase in F arising from a reduction factor in D of 2.5.

74. The head loss lines show the exit loss in terms of water depth for an assumed value of $\Delta s/s$ of 0.026. The Authors suggest an exit loss of not more than about 0.5 m for economy in operation, so that $(h_e/Y_0)_{\max}$ would be 0.1 in 5 m of water, 0.05 in 10 m of water and 0.02 in 20 m of water. In practice, therefore, the optimum design points for calm water shown in Fig. 2 are unattainable unless there is unusually high velocity at the outlet nozzles. Bearing in mind that an outfall cannot operate at $F < 1$, the practical part of the Abraham plot lies in the region ABCDE on Fig. 12.

75. The dimensionless discharge grid on Fig. 12 shows that to increase surface dilution to a prescribed value it is necessary to reduce the value of $4Q/\pi n\sqrt{g'Y_0^{2.5}}$. Nozzle diameter has a secondary effect. Hence it is by increasing n , and especially Y_0 which occurs to the power of 2.5, that the required calm water dilution factor can be achieved. Increasing Y_0 is usually expensive, because it means extending the outfall to deeper water, but may be unavoidable. A ten outlet diffuser can yield the same surface dilution as a single outfall in water 2.5 times as deep ($2.5^{2.5} = 10$) when operating at the same exit head loss.

76. The grid of dimensionless discharge lines added to the Abraham plot illustrates a principle that governs the convenience in practice of any design charts; the arguments used should separate the engineering variables. Indeed the Abraham plot is inconvenient because both Y_0/D and F are dependent on the nozzle diameter. If the calm water diagram of Fig. 12 were to be combined with the velocity ratio function of

Fig. 12. Initial dilution in a calm fluid, showing dimensionless discharge and head loss

Fig. 11, the engineering variables could be separated by re-writing equation (9) in the form

$$S_0 = f \left\{ \frac{4Q}{\pi n \sqrt{g' Y_0^{2.5}}}; \frac{D}{Y_0}; \frac{V_a}{\sqrt{(g' Y_0)}} \right\} \quad (11)$$

Basically this requires a series of charts for a sequence of values of $V_a/\sqrt{(g' Y_0)}$, the ambient flow condition in the form of a Froude number. Each chart would then have a family of curves of equal surface dilution S_0 , with dimensionless discharge and outlet diameter as arguments. Thus a direct solution for D in terms of S_0 (or vice versa) would be possible for a given outfall location and design discharge Q/n . Such a set of charts has been presented by Agg and White.³⁶

77. An alternative approach is possible using Cederwall's equations³⁷ for the dilution in calm waters

$$S_0 = 0.54F \left(\frac{Y_0}{DF} \right)^{7/18} \text{ for } \frac{Y_0}{D} < 0.89F \quad (12)$$

$$S_0 = 0.54F \left(0.38 \frac{Y_0}{DF} + 0.68 \right)^{5/3} \text{ for } \frac{Y_0}{D} > 0.89F \quad (13)$$

These equations agree closely with Abraham's results and can be represented as a two-variable relationship, i.e. a curve rather than a family of curves. Only equation (13) is relevant as the practical zone of Fig. 12 lies to the left of and above the line $Y_0/DF = 0.89$.

$$\frac{S_0 D}{Y_0} = 0.54 \left(\frac{FD}{Y_0} \right) \left\{ 0.38 \left(\frac{FD}{Y_0} \right)^{-1} + 0.68 \right\}^{5/3} \quad (14)$$

This can be combined with the empirical function of Fig. 11 in the form

$$\left(\frac{\alpha S_0}{F} \right) = f \left\{ \frac{V_a}{\sqrt{(g' Y_0)}}; \frac{1}{\sqrt{F}}; F \left(\frac{D}{Y_0} \right) \right\} \quad (15)$$

where α is the additional dilution taken from Fig. 11. Equation (15) is a family of curves on a single chart (Fig. 13). It remains inconvenient: a direct solution for D

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does not emerge as the variables cannot be separated in a three parameter plot. However, with F as a repeating variable, direct solutions are possible for any given densimetric Froude number at the nozzle, so that several alternative values of F of 1 or more may rapidly be compared. In practice there will be few cases where ambient currents are so low that increasing F by reducing nozzle diameter will enhance surface dilution. Two examples are compared in Table 4, the design points being marked on Fig. 13.

78. Having shown that in the presence of moderate tidal currents present evidence suggests that outfalls should be designed to operate at a low densimetric Froude number, the limitations on F must be considered. If the flow is pumped intermittent-

Fig. 13. Initial dilution taking account of ambient current : values of $\alpha S_0/F$

ly at a minimum rate, it would seem reasonable to design the outfall to operate with F between 1 and 2 at that discharge. In a gravity system, night-time flows might be so low that the nominal value of F inevitably drops below 1. This means that sea water would then enter through the lower section of the outlet, but it would seem unwise to design for $F < 1$ at dry weather flow. A further restriction arises with multiple outlets. In order to balance the flow between the outlets there will be a minimum head loss; this implies operating at higher values of densimetric Froude number. Moreover, there will be minimum velocity criteria to be met in the main outfall pipe and in the diffuser system. These are all factors that the designer must take into account.

79. It is not easy to follow the Authors' recommendations concerning multiple outlet diffusers (§§ 16–18). It seems logical to design each outlet separately for a discharge Q/n , and then to space the outlets so that there is little overlap when the individual plumes reach the surface. In that way the initial dilution is not diminished by mutual interference. On this basis, Rawn and Palmer³ suggested that, in calm water, diffuser outlets should be spaced at $L_0/4$, where L_0 is the plume length. Pearce³³ suggested $L_0/2.27$ on the basis of the expansion of the fringe of the plume. The plume trajectory lengthens when there is a current, and must also enlarge as extra dilution occurs. Thus $L_0 \gg Y_0$. In this case, a convenient rule of thumb for generous diffuser spacing might be $\sqrt{\alpha} Y_0/2$, but the effect of overlapping on predicted dilution at the surface should be small at half this spacing, i.e. $\sqrt{\alpha} Y_0/4$.

80. In § 16 the Authors give the equation $S = S_0 \sqrt{(X/X_0)}$, without deriving it. Pearce's derivation of the same equation is based on Rawn and Palmer's observation of radial surface velocity where the rising plume becomes an expanding surface field. He makes the assumption that the field thickness remains constant. It would be interesting to know whether this square root relationship for concentration had been confirmed by direct measurements, because it implies interfacial entrainment that is independent of stratification arising from a high density difference where initial dilution is low. The experiments by Sharp²⁰ are also relevant.

81. The Authors are rightly cautious about the success that can be achieved with present knowledge in predicting the ultimate dispersion of conservative pollutants. The main problem is to describe adequately in the system of equations the turbulent dispersion induced by tidal currents. There is already a vast literature about dispersion in coastal waters, but no consensus of opinion about the best equations to use.

Table 4. Comparison of alternative designs; discharge $0.5 \text{ m}^3/\text{s}$ through 12 outlets, $\Delta\rho/\rho = 0.026$; $D = (4Q/Fn\pi\sqrt{g'})^{0.4} = 0.406/F^{0.4} \text{ m}$

F $D, \text{ m}$		1.0 0.406	2.0 0.308	5.0 0.214	10.0 0.162	20.0 0.123
Example 1:	V_a					
	$\sqrt{(g' Y_0 F)}$	0.050	0.035	0.022	0.016	0.0112
$V_a = 0.10 \text{ m/s}$	FD/Y_0	0.027	0.041	0.071	0.108	0.164
$Y_0 = 15 \text{ m}$	$\alpha S_0/F$	150	44	17	9.4	5.0
	Dilution	150	88	85	94	100
Example 2:	V_a					
	$\sqrt{(g' Y_0 F)}$	0.133	0.094	0.059	0.042	0.030
$V_a = 0.30 \text{ m/s}$	FD/Y_0	0.020	0.031	0.053	0.081	0.123
$Y_0 = 20 \text{ m}$	$\alpha S_0/F$	880	300	44	15	7.7
	Dilution	880	600	220	150	154

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Joseph and Sendner³⁸ and Brooks,³⁹ among others, have made valuable contributions. Tests at the outfall site are most valuable because of this uncertain dispersion.

82. It is perhaps because of the problems facing the mathematical modeller that the Authors suggest physical modelling. However, the representation of turbulent dispersion in physical models provides no easy solution. Usually tidal models have to be vertically exaggerated, and some of their shortcomings in pollution studies have been described by Ackers and Jaffrey.⁴⁰ Crickmore⁴¹ has compared model with prototype.

83. Although I would support the Authors' plea for further research, I believe there is an even more urgent need. A great deal of past research has not yet been assimilated into design practice, and this is as much a fault of some of the researchers themselves—through not fully appreciating the need for clear explanations of how their work is to be applied—as of the engineers, who may be impatient with theoretical and laboratory studies that appear remote from real outfalls in their scale and complexity.

84. Each outfall proposal requires a field survey if performance is to be reliably predicted. Information about tidal currents, on residual drifts, on turbulent dispersion and on the long-term exchange of water with the open sea is necessary. Biological factors including bacterial die-off are no less important than hydraulic factors. The sea's function is not merely to absorb and disperse pollution; it also eliminates many pollutants such as coliforms and biochemical oxygen demand.

Dr W. R. White, Hydraulics Research Station

Initial dilution is an important aspect of any design because it can influence maximum surface concentrations of effluent, the formation of slicks and the degree of subsequent

Fig. 14. Initial dilution data after Abraham⁵ (horizontal jet into still water)

dispersion. Hence it seems appropriate to clarify some of the points made in §§ 10–14 and to mention the effects of an ambient current on initial dilution levels. The Authors suggest that the tidal current is considered to affect only secondary dilution, but research by Pearce³³ and Agg and Wakeford³⁵ has given the engineer information with which he can calculate initial dilution levels in moving water.

86. First, it is necessary to examine the working range in terms of densimetric Froude number. The highest value achievable coincides with the minimum acceptable diameter, normally taken as 0.2 m to avoid blockage, and the highest acceptable velocity, probably about 3 m/s to avoid undue head losses on exit. These two design figures yield a maximum densimetric Froude number of about 13 with a typical density difference ratio of 0.026. At the other end of the scale the minimum operational densimetric Froude number is between one and two because lower values would allow sea water to enter the outfall. Many of the figures quoted in Tables 2 and 3 fall outside this working range.

87. There are difficulties in interpreting Figs 1 and 2 from the point of view of a design engineer who knows the quantity of effluent to be discharged and has to determine the optimum diameter for the outfall. In view of the fact that

$$Fr = \frac{u_i}{\sqrt{\left(gD \frac{\Delta s}{s}\right)}} = \frac{4Q}{\pi D^{5/2} \sqrt{\left(g \frac{\Delta s}{s}\right)}} \quad \dots \quad (16)$$

where Fr is densimetric Froude number, Q is effluent discharge and u_i is velocity of ambient current, the diameter of the outfall occurs in both the ordinate and abscissa in Figs 1 and 2. This is inconvenient and masks the effect of diameter.

88. Agg and White³⁶ suggest an alternative presentation for dilution data in which, for the general case of a jet issuing into a moving ambient fluid, the traditional parameters are replaced by three new but related parameters

$$\frac{D}{y} = \left(\frac{y}{D}\right)^{-1} \quad \dots \quad (17)$$

$$\frac{u_a}{\left\{gy \frac{\Delta s}{s}\right\}^{1/2}} = \left(\frac{u_a}{u_i}\right) (Fr) \left(\frac{y}{D}\right)^{-1/2} \quad \dots \quad (18)$$

$$\frac{Q}{y^{5/2} \left\{g \frac{\Delta s}{s}\right\}^{1/2}} = \frac{\pi}{4} (Fr) \left(\frac{y}{D}\right)^{-5/2} \quad \dots \quad (19)$$

where u_a is velocity of ambient current and y is the height above outfall.

89. In stagnant water $u_a=0$ and the two parameters on the left-hand side of equations (17) and (19) suffice to describe the phenomenon. Fig. 14 shows the new plot for a horizontal jet issuing into still water based on the data of Abraham.⁶ Fig. 15 shows a typical moving water plot based on the field-work of Agg and Wakeford³⁵ in which the value of the third parameter (see equation (18)) is 0.15. Agg and White³⁶ prepared a series of charts to cover a range of ambient conditions and these can be used to determine initial dilutions in the range $0.1 < u_a/u_i < 2.0$ and $1 <$

Table 5

Effluent flow	0.05 m ³ /s
Depth of outfall	10 m
Acceleration due to gravity	9.81 m/s ²
Density difference ratio	0.026
Range of ambient currents	$0 < u_a < 0.24$ m/s ($\approx \frac{1}{2}$ knot)

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Fig. 15. Initial dilution data after Agg and Wakeford³⁵ (horizontal jet into moving water)

$Fr < 10$. The outfall diameter occurs only in the D/y parameter in Figs 14 and 15 and this simplifies its interpretation.

90. Figures 14 and 15 show contours for Fr as

$$Fr = \phi^{(1)} \left\{ \frac{D}{y}, \frac{Q}{y^{5/2} \left\{ g \frac{\Delta s}{s} \right\}^{1/2}} \right\} \dots \dots \dots (20)$$

and Fig. 15 shows contours for u_R/u_1 as

$$\frac{u_a}{u_j} = \phi^{(11)} \left\{ \frac{u_a}{\left\{ \frac{\Delta s}{gy} \right\}^{1/2}} \frac{D}{y}, \frac{1}{Fr} \right\} \cdot \cdot \cdot \cdot \cdot \quad (21)$$

91. To illustrate the effects of outfall diameter within the working range of densimetric Froude numbers, say $1.5 < Fr < 13$, it is useful to consider an example, the details of which are given in Table 5. The discharge parameter $Q/\{y^{5/2}(g\Delta s/s)^{1/2}\}$ (see equation (19)) is in this case 3.13×10^{-4} . The ambient current parameter $u_a/(gy\Delta s/s)^{1/2}$ (see equation (18)) is zero when $u_a=0$ and 0.15 when $u_a=0.25$ m/s. Hence Figs 14 and 15 apply at the two extremes of the range of the ambient current and the relationships between initial dilution at height y above outfall S_M and D/y can be read directly from the charts. Having assumed a depth of $y=10$ m, the relationships between diameter D and initial dilution S_M are obtained for the still and moving water cases.

92. The results for this example are plotted in Fig. 16, but before comparing the still and moving water cases it is necessary to modify either the Agg and Wakeford³⁵ or the Abraham⁵ initial dilution values to make them comparable. Abraham's

theoretical curves predict initial dilutions based on concentrations along the centre line of the the plume and Agg and Wakeford reported initial dilutions based on average concentrations across the plume. The difference is probably a factor of about 1.7 in S_M with the Agg and Wakeford results appearing more optimistic in terms of initial dilution.

93. Curve A in Fig. 16 is for the still water case based on Abraham's theoretical curves relating initial dilutions in terms of concentrations along the centre line of the plume. Curve B is for the moving water case (after Agg and Wakeford³⁵), based on average concentrations across the plume. Curve C is a revised version of curve B in which the S_M values of curve B have been divided by the factor 1.7. Curves A and C are thus comparable. The separate scale at the top of Fig. 16 indicates how Fr varies with D and the variation of the ratio u_a/u_j with D for curves B and C is also shown.

94. Conclusions to be drawn from Fig. 16 are that, within the working range of densimetric Froude numbers

- in still water there is a marginal advantage in terms of initial dilution levels in choosing the smallest practical diameter for the outfall and designing it to run at a densimetric Froude number of around 10; head losses may, of course, be unacceptable
- in an ambient current initial dilutions are significantly higher than the equivalent still water values; further, the initial dilutions obtained in moving water increase dramatically with increasing outfall diameter, largely because of the effect of the u_a/u_j ratio.

Fig. 16. Design example: initial dilution factors in terms of outfall diameter

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95. Thus it appears that an ambient current should certainly be taken into account when estimating initial dilution levels and that, where an ambient current exists, the designer should increase the outfall diameter to the practical limit and operate at low densimetric Froude numbers, say in the range $1 < Fr < 3$.

Mr A. R. Agg, Water Pollution Research Laboratory

The Authors have drawn on a variety of published data relating to the dilution of jets of waste water discharged from submerged outfall pipes and to the subsequent dilution of the effluent field formed near the water surface. Their interpretation of these data may be misleading. Fig. 1 compares Abraham's theoretical predictions²⁷ S_m for dilution along the axis of a three-dimensional jet in an infinitely deep body of water with experimental observations by Rawn and Palmer³ of dilution at the water surface S_{surf} as presented 30 years later by Rawn *et al.*⁴ The experimental results were obtained with fresh water, containing dye, discharged through horizontal nozzles suspended below a raft moored in a basin in Los Angeles harbour. Similar data were also obtained for vertical jets. There was no significant tidal velocity so that both sets of curves refer to still or slack water conditions. The dilution factors at the water surface in Rawn and Palmer's study were determined by taking grab samples of the shifting effluent boil and averaging the results. This procedure gives values of S_{surf} which are greater than the predicted centre line values S_m at the surface, especially at higher densimetric Froude numbers.

97. The interpretation of this graphical relation between the dimensionless initial dilution parameters was adequately described by Rawn *et al.*⁴ by the use of Fig. 17 which shows the curves for dilution factors of 10 and 40. As an example they

Fig. 17. Different methods for increasing dilution (after Rawn *et al.*⁴)

considered a discharge of 5.5 m³/s from a 1.8 m dia. pipe at a depth of 33 m ($y_0/D=18$ and $F=3.0$) corresponding to point A. Increasing the depth to 90 m while keeping other conditions constant would correspond to point B on the $S_0=40$ curve. Similarly, decreasing the diameter to 0.7 m could attain point D, but this would entail an increase in the discharge velocity from 2.1 to 13.4 m/s with a corresponding increase in the velocity head loss from 0.2 to 9.4 m. A third way involved providing a number N of smaller outlet ports so that the value of D was decreased but the individual jet velocities remained constant. Assuming no mutual interference between the individual jets, a field dilution of 40 (point C) would be achieved by means of 10 or 11 outlets. There can therefore be no general optimum choice of dilution parameters; account must be taken in any calculation of the limiting conditions of the controlling parameters of water depth, effluent flow, velocity of flow in the pipe, density difference between the effluent and the receiving water, and whether the discharge is to be pumped or can flow by gravity.

98. Water movement past the outfall is a necessary requirement for the discharged effluent to be removed from the vicinity of the outfall. At sites in the UK, slack water conditions exist for only a small part of each tidal cycle so that calculations for such conditions, while representing worst dilutions, are not representative of outfall performance at other tidal states. Empirical observations by Agg and Wakeford⁹⁵ of dilution at the water surface at four outfalls showed that additional dilution between the outfall and the water surface, over that predicted by Abraham⁵ for slack water, was related to the dimensionless ratio between the ambient water velocity and the velocity of the jet leaving the outfall. These data also indicated that Abraham's predictions were applicable when the ambient velocity was zero. Agg and White⁹⁶ used Abraham's predictions and the data from Agg and Wakeford⁹⁶ to construct new design charts by re-arranging the parameters in a more convenient form in which outlet diameter D was implicit in only one of the dimensionless groups. This new presentation showed that when the ambient velocity was greater than zero, dilution increased as the ratio of diameter to depth increased. The limit of a densimetric Froude number of unity is imposed to ensure that the submerged pipe flows full of effluent at all times. This analysis therefore shows that, subject to the limitation imposed by the densimetric Froude number of unity, a larger diameter outlet is more beneficial when the receiving water is moving.

99. In discussing subsequent dilution due to diffusion the Authors have quoted an investigation by Thomas⁹ at a sewage outfall site in New Zealand. The result given in equation (1) was derived from a more general equation which describes the value of D at a point on the surface on the centre line of the sewage plume, at a distance y from the point at which $D=D_0$, for a constant value of the diffusion coefficient k

$$D = \frac{3.54Vh}{q} \left\{ \frac{ky}{V} + \left(\frac{D_0q}{3.54Vh} \right)^2 \right\}^{1/2} \quad \dots \quad (22)$$

The coefficient k was then assumed to vary in terms of the standard deviation σ of the dispersing sewage patch, so that

$$k = p(2\sigma)^s \quad \dots \quad (23)$$

where p and s are constants. A substitution for σ in equation (23) was next made assuming $\sigma = \sqrt{(2kr)}$. This appears to be invalid as a statement of the general case because the relation is true only when k is constant and $s=0$.

100. Thomas⁹ described the outfall as consisting of two pipes, 0.23 m and 0.15 m in diameter, discharging only about 1.2 m below the surface at 45° below the horizontal into a pool, some 9 m wide, among rocks exposed at all states of the tide. He recognized that his equation for the lower limit of dilution with respect to time of drift from the outfall area (equation (2)) was applicable only to the particular conditions at the site investigated in Tahiti Bay.

101. The disappearance of coliform bacteria was found to be such that there was a 90% reduction in count (T_{90}) in 40 min. Thomas acknowledged that this was much more rapid than values quoted by Pearson⁴² and Gaultier.⁴³ Since 1962 the rate of disappearance of coliforms has been studied in different parts of the world under varying ambient conditions. In the UK mortality of coliforms has been studied in samples kept ashore both in the light⁴⁴ and in the dark,⁴⁵ in bottles immersed at different depths in the sea⁴⁶ and in situ experiments at sea.⁴⁷ Incident radiation has been found to be a significant factor in the rate of mortality and values of T_{90} for in situ experiments reported by Gameson⁴⁸ vary from 34 min. to 9 h with a median value of 3.2 h. No measurable die-off was observed in these experiments in the dark. Clearly a T_{90} of 40 min. is applicable only in conditions of bright summer sunshine.

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The Authors' approach to the problem of outfall design and their conclusions are an indication of the shortcomings of practical knowledge of effluent dispersion. They show (§§ 8–23) that a fair amount of work has been done on the dilution within a buoyant plume, and knowledge thus far is adequate for practical design purposes, except that dilution may differ in character in areas of density (salinity) gradient from areas where there is uniform vertical density.

103. The subsequent dilution of the effluent due to tidal action presents the designer with a much more complex problem. This part of the dilution or dispersion process can be divided into three phases.

104. First is the initial dispersion due to turbulent mixing in the spreading plume. This phase will usually operate unidirectionally within the time limit of half a tidal cycle (approximately six hours).

105. The second phase may occupy a time span of usually one, but occasionally two or three tidal cycles. The large-scale rotational motion that frequently occurs at the turn of the tide produces non-uniform transverse mixing in a limited body of water such as an estuary. What may be acceptable intermediate concentration of effluent in a plume under flood or ebb conditions can be transferred, with little change in dilution, to a part of the estuary or coastline where such dilutions are not acceptable. In an estuary or confined water this process plays a large part in producing the long-term transverse dispersion coefficients which will, as a consequence, vary longitudinally.

106. The third phase is that of a long-term two-dimensional dispersion in a body of water when a discharge is considered to be distributed by multiple tides, and to be progressively flushed, with fresh water input, out of the estuary, or along the coast. This phase of the dilution, dispersion and flushing process determines the ultimate capacity of a body of tidally flushed water to deal naturally with an effluent, not forgetting questions (a) and (b) in § 50.

107. Dispersion coefficients are very important in numerical models but unfortunately they incorporate complex physical phenomena which are not easily measured in the field. Moreover dispersion coefficients derived from model experiments cannot be relied on because of inequalities of similitude. Harleman⁴⁹ showed that the criterion for mixing similarity is not satisfied in distorted models and, because of this, many dispersion coefficients derived from models are on the low side. Field studies are difficult as dispersion coefficients are local in nature, being affected by turbulence, shear, gravitational circulation and density gradients that are present in tidal waters. Fischer⁵⁰ presented an assessment of these factors.

108. Care should be exercised in the use of numerical models as computational errors can be inherent in them. For example, in the use of finite element models the accuracy of the solution is affected by the number of sub-divisions used to approximate the continuous domain, as well as the order of the approximating model.⁵¹

109. At the University of Dundee's Tay Estuary Research Centre fluorometric tracking techniques are being used to measure concentration distribution in sewage

plumes (first phase) and these are being related to local dispersion coefficients through a fine mesh finite element model.⁵²

110. The same field technique is proving useful in tracking the short-term transverse displacement of partially diluted sewage (second phase), but as this is a unique situation it is unlikely ever to be amenable to mathematical prediction. It therefore looks as though careful site investigation will always be required, as it is at the second phase that local coastal pollution problems occur.

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Any furtherance of knowledge relating to this subject, which is of considerable importance in regard to the growing fears of irreparable ecological damage to the environment, is particularly worthwhile and of direct use to the design engineer.

112. From the theorist's viewpoint, the problem of predicting events—associated with the design of outfalls—presents hitherto insurmountable difficulties. These are to some extent alleviated by the incorporation of measurable parameters obtained from site. However, the complications incurred from the highly transient physical conditions pose problems which are outside the scope of current solution techniques and available computational hardware. The highly transient nature of the problem is related to variations in effluent flow and strength, stratification, short-term and long-term currents and tidal levels, and the timewise build-up or decay of local toxicity levels. These place the solution outside the bounds of an exact science. Recourse is therefore made by practising engineers to a semi-empirical approach.

113. At present it appears that no amount of theory can replace practice. This is particularly apparent from the contributions by both **Mr Ackers**, and **Dr Charlton** and **Mr Apostol** which suggest that future work should be directed towards the study of existing outfalls and the monitoring of future outfalls, and not merely to an extension of theory. It is hoped that the Water Research Centre will continue to devote time and resources to this practical side of their work.

114. In reply to **Mr Green**, the justification of the use of a two-dimensional model for predicting the velocity distribution in a well mixed estuary is a widely accepted assumption. If these assumptions are not valid, as in the case of salt wedge movement, then a full three-dimensional analysis including stratification would be necessary. We did not claim a generality of approach and this is immediately apparent from the governing equations used.

115. As always in the type calculations advocated the availability of instantaneous comprehensive data is rare. This is because of the practical difficulties involved. The field measurements are therefore not comprehensive enough for a full analysis. This is also the situation relating to further examples presented in reference 53 where a limited comparison can be made.

116. Convergence of the numerical results was not sought for the Paper, the purpose of which was to show that such a technique exists. Obviously convergence would be necessary if a comprehensive analysis were to be undertaken.

117. The accumulation of toxic matter and likely permanent damage to fauna was not our concern. This is left to the biologists who could use the data from an analysis such as that indicated. Until such research is undertaken by the natural scientists no real measure of the toxicity of effluents can be formulated.

118. In reply to **Dr Sharp**, the Paper was not intended as a review of previous work; the main object was to present a sequential procedure for design. Indeed, a similar combination of equations can be used for the thermal problem mentioned.

119. The term optimum dilution was unfortunate; a better term would have been critical dilution, i.e. the dilution one could be sure of obtaining for a wide range of F numbers. In practice an outfall has to deal with flows ranging from say 1 d.w.f. to 7 d.w.f., unless balanced or stored, with velocities ranging from 25 ft/s to 17.5 ft/s and this combined with a minimum outlet diameter of 5 in. This means a range in the F number from 4.5 to 32, or with jet velocities at twice the pipe velocities, 5–35 ft/s,

F numbers from 9 to 63. At 7 d.w.f. an initial dilution in the outfall of say 2:1 is probable but at 3 d.w.f. peak flows would give a sewage of normal strength. The discussion and this point to the need for a careful consideration of all conditions at each outfall, but do not detract from the statements made in the Paper.

120. We do not know of a scheme to incorporate local conditions regarding turbulence. Each site must be considered individually and local effects taken into account if possible.

121. There are, for obvious reasons, difficulties in obtaining L , especially if it is desired to cover all circumstances. We assumed that a parabolic shape giving approximately $L = 1.33 Y_0/D$ would suffice for most practical purposes. This will indeed vary with the velocity at the jet. Rawn and Palmer⁵³ give two approximate formulae for L , and also give a proof of the relationship $S = S_0 \sqrt{(x/x_0)}$ together with details of practical observations which go some way towards confirming the theory.

122. Mr Ackers' contribution indicates some of the many ways in which the Paper can be augmented. The simplification of the calculations suggested in the comparison of a ten outlet diffuser with a simple outlet in water 2.5 times as deep yielding the same surface dilution is particularly useful.

123. We are grateful to Mr Agg for pointing out particularly relevant information regarding initial dilution. The fact that the T_{90} dilution by Thomas⁶ was only applicable in the bright sunlight conditions in Tahiti Bay and Gameson's alternative⁴⁸ was for the UK must be taken into account. There appears to be room for further work on the combined effect of dilution, dispersion and the die back of the bacteria. This could possibly follow the work done by Covill.⁵⁴

124. Dr Charlton and Mr Apostol have introduced a conceptually simple but mathematically extremely complex phenomenon. Again the importance of field data is highlighted which is, undoubtedly, the area where considerable effort must be directed in order to achieve meaningful results.

125. We agree with their comment regarding the accuracy of a finite element solution. However, an indication of convergence was not an objective of the Paper.

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Corrigendum

Equation (1) *should read*

$$D = \frac{3.54}{W} \left\{ \frac{t}{25} + \left(\frac{D_0 W}{3.54} \right)^{2/3} \right\}^{3/2} \quad . \quad . \quad . \quad . \quad . \quad (1)$$